THE PERFORMANCE OF STABILIZED AND UNSTABILIZED MINESTONE IN AN AQUEOUS ENVIRONMENT WITH EMPHASIS ON ITS EROSION RESISTANCE

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THE UNIVERSITY OF ASTON IN BIRMINGHAM

THESIS SUMMARY

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Britain's sea and flood defences are becoming increasingly aged and as a consequence, more fragile and vulnerable. As the government's philosophy on resources shifts against the use of prime quarried and dredged geo-materials, the need to find alternative bulk materials to bolster Britain's prone defences becomes more pressing. One conceivable source for such a material is colliery waste or minestone. Although a plethora of erosion-abrasion studies have been carried out on soils and soil-cements, very little research has been undertaken to determine the resistance of minestone and its cement stabilized form to the effects of water erosion.

The thesis reviews the current extent to which soil-cements, minestone and cement stabilized minestone (CSM) have been employed for hydraulic construction projects. A synopsis is also given on the effects of immersion on shales, mudstones and minestone, especially with regard to the phenomena of slaking.

A laboratory study was undertaken featuring a selection of minestones from several British coalfields.The stability of minestone and CSM in sea water and distilled water was assessed using slaking tests and immersion monitoring and the bearing on the use of of these materials for hydraulic construction is discussed. Following a review of current erosion apparatus, the erosion/abrasion test and rotating cylinder device were chosen and employed to assess the erosion resistance of minestone and CSM. Comparison was made with a sand mix designed to represent a dredged sand, the more traditional, bulk hydraulic construction material.

The results of the erosion study suggest that both minestone and CSM were more resistant to erosion and abrasion than equivalently treated sand mixes. The greater resistance of minestone to the agents of erosion and abrasion is attributed to several factors including the size of the particles, a greater degree of cement bonding and the ability of the minestone aggregate to absorb, rather than transmit shock waves produced by impacting abrasive particles. Although minestone is shown to be highly unstable when subjected to cyclic changes in its moisture content, the study suggests that even in an intertidal regime where cyclic immersion does takes place, minestone will retain sufficient moisture within its fabric to prevent slaking from taking place.The slaking study reveals a close relationship between slaking susceptibility and total pore surface area as revealed by porosimetry. The immersion study shows that although the fabric of CSM is rapidly attacked in sea water, a high degree of the disruption is associated with the edges and corners of samples (ie. free surface) while the integrity of the internal fabric remains relatively intact. CSM samples were shown to be resilient when subjected to immersion in distilled water.

An overall assessment of minestone and CSM would suggest that with the implementation of judicious selection and appropriate quality control they could be used as alternative materials for flood and sea defences. It is believed, that even in the harsh regime of a marine environment, CSM could be employed for temporary and sacrificial schemes.

KEYWORDS Minestone, Cement Stabilization, Abrasion, Erosion, Sea Defences

(Bevan Boy Blues)

To my Mother, Father, Brother and Sister .....

In memory of Tim who died fighting for a life
but lived life fighting.

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1. INTRODUCTION

Despite the relentless decline of the British coal industry, and a dramatic decrease in the overall production of coal, the problem of colliery waste will remain prominent for the foreseeable future. The spoil heaps that overshadow vast tracts of land throughout Britain, though a testament to a productive past, nevertheless provide a detrimental legacy for the present generation. Today, an estimated 4000Mt of colliery waste (commercially and geotechnically known as minestone) idle on the surface of the coal mining regions and, until the devastating rationalization of the past decade, an estimated 50Mt was being added to these stockpiles each year (Glover, 1984). Such stockpiles of minestone tie up valuable agricultural and prospective development land, cause concern as health and safety hazards, can be detrimental to their surrounding environments and may blight the aesthetic appeal of the local landscape.

With the advent of nationalisation in 1947, the National Coal Board (NCB) was given the responsibility for managing, not only the waste arising from post-1947 mining, but those accumulations amassed prior to nationalization. Thus, nationalization instantly gave the NCB the mantle of being the largest source of waste material in Britain, both in terms of historic stockpiles and of present production (Rainbow, 1982). The utilization of colliery waste is beneficial both to British Coal (the NCB became British Coal in 1985) and the general community, so in 1971 the NCB established the Minestone Executive to research and develop the potential of 'Minestone' (Turnbull, 1984). Minestone is the commercial name given to unburnt colliery spoil to distinguish it from burnt colliery spoil that has been affected by heap combustion, commonly referred to as Burnt Colliery Shale. Minestone Services, the successor to the Minestone Executive, have been given the task of promoting the commercial exploitation of Minestone, Burnt Colliery Spoil and all surface minerals other than coal, including sandstone, limestone, fire clay, special sands, sands , gravels and clays. They have been fully integrated with the parent organization, British Coal, both for U.K. and international activities. Minestone Services have promoted the use of minestone in a wide range of applications including cement stabilization, reinforced earth structures, recultivation and reclamation, energy recovery, brick making, specialized aggregate production and general landfill.

While the Netherlands has extensively used minestone for coastal defences and other hydraulic applications and the USA has relied heavily upon comparatively low quality soil-cements for water storage purposes, Britain's experience with either materials for similar purposes has been negligible. The compounded threat of rising sea levels and isostatic subsidence of the threatened regions (New Scientist, 1993a), the ever increasing price of traditional dredged and quarried materials (Times, 1993), concern over the environmental impact of dredging and quarrying and the increasing favourability of 'soft engineering' and 'managed retreat philosophy' for coastal defenses (MAFF, 1992), all act as incentives for considering the use of a less traditional material. Indeed, municipal thought, in particular that of Essex County Council, has already shifted towards the possible use of waste materials to bolster the ever increasing fragility of Britain's coastline (New Scientist, 1993b). The sheer quantity of minestone available in the UK, its proven success as a bulk material and its relative cheapness would suggest that minestone would be an ideal candidate for large scale hydraulic projects.

Although some aspects concerning the performance of minestone (both unstabilized and stabilized) in the presence of water have been partly addressed in previous studies (Rainbow, 1984; Laan *et al.* 1984; Sleeman, 1984; McNulty 1985; Thomas, 1987), research into the erosion resistance of this material in an aqueous environment has been scant. Even in the Netherlands and Germany where minestone has been looked upon in a more original and innovative light than in the UK, little research has been carried out to assess its erosion resistance compared to more traditional hydraulic construction materials.

This study investigates the performance of both unstabilized and stabilized minestone in an aqueous environment. Both distilled water and sea water were used in the research, to represent the two major aqueous environments, namely freshwater and marine. Eight minestone samples were collected from Welsh and English collieries, being chosen for either their association with aqueous environments, the likelihood of their use, if proven suitable, for a hydraulic application or, in the case of the Hatfield minestone, its notorious behaviour when immersed in water. The chemical, mineralogical and physical characteristics of each minestone were determined using various analytical techniques. The slaking characteristics of each minestone were determined using a number of techniques including tests that simulated the various micro-environmental zones associated with an aqueous structure. The stabilized minestone samples were subjected to strength and durability tests including tests to determine the effect of long term immersion. The rotating cylinder shear device and the erosion/abrasion apparatus were chosen to determine the erosion resistance of both unstabilized and stabilized minestone. Both pieces of apparatus had been used in previous research into the erodability of soil-cement in aqueous environments. Comparisons between minestone and a laboratory sand mix, that simulated a traditional hydraulic dredged sand, were made using the erosion/abrasion apparatus.

2. MINESTONE

2.1 Introduction

Coal reserves in Britain are predominantly of Carboniferous age and mainly belong to a diachronous series of rocks known as the Coal Measures. The Coal Measures are characterised by cyclothemic sequences which usually culminate in coal seams. A typical coal cyclothem is shown in *Figure 2.1*. Cyclothems represent the processes that predominated in the low energy, oxygen depleted coastal swamps and deltaic environments that existed in abundance during the Coal Measures. The resultant lithologies have a profound effect upon the engineering properties of the colliery spoil derived from working the coal seam. As a consequence of the low energy, depositional environment, siltstones, mudstones and shales will dominate the cyclothemic sequences. Coal only forms in an oxygen depleted environment but, such environments also allow syngenetic pyrite to develop in sizable quantities. It is the nature of these fine grained lithologies, together with pyrite that will govern the physical and chemical properties of the colliery spoil and hence its subsequent engineering capabilities.

Colliery spoil is mainly composed of the shales, siltstones and mudstones that form the roof and floors of coal seams and which may even form laminae within the seam itself. Other rocks won from the driving of shafts, drifts and 'roads' are also incorporated into the resultant colliery spoil. The bulk of this colliery spoil is tipped onto spoil heaps. It is estimated that there are approximately 4000Mt of colliery spoil stored on the surface in the UK, in the form of operational tips, disused tips and restored sites (Sleeman, 1990). The advancement of extractive technology has had a major influence on the non-coal content of the excavated material (Glover, 1984). Today, 35 per cent of all extracted material would be considered as being non-coal whereas, in 1920, only 5 per cent of the extracted material would have been classified as non-coal. An estimated 50 to 60Mt of newly produced spoil is created each year, of which 74 per cent is tipped at the surface, 6 per cent per cent is used in underground packing, 9 per cent is dumped at sea and 11 per cent used by power stations. It is estimated that approximately 3Mt of colliery spoil stored on the surface is used each year (New Civil Engineer, 1991), while an estimated 60Mt of spoil has been put to constructional use since 1968 (British Coal, 1990). If possible incentive schemes are implemented, future estimates for the usage of minestone range from the present quantity to a projected estimate of 15Mt (New Civil Engineer, 1991)

2.2 Colliery Spoil Classification

There are three basic categories of colliery spoil which can be considered for commercial use (Sleeman, 1987) :

1. Burnt colliery spoil (burnt minestone).
2. Modified/ processed colliery spoil.
3. Unburnt coarse colliery spoil (minestone).

2.2.1 Burnt Colliery Spoil

Prior to the Aberfan disaster and especially before the post war advances in mining technology, all spoil tips were constructed by loose tipping with the resultant structures being uncompacted, loose textured, air permeable heaps which were prone to combustion. Although spontaneous combustion, as a result of the exothermic degradation of pyrite and coal, may have ignited some spoil heaps, the majority have been ignited by the tipping of hot ashes from colliery boilers and coke ovens (Collins, 1976). The residual calcined rock remaining within these fired heaps are now known as burnt minestone. British Coal is today bound by legislation to prevent new ignition of spoil heaps. This has been achieved by improved spoil heap management and monitoring, spoil compaction and improved washery techniques to reduce the percentage of residual coal. With these improved techniques of spoil disposal it is thought that no new stocks of burnt minestone will be added to the present limited reserves (Tanfield, 1971).

Today, burnt minestone is reserved for more technically demanding applications for which minestone is thought unsuitable. It is used as a special fill material for use below formation level in roadworks, a free drainage material, an alternative to hardcore or type 1 and 2 sub base material, and as surface aggregate at speedway circuits and sport grounds (British Coal, 1990). Although burnt minestone may be superior in some respects to minestone, the present research concentrates on minestone as the vast majority of colliery spoil reserves can be so classed. Burnt minestone will play no further part in the following study.

2.2.2 Modified and Processed Colliery Spoil

Sleeman (1987) acknowledged that the majority of applications for burnt and unburnt minestone would be for infill purposes. However, by modifying and processing these materials more specialised and demanding applications could be approached. Cement stabilized minestone used as a structural material, reinforced minestone as earthwork structures, brick making and cement manufacture are all forms of modified or processed minestone.

2.2.3 Unburnt Colliery Spoil

Once coal separation has occurred in the washeries, colliery discard can be divided up into two fractions:

1. Coarse discard (minestone).
2. Fine discard (slurry and tailings).

Generally fine discard has a maximum particle size of 0.5mm, however actual rock particle size may range from clay to medium gravel grade (Rainbow, 1982). Due to the high moisture content, the fine discard is usually allowed to de-water in sedimentation lagoons. Another significant characteristic of the fines is a very high organic carbon content, which on average approaches 50 per cent by weight of the fine discard (Sleeman, 1987). Fine discard is regarded as being a major logistical and economical problem for British Coal and, though research into possible outlets continues, at present fine discard is recognised as being an unmarketable 'resource' (Sleeman, 1990).

Coarse discard (minestone) is generally black or grey in colour and is a heterogenic mixture of rock debris and mineral fragments. The general composition of a minestone is mainly a function of the geographical location and geological setting from which the discard has been excavated (Thomas, 1987). There is a high degree of variation within spoil heaps which is to be expected given that some heaps have been operational for several decades in which time mining technology has changed and seam switching at the colliery will have occurred on a number of occasions. One spoil heap may service several 'faces' which may be several miles apart and possibly of differing geological aspects. Furthermore it is not unknown for one spoil heap to service two or more collieries during its operational life and so local geological factors may come into effect allowing a minestone to have varying engineering properties despite being sampled from a single spoil heap.

2.3 Properties of Minestone

2.3.1 Physical Properties

Minestone has been described as a problematic rock soil material (Rainbow and Nutting, 1986). It neither fits snugly into a soil classification, as seen from the point of view of a civil engineer nor does it classify as a rock when considered by a geologist. Minestone has the misfortune of overlapping both classifications and thus often falls foul of 'sacrosanct' specifications that are associated with the more standard construction materials. However, despite these difficulties, some minestones, if carefully selected, can meet existing specifications and compete with the more traditional materials.

The grain size distribution for British minestones is depicted in *Figure* 2.2 (Rainbow, 1982). Modern mining, washery and tipping practices ensure that only a minor quantity of discard exceeds 200mm in size (Rainbow, 1983; Canibano and Leininger 1987). Minestone generally falls into the ranges of silty sand / gravels and medium to coarse gravels with cobbles (Sleeman, 1987). When using the Casagrande plasticity chart the finer portions of the minestone can be classified as being silts and inorganic clays with medium plasticity. However, it has been estimated that up to 50 per cent of the finer fractions of both coarse and fine discard are non plastic in nature (Sleeman, 1987).

The mean natural moisture content of tip samples is 11 per cent and the mean bulk density for U.K. minestone is 1902kg/m^3. The specific gravity of minestone ranges from 1.08 to 2.90 with the mean being 2.30 (Rainbow, 1983; British Coal, 1989). It is thought that the relatively low specific gravity values for some minestones reflect the high coal content of the spoil material. Coal has a specific gravity of between 1 and 1.8 (Read, 1976). This range counteracts the higher specific gravities possessed by quartz and the clay minerals which are found in minestone (Rainbow, 1983; Morton *et al.* 1984).

2.3.2 Chemical Composition

The pH value of freshly wrought minestone is generally neutral to slightly alkaline. However, if extensive oxidation of pyrite occurs during the process of weathering, its pH will drop as it becomes more acidic with age. The pH of minestone exposed in tips ranges between 2.7 and 9.6 and has a mean value of 7 (Rainbow, 1983; Rainbow and Sleeman, 1984). A comprehensive database relating to the chemical composition of UK minestone has been collated by Minestone Services (British Coal, 1989). The content range of the major chemical constituents are given in oxide form below :

Silicon	SiO_2	18-63 %
Iron	Fe_2O_2	2-40 %
Aluminium	Al_2O_3	6-27 %
Carbon	CO_2	5-21 %
Calcium	CaO	0-10 %
Sulphur	SO_2	0-7 %
Potassium	K_2O	0-5 %
Magnesium	MgO	0-4 %
Titanium	TiO_2	0-3 %
Sodium	Na_2O	0-2 %

The aforementioned database also presents the chemical ranges for individual mining regions throughout the United Kingdom. The information within this database indicates that the same chemical trends occur in all British coal regions. A nationwide study suggested that, with the exception of carbonaceous material/organic content, no major differences exist between the spoil heaps of different regions (Collins, 1976).

It is well known that concrete and cement stabilized materials will be detrimentally affected by the presence of sulphates at certain concentrations. A sulphate concentration of 1:1 shale : water extract should not exceed 0.2 per cent if used adjacent to a structure in which ordinary portland cement plays an integral role (British Coal, 1989). Mild steel reinforcement straps for reinforced earth structures are also attacked if sulphate concentrations exceed 1000ppm (0.1 per cent) (Rainbow and Sleeman, 1984). A concentration of less than 0.8 per cent acid soluble sulphate was one of the criteria thought necessary for cement stabilized minestone to achieve specified compressive strength and durability values (Thomas, 1987). The mean value for water soluble sulphate for British minestones is 0.1 per cent. Whilst most have acid soluble sulphate concentrations of less than 1 per cent some individual samples may exceed a concentration level of 3 per cent (Rainbow, 1983; British Coal, 1989). The 1989 database also indicates that total sulphur content (expressed as SO_3) for British minestones ranges from 0.2 per cent to 9.9 per cent, but with the vast majority of samples falling below a concentration of 5 per cent . The effects of sulphate attack upon cement stabilized materials will be discussed in *Section 4.4.*

2.3.3 Mineralogy

Minestone is dominated, mineralogically, by the hydrous aluminium silicates, more commonly known as the clay minerals. The chief detrital clay minerals are kaolinite, illite, montmorillonite, chlorite and the mixed layer clays. A detailed mineralogical study of minestone is important, as chemical analysis alone fails to illuminate any major differences between spoil from different collieries with the exception of carbon content (Collins, 1976). The favoured technique for mineralogical determination is X-ray powder diffraction analysis (Shultz, 1964; Spears *et al.* 1970; Spears and Taylor,1972; Collins, 1976; Thomas, 1987). Some early studies had limited success in quantitative mineralogy using optical microscopy (Dixon *et al.* 1964). Other analytical techniques that can be used in the study of the clay minerals within minestone include infrared spectroscopy, thermal methods, transmission electron microscopy (TEM), scanning electron microscopy (SEM) and chemical analysis (Wilson, 1987).

A nationwide study of minestone (Collins, 1976) revealed that illite and kaolinite are the most abundant clay minerals, accounting for up to 36 per cent and 38 per cent of each sample respectively.The English and Welsh spoils are characterised by illite dominance, whilst Scottish spoil is characterised by a dominant kaolinite content. Mixed-layer clays form the next most abundant group of minestone clays. In Britain, these can be regarded as the most important clay group where engineering projects are concerned as they are the most sensitive to variations in moisture content. Although montmorillonite is a very important clay mineral in minestones from other countries (e.g. Masatomo, 1989), it has not been recognised as a separate mineral phase in British Coal Measure rocks (Spears and Taylor, 1972; Collins, 1976). It has however, been noted as a component of some mixed layer clays.

2.3.4 Lithology

All rock types present in the Coal Measures are present in minestone (Taylor and Spears, 1970). Both the floor and roof of coal seams are argillaceous in nature leading to a predominance of clay based rocks over quartz based sedimentary rocks in mining spoil. This is a global phenomenon and not just a feature of British minestone (Knutz and Plogmann, 1979; Leininger *et al.* 1987). However, there are exceptions such as the South Yakutia Basin, in what was the Soviet Union, where between 60 per cent and 70 per cent of the spoil is sandstone and South Korean colliery waste which is composed of at least 60 per cent sandstone (Canibano and Leininger, 1987; Youn and Ki, 1990).

There are four lithological types in minestone (Lawrence, 1972), these are mudstone, siltstone, sandstone and seat-earth. Further classification may be carried out using various criteria to subdivide the major lithological types. Mudstones become carbonaceous if appreciable amounts of free carbon matter are present and, if they possess the property of fissility, they may described as shales (Grainger, 1984). The paler the colour of minestone the less likely it will contain appreciable amounts of combustible material. The darker mudstones, such as the black shales, are thought to contain the highest amount of carbonaceous, and hence, combustible material. At the lower end of Lawrence's classification (1972) based upon impact strengths, are slickensided mudstones which have undergone deformation during faulting.

Seat-earths are the fossilised root beds of the Coal Measure plants and hence are to be found directly beneath the coal seams. They are characterised by abundant fossilised rootlets set in unlaminated siltstones and mudstones (Taylor and Spears, 1970). Sandstones may vary from being well-sorted, mature and quartzitic to being poorly-sorted, immature and muddy. In addition to the major lithologies, limestone may be present, either derived from marine bands within the Coal Measure strata or as part of the overburden covering the Coal Measures rocks. Coal may also be described as a lithological type and is an important constituent of spoil heaps. It is estimated that the national average coal content of spoil heaps is about 11 per cent but in older tips the coal content may be significantly higher due to the large quantities of unsaleable 'small coals' (Sleeman, 1987).

2.4 The Location of Minestone Resources

The location of minestone stockpiles will obviously mirror the location of the coal mining regions. All British coal mining districts, with the exception of the Kent and Somerset regions, lie north of a line drawn between the Severn Estuary and the Wash (*Figure 2.3*). The southeastern sector of Britain is noticeably devoid of coalfields. Considering that a large proportion of British development is concentrated in the Southeast, it seems that minestone is unfortunately a misplaced resource. At present the bulk of fill materials and construction aggregate for southeastern development is derived from local borrow pit schemes. As time passes and the more accessible reserves of traditional construction materials are used up, minestone may become more competitive especially if local rail links are available. Borrow pit schemes are frequently becoming undesirable as environmental concerns are increased, helping minestone to become a more attractive material. The ability of minestone to be a competitive resource has already been shown and there are examples where it has shown commercial viability at distances of up to 210km from the source site (British Coal, 1990). *Table 2.1* gives the location of minestone resources in relation to areas within Britain that have the potential to use large quantities of minestone for possible hydraulic schemes and it also outlines possible transport considerations.

2.5 The Weathering of Minestone

Minestone may experience three types of weathering, namely physical, chemical and biological. Immediately following excavation, physical disintegration is the most important weathering process but, with time, particle surface area increases allowing chemical and biological weathering to play an increasingly larger role in minestone degradation.

2.5.1 Physical Weathering

Natural degradation of minestone will have already begun even before the spoil has reached the spoil heap. Following excavation a certain degree of stress relief occurs within the minestone, which may account for some of the cracks that run perpendicular to bedding structures (Nichols, 1980; Perry and Andrews, 1983). Physical degradation of minestone is the predominant form of weathering on spoil heaps (Spears *et al.* 1970; Laan *et al.* 1984). Close examination has shown that this degradation process only occurs within the top few metres of the heap's surface, the inner sanctums of the spoil heap being unaffected by physical processes (Spears et *al.* 1970, Laan *et al.* 1984). The outermost layer (1-2m) of a tip is subjected to continuous changes in its physical environment, such as fluctuations in moisture content and temperature, all of which are conducive to physical weathering. The minestone beneath this outer 'protective skin' is subjected to a more constant environment, where temperature and moisture fluctuations are uncommon.

Taylor and Spears (1970) and Taylor (1988) discussed the role of geological structures in the breakdown of Coal Measure rocks. They concluded that degradation occurs along planes of weakness such as bedding and lamination planes, jointing, microfaults and listric surfaces. Parallel to sub-parallel orientation of clay minerals to the original bedding surfaces also provides major lines of weakness along which degradation may occur. The importance of fabric in relation to physical degradation has been further discussed by Russell (1982) (*see Section 4.2.2.1*).

The two most significant forms of physical weathering are freeze/thaw cycling and wet/dry cycling. By differing means, both involve the production of stresses within and between the rock components. The mechanisms of stress development during wet/dry cycling are discussed in *Section 4.2* which deals with the effects of immersion upon minestone. Freeze/thaw cycling (frost weathering) produces stresses through the growth of ice crystals within small rock fractures, joints and pores (Ollier, 1984; Fookes *et al.* 1988). The anomalous expansion and density decrease associated with water freezing, causes a volume increase of approximately 10 per cent which generates tensile stresses sufficient to rupture most lithological types whilst the development of clusters of parallel ice crystal needles, growing normal to the freezing surface, can generate stresses that are ten times greater than those arising from simple volume expansion (Leeder, 1983). Conventional frost action, however, is thought to play only a minor role in the degradation of argillaceous rocks when compared to the sorbtive interactions of water vapour or water in its liquid phase (Dunn and Hudec, 1972). Water, which is a highly polar fluid, is attracted to the surface of minerals by Van der Waal forces. Essentially this adsorbed water is unable to be frozen (Trehaile and Mercan, 1984). The relative water vapour pressure within argillaceous rocks is high as a consequence of a decreased rock capillarity value. High water vapour pressure inhibits freezing and as a result, water in argillite pores tends to be super cooled but not frozen (Hudec, 1973). The low effective porosity of argillaceous rocks will also lead to a lower susceptibility to frost shattering (Lautridou and Ozouf, 1982). With decreasing temperatures a certain degree of adsorbed water will be expelled, this expulsion may lead to fabric damage. In cool, temperate climates, alternating increases and decreases in temperature may lead to damaging pressures being set up as a result of water adsorbtion and expulsion (Dunn and Hudec, 1965; Litvan, 1972; Hudec, 1977).

Lautridou and Ozouf (1982) suggested that damage due ice formation during freeze/thaw cycles was minimal, but damage as a result of adsorbtion/expulsion cycles could be significant. It is thought (Sleeman, 1987) that frost action is only a very minor factor in the breakdown of minestone. Furthermore if minestone is to be used for coastal structures, any susceptibility to frost degradation could be further diminished by the depression of the freezing point of water when salt is present. The number of freeze/thaw cycles in a given period would probably be reduced as a result of freezing point suppression (Dunn and Hudec, 1965; Lasan and Cody, 1967; Dunn and Hudec, 1972; Litvan, 1975; Skarzynska and Porebska, 1990).

The physical effects of salt crystallisation have been underestimated as an agent of weathering. Salt expansion may produce stresses in three ways, hydration, heating and crystal growth (Goudie *et al.* 1970; Cooke, 1979), the latter two seeming to be the most important (Leeder, 1983), occurring in coastal areas at all latitudes where salts concentrate and where dews, mists, sprays and rainfall provide the necessary aqueous phase. Despite this, weathering due to salt crystallisation has been shown to be far more effective in hot, semi-arid climates than in temperate regions (Fookes *et al.* 1988; Obika *et al.* 1989). Thermal cyclicity may also set up stresses within a rock as a result of differential thermal expansion of component minerals.

In a marine/coastal environment physical processes would have the greatest effect at the water/structure/atmosphere interface. This zone is characterised by intermittent submersion and subaerial exposure producing extreme fluctuations in moisture content and temperature (*see Section 5.3*).

2.5.2 Chemical Weathering

Fundamentally, chemical weathering (as with all chemical reactions) consists of reactions that occur because of a lack of equilibrium between substances and their immediate environment (Curtis, 1976). In order for normal chemical reactions to take place, it is essential for an aqueous phase to be present. A marine environment, with its aggressive sea salts and permanent aqueous phase can be regarded as one of the most stressful environments available for promoting and sustaining chemical attack. The rate of chemical weathering can be accelerated by increasing the surface area, increasing the temperature or by the interaction of a catalyst which could be a biological (*see Section 2.5.3*) or chemical agent.

The susceptibility of a minestone mineral to chemical weathering depends upon how far out of equilibrium it is in relation to the surface environment. Minestone minerals can be divided into two groups depending upon the state of equilibrium, the detrital minerals and the syngenetic/authigenic minerals.

Quartz and the clay minerals are the dominant detrital minerals in minestone. They have undergone previous cycles of weathering and thus have been preconditioned to the surface environment and its processes. These minerals can be regarded as being relatively stable and inert and possess a high resistance to chemical degradation. However, there are extreme instances where quartz may come under attack. In highly alkaline environments, silica may dissolve leading to a weakening of the rock fabric. Detrital quartz is thought to be unaffected by high levels of alkalinity, but stressed quartz can be highly sensitive to alkaline reaction. In the Coal Measures, quartz can become stressed in shear and fault zones, but quantitatively, these would constitute an insignificant amount of the overall material. Extreme chemical conditions such as the acidification of spoil heaps resulting from extensive pyrite decomposition, could also cause rock fabric damage by the structural breakdown of the clay minerals (Taylor and Spears, 1970; Pye and Miller, 1990).

Clay minerals may be affected to a minor degree by interlayer cation substitution and the formation of isomorphic equivalents of the original mineral. The phenomenon of cation exchange refers to the property that clays have of replacing one metallic cation with another. This form of hydrolysis (Williams, 1982) has a strong bearing upon chemical weathering and engineering properties of a geo-material (Grim, 1968). The cation exchange capacity (CEC) is a measure of the type and amount of clay minerals present within a soil and is defined as the number of milliequivalents of exchangeable cations per 100 gm of dry soil (Ariathurai and Arulanandan, 1978). Clay minerals differ somewhat in surface charge densities (i.e.the number of exchange sites per unit area of particle surface), hence there will be a great difference between the cation exchange capacity of different clay minerals (Hillel, 1971). The CEC of a clay will to some extent control its swelling properties. Generally, the higher the cation exchange the greater the ability of the clay to adsorb water and swell. Montmorillonites have relatively high CEC's, illites have intermediate capacities, whilst kaolinites have a relatively low capacities (Grim, 1968).

Depending upon the state of hydration and the composition of their exchangeable cations, clay particles may either flocculate or disperse according to the environmental conditions prevailing (Wilson, 1987). Dispersion generally occurs with monovalent and highly hydrated cations e.g sodium, whilst flocculation occurs at high solute concentrations and/or in the presence of polyvalent cations such as calcium (divalent) or aluminium (trivalent). Dehydrated dispersed clays form dense hard crusts, whilst dehydrated flocculated clays form loose crumbly assemblages of aggregated clay. When wet, dispersed clays are likely to be muddy, more impermeable and more highly erodable than flocculated clays (Hillel, 1971).

The attraction of a cation to a negatively charged clay platelet generally increases with increasing valency. The cation preference order for exchange reactions is :

$$Al^{3+} > Ca^{2+} > Mg^{2+} > K^{+} > Na^{+} > Li^{+}$$

The highly hydrated cations, those furthest away from the platelet surface, are also more easily substituted than less hydrated cations.

The syngenetic and authigenic minerals can be regarded as primary as they have not undergone any previous cycles of weathering, unlike the detrital minerals. They either form immediately following sedimentation (syngenetic) or following lithification (authigenic or diagenetic minerals). Pyrite may be used as an example for both groups as it may form at the time of sedimentation or as a diagenetic mineral. In both cases pyrite forms in an environment alien to normal surface conditions. In the case of syngenetic pyrite, formation occurs in a reducing environment rich in organic matter (Coal Measure environment). Once excavated the pyrite finds itself out of equilibrium with the oxygen rich atmosphere to which it has become exposed. Hence the chemical weathering of pyritic minestone will be a function of oxidation (Sleeman, 1987). The chemical oxidation of pyrite by molecular oxygen is a sequence of three steps :

1.Pyrite degradation to sulphate and ferrous iron

$$2FeS_2 + 7O_2 + 2H_2O \longrightarrow 2FeSO_4 + 2H_2SO_4 \qquad \textit{(Equation 2.1)}$$

2.Ferrous sulphate oxidised to ferric sulphate.

$$4FeSO_4 + O_2 + 2H_2SO_4 \longrightarrow 2Fe_2(SO_4)_3 + H_2O \qquad \textit{(Equation 2.2)}$$

3.Oxidation of further pyrite by ferric iron sulphate (Garrells and Thomson 1960)

$$FeS_2 + Fe_2(SO_4)_3 \longrightarrow 3FeSO_4 + 2S \qquad \textit{(Equation 2.3)}$$

No chemical catalyst is known that enhances the reaction rate for the initial step, however the rate of reaction does increase as pH increases (Lowson, 1982). Steps one and two are self regulating, if considered on a purely chemical basis, as the oxidation of iron ceases almost completely in acid media (Penner *et al.* 1970).

The availability of calcium in the presence of sulphuric acid may lead to the formation of gypsum following the reaction expressed in *Equation 2.4.* The sulphuric acid is provided by the reaction in step one, the calcium may be derived from limestone, marls, ankerite, calcitic jointing or from the clay minerals themselves following cation mobilization caused by the action of sulphuric acid.

$$CaCO_3 + H_2SO_4 + 2H_2O \longrightarrow CaSO_4 + 2H_2O + CO_2 \qquad \textit{(Equation 2.4)}$$

Siderite is another Coal Measure mineral which is metastable in atmospheric conditions (Seguin, 1966). Siderite may disassociate to FeO which is subsequently oxidised to haematite :

$$2FeCO_3 + \frac{1}{2}O_2 \longrightarrow Fe_2O_3 + 2CO_2 \qquad \textit{(Equation 2.5)}$$

In moist environments siderite may dissolve to release ferrous iron which can oxidise and reprecipitate, forming a skin of yellowish brown goethite. Oxidation and slow hydrolysis of the released ferrous iron results in the formation of further goethite (Seguin, 1966; Senkavi *et al.* 1986)

The CO_2 released in *Equation 2.5* is dissolved in water to form carbonic acid which, in turn, is capable of dissolving further quantities of calcium carbonate :

$$CaCO_3 + H_2CO_3 \longrightarrow Ca^{2+} + 2HCO_3. \qquad \textit{(Equation 2.6)}$$

Carbonic acid may also lead to the dissolution of dolomite :

$$CaMg(CO_3)_2 + 4H^+ \longrightarrow Ca^{2+} + Mg^{2+} + 2H_2O + 2CO_2{}^{1} \qquad \textit{(Equation 2.7)}$$

Taylor and Spears (1970) concluded that the steady state prevailing within spoil heaps (with the exception of the outer skin) helps to make the processes of chemical weathering of negligible importance in the British Isles. However, it is believed that, when assisted by biological weathering, chemical weathering gains credence as an extremely important process in the breakdown of minestone and an important factor when considering the viability of engineering structures constructed with minestone.

2.5.3 Biological Weathering

Biological weathering of minestone, in particular pyrite, is mainly concerned with the biochemical processes associated with autotrophic, sulphurphile bacteria. Such autotrophs act as catalysts to speed up the chemical oxidation of pyrite for, without the benefits of these catalysts, oxidation by purely chemical means could be very slow (Leathen *et al.* 1956; Lowson, 1982). Ashmead (1955) has cited a reaction rate that was 1 x 10^6 times greater for biological degradation compared to chemical degradation when some species of autotrophs were involved. Experiments performed by Ashmead (1955) suggested that for every ton of sulphuric acid released by purely chemical means four tons were produced when bacterial action was involved.

Two groups of autotrophs exist, anaerobic species that extract oxygen from sulphates (sulphate reducing) and deposit sulphides or elemental sulphur and aerobic species (sulphide oxidants) which help to convert elemental sulphur and sulphides into sulphates and sulphuric acid. When anaerobic soil structures are intermittently flushed by aerated water or are subjected to periods of wetting and drying, cyclic blooms of aerobes and anaerobes may occur. This may happen when the water table changes seasonally or on a diurnal time scale as in the case of tidal cyclicity (Rainbow, 1984). The release of sulphuric acid following aerobe attack may lead to the degradation of feldspars and other silicates, helping to convert them into clay minerals (Loughnan, 1969). Two mechanisms are thought to exist for bacterial oxidation of pyrite, operating concurrently (Silverman, 1967) :

1. Direct contact mechanism between the bacteria and the actual pyrite particle.

2. Indirect contact mechanism where the bacteria oxidise ferrous ions to the ferric state, thereby regenerating the ferric ions required for the chemical oxidation of pyrite.

Although the chemical decomposition of pyrite and ferrous iron is unable to proceed in an acidic environment (Penner *et al.* 1970), it was found that Thiobacillus ferrooxidans could transform ferrous sulphate into ferric sulphate even in high concentrations of sulphuric acid (Temple and Colmer, 1951) :

$$4FeSO_4 + 2H_2SO_4 + O_2 \longrightarrow 2Fe_2(SO_4)_3 + 2H_2O \qquad (\textit{Equation 2.8})$$

The reaction in *Equation 2.8* temporarily reduces the acidity of the environment, since two molecules of sulphuric acid are required. However, this is short lived as hydrolysis occurs releasing further quantities of sulphuric acid (Kuznetzov *et al.* 1963; Hawkins and Pinches 1987) :

$$Fe_2(SO_4)_3 \longrightarrow 2Fe_2(OH)_6 + 6H_2SO_4 \qquad (\textit{Equation 2.9})$$

or :

$$Fe_2(SO_4)_3 + 2H_2O \longrightarrow 2Fe(OH)SO_4 + H_2SO_4 \qquad (\textit{Equation 2.10})$$

In the presence of excess amounts of sulphur, ferric sulphate may be hydrolysed to ferrous sulphate with the aid of microbial action (Temple and Delchamps, 1953) :

$$2S + 6Fe_2(SO_4)_3 + 8H_2O \longrightarrow 12FeSO_4 + 8H_2SO_4 \qquad (\textit{Equation 2.11})$$

The degree of hydrolysis and production of hydroxide $Fe_2(OH)_3$ are dependant upon the ratio of ferrous to ferric oxide, the sulphate content, acidity and the water buffering capacity (Leathen *et al.* 1956; Kuznetzov *et al.* 1963).

The exchange of hydrogen ions for interlayer K, Mg, Fe and Al in micas and illites may lead to the formation of kaolinite (Ivarson *et al.* 1978; Pye and Miller, 1990) :

$$K_2Al_4Si_6Al_2O_{20}(OH)_4 + 2H^+ + 3H_2O \longrightarrow 3Al_2Si_2O_5(OH)_4 + 2K^+$$

(*Equation 2.12*)

Eventually kaolinite may well dissolve:

$$Al_2Si_2O_5(OH)_4 + 6H^+ \longrightarrow 2Al^{3+} + 2Si(OH)_4 + H_2O \qquad \textit{(Equation 2.13)}$$

For the successful development of Thiobacillus ferrooxidans colonies and colonies of related species, optimum conditions must prevail. Being obligate acidophiles, they require a pH range of between 2.0 and 4.5 to enable them to oxidise ferrous iron (Leathen *et al.* 1956). The optimum pH for T. ferrooxidans is 3.5 while for T. thiooxidans the optimum pH for maximum growth and maximum ferrous oxidation is between 2 and 3 (Brock, 1975). The optimum temperature for colony growth lies between 20^0C and 25^0C. However, thermophilic bacteria have been discovered that can thrive and metabolize at much higher temperatures. It has been reported that some thermophiles can oxidise sulphides at temperatures reaching 80^0C (Lowson, 1982). When temperatures decrease below the optimum, the rate of growth and metabolic process decrease and leads to a reduction in the degradation rate of the sulphurous minerals. Both T. ferrooxidans and T. thiooxidans are obligate aerobes, their metabolism being dependant upon adequate supply of oxygen for normal growth to take place. Like other common soil bacteria, these aerobes need a fairly high water potential to carry out normal activity. Brock (1975) has pointed out that "because increased water availability restricts the passage of air, which is so necessary for their development and because conditions promoting best aeration will lead to restricted water availability, many habitats which might otherwise be suitable for the development of these bacteria, might essentially be uncolonized". In older spoil tips such as those studied by Belly and Brock (1974), the breakdown of silicate minerals following sulphuric acid attack may have helped to lower, what they term the water potential of the soil, hence restricting the availability of free moisture necessary for successful colonization. In such a way, the autotrophic bacteria responsible for the acidic conditions could bring about a limitation to their own environment.

Initial development of T. ferrooxidans colonies has been shown to be inhibited by a low ratio of sulphate ions to chloride and other anions. A high concentration of sulphate ions is necessary for the oxidation of iron pyrite (Lazaroff, 1963). Eventual mutation of new strains can occur which are capable of oxidising sulphurous compounds at high chloride concentrations. Colony growth within a tip will initially show an exponential rate of growth, with very little bacterial activity at first due to the low sulphate content but as chemical weathering releases sulphates, bacterial growth will increase, releasing more sulphates into the environment to be acted upon by further increases in the bacterial colony. This conflicts with the findings of Pugh *et al.* (1984) who state that initial sulphate concentration has little effect upon reaction rates. The presence of high organic matter (greater than 0.5 per cent) has been shown to have a retarding effect upon the oxidation of iron pyrite by cultures of T. ferrooxidans (Leathen *et al.* 1956).

The discrepancies highlighted by Thomas (1987) between the slow oxidation process of Taylor and Spears' study (1970) and the rapid decomposition of sulphides as reported by Hawkins and Pinches (1987) and others, (Vear and Curtis, 1981; Caldwell *et al.* 1984; Wilson 1987) can probably be explained by a varying degree of microbial action present within the samples studied. Microbial action may have vastly increased the rate of pyrite degradation in the latter two studies, as is the explanation in the cases reported by Quigley and Vogan (1970), Quigley *et al.* (1973) and Gillot *et al.* (1974).

The dramatic rapidity of pyrite decomposition by bacterial action and the serious consequence of such action was exemplified by the failure of a shale embankment dam in Derbyshire which was still under construction. It was shown that bacterial action had seriously altered the shale with evidence that a 10 per cent removal of mass in some zones of the structure resulted from carbonate dissolution by bio-derived sulphuric acid (Pye and Miller, 1990). Even when shales are inoculated with autotrophic bacteria, decomposition rates of pyrite will vary depending upon how idyllic the conditions are for bacterial colonization and the type and morphology of the pyrite. Relative reaction rates have a direct relationship with surface area. It has also been found that marcasite (a form of iron pyrite) is more susceptible to bio-degradation than normal pyrite (Temple and Koehler, 1958). The order of susceptibility to decomposition based upon morphological type is as follows (Pugh *et al.* 1984) :-

marcasite > framboidal pyrite > impure pyrite > crystal pyrite

The finely divided nature of framboidal pyrite with its high surface to volume ratio makes it far more susceptible to biochemical oxidation than massive pyrite which has lesser surface areas (Kuznetzov *et al.* 1963; Caruccio *et al.* 1976).

In addition to natural weathering by the meteorological elements, 'mechanical weathering' can be an important factor in the degradation of geo-materials in 'engineering time' (Fookes *et al.* 1988). Stockpiling, loading, transportation, unloading, mixing, spreading and compaction may all lead to disintegration of minestone if used for an engineering application (Fookes, 1991).

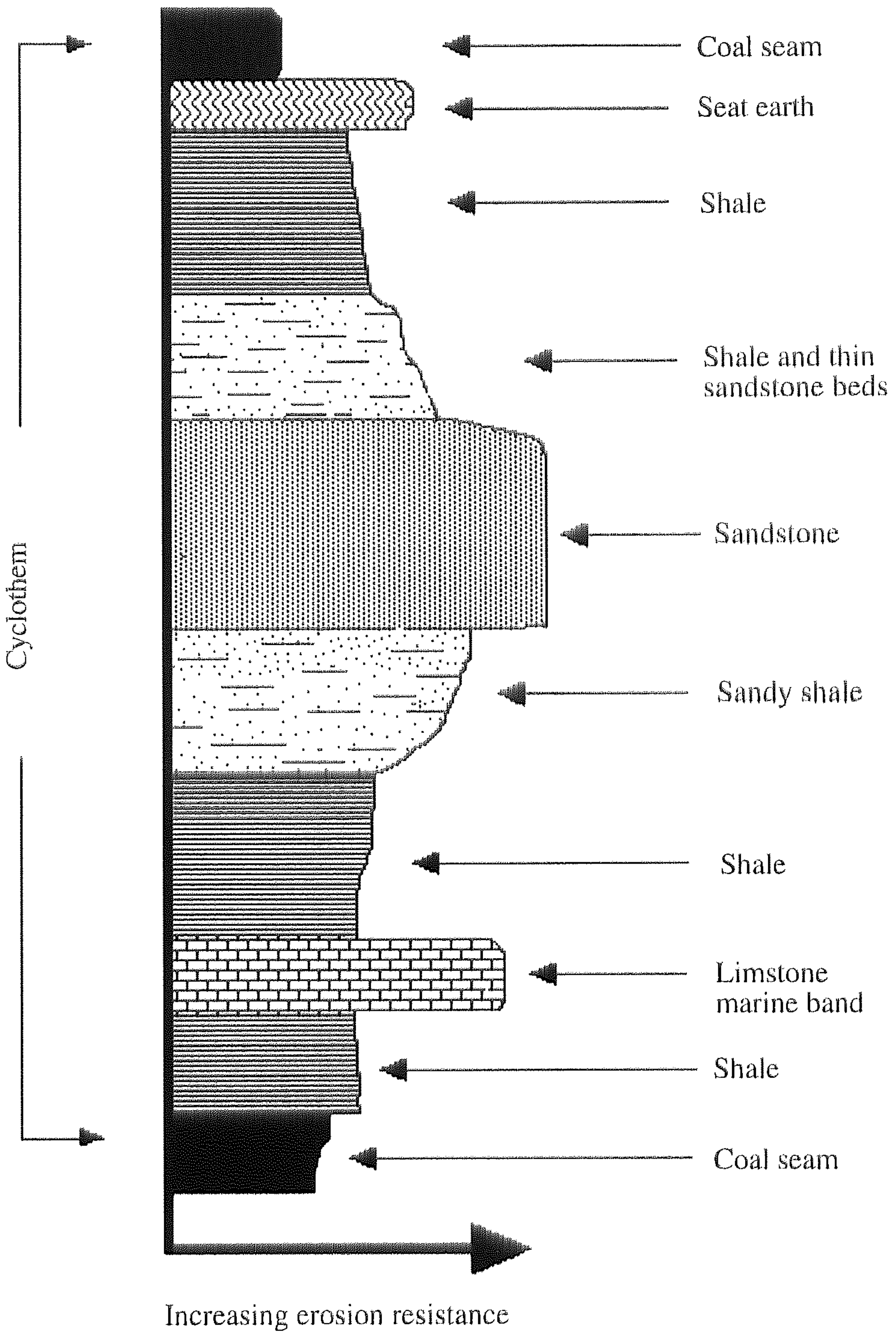


Figure 2.1 Schematic representation of *in situ* Coal Measure rocks.

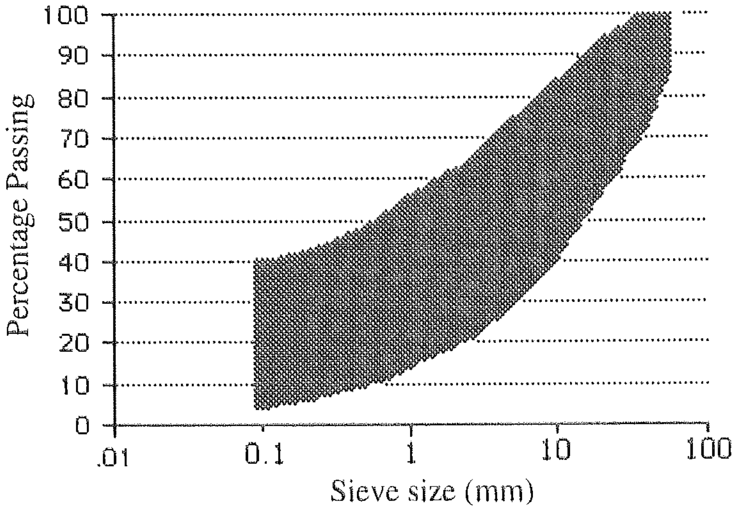


Figure 2.2 The national grading envelope for British minestone.

Coalfield Region	**Areas covered for possible hydraulic development**	**Transport**
South Wales	Severn Estuary and Bristol Channel	Rail, Barge
North East	East Coast of Britain	Barge
Scottish	Firth of Forth and Clyde Estuary	Rail, Barge
Somerset	Severn Estuary and Bristol Channel	Rail, Barge
Kent	S. Coast , Thames Estuary	Rail, Barge
North Wales	Mersey Estuary and North Wales Coast	Rail, Barge
South Yorkshire	Humber Estuary and East Coast	Rail
Derby and Nottingham	Humber Estuary, The Wash	Rail
Northumberland	Morecambe Bay	Barge
Merseyside and Lancs.	Mersey Estuary.North Wales Coast. North West Coast	Barge
Midlands	Midlands	Road, Rail, Canal

Table 2.1 Minestone sources, areas of potential use and possible modes of transport.

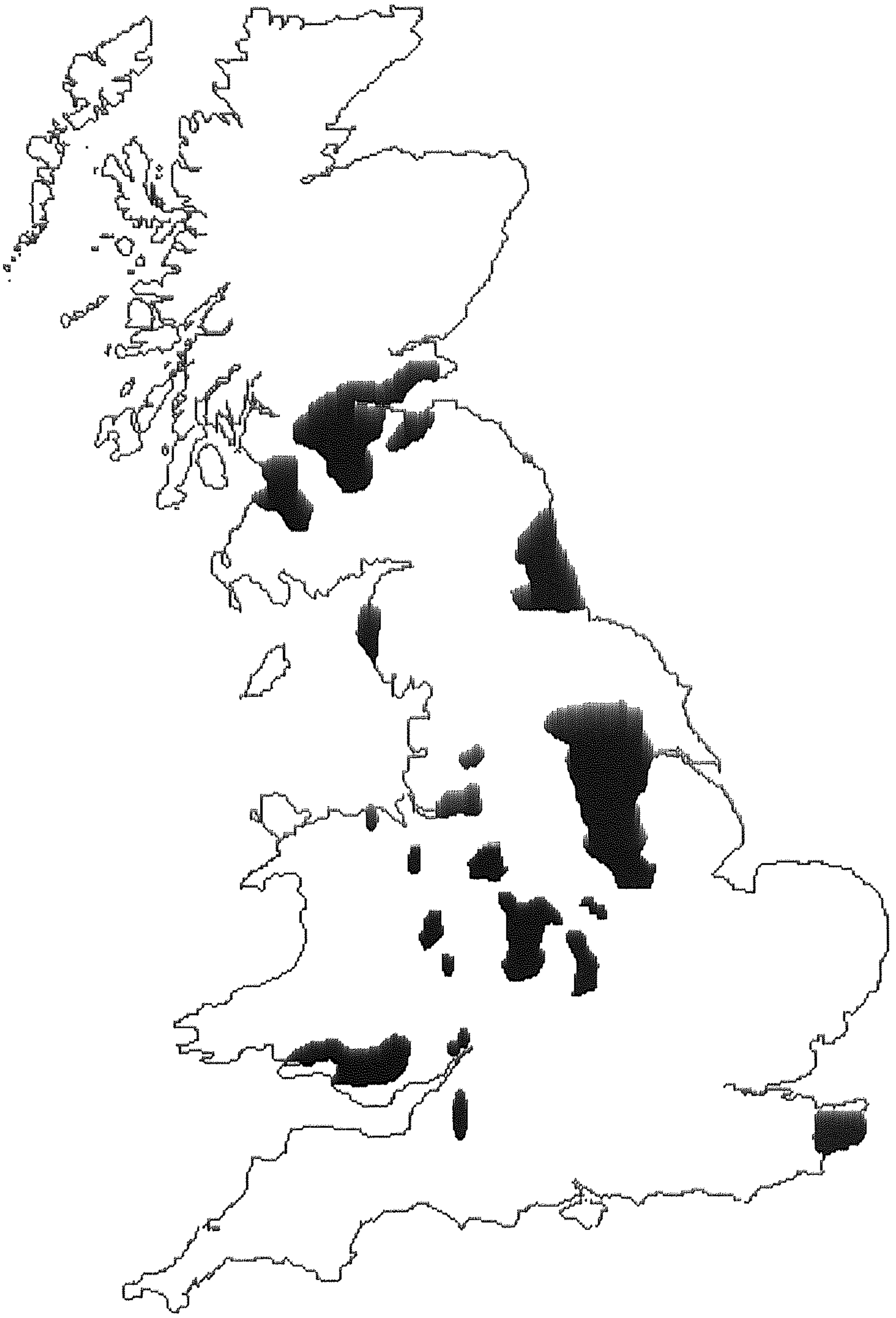


Figure 2.3 Location of coalfields and minestone stockpiles in Britain.

3. STABILIZATION

3.1 Introduction

Dating back to the earliest civilizations, soil has been used for construction purposes. However, if untreated, it will generally lack the strength and durability to be considered a successful building material (Ransom, 1963). The treatment process by which the strength, cohesion, durability and water resistance are enhanced is known as stabilization. Soil stabilization may be achieved in three ways: by bonding particles together, by waterproofing the particles or by a combination of the two (Sherwood, 1961). The ancient Egyptians employed the techniques of stabilization to make mud bricks by thermally altering clays using the rays of the sun. Both the Romans and the ancient Chinese used lime and pozzolans to modify soils for road and wall construction, whilst the ancient Shersi constructed some of the earliest pyramids from lime stabilized clays, which have stood for over 3000 years as testament to the ability of stabilized materials (Livesey, 1988).

The definition of a soil can be based upon particle size as well as the origin of its source (Davidson, 1961). By employing this broad definition, such materials as cinders, slag, caliche, crushed stone and minestone may be categorised as a soil type material. The improvements required of stabilization agents are numerous and it is unlikely that a single stabilizer can fulfil all such requirements. The main factors to be considered are listed below:

1. Universal applicability.
2. Ease of incorporation into a soil.
3. Improvement in the workability and compaction.
4. Need for minimum curing and conditioning period.
5. Permanency of improvements.
6. Increased strength and durability.
7. Reduction in permeability.
8. Low initial and transportation costs of the stabilizer.

Stabilization may be grouped into three categories depending upon how stabilization is achieved :

1. Mechanical.
2. Physical.
3. Chemical.

Mechanical stabilization involves the blending of two soil types to improve the grading quality. Physical methods of soil stabilization can be defined as those that bring about an improvement of the soil without extensive chemical reactions. Examples of physical stabilization include compaction, vibroflotation, thermal and electrical treatment, bituminous waterproofing and compaction by explosion. Chemical stabilization may be defined as those mechanisms that bring about improvement to the soil following induced chemical reactions which may involve the soil grains. Both cement and lime may be considered as chemical stabilizers, for both cement-soil and soil-lime reactions play a significant role in the stabilization of a soil apart from the more obvious stabilizing effect of physical binding following cement hydration. Cement stabilization is probably the most widely used method of improving natural and 'man made' soils in the U.K. and internationally.

3.2 Soil-Cement

The American Concrete Institute defines a soil-cement as being "a mixture of soil and measured amounts of portland cement and water compacted to a high density" (ACI, 1990). It is a material produced by blending, compacting and curing to form a hardened material with specific engineering qualities (Adaska, 1991). Other synonyms that have been used for the material are, stabilized soils, cement stabilized soils, cement treated aggregate base and rammed earth.

3.3 Materials Used in Soil-Cements

3.3.1 Soils

Soil may be defined as an aggregation of rock and mineral fragments with the possible presence of colloidal organic matter. Sherwood (1961) classified the components of the soil into three categories :

1. Inert constituents.
2. Beneficial constituents.
3. Deleterious constituents.

3.3.1.1 Inert Constituents

Constituents within this group are generally chemically inert. Their interaction with stabilizing agents usually involves physical action rather than one of a chemical nature. The inert constituents mainly make up the particles in the gravel to silt range and are dominated by the mineral quartz. Only under extreme conditions will the inert constituents react and contribute to the overall stabilization of a soil.

3.3.1.2 Beneficial Constituents and the Clay Minerals

These constituents dominate the finer portions of a soil and include the clays, the carbonates and some oxides. The clays are by far the most important member of this group. Though these components may be regarded as beneficial, if present in excess quantities, they may result in the impairment of a stabilized soil.

The clay fraction is composed of all particles smaller than 2µm. The distinctive character of a clay is governed by the chemical and mineralogical composition together with the ion exchange capacity (Ransom, 1963). These properties will, in turn, influence both the engineering properties of the soil and the subsequent response to stabilization. Clays are dominated mineralogically by the hydrous aluminium silicates. The chief detrital clay minerals are kaolinite, illite, montmorillonite, chlorite and the mixed layer clays. Structurally, the clay minerals are phyllosilicates whose basic structure involves alternating octahedrally co-ordinated, gibbsite sheets ($Al(OH)_3$) and tetrahedrally arranged silica sheets (SiO_2). Substitution for Al or Si may occur in both types of sheets leading to a charge deficiency which is balanced by interlayer cations. Schematic representations of the structures of the more common clay minerals are illustrated in *Figure 3.1*.

Kaolinite

The basic formula for kaolinite is $Al_4 Si_4 O_{10} (OH)_8$, which is derived from the breakdown of feldspars under acid weathering conditions :

$$2K\,Al\,SI_3\,O_8 + H_2CO_3 + H_2O \longrightarrow K_2CO_3 + Al_2\,Si_2\,O_5(OH)_4 + SiO_2$$

(*Equation 3.1*)

Only minor Al/Si substitution occurs and the charge remains adequately balanced to eliminate the necessity of interlayer cations. Individual clay units have a spacing thickness of approximately 10 Å

Illite

Illite, $K_y\,Al_4\,(Si_8 - yAl_y)\,O_{20}\,(OH)_4$, is the result of the weathering of muscovite and feldspars. Al^{3+} substitutes for Si^{4+} in a ratio of 1:7 within the silica sheets. The net negative charge created by this substitution is balanced by K^+ cations which provide strong ionic bonds between adjacent silica-gibbsite-silica packages. Other cations and H_20 are prevented from entering the structure by this bonding so that the exchange capacity is confined to the edges of individual crystals. The spacing between individual sheets is approximately 7 Å.

Montmorillonites and other Smectites

The general formula for montmorillonites is $Al_4\,Si_8\,O_{20}\,(OH)_4\,n(H_2O)$. As a result of the substitution of Al for Mg, positive ions such as Na^+ and Ca^{2+} become attached to the surface or edges of crystals to balance the consequent negative charge. These positive ions are exchangeable bases and account for the high exchange capacity of the mineral. Although montmorillonite has the same basic structure as illite, it lacks the strong K^+ bonds and therefore allows the entry of H_2O between the layers resulting in a volume increase and mineral swelling. Layer spacing ranges between 10 and 15 Å and is generally dependant upon the nature of the exchange cations and the relative humidity (Wilson, 1987). Smectites, when present are nearly always found in the finer fractions of a clay sample.

Chlorite

Chemically this group of clays is very variable, but can be represented by the formula :

$$(Si_{8-x}Al_x)\,((Fe, Mg, Al)_{12-y}Al_y)\,O_{20}\,(OH)_6$$

Chlorite has a complicated structure, but basically consists of octahedrally co-ordinated gibbsite sheets sandwiched between silica sheets with alternating sandwiches being linked together by brucite sheets ($Mg(OH)_6$. Minor substitutions of Al^{3+} for Si^{4+} and Fe^{2+} or Al^{3+} for Mg^{2+} can also occur. Detrital chlorite can only occur in regions where chemical weathering is reduced or absent, for the Fe^{2+} in the brucite layers is easily oxidised resulting in the decomposition of the chlorite structure.

Mixed-Layer Clays

The most important mixed layer clays are the illite-montmorillonite combines. Layers may have regular or irregular alternations between the two clay minerals. Chlorite-montmorillonite is another important mixed-layer clay. The layer spacing will depend upon the type of mineral present together with the degree of swelling in the expansive component.

For engineering purposes, the quickest and simplest way of classifying the finer fractions of a soil (< 425μm) is by way of the Atterberg limits. These limits denote particular moisture contents at which definite physical changes occur in the properties of a clay. A close correlation exists between the activity of a clay, in relation to its physical reactivity to water, and the cation exchange capacity. Depending directly upon these properties is the degree of success in achieving satisfactory stabilization of a soil containing clay. The more active the clay the greater the effort necessary (usually in the way of higher cement ratios) to stabilize a soil and hence the more expensive the stabilization process.

Carbonates are beneficial in the process of cement stabilization as they deactivate the deleterious effects of organic compounds (Sherwood, 1968). Furthermore, any soil rich in calcium carbonate will tend to be poor in organic matter, as the plants responsible for the production of the deleterious organic compounds following decomposition, cannot grow in alkaline soils. Certain oxides such as iron and aluminium may be beneficial to the stabilization of a soil by acting as a cement to hold larger particles together and also taking part in pozzolanic reactions. This type of soil stabilization is uncommon in temperate Britain as free Fe and Al is only found in small quantities. In tropical climes, where chemical weathering is dominant, an abundance of these oxides is present within the soils and so the ability of a soil to be stabilized is greatly enhanced.

3.3.1.3 The Deleterious Components

The deleterious components of a soil may impair cement stabilization by either inhibiting complete cement hydration as in the case of organic compounds, or attacking the products of hydration as in the case of certain sulphates. Organic matter is usually derived from the decay of vegetable matter and is in the form of tannic acid and its derivatives (Neville, 1981). Not all organic compounds have a detrimental effect upon cementitious products so, though the organic test for soils (BS 1377:1975a Test 8) may give the organic content, test samples are considered necessary to determine the actual effect of organic matter upon a cement stabilized soil.

The ability of sulphate ions to impair the fabric of cement products is well known. The most widespread sulphate in British soils is the calcium sulphate, gypsum ($CaSO_4.2H_2O$). Sodium and magnesium sulphate are less common and confined to specific areas (Bessey and Lea, 1953). Sulphates are usually confined to clay based soils, gravels and sand have the property of free drainage and hence any sulphates that may have been present would have been leached away. Sulphates are usually derived from the decomposition of iron pyrites. Although iron pyrites may be deemed a potentially deleterious compound to a soil-cement mix (Thomas, 1987), not all forms of pyrite are reactive (Neville, 1981). Sulphate resistance is a function of the sulphate concentration in the clay and the percentage of clay present within a soil (ACI Committee, 1990). Sherwood (1962) has indicated that sulphate-clay reactions are more detrimental than sulphate-cement reactions and hence fine grained stabilized soils will be affected to a greater degree than coarser grained stabilized soils. The reactions associated with sulphate attack upon cement are discussed fully in *Section 4.4.*

In concrete technology, coal grains are regarded as being unsound particles and for the same reasons may contribute to the impairment of a soil-cement. Besides being soft and hence susceptible to abrasive wear, they have also the potential to swell and, if present in large quantities of comminuted form, could affect the hardening process of cementation.

3.3.2 Soil Suitability

Almost all types of soil can be used for soil-cement, the only exceptions being highly plastic soils, organic soils and poorly reacting sandy soil (ACI Committee, 1990). The more granular the soil, the more economical it is to stabilize as mixing and pulverization is more successfully achieved enabling a lower cement content to be used. Gradation requirements are usually less restrictive than those set for conventional concrete.

Soils that have been typically used for stabilization include silty sands, processed crushed and uncrushed sands and gravels and crushed stone. Fine grained soils need a greater quantity of cement to be added to allow sufficient strength to be gained. For slope protection applications it has been shown that erosive susceptibility is inversely related to the coarseness of the soil grading (Nussbaum and Colley, 1971). Furthermore, the presence of clay balls, within a stabilized granular soil, may be detrimental to a hydraulic structure as they are susceptible to washout leading to a weakening of the soil-cement structure. Finally any soil with an organic content greater than 2 per cent is considered unacceptable for stabilization.

3.3.3 Cement and Admixtures

Type I and II cements are the commonest cements used for stabilization (ACI Committee, 1990). The percentage of cement by dry soil weight used for stabilizing soils ranges from 4 per cent to 16 per cent, usually a higher clay content necessitating a higher percentage of cement. The amount of cement required will depend upon the soil type and the desired properties needed for the stabilized structure.

Both pozzolans and hydrated limes have been used to improve a soil prior to cement stabilization. Lime is used to reduce plasticity and to render the soil more friable hence aiding pulverization and subsequent mixing with the cement. Chemical admixtures have rarely been used in full scale soil-cement structures.

3.3.3.1 Chemical Composition of Portland Cement

Portland cement is obtained by the process of calcination of an intimate mix of calcareous and argillaceous materials followed by the crushing of the resultant clinker to give a fine anhydrous powder. A typical portland cement is composed of the following oxides in representative proportions:

CaO	63%
SiO_2	20%
Al_2O_3	6%
Fe_2O_3	3%
MgO	1.5%
SO_4	2%
K_2O	<0.1%
Na_2O	<0.1%

The bulk of these oxides are incorporated into four compounds (Neville, 1981) :

Dicalcium Silicate	$2CaOSiO_2$	C_2S	17%
Tricalcium Silicate	$3CaOSiO_2$	C_3S	54%
Tricalcium Aluminate	$3CaOAl_2O_3$	C_3A	11%
Tetracalcium Aluminoferrite	$4CaOAl_2O_3Fe_2O_3$	C_4AF	9%

Several minor constituents such as the oxides of sodium, potassium, magnesium, titanium and manganese, together with calcium sulphate make up the remaining compounds of cement.

3.3.3.2 Cement Hydration

With the addition of water (H in cement technology nomenclature), hydration of the four main cement compounds occurs. It is the product of this hydration which develops into the hardened cement paste that provides the strong bonding properties for which portland cement is renowned. Of the four main anhydrous components of cement, the two calcium silicates give rise to the main cementitious compounds. The reactions involving the hydration of the calcium silicates are as follows :

$$2C_2S + 4H \longrightarrow C_3S_2H_3 + Ca(OH)_2 \qquad (\textit{Equation 3.2})$$

and :

$$2C_3S + 6H \longrightarrow C_3S_2H_3 + 3Ca(OH)_2 \qquad (\textit{Equation 3.3})$$

The assumption that $C_3S_2H_3$ is the final product of calcium silicate hydration, allows the above reactions to be used only as a guide and not stoichiometric definitions. The products of C_3A and C_4AF hydration are as follows :

$$C_3A + 6H \longrightarrow C_3AH_6 \qquad (\textit{Equation 3.4})$$

and :

$$C_4AF + 6H \longrightarrow C_3AH_6 + \text{indeterminate compound} \qquad (\textit{Equation 3.5})$$

Again these equations only offer a guidance and are not stoichiometrically accurate. One of the most fundamentally important properties that the resultant hydrated compounds impart upon hardened portland cement is insolubility in water. This very low solubility has played a major part in the success of the material especially when used in hydraulic applications.

3.3.4 Water

The water in both concrete and soil-cement usually comes from two sources: added water and moisture in the aggregate (Owens, 1989). Water is essential in soil-cement for both maximum compaction and cement hydration. Potable water is preferred for mixing, or any other relatively clean water, free from acids, alkalies and organic compounds. Sea water has been shown to give satisfactory results when used in concretes, especially when the tricalcium aluminate (C_3A) level of the portland cement is above 8 per cent (Steinour, 1960). Sea water has also been shown to give a satisfactory product when used in the stabilization of soils (ACI Committee, 1990; Hansen, 1991). When sea water is used for mixing concrete the early strength is increased owing to the increased chloride content (Owens, 1989) but a lower long term strength is achieved though usually strength loss is no greater than 15 per cent (Neville, 1981). Although high alumina cements have the ability to resist sulphate attack and perform extremely well in sea water, it should not be used as the mixing water as the setting and hardening process is adversely affected. Sea water contains up to 3.5 per cent (35,000 ppm) dissolved solids of which 78 per cent is NaCl and 15 per cent $MgCl_2$ and $MgSO_4$. The adverse effects that magnesium sulphate may have on cementitious compounds will be discussed in *Section 4.4*.

3.4 Soil-Cement Interactions

3.4.1 Cement-Aggregate Interaction

The mechanisms by which cement stabilizes soil can be broadly placed into two categories, the primary reactions involving the initial hydrolysis and hydration of cement and the secondary reactions leading to the formation of secondary cementitious compounds as a result of clay cement interaction (Mitchell and El Jack, 1965).The interaction between cement and the aggregate will depend upon both the physical and chemical properties of the particulate matter. For soil-cement differs from portland cement concrete (PCC) in the manner in which the aggregates or soil particles are held together. A PCC mix contains sufficient cement paste both to coat the entire surface area presented by the aggregate and to fill the voids between the aggregate. In soil-cement mixtures the quantity of paste is insufficient to fulfil either role (Adaska, 1991). The type of structural fabric that develops from the stabilization of soil with cement will depend upon the ratio of the cohesive component to the non-cohesive component and minestone, like many natural soils, is a mixture of cohesive and non-cohesive particles.

3.4.1.1 Non-Cohesive Aggregate

The most important characteristic of this group, in terms of cement interaction, is that the size of the aggregate particles are several orders of magnitude greater than the cement particles. The size of the particles within this group range from fine sands to cobble grade, the grades that dominate minestone. Cement will partially or fully coat the surface of the aggregate, however bonding will only occur at contact interfaces between adjacent aggregate units, as occurs in traditional PCC. By increasing the initial surface contact a greater degree of bonding is attained. To maximise this initial degree of bonding, a well graded material is desirable (Lilley, 1971) together with a high degree of compaction. Surface roughness of the aggregate also plays an important part in the bonding process for the rougher the surface, the greater the degree of interlocking between the aggregate and the cement paste. A closed impenetrable texture is not beneficial to achieving a good aggregate-paste bond.

3.4.1.2 Cohesive Material

The particles that compose this type of material, generally being of fine silt or clay grade, are characteristically the same size or smaller than cement particles. The enormous surface area presented by this material, together with its cohesive nature, allows only a minute proportion of the whole to come into contact with a cement particle. Whereas the fundamental unit in a cement stabilized granular soil is an individual rock or mineral fragment, the fundamental unit in a stabilized cohesive soil is an aggregation of clay particles referred to as 'domains' (Herzog, 1963). A clay domain contains coherent, parallel-orientated, clay platelets in close packing and are thought to behave as integrated structural units. Several theories have been proposed to explain how these aggregations are bound together.

3.4.2 Passive Matrix Theory

The basic concept of this theory is illustrated in *Figure 3.2*. Each individual aggregation of domainal clay becomes coated with cement. The cement forms an interconnected skeletal framework binding other individual aggregations together. The strength of this fabric is attributed entirely to the actual cement framework, with the clay aggregations playing only a passive role in the overall structure of the fabric (Davidson, 1961; Highway Research Board 1961; Lilley, 1971; Lilley, 1979). The strength exhibited by such a fabric depends on the size of the individual aggregations, the degree of waterproofing that the cement coating affords to the aggregations and the extent of the actual skeletal framework, which is controlled by cement content and the degree of pulverization and mixing the soil-cement has undergone. A follow-on theory, ascribed to Stocker (1963), also suggests a primary skeletal framework, but describes the development of a secondary framework of lime that penetrates the individual aggregations following primary hydration (*see Figure 3.3*).

3.4.3 Active Matrix Theory

In this theory the matrix plays an active role in imparting strength to the overall soil-cement structure. Herzog (1963), following Stocker (1963) postulated that extensive secondary cement development occurred in clay-cement mixes as a result of clay-cement interaction. However, instead of suggesting that cement would coat individual aggregations to form a continuous interconnecting framework, Herzog suggested that individual grains of cement hydrate separately whilst surrounded entirely by the clay matrix. As primary hydration and simultaneous secondary cement mineral growth occurs, a 'sea urchin like body' begins to develop projecting spindles of cementitious material into the surrounding matrix and increasing the bonding effect (*see Figure 3.4*). The cement grain is hydrated by pore water which propagates the formation of hydrated gel. The hydrated gel is expansive in nature so it pushes outwards moulding itself around the surrounding clay framework. As illustrated in *Figure 3.5*, this keying action results in a hydrated cement particle being surrounded by embedded domains of clay particles. During this process the hydroxide ions, released by hydration of the cement, attack the clay minerals releasing free silica and alumina into the pore spaces. These elements react with calcium ions of the released lime to form secondary cementitious compounds throughout the micro and macro pore vicinity of the hydrated cement grain. Again a high degree of pulverization is beneficial as it allows the cement particles to be more uniformly and intimately mixed with the matrix (Grimer and Ross, 1957; Mueller, 1991).

3.4.4 Secondary Cement Reactions

Primary hydration of cement can occur for decades following the initial mix with peak strength in concrete being attained at between 10 and 25 years (Washa and Went, 1975). In stabilized soils, especially if a significant quantity of clay is present, secondary reactions may occur simultaneously with primary hydration further enhancing the cementitious properties of the soil-cement fabric.

The microscopic nature of clay crystals, together with their essentially two dimensional structure, is responsible for their high surface to volume ratio. The prospect of chemical reactions occurring between cement and clays is greatly increased as a result of this vast, potentially reactive surface. Through the use of X-ray diffraction, differential thermal analysis and electron microscopy, several studies have revealed that the principle reactant involved in beneficial secondary reactions is calcium hydroxide, $Ca(OH)_2$ (Herzog and Mitchell, 1963; Mitchell and El Jack, 1965; Moh, 1965; Diamond and Kinter, 1966).

Initially the Ca^{2+} from calcium hydroxide substitutes for the exchangeable K^+ and Na^+ upon the surfaces of the clay crystals. This change in surface charge leads to the flocculation of the clay platelets (or domains), effectively reducing the clay size fraction and ultimately the plasticity of the soil (Noble, 1967). This initial process is known as 'lime modification' and occurs within the first few hours following cement application. Clays with a high cation exchange capacity will have a greater receptivity to Ca^{2+} and will hence respond more favourably to lime modification than would clays with a lower exchange capacity. There is a risk that calcium depletion may occur with the stabilization of reactive clays, such as montmorillonite, for as calcium is adsorbed, the pH of the mix system is reduced threatening the progress of primary hydration (Croft, 1967).

As time proceeds, more permanent effects are produced following the reaction of calcium hydroxide with the clay minerals. These long term reactions are known as 'lime stabilization' and result in the formation of secondary cementitious compounds. Initially some calcium disassociates from the hydroxyl component :

$$Ca\,(OH)_2 \longrightarrow Ca^{2+} + 2(OH)^- \qquad \textit{(Equation 3. 6)}$$

The presence of $Ca(OH)_2$ helps to increase the pH of the soil-cement system. Highly alkaline conditions enable the mobilization of soil silica (S) and alumina (A) which are able to react with the calcium and the calcium hydroxide to form calcium silicate hydrate and calcium aluminate hydrate (Andrews, 1966).

$$Ca^{2+} + 2(OH)^{-} + Si + Al \longrightarrow CSH + CAH \qquad \textit{(Equation 3.7)}$$

If present within the soil, sodium silicate (NS) and sodium aluminate (NA) may react with the disassociated calcium :

$$Ca^{2+} + NS \longrightarrow CSH + Na^{+} \qquad \textit{(Equation 3.8)}$$

$$Ca^{2+} + NA \longrightarrow CAH + Na^{+} \qquad \textit{(Equation 3.9)}$$

Calcium hydroxide may also react with carbonic acid (H_2CO_3), derived from rainwater (or soil water), to form calcium carbonate which has weak cementitious properties :

$$Ca(OH)_2 + H_2CO_3 \longrightarrow CaCO_3 + 2H_2O \qquad \textit{(Equation 3.10)}$$

Though clay may be beneficial to the stabilization of a soil through the processes of secondary cement development, its physical behaviour is considered to be detrimental to soil stabilization even in quantities as low as six per cent (Sherwood, 1968).

3.5 Soil-Cement Properties

The performance of a soil-cement will depend upon a number of factors, including type and proportion of soil, cementitious materials, water content, compaction, uniformity of mixing, curing conditions and age of the compacted mixture. As a result of the high number of influencing factors, a wide range of values for specific properties may exist (ACI Committee, 1990).

3.5.1 Density

The density of a soil-cement mix is usually measured in terms of dry density (Test 5 BS 1924:1975b). Clays have a tendency to increase the optimum moisture content and, as a corollary, the maximum dry density for a given compactive effort will be lowered. By adding cement to a soil the optimum moisture content and the maximum dry density achieved for a given compactive effort, will be altered. Predicting the change is difficult because of two counteracting factors, lime modification caused by initial cement hydration will lead to a flocculation of clay particles which results in an increase in the optimum moisture content and a decrease in the maximum dry density. Counteracting this, the higher specific gravity of cement relative to the soil particles will result in higher dry densities.

Given the influence of clay content on compacted density, for a given cement content, a sandy or gravelly soil, will have a higher strength than a stabilized clay assuming that secondary cementitious reactions are kept to a minimum (Sherwood, 1968). Clay based stabilized soil will usually require a greater cement content to attain the same strength as a coarse grained soil. A direct relationship exists between dry density and the compressive strength of a soil-cement. However, significant strength loss will occur if a delay occurs between mixing and compaction (Arman and Saifan, 1967; Arman and Danten, 1969). Cement hydration and bonding begins immediately after the addition of water and so a failure to promptly compact a soil-cement mixture will lead to a loss of some of the hardening effects of the cement. Compaction will then require an increased effort to break the initial cement bonds to achieve a satisfactory density. A delay greater than 2 hours has been shown to decrease the strength of a soil-cement significantly (West, 1959). The degree of strength loss will depend upon the rate of cement hydration, which is a function of the cement itself, temperature, and the chemical composition of the soil. Hydration retarders may be used to extend the time between mixing and compaction (Felt, 1955) though using them is not a preferred practice (ACI Committee, 1990), and increases costs.

3.5.2 Compressive Strength and Durability

The unconfined compressive strength of a stabilized soil indicates the degree of reaction between the soil-cement-water mix and also the rate of hardening of the mixture. It is the most widely referenced property of soil-cement and serves as a criterion for determining the cement requirements to achieve the required performance. The unconfined compressive strength is usually measured according to BS 1924 (1975b) Test 10 or 11, depending upon the grade of soil. In the USA, the procedure is different in that samples are soaked prior to strength testing since most soil-cement structures may become permanently or occasionally saturated during their lifetime (ACI Committee, 1990). Saturated samples will usually provide a lower strength than so called 'dry' samples as a result of a number of mechanisms (*see Section 4.2*).

The medium term strength of cement stabilized materials (1-2 months) normally exhibits a linear trend when plotted against the logarithm of the age of the sample (Grimer and Krawczyk, 1963; Sherwood, 1968). Soil type will influence the rate of strength development, with stabilized fine grained soils exhibiting a slower rate of strength development than granular soils (ACI Committee, 1990). A change from a linear to a non-linear relationship is usually experienced after approximately 2 months if clay is present

(Sherwood, 1968). This change is a result of secondary cementitious products becoming increasingly influential in strength development.

Durability is the measure of the ability of a material to withstand the detrimental forces of weathering and erosion. The addition of cement to a soil is expected to greatly increase its durability. As with compressive strength, a relationship also exists between dry density and durability. Denser materials usually show greater durability because water has less opportunity to enter the fabric of that material, for it is water and, particularly, dissolved solutes that are most likely to disrupt the stabilized material. The durability of soils rich in silts and clays benefit the most from an increase in density. However, care must be taken to avoid over-emphasising the relationship between dry density and both compressive strength and durability (Lightsey and Callihan, 1970). They showed that, with sandy loam, an increase in dry density was experienced if a delay in compaction was allowed to take place even when the moisture content was below optimum. At these low moisture contents, the strength and in particularly durability were much lower than those achieved at higher moisture contents despite similar dry density values. In the same study, excess moisture content, though generally no greater than 2 per cent above optimum, and a delay in compaction was shown to improve durability. Two mechanisms were cited to explain these results, firstly excess moisture improved primary hydration and secondly by an improvement in dry density, following the lubrication of cemented aggregations formed during the delay between mixing and compaction. Other studies have indicated that excess moisture content improves the durability and resistance to the effects of frost action, immersion and cyclic wetting and drying (Felt, 1955; McNulty, 1985; Thomas, 1987). Increased hydration, clay preconditioning and increased densities (in the case of delayed compaction) have all been cited to explain the increased durability of soil-cement mixes using excess moisture content.

3.5.3 Permeability and Shrinkage

Permeability and shrinkage are important properties of a material used for hydraulic applications. The permeability of a soil is expected to decrease if cement is used to stabilize the material. Large scale tests by the US Bureau of Reclamation (DeGroot, 1971) indicated that soil-cement permeability decreased with time. This was attributed to the filling of shrinkage cracks by sediment and the propensity of cracks to self heal. The permeability of a hydraulic structure will depend upon soil type, cement content, degree of compaction and method of construction. It has been shown that permeability is 2 to 20 times greater parallel to compacted lifts than flow normal to the compaction planes (Nussbaum and Colley, 1971) Seepage was found to be ten times greater in the cold winter months than during the summer periods and it was thought that thermal expansion and algal growth played a major role in seepage reduction during the hot summer months. Shrinkage plays an important role in seepage control for, like permeability, it too depends on soil type, cement content, water content, degree of compaction and curing conditions. Clays are more susceptible to shrinkage though shrinkage crack width will be smaller than those that form in more granular soils.

3.5.4 Erosion/Abrasion Resistance

The resistance of a soil-cement to erosion and abrasion will be dependent upon a number of critical factors including soil type, cement content, compressive strength, density and sample/material soundness. With increasing cement content, erosion and abrasion rates have been shown to decrease. In a study into the use of soil-cement for dam facings, it was shown that by increasing the cement content of a stabilized sandy soil from 3 per cent to 7 per cent, the erosion rate was reduced by over 50 per cent (Nussbaum and Colley, 1971). Work on stabilized loams has demonstrated that the resistance to weathering, and subsequent erosion, increased with increasing cement content. A minimum of 12 per cent cement was thought necessary to impart durability and erosion resistance upon a stabilized loam soil (Shen and Akky, 1974). An associated study into cement stabilized sandy soils also showed that increased cement content significantly increased the resistance to erosion. In situations where no abrasive element was associated with the flow, such as in a canal, the quantity of cement necessary to stabilize a soil was drastically reduced. Nussbaum and Colley (1971) found that just 1.5 per cent cement for fine grained soils and 0.75 per cent cement for coarser grained soils was sufficient to reduce erosion to zero in flume tests involving flows of up to 16,000 gallons of water per day.

Erosion/abrasion resistance has been shown (Akky, 1974) to be directly related to compressive strength which is not surprising given that both properties are also directly related to cement content For a sandy soil-cement, the erosional losses were shown to decline exponentially with increasing compressive strength (Oswell and Joshi, 1986). In the same study, the compressive strength necessary to achieve negligible erosion ranged from 16 MPa, at relatively low impact pressures (70 kPa), to 40 MPa at the highest impact pressures (280 kPa). Abrasive loss correlated well with compressive strength, with the results being somewhat independent of cementitious content or water-cement ratio of the soil-cement mixtures. Smooth functional relationships between erosion rate, critical shear stress and compressive strength were shown to exist for various stabilized soils when samples were tested using as rotating cylinder device (Akky, 1974) (*see Section 7.4*). These relationships were considered to present a very practical tool for the design of soil-cement slope protection.

Soil type is a very important factor in controlling the rate and amount of erosion and abrasion,with stabilized fine grained soils having a lower resistance to erosion and abrasion than similarly treated granular soils. In flume tests incorporating a heavy abrasive element, fine grained soils (type A-4 in the American Association of State Highways Publication No N145) stabilized with 13.5 per cent cement were found to be significantly less resistant than granular soils (A-1b) stabilized with only 5 per cent cement. Modification of alluvial and fine grained soils by introducing granular materials into the mix has been shown to reduce erosion and abrasion. Sand was mixed with alluvium to increase its durability and erosion resistance (Litton, 1982) whilst gravel (> 6mm) was successfully introduced into a fine grained soil to improve the abrasion resistance of the resultant soil-cement in a study investigating hydraulic slope protection (Nussbaum and Colley, 1971). A gravel component greater than 20 per cent was thought necessary to significantly improve the abrasion resistance of a fine grained soil-cement. A comparison between stabilized loess and stabilized loess-sand mixtures indicated that a 20:80 sand-loess mixture, stabilized with 3 per cent cement exhibited only one third of the erosional losses of the loess soil stabilized with 7 per cent cement. The influence of the additional sand has major implications upon the economics of cement stabilized structure as less cement is necessary to achieve the required performance than when just cement was used (Litton and Lohnes, 1982).The ability of a concrete to resist abrasive action has been shown to increase by 30 to 40 per cent if shrinkage-compensating cement is used (ACI committee, 1990). This may be expected to occur with soil-cements, though the increased cost of the cement would eliminate its use on economic grounds. However, the failure of hydraulic structures constructed from soil-cement has usually been as a result of structural failure, due to design and/or construction shortcomings, rather than to the deterioration of the soil-cement fabric (Davis *et al.* 1973).

3.6 The Application of Soil-Cement

Following the invention of 'portland cement' over 150 years ago, it was inevitable that experimentation would lead to the eventual application of cement stabilized soils. The use of soil-cement has usually been hastened by the scarcity of more traditional material and the consequent economic considerations. The first recorded use of soil-cement was in 1915, when a street in Sarasota, Florida was ploughed, treated with cement and compacted (ACI Committee, 1990). The first recorded use of soil-cement in Britain was also to stabilize roadways, upon Salisbury Plain in 1917 (Williams, 1986). To date over 100,000 miles of soil-cement roads have been built in the United states. Apart from highway construction, soil-cements have been used for a multitude of other purposes ranging from airfield construction, heavy industrial pavement projects (Portland Cement Association, 1975b), foundation stabilization for nuclear power plants (Dupas and Packer, 1979), coal handling and storage berms, backfill material for waste water treatment plants (ACI Committee, 1990) and rammed earth houses (Little *et al.* 1983; Berglund, 1986). Another major application for soil-cement is for slope protection and lining for hydraulic structures. The United States have taken the initiative in this field, with over 300 major soil-cement, slope protection projects since 1961 together with numerous smaller projects such as channel protection, spillways, coastal protection and reservoir embankments (ACI Committee, 1990).

The impetus that dramatically brought about the extensive use of soil-cement for hydraulic applications was a rapid expansion of water resource projects in the Mid-West and South Central USA, regions where insufficient quantities of quality rip-rap rock exist (Adaska, 1991). The driving force behind the initial implementation of soil-cement was the United States Bureau of Reclamation (USBR) which undertook extensive and long term monitoring of soil-cement under field conditions. The Bonny Reservoir test section was constructed in 1952 in eastern Colorado and was observed for ten years whilst it was subjected to severe natural conditions including storm waves, ice, freeze-thaw cycles, wet-dry cycles and abrasion. By 1961 the USBR was finally convinced of soil-cement suitability as an alternative to rip-rap and built the Merritt dam, Nebraska, which became the first to be commissioned with soil-cement slope protection, soon to be followed by Cheney Dam, Kansas (Casias, 1991). The State of Texas has regularly incorporated soil-cement in water resource projects to become recognised as a major innovator in soil-cement technology (Hansen, 1991). More than 30 per cent of all the soil-cement, water resource projects in the United States have been carried out in Texas using 40 per cent of the entire volume of soil-cement that has so far been used in the USA. The proliferation of soil-cement structures in Texas is a result of several factors. Quality rip-rap is scarce, the local availability of very experienced soil-cement contractors and satisfaction with the excellent performance of the finished projects have all contributed towards raising the popularity and confidence in soil-cement in Texas. The largest soil-cement, water control project in the world is the South Texas Nuclear Cooling Reservoir, where 918,000m^3 of soil-cement have been used to protect the exterior embankments and interior dykes from wave action which may reach heights of 5m generated by 175mph tornado winds.

Soil-cement has been a popular material for reservoir lining since 1945 when a reservoir in Port Isabel, Texas was lined with a 100mm thick soil-cement mixture containing 12 per cent cement. At least 17 small irrigation reservoirs in the southern Californian desert have been lined with soil-cement by the Soil Conservation Service enabling poor quality land to be farmed (PCA, 1986). Soil-cement has been used to protect channels and drainageways from erosion and water loss. Flumes for carrying cooling water at power stations have been constructed out of 150mm thick soil-cement linings in Florida, while, in Arizona, storm channels made from 10 to 14 per cent cement treated soils have been designed to withstand flows in excess of 6 metres per second carrying copious quantities of abrasive material.The value of soil-cement channel protection was demonstrated in 1983, when conventional rip-rap and gabions failed during storm flooding yet 30 soil-cement projects performed admirably (PCA, 1986). Soil-cement has been popular for lining aeration lagoons for industrial and municipal wastewater. The credibility of soil-cement as a construction material was greatly enhanced when a Michigan waste management project, that utilized soil-cement, was voted one of the top ten engineering achievements of 1972 by The National Society of Professional Engineers.

3.7 Soil-Cement Facing Construction

Usually soil-cement facings for embankments and dams are constructed in successive horizontal layers either concurrently with the main construction or subsequently, after the construction of the main body of the dam (*see Figure 3.6*) (PCA, 1988). Details for designing stair step slope protection may be found in the PCA publication IS173W Soil-Cement Slope Protection for Embankments (1984).

Central plant mixing is the most commonly used method, being the preferred method of the USBR on 14 major water resource projects (Casias and Howard, 1984; Casias, 1991). Once mixed and transported to the site, the material is spread over a pre-moistened, firm sub-grade. Bonding of successive layers is an important requirement for applications in hydraulic environments. Davis *et al.* (1973) noted that failure of soil-cement slope protection was usually a result of poor layer bonding, rather than fabric failure. Bonding between successive layers is affected by the time between compacting the underlying lift and covering it with the subsequent layer, surface conditions (cleanliness, roughness and moisture content) and type of bonding agent (i.e. none, dry cement, cement paste or cement slurry) (Casias, 1991). It cannot be stressed too highly that bonding is one of the most important factors influencing the life span of a such structures and that the compressive strength and durability of the actual material are not directly related to the ability of the structure to withstand erosional and weathering elements.

Once spread, compaction takes place as soon as possible, generally within 2 hours of the initial mix, with no area of an active section being left unworked for longer than 30 minutes during the compaction process. Compaction usually takes place at a moisture content slightly above the optimum value. The type of compactor used depends primarily on the type of soil involved. Fine soils are compacted using a tamping or sheepsfoot roller and finished with a multiple wheel, rubber tyred roller whereas, for granular soils, a vibratory steel wheeled or heavy rubber tyred roller would generally be used (PCA, 1988; ACI, 1988, 1990). Regardless of the lift thickness, it is vital that the layer attains the specified maximum dry density throughout the lift, hence the type of compactor will be governed by its ability to achieve this density. Once compacted the surface of the soil-cement is shaped to the desired design line, grade and cross-section. It may need scarification to remove the traces of plant tracks before being compacted for a final time by a non vibratory steel or rubber wheeled roller.

Curing is very important to the final strength and durability of the soil-cement. Both bituminous and water coatings applied by sprinklers are popular curing methods. Sand may be sprinkled onto a completed section to allow light traffic to pass over a section if necessity dictates. Bituminous curing is only allowed when no further lifts are destined to be laid. Protection against freezing in the manner of insulation blankets, straw or soil may be necessary if freezing is likely to occur during the curing period. Finally, all heavy plant is kept off the compacted material for 3 to 7 days.

3.8 Soil-Cement Specifications for Hydraulic Structures

Aggregate gradation requirements are not as strict as for portland cement concrete (PCC), for almost any type of inorganic soil can be used for soil-cement. Normally the maximum particle size is limited to 50mm, with at least 55 per cent passing the No.4 size sieve (4.75mm) (ACI, 1990; Adaska, 1991). Recently the USBR have permitted up to 60 per cent gravel (> 4.75mm) to be incorporated into a soil-cement mix (Casias, 1991). For economical and practical reasons, an upper limit of 35 per cent 'fines' (<75μm) is imposed (PCA 1976, 1986). Furthermore clay balls greater than 25mm should be removed and those that are smaller than this grade are limited to 10 per cent of the overall dry soil weight. The PCA material specifications for soil-cement linings exclude any soil which contains material retained upon a 50.8mm (originally 2 inch) sieve and limits any material retained on a 4.75mm sieve to 45 per cent. At least 5 per cent of the soil must pass the 75μm sieve. The distribution of the material must be such that the constructed lining will not contain lenses, pockets, streaks or differential layering within the structure of the soil-cement (PCA, 1975a; 1975b). Before the cement is mixed into the soil, the soil must be thoroughly pulverized so that at the completion of moist mixing 100 per cent (by dry weight) passes the 25.4mm (1 inch) sieve and no less than 80 per cent passes the 4.75mm sieve. The same reasons exist for avoiding high sulphate levels in soil-cement used for hydraulic structures as in highway construction. In Britain the total sulphate content shall not exceed 1 per cent (as SO_3) in granular material or 0.25 per cent in cohesive soils.

The principal structural requirements of a soil-cement fabric are strength and durability (Adaska, 1991). Whereas in highway construction, strength is the most important property, durability is the key requirement for hydraulic structures (Litton and Lohnes, 1982; 1984). Though British specifications exist for the use of soil-cement in highway construction, namely Road Note 29 (Department of the Environment, 1970) and Clause 1036 (Department of Transport 1986), no national requirements exist for the use of soil-cement in hydraulic applications in Britain. British Standard tests relating to stabilized soils are laid down in BS 1924 :1975b, whilst other useful tests can be found in Methods of Test for Soils for Civil Engineering Purposes (BS 1377:1975a).

In the United States several organisations have an interest in soil-cement and specify accordingly. The Portland Cement Association (PCA), the US Army Corps of Engineers (USACE) and The US Bureau of Reclamation have all issued specifications and soil-cement criteria for durable mixes. The PCA specifications for durability (PCA, 1971), based upon wet-dry and freeze-thaw cycling, depend upon the soil type. On testing, the selected cement content must be sufficient to reduce weight loss to below a level ranging from 7 and 14 per cent, depending upon soil type, for the soil-cement to be considered durable. To produce a durable soil-cement slope the PCA criteria governing the cement content for pavement construction may be used for protecting slopes that are at least 1.5m (5ft) below the minimum water level. For slopes above this elevation, the cement content should be increased by 2 per cent. At present, the USACE gives interim guidance in an unnumbered draft engineering technical letter (USBR, 1986). The USBR design criteria for slope protection allows a maximum weight loss from durability tests of 8 per cent for freeze-thaw and 6 per cent for wet-dry cycling. Furthermore minimum compressive strength values are specified at 7 and 28 days, 4.1MPa and 6.03MPa respectively. In practice an additional 2 per cent cement is added to a laboratory mix design that meets these criteria to allow for variability in the field. Prior to compaction the texture of a soil-cement must be loose throughout the placed layer. The specifications insist that a soil-cement be uniformly compacted to at least 96 per cent of maximum density as determined by the standard tests.

3.9 Cement Stabilized Minestone (CSM)

A number of stabilizers have been investigated in conjunction with minestone, namely lime, cement, tar and bituminous compounds, and of these the addition of ordinary portland cement has been the most successful so that it has now become the most commonly used binder for stabilizing minestone (Rainbow, 1982; Rainbow and Sleeman, 1884; British Coal, 1990). By far the biggest outlet for cement stabilized minestone has been the road construction industry and this is considered to be the greatest potential user for future stock (Rainbow and Sleeman, 1984). The variety of applications using cement stabilized minestone includes roads, airfields, hard shoulders, site roads, factory floors, hardstandings, material stocking areas and oversite and blinding 'concrete'.

By focusing upon the application of CSM for road infrastructure it is not surprising that the procedures relating to the control of CSM have developed from the conditions laid down by the Department of Transport in "Specifications for Road and Bridge Works" (1986). Although these specifications relate to the construction of road infrastructure, they may be extended to provide a basis for controlling the use of CSM in more diverse applications. Formerly the requirements for cement stabilized minestone were governed by Clause 805 of the 1976 edition of the DoT specification for Road and Bridge Works. More recently these requirements have been modified and are detailed in Clauses 1035, 1036 and 1037 of the 1986 edition of the specification. Both the raw material and the resultant mix are governed by the specifications, the salient points of which are outlined below :

Grading. The material to be stabilized must be well graded, must have a uniformity coefficient of not less than 5 with the grading being within specified confining limits (tables 10 and 11 of the specification).

Consistency Limits. Specifications for consistency limits have been dropped from the present DoT. specifications. Previously, materials passing the 425μm mesh were required to have a plastic limit no greater than 20 per cent and a maximum liquid limit of 45 per cent.

Sulphate Content. The total sulphate content of the material shall not exceed 1 per cent (as SO_3) for granular material or 0.25 per cent for cohesive materials.

Moisture Content. The soil-cement mix should have a moisture content which does not fall below the optimum required to achieve maximum dry density as determined by the vibrating hammer method (Test 5 BS 1924b) nor should the moisture content be higher than 2 per cent above the optimum.

Crushing Strength. For cubical specimens, an average 7 day crushing strength of 4.5 MPa must be achieved for each group of five cubes, with a minimum crushing strength of 2.5 MPa being stipulated for individual cubes.

Immersion Test. In addition to the 7 day compressive strength requirements, the average compressive strength determined following immersion in water of 5 test specimens should not be less than 80 per cent of the average compressive strength of 5 control specimens when subjected to the test procedure stipulated in Test 12 of BS 1924. No cracking or swelling must be evident after the 7 day immersion period.

In a previous study of cement stabilized minestones several criteria were pinpointed as being important factors in governing the ability of a minestone to attain the required strength and durability requirements when stabilized. (Thomas, 1987) :

1. A sub 63μm content less than 20 per cent.
2. A plastic limit less than 22 per cent.
3. A plasticity index less than 15 per cent.
4. A modified slake index greater than 60 per cent.
5. An acid soluble sulphate content less than 0.3 per cent (as SO_3).

3.9.1 Compressive Strength

Nearly all the properties required of cement stabilized minestone can be achieved with sufficient quantities of stabilizing agent. However, economic constraints will limit the maximum amount of cement that can be used in particular circumstances. Kettle and Williams (1977) suggested an upper cement content of 10 per cent although Kennedy (1979) believes that in some conditions a higher percentage of cement can be justified. Indeed, by maintaining the same comparison between minestone and soil-cement as suggested by Rainbow (1982) it should not be out of the question to implement the same quantities of cement employed to stabilize a soil. In some circumstances in the USA up to 16 per cent cement has been used for particular hydraulic applications (ACI, 1990).

The same factors that influence the strength development of a cement stabilized soil will also influence the strength of a minestone stabilized with cement. *Section 3.5.2* dealt with these factors, including the effects of reactive clays, the inhibitory action of organic matter, and the effects of sulphates on strength development. The organic content of a minestone is usually of the type that produces little or no adverse effects on strength development. Finely comminuted coal may disrupt the hardening process of cementitious material if present in significant quantities (Neville, 1981). Previous studies have indicated that the coarser, less plastic, minestones have the greatest potential for good strength development (Kettle and Williams, 1977; Kettle, 1978; McNulty, 1985; Thomas, 1987). The actual properties of the minestone selected for stabilization are not the only factors influencing the performance and strength development of the completed product. Both the mix design and the competence of workmanship can seriously influence the performance of a CSM structure. In particular, Thomas *et al.* (1990) have shown that mix moisture contents both above and below the optimum values can significantly reduce the strength. It has also been reported that, as the maximum grain size of a given minestone decreases, the average strength of the stabilized minestone will increase. This is true at least for maximum grain sizes down to 5mm in diameter (Kettle, 1990).

3.9.2 Durability

For hydraulic structures the durability requirements for a cement stabilized material will be more significant than the actual strength value of the material (Litton and Lohnes, 1982; 1984). Although there is a paucity of research dealing with the performance of CSM in a hydraulic environment, the similarity between soils and minestone, from an engineering point of view, would suggest that the view of Litton and Lohnes (1982) would also be valid for CSM. The durability of CSM is usually assessed using the Immersion and Freeze-Thaw cycling tests.

3.9.3 Resistance to Frost Action and Freeze and Thaw.

Several previous studies have investigated the effects of freezing and freeze-thaw cyclicity upon stabilized minestone. These include a study into the frost susceptibility of unstabilized and stabilized minestone (Kettle and Williams 1969; Kettle, 1973), a study into the durability of cement stabilized minestone which investigated the effects of cement content on freeze-thaw resistance and a study into the performance of cement stabilized minestone involving the use of vacuum saturation to simulate the effects of freeze-thaw action (McNulty, 1985). These studies have shown that CSM can be produced with an adequate resistance to satisfy the acceptance criteria associated with these tests. Indeed, given the fissile nature of the raw minestones, it is considered that the adverse effects of freeze-thaw cycles upon a stabilized minestone used for hydraulic applications would probably be secondary to the effects of the deleterious mechanisms associated with wetting and drying cycles and hence the present study has concentrated efforts into researching the effects of wetting and drying rather than the effects of freeze-thaw action. It is acknowledged that the effects of excess moisture in the thawing stage can result in expansion following interparticle swelling or sulphate attack (*see Sections 4.2* and *4.4*).

3.9.4 Immersion Durability

The immersion test was incorporated into the latest edition of the DoT specifications (1986) in order to reveal any adverse effects that the presence of certain constituents (such as sulphates and reactive clays) may have on stabilized materials when immersed in water. The Specification insists upon an immersion ratio of 0.8 i.e. a compressive strength loss no greater than 20 per cent when compared to samples subjected to standard curing. A drawback to the test is that it does not indicate when, or if, deterioration has ceased (McNulty, 1985).

A relationship between the grading of the material used for CSM and the immersion resistance has been shown by Kettle and Williams (1977) who suggested an upper limit of 30 per cent on the sub 75μm material to ensure successful stabilization. The expansion potential of minestone has been shown to decrease with the addition of sand (Carr and Withers, 1987), though the quantities necessary (>30 per cent) may economically prohibit its use. However, it was shown that by removing the finer fraction of the minestone (< 0.5mm), a 50 per cent reduction in the expansability of CSM could be achieved together with a 22 per cent increase in compressive strength. The durability of CSM was improved when samples were prepared at moisture contents up to two per cent above optimum. Both the expansion potential and the freeze-thaw susceptibility were reduced, probably as a result of the increased moisture content causing potentially damaging expansion to occur prior to compaction (Thomas *et al.* 1990). It was shown by Carr and Withers (1987) that the use of sulphate resisting cement made a negligible difference to the strength development and the resistance to swelling of CSM. However, they were encouraged by the results obtained when stabilizing minestone with an OPC-PFA blend. An addition of 10 per cent PFA was shown to improve the long term strength development by 70 per cent and to reduce the expansion by as much as 60 per cent.

With CSM, Rainbow and Sleeman (1984) could detect no direct relationship between cement content and the immersion ratio value and, furthermore, no direct relationship was revealed between the expansive clay content and the immersion ratio. CSM from high and low ranking coal spoil had lower immersion ratios compared to CSM from middle ranking coal spoil (400-700 rank). The immersion test was considered too harsh to be applied to pavement materials for it was thought that these would rarely be inundated (Thomas, 1987). The use of the immersion test may be more appropriate for assessing the use of CSM for hydraulic applications.

3.9.5 Erosion Resistance of CSM

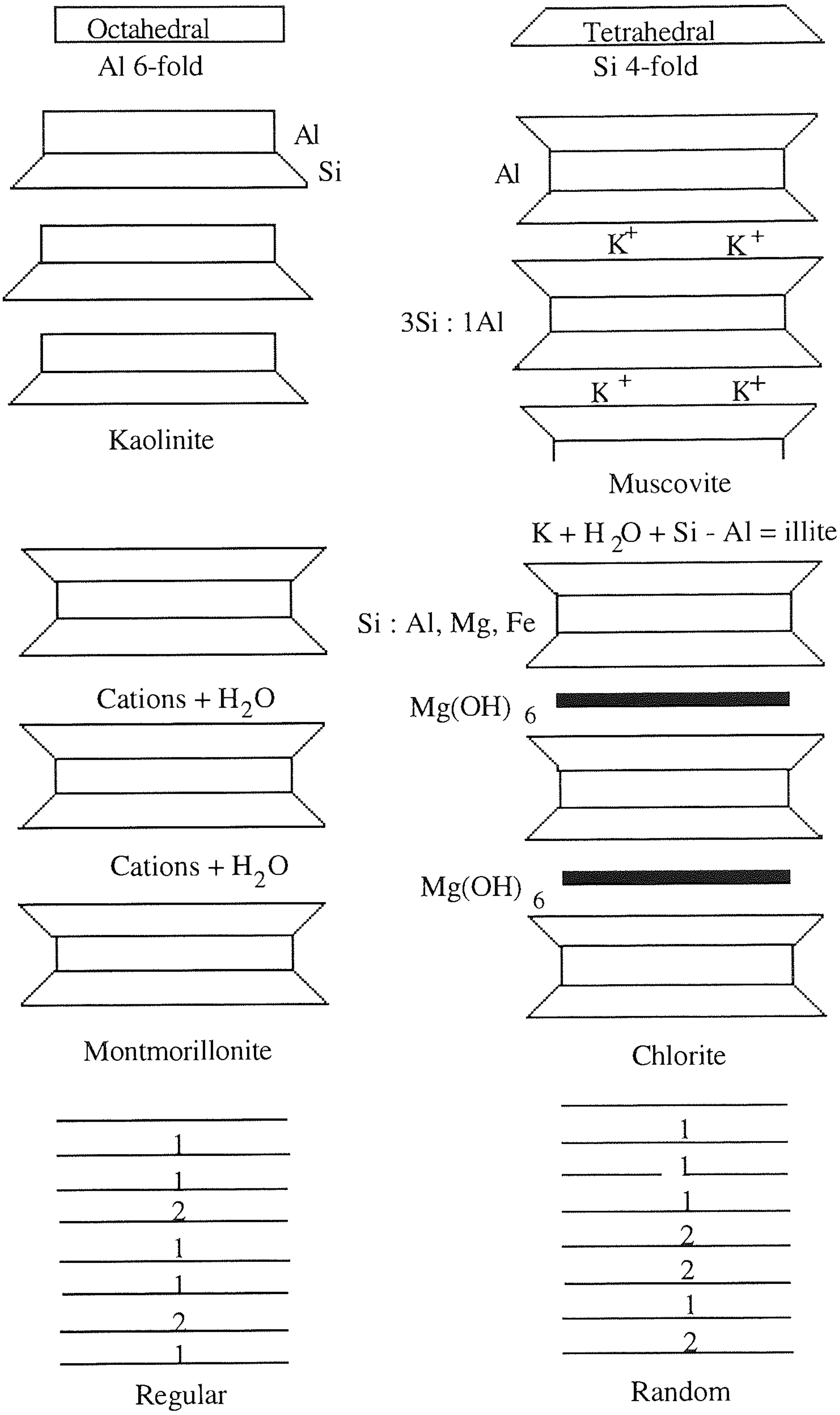
Though the effects of sea water on cement stabilized fly ash have been studied for a number of years (P.M Lockwood, Southampton University, Personal Communication; Hockley and Van der Sloot, 1991) very little, if any, research has been carried out into the effects of sea water on cement stabilized minestone. Likewise, the research into the erosion susceptibility and abrasion resistance of cement stabilized minestone in respect to its possible usage for hydraulic structures has been equally sparse.

It has been stressed previously that cement stabilized minestone, in common with most other cement stabilized materials, is designed to perform primarily as a structural layer and will usually require an overlying wearing course to ensure long term durability and performance (British Coal, 1990). If the comparison between minestone and soil-cement is extended, cement stabilized minestone could possibly be employed in a similar capacity to the soil-cements now extensively employed in the United States for the wearing layers of hydraulic structures. The relative poor quality of cement stabilized minestone does not necessarily rule out its use in such a capacity. Furthermore, if a change in engineering philosophy is implemented, both untreated minestone and cement stabilized minestone could possibly be used as sacrificial material for protecting hydraulic structures. This philosophy is in step with the current trend that emphasises a shift towards the 'soft engineering option' to coastal management (ICE, 1989, MAFF, 1992). This approach involves the removal of reliance upon a solid structure of sea defence works and concentrates efforts upon beach replenishment. The 'soft option' philosophy may be applied to the use of minestone for constructing offshore breakwater reefs. Intermittent replenishment may be possible owing to the cheapness of minestone and, in certain circumstances, this could apply to cement stabilized minestone.

3.9.6 Summary

Cement stabilized minestone has been successfully and economically employed for road and pavement construction for many years. As a consequence, research into the material has concentrated upon examining its performance and durability in relation to the conditions expected as an integral part of the pavement structure. Very little research has been undertaken to determine the performance of cement stabilized minestone in a hydraulic environment, relating to such actions as erosion, abrasion and stability of minestone, especially when immersed in sea water, this lack of data also being applicable to the performance of unstabilized minestone.

Although specifications exist for the quality control of soil-cements used for hydraulic applications, albeit based on experience in the USA, little guidance is available for such applications in Britain. The specifications governing the use of CSM for road and bridge works in Britain may, however, be used as a basis for such standards as these specifications relate both to the general qualities of a material and the specific properties required to produce a successful and durable pavement structure. Although short term durability testing and standard property classification can determine whether a material is suitable to achieve adequate performance in the short term, these tests in themselves are inadequate to reveal the long term durability and performance of a cement stabilized minestone. Previous research has gone a long way to address these inadequacies by determining the mechanisms of failure and thereby relating them to the intrinsic properties revealed by short term durability tests and property classification.



Schematic mica (1) montmorillonite (2) mixed layers

Figure 3.1 Schematic representation of the structure of common clay minerals (After Pettijohn *et al.* 1972).

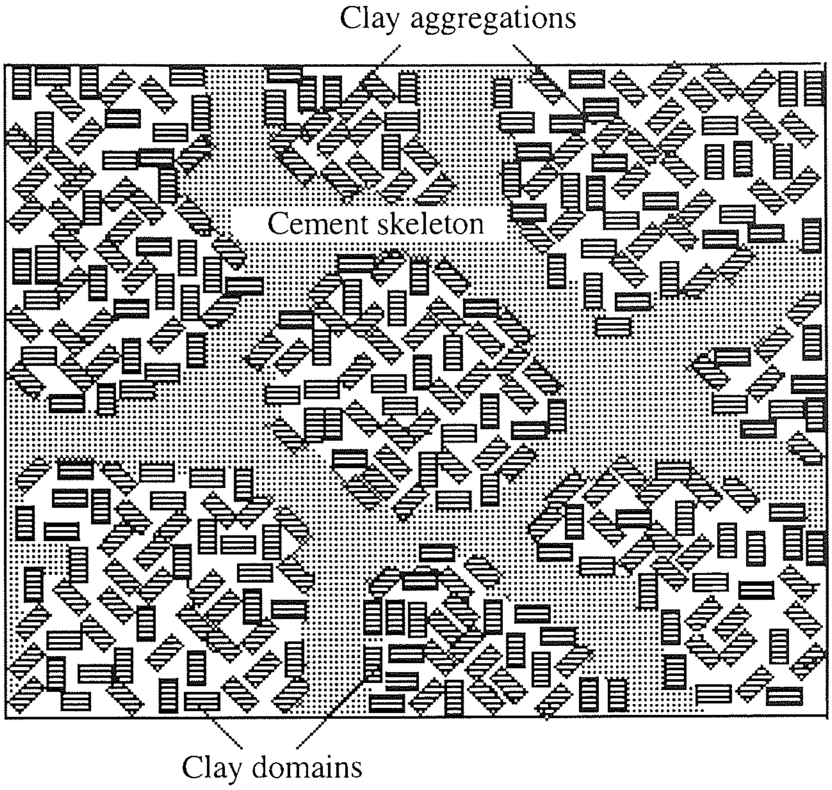


Figure 3.2 Aggregations of clay domains passively fixed in a framework of cement.

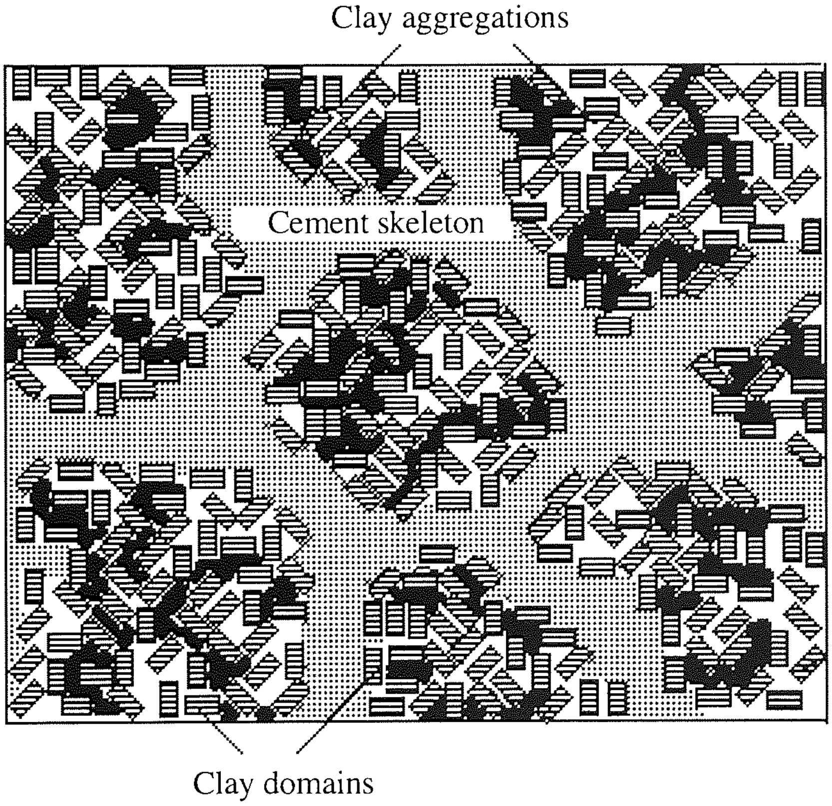


Figure 3.3 The formation of a secondary binding framework following the migration of secondary cementitious compounds.

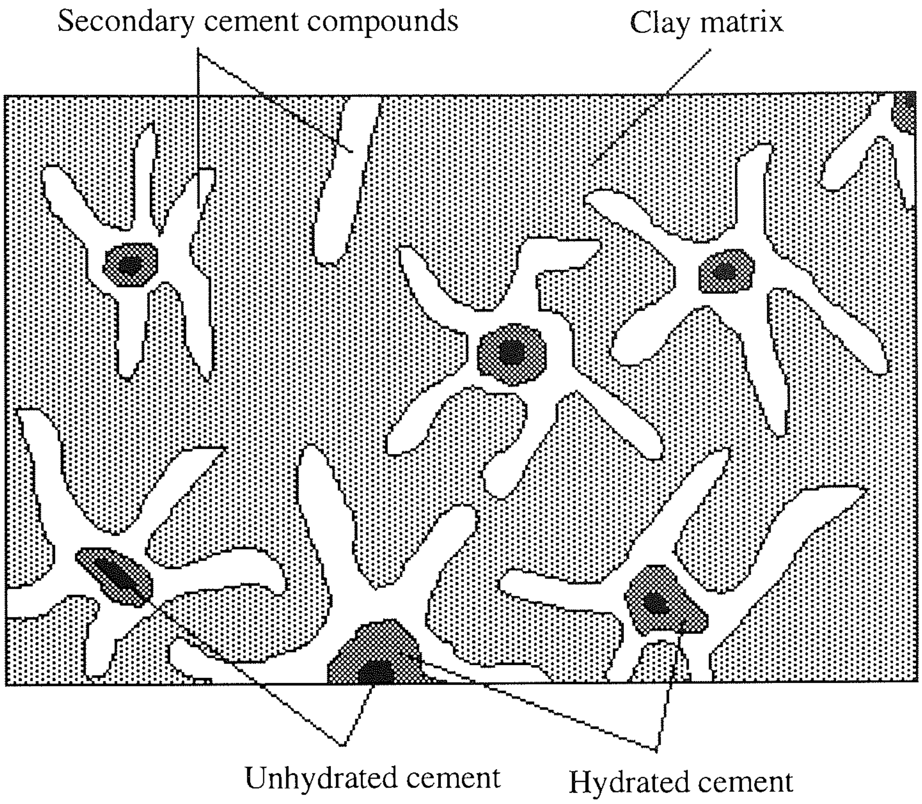


Figure 3.4 The relationship between secondary cement development and the matrix

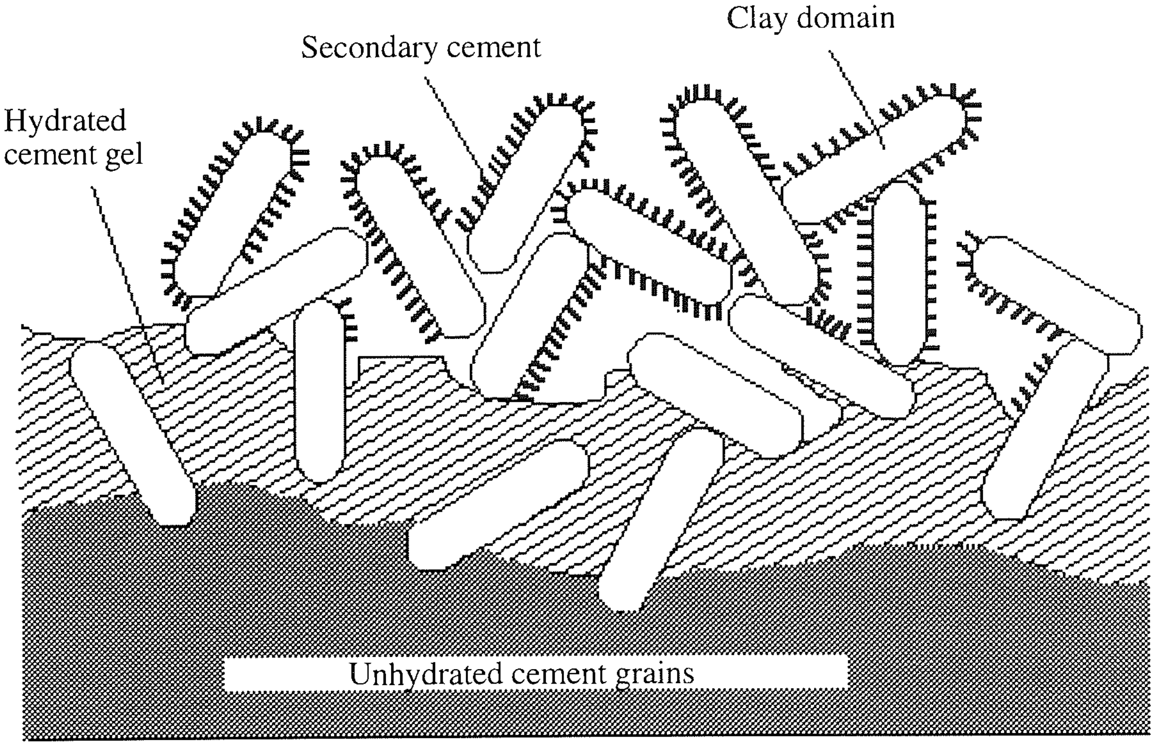


Figure 3.5 The physical interaction of clay with a hydrating cement grain showing the keying mechanism of the clay domains.

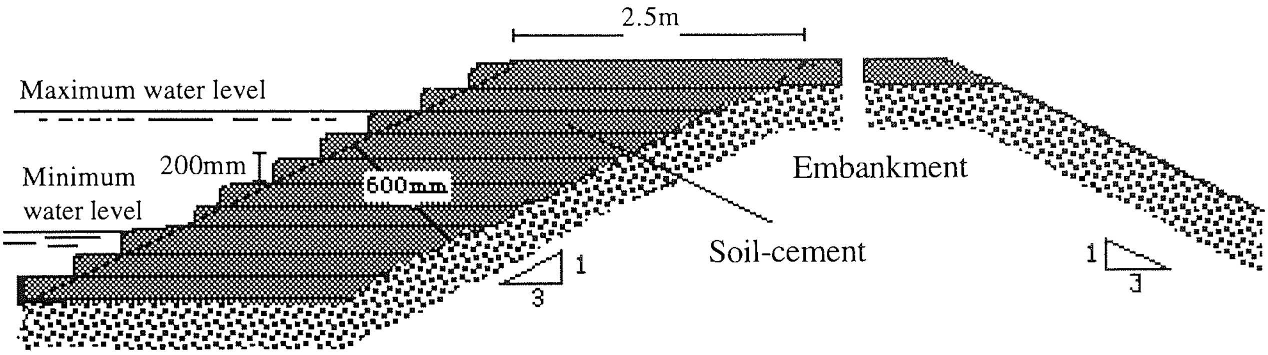


Figure 3.6 Typical method of construction of soil-cement facing on dams and embankments.

4. UNSTABILIZED AND STABILIZED MINESTONE IN A HYDRAULIC ENVIRONMENT

4.1 Introduction

This chapter reviews research into the hydraulic stability of minestone and related materials such as mudstones and shales together with their stabilized equivalents. The success of unstabilized and stabilized minestone as a hydraulic construction material will depend upon the way it responds when immersed in water. Both short term and long term reactions will govern the suitability of both the raw and stabilized minestone for hydraulic applications.

The stability of stabilized minestone in water will depend upon the stability of the individual components that make up CSM, namely minestone and ordinary portland cement. The individual characteristics of these components together with the actual relationship that exists between the two, will determine the stability of CSM. A combination exists within CSM, of a weaker, potentially disruptive material (shale) and a strong, binding material that is cement. The ratio of these two components will govern the performance of cement stabilized minestone when immersed in water.

4.2 The Stability of Minestone in Water

It is well established that geo-materials are weaker when tested 'wet' than when tested 'dry' (Walker and Bloem, 1960; Dulaney, 1960; Price, 1960; Colback and Wiid, 1965; King, 1968; Van Eekhout, 1976). It has been observed that even changes in humidity levels may alter the strength and other properties of the material (Grice, 1968; Pihlajavaara, 1974; Van Eekhout and Perg, 1975; Olivier, 1979; Venter, 1981; Tandanant, 1987). There appears to be no general consensus of opinion to explain the influence of moisture on rock strength, however several strength loss mechanisms have been widely quoted as way of explanation (Van Eekhout, 1976). These are:

1. Fracture energy reduction.
2. Capillary tension decrease.
3. Pore pressure increase.
4. Frictional reduction.
5. Chemical and corrosive deterioration.

Although none of these mechanisms can be discounted outright, it is likely that some will predominate over others depending upon the type of loading, the loading environment and the rock type. For example, the argillaceous rocks, such as shales, mudstones and clays, are particularly susceptible to capillary tension decrease as a result of their high capillarity values. Argillaceous rocks are the most susceptible to chemical and corrosive deterioration, especially if expandable minerals such as montmorillonite are present. Van Eekhout and Perg (1975) found that, for *in situ* coalmine shales, strength reduction caused by humidity fluctuations was a function of two mechanisms, the expansion/contraction characteristics which lengthen internal cracking and the reduction of fracture energy as a result of increased moisture.

Argillaceous rocks are also very susceptible to the phenomena of slaking. Slaking is a general term for the cracking and disintegration of rock when introduced to atmospheric conditions. However, there is no universally accepted definition for the term 'slaking', Perry and Andrews (1983) have listed several definitions and have themselves proposed a long but reasonable definition of slaking "as the short term disintegration of a geo-material following removal of confining stresses. Breakage may result either from the establishment or the occurrence of sufficient stresses within the material or from a decrease in structural strength". However, Surendra *et al.* (1981) believe that all slaking mechanisms require water to penetrate the pore spaces and joints of the shale, which would rule out stress relief as a slaking mechanism. It is accepted that water plays the predominant role in the slaking process. Slaking cannot be attributed to one mechanism of degradation, but must be regarded as the effect of several mechanisms acting in unison to varying degrees of effectiveness depending upon mineralogy, rock fabric and environmental conditions. Although there are several mechanisms through which slaking occurs, there are two basic causes, either an increase in the applied stress above the failure threshold force or the resistance threshold is lowered to the level of the applied force (Richardson and Long, 1987).

4.2.1 Slaking Mechanisms

4.2.1.1 Interplanar and Osmotic Swelling

Any argillaceous rocks, including minestone, that contain lithium and sodium exchange forms of montmorillonite are prone to continuous and essentially unlimited water uptake and consequently unlimited swelling, referred to as osmotic swelling (Wilson, 1987). Other exchange forms of montmorillonite, such as those of higher cation exchange and larger monovalent cations (i.e. potassium), permit only a limited swelling of the smectite layers as energy barriers exist that prevent extensive interlayer separation from taking place, so that this swelling is usually anisotropic in nature (Murayama, 1966). Such differential swelling leads to stresses being set up in the rock fabric, which will ultimately lead to slaking. Japanese coal bearing rocks have been shown to be highly prone to montmorillonite swelling and slaking. The sensitivity of these rocks is illustrated by a recorded volumetric expansion of between 2 and 12 per cent at a relative humidity of 75 per cent (Masatomo, 1989).

Hudec (1983) has recognised another form of osmotic swelling arising from the water penetration of the shale. He suggested that as the pore sizes of shales are extremely small, the water vapour pressure within them will be lower than that of the surrounding bulk water and so water will be impelled to enter the rock. As water enters, the pore pressure increases leading to tensile stresses, expansion and eventually slaking. The smaller the capillary radius the greater the capillary suction hence there will be increased pore pressures and a greater rate of failure (Hudec, 1989).

Whereas smectites (including montmorillonites) are able to absorb water on both external and internal surfaces, kaolinite can only adsorb water on its external surfaces and edges. Interplanar swelling will as a consequence be low and kaolinite rich mudstones exhibit far greater resistance to interplanar swelling and slaking. Illites have only a slightly greater affinity for water and also have the ability to resist serious interplanar swelling. Chlorite is similar to illite in its slaking resistance. British minestone is dominated by kaolinite in the north (Raybould, 1966) and illite in the English and Welsh fields (Taylor and Spears, 1970). Montmorillonite is found only in mixed layer clays so its detrimental affects are diluted to a great extent. Although mineralogically related slaking is important, the stability of minestone mineralogy in Britain would suggest that other mechanisms play a more important role in the slaking process of British minestones.

4.2.1.2 Air Breakage

Terzaghi and Peck (1967) attributed slaking to the compression of entrapped air within pores and microcracks, following water uptake by capillary action. The resultant stresses cause failure along planes of weakness within the mineral skeleton, consequently exposing a larger surface area for further sequences of capillary compression and failure to take place (Taylor and Smith, 1986). Several studies have cited air breakage as being one of the mechanisms for slaking (Nankano 1967, Taylor and Spears, 1970; Moriwaki, 1975; Caswell and Trak, 1985; Masatomo, 1989). Another study showed that when saturated minestone was immersed in water, slaking did not occur, whilst lower levels of resistance were observed for desiccated minestone immersed in water (NCB, 1964). Cyclic wetting and drying would maximise the effect of this slaking mechanism.

4.2 1.3 Ion Adsorption

The structuring of water molecules in a clay/water mix causes the well known thixotropic effect, in which the viscosity of the water increases as reorientation and restructuring occurs as the clay surface is approached (Terzaghi and Peck, 1967; Hudec, 1973; Hudec, 1983). These layers of ordered water are able to exert pressure on confining surfaces, causing disruption of the material fabric. Although the pressure is relaxed by drying, the rock does not return to its original state, as some of the pores, joints and microfractures have been permanently enlarged. Repeated wetting and drying causes expansion and contraction, tensional fatigue and rock failure (Moss *et al.* 1981; Mugridge and Young, 1983). The expansion of cement gels and cement stabilized minestone in various environmental conditions has been explained by structured surface water (Hudec, 1983; McNulty, 1985). The ability of a shale to swell is exacerbated by the massive internal surface area associated with clay minerals. The effective surface area (m^2/g) provides massive potential for the hydrogen bonding of water which in turn creates great potential for the volume increase of the rock (Dunn and Hudec, 1972).

4.2.1.4 Ion Exchange and Disassociation

In their natural state, clay minerals will be accompanied by an associated 'atmosphere' of adsorbed cations, usually hydroxyl ions, which are only weakly bonded to the clay particles. If a solution of negatively charged ions comes into contact with the clay particles, dispersion of the ions will occur leaving the particles negatively charged and in a state of repulsion. The dispersing stresses arising from ionic disassociation have been cited as a mechanism of slaking (Franklin and Chandra, 1972; Moriwaki and Mitchell, 1977). The dispersive potential of the dispersing fluid (dialectric constant) together with the type of exchangeable cation are the controlling factors concerning dispersion efficiency. Sodium cations are the easiest to disperse followed in decreasing order by potassium, magnesium and barium cations (Badger *et al.*1956; Franklin and Chandra, 1972). Nankano (1967) considered slaking from a thermodynamic point of view asserting that the slaking process was due to chemical dissolution of hydrogen bondings between clay particles.

Very little research has been carried out into the slaking effects on clays, shales, mudstones or minestone following sea water immersion. Although some slaking mechanisms would be expected to operate in a similar manner in sea water as in fresh water, the rate and extent of slaking could differ as a result of the solute content of sea water. Taylor and Spears (1970) suggested that the internal and external surface tension of shales and mudstones would be altered by the effects of electrolytes, this would in turn reduce capillarity and could thus reduce the effects of 'air breakage'. Also, there may be a reduction in ionic dispersion suffered by a clay or mudstone when immersed in sea water. Ionic dispersion occurs as a result of adsorbed cations disassociating in a dispersing fluid thus setting up repulsive forces between clay colloids. Sea water tends to flocculate rather than disperse clays which would suggest that the mechanism of slaking caused by ionic dispersion would be less important in sea water than in fresh water.

4.2.2 Factors Influencing Slaking Susceptibility

Slaking is affected by many complex and interrelated variables and the importance of individual slaking mechanisms will depend upon a number of factors :

1. Rock fabric.
2. Mineralogy
3. Environmental and hydrological considerations

4.2.2.1 Rock Fabric

Porosity and permeability are very important controlling factors, since these govern the entry and retention of slaking fluids and also the mobility of this fluid once it has penetrated the rock. It has been found that the smaller the capillary radius, the greater the capillary suction pressure experienced and consequently the faster the rock becomes saturated (Hudec, 1989). However, there may be a failure to achieve a rapid increase in capillary suction pressure because of the restricted movement of the water front as a result of low permeabilities, especially if expansive clays are present (Emerson, 1977). A rapid increase in pressure is required for air breakage to occur and usually this rapid pressure increase will be related to the pore size distribution and permeability of a fine grained rock. The rocks which exhibit high saturation rates are found to be associated with early compressive strength failures. Fabric characteristics such as mineral alignment and microfracturing, both common in minestone, also control the resultant geometry of the slaked debris. Slaked particles will be elongated if there is sub-parallel alignment of platey clay particles whilst, if the clay fabric has a more random nature, cracking will usually have a more curved form producing smaller particles of a regular shape (Russell, 1982).

Pore and capillary size is directly related to grain size. Inherently, fine grained rocks will be more prone to break down, and more rapidly, than coarse grained rocks (Perry and Andrews, 1983). The degree of diagenesis and induration may play a major role in the resistance to slaking. An association between low rank coals (representing relatively low levels of induration) and slaking susceptibility has been found (Badger *et al.* 1956; Taylor, 1984; 1988). This may be as a result of enhanced porosity stemming from lower induration. However, it is possible for some shales, associated with high rank coal, to suffer severely from slaking. This can possibly be explained by increased tectonic influence resulting in a fabric riddled with microfractures and thus enhancing the permeability of the rock.

4.2.2.2 Mineralogy

Emphasis has already been placed upon the susceptibility to moisture of montmorillonite and mixed-layer rich argillites and has been reported elsewhere (Skarzynska *et al.*1990). Even small amounts of these reactive minerals can influence behaviour through water absorbtion and ion adsorbtion. Diagenetic cements are also a controlling factor in mudstone durability. In a study of some Canadian shales (Russell, 1982) it was found that slaking durability was controlled entirely by calcitic cements when present, but when absent, the durability of shale was controlled by the clay content. Dissolution of cements has been evoked as a slaking mechanism (Badger *et al.* 1956; Moriwaki, 1975; Surendra *et al.* 1981). However, this evocation may be debatable, for the removal of cement by dissolution would undoubtedly lower the capacity of a rock to resist disruptive forces associated with various slaking mechanisms. Varley (1990) attempted to link mineralogical phases in shales to degrees of weathering and subsequently the degree of slaking resistance in a study relating to tunneling in coal mine shales.

4.2.2.3 Hydrological and Environmental Factors

Slaking is maximised when the hydrological aspect of the environment frequently fluctuates between extremes and prolonged periods of equilibrium are seldom achieved. Wetting and drying cycles and cyclic humidity changes are examples of fluctuations that prevent hydrological equilibrium from being attained. If minestone is placed in an environment where equilibrium is maintained, slaking will cease to be an influential factor in its degradation. Such conditions include constant immersion (Laan *et al.* 1984) or the constant environment provided by the interior of spoil heaps. When encountering these environments, minestone may undergo an initial period of disintegration before equilibrium is reached. Once the effects of this initial 'shock' have been felt, the minestone will reach a stable condition where no further slaking will take place.

The hydrological factors influencing slaking, include the initial moisture content and the dialectric constant of the slaking fluid. A relationship was also found to exist between natural moisture content and the slake durability of argillaceous rocks. The higher the natural moisture content the more likely that the shale will exhibit a lower slaking resistance (Hopkins and Deen, 1984). Both Kennard *et al.* (1967) and Laan *et al.* (1984) have demonstrated that minestone which has not suffered desiccation and maintains its natural moisture content will not slake when immersed in water. In a study of Japanese mudstones (Okamoto *et al.* 1981), pre-immersion drying temperatures, length of drying and initial moisture content were found to have a strong influence on the swelling and the slaking properties of mudstone. It has been suggested (Okamoto *et al.* 1981) that at high temperatures and when prolonged drying is experienced, the internal structure of the mudstone is 'hardened' lessening the likelihood of slaking.

4.3 The Effects of Fresh Water on Cementitious Compounds

Cement based products possess remarkable stability when in contact with 'fresh' unadulterated water. This is one of the major reasons why ordinary portland cement has become one of the most popular and successful construction materials in use today. Some leaching of the products of hydration may occur if the cement product is immersed in water that is not static, though the effect upon durability and strength will be minimal (Neville, 1981). Detrimental leaching may occur if the immersing water is fairly acidic in nature which could lead to a weakening of the cement fabric though the acidity levels required for this to take affect would not warrant the description of the water as being fresh.

4.4 The Effects of Sea Water on Cementitious Compounds

Sea water is a highly aggressive solution which has high potential for promoting chemical attack upon cement based products. The sulphate based compounds within sea water are the principle aggressive chemicals and have the ability to attack hardened cement pastes. Of the sulphates found in sea water, magnesium sulphate ($MgSO_4$) is considered to be the prime aggressor threatening the structure of cement based materials. Unlike other sulphates, where chemical reaction only occurs with the calcium aluminate hydrate component of cement, magnesium sulphate has the further deleterious property of being able to decompose both calcium hydroxide and calcium silicate hydrate.

The processes in which magnesium sulphate disrupts hardened cement paste follow a series of set reactions resulting in the formation of a number of deleterious by-products. These reactions are:

$$3(MgSO_4 . 7H_2O) + 2(3CaO . Al_2O_3 . 12H_2O) \longrightarrow$$

$$3CaO.Al_2O_3.3CaSO_4.31H_2O + 2Al(OH)_3 + 3Mg(OH)_2 + 8H_2O$$

(*Equation 4.1*)

$$MgSO_4 + Ca(OH)_2 + 7H_2O \longrightarrow CaSO_4.2H_2O + Mg(OH)_2 + 5H_2O$$

(*Equation 4.2*)

The gypsum thus produced will subsequently react with calcium aluminate hydrate producing further quantities of ettringite (calcium aluminium sulphate hydrate)

$$3(CaSO_4.2H_2O) + 3CaO.Al_2O_3.12H_2O + 13H_2O \longrightarrow 3CaO.Al_2O_3.3CaSO_4.31H_2O$$

(*Equation 4.3*)

As previously stated magnesium sulphate has the ability to decompose calcium silicate hydrate, the resultant products being gypsum, magnesium hydroxide and silica gel:

$$MgSO_4 + 3CaO.SiO_2.nH_2O + 7H_2O \longrightarrow CaSO_4.2H_2O + Mg(OH)_2 + SiO_2.nH_2O$$

(*Equation 4.4*)

As a result of the low solubility of magnesium hydroxide the pH of the cement is lowered, and as a consequence the reaction shown in *Equation 4.4* may proceed to completion (Lea, 1970; Neville 1981; Mather, 1982). A further consequence of the decrease in the alkalinity of the micro-environment, is the inability of ettringite to find equilibrium and hence stabilize. Thus the continual presence of magnesium sulphate may lead to the conversion of the unstable ettringite into gypsum. With an increase in the level of gypsum following the reactions of *Equation 4.2* further decomposition of calcium aluminate hydrate can be expected following the reaction of *Equation 4.3*. The magnesium hydroxide produced in the preceding reactions has the potential to react with the silica gel which is released during the reaction of *Equation 4.4* to produce magnesium silicate hydrate (Serada and Ramachandran, 1975) :

$$4Mg(OH)_2 + SiO_2.nH_2O \longrightarrow 4MgO.SiO_2.8.5H_2O + n\text{-}4.5H_2O$$

(*Equation 4.5*)

The reactions between the other members of the sulphate family and cement compounds can be epitomised by the reactions involving sodium sulphate. Calcium aluminate hydrate is affected in the following way :

$$2(3CaO.Al_2O_3.12H_2O) + 3(Na_2SO_4.10H_2O) \longrightarrow 3CaO.Al_2O_3.3CaSO_4.31H_2O + 2Al(OH)_3 + 6NaOH + 17H_2O$$

(*Equation 4.6*)

Whilst calcium hydroxide reacts in the following manner :

$$Ca(OH)_2 + Na_2SO_4.10H_2O \longrightarrow CaSO_4.2H_2O + 2NaOH + 8H_2O$$

(*Equation 4.7*)

Calcium sulphate is slightly less disruptive than magnesium and sodium sulphate, as it is unable to react with calcium hydroxide, though like sodium sulphate it does react with calcium aluminate hydrate.

The principle products of these sulphate reactions are gypsum and ettringite. These minerals have lower densities than the minerals they replace so these reactions are accompanied by an increase in volume. The resultant expansion of the cement fabric in an attempt to accommodate this volume increase leads to swelling, disruption and sometimes failure of the cement paste. Despite this, even though cement based products such as concrete have the potential to swell when introduced to a marine environment, this potential is seldom realised. The absence of fabric expansion in concrete marine structures may be attributed to the inhibitory effects of the high concentrations of chloride ions present in sea water. At high chloride concentrations (sea water having a chloride concentration of 0.55 mol/litre) gypsum and ettringite exhibit a higher degree of solubility than when subjected to the lower concentrations of 'fresh water' (2.2×10^{-4} mol/litre). This higher degree of solubility allows the expansive minerals to be leached away by the sea water and hence leads to the preservation of the cement fabric (Lea, 1970).

Other potentially aggressive constituents of sea water are chlorides, carbonates, bicarbonates and magnesium ions. Chloride ions may participate in similar chemical reactions to those involving sulphate ions, with the eventual production of calcium chloroaluminates and other chloride based salts (Mather, 1982). The penetration of magnesium and chloride ions into cement paste may produce a zone where magnesium silicate hydrates are extremely weakened and which could lead to the spalling of surface layers (French, 1991). Both carbonate and bicarbonate may react leading to the carbonation of the calcium ions and calcium hydroxide formed during the hydration of cement. Dissolved carbon dioxide, in the form of carbonic acid, may lead to localised weakening and softening of the cement paste fabric. Carbonation is intrinsically a dehydrating process that leads to internal fabric shrinkage and contraction cracking. Carbonation may lead to an increase in strength as a result of slight shrinkage but deep carbonation could lead to an increase in permeability following the opening of contraction cracks (French, 1991). The maximum rate and extent of carbonation occurs when the hydration products are subjected to relative humidity levels of about 50 per cent (Verbeck, 1958). Carbonation is likely when only very occasional wetting of the cementitious material occurs, such as in the splash zone.

If continual immersion and hence saturation are features of a structure's environment, it is thought unlikely that carbonation shrinkage will play a major part in the deterioration of a cement based material (Mather, 1982).

In addition to the chemical perspective of sea water attack, there are also physical aspects to be taken into account. Crystallization of salt within the pores and fissures of a cement stabilized material may lead to disruption owing to the stresses exerted by crystal growth. This also is more likely to occur near the high water mark and within the splash zone (Allen and Terrett, 1968; Goudie *et al.* 1970; Neville, 1981; Fookes *et al.* 1988). Crystal growth stresses are particularly dependent on the pore structure of the material. The disruption is most effective in high porosity fabrics. Although various salts have differing abilities to exert crystal growth stresses (Allen and Terrett 1968; Leeder, 1983), the most effective salts are sodium and magnesium sulphate. The magnesium sulphate may crystalize directly from sea water whist the sodium sulphate is derived from the simultaneous decomposition of sodium chloride and gypsum (Allen and Terrett, 1968). However, magnesium hydroxide has the ability of slowing down the progress of crystal growth by blocking the pores and reducing the permeability of a concrete fabric (Neville, 1981).

Enough knowledge is available today to preclude the occurrence of concrete deterioration following its exposure to sea water (Mather, 1979), but it is essential that structures are constructed using the highest quality materials and the appropriate precautionary procedures. Any cement stabilized structures that have been constructed out of materials of indifferent quality can be expected to succumb, in varying degrees, to the rigours of the aggressive chemical environment promoted by sea water (Allen and Terrett, 1968). The nature and quality of minestone as a construction aggregate can be described as indifferent so it can be assumed that cement stabilized minestone will suffer to a greater degree from the effects of sea water attack than a traditional concrete, especially when the slaking susceptibility of minestone is taken into account.

4.5 Counteracting the Effects of Chemical Attack upon Cement Based Products

Although the type of cement has a strong bearing on the resistance of a cement based material to sulphate attack, it must be realised that the prime factor influencing durability is the nature of the fabric, particularly as exemplified by the density and permeability of the material. Resistance to sulphate attack may be increased by the use of a sulphate resistant cement which is characterised by a low percentage of tricalcium aluminate (C_3A) so that less ettringite will form as a consequence of sulphate attack. Pozzolanic addition, substitution and replacement, may impart a certain degree of sulphate resistance to a cement based fabric. These pozzolans render inactive the alumina bearing phases by reacting with the free calcium hydroxide ($Ca(OH)_2$) to produce stable calcium silicate hydrates, thus preventing the development of gypsum (Kalousek *et al.* 1972). The role played by clay with respect to the sulphate resistance of a cement stabilized soil has been examined by Sherwood (1958; 1962). It was found that a reaction between gypsum and clay minerals allowed ettringite to form, despite the use of sulphate resistant cements. It was thought that excess water and free lime provided the ideal conditions for this reaction. However, this type of reaction may also be inhibited by the use of pozzolans which again react with the excess lime, leading to a lowering of alkalinity.

Where concrete is concerned, by increasing the compaction and density of the fabric and hence lowering its permeability, resistance to sulphate attack is greatly enhanced. A lower permeability also ensures that the leaching of the soluble constituents of cement is kept to a minimum (Mather, 1982). As previously mentioned the high chloride concentration of sea water prevents the crystallization of gypsum and ettringite, but if stagnation of the pore water occurs, super-concentration of these minerals will build up eventually leading to crystallization and further fabric disruption. This situation can be observed in laboratory immersion tests where stagnation is allowed to take place and leaching is hindered. A certain degree of circulation is thus required to flush out the expansive leachates (Neville, 1981; Mather, 1882).

4.6 Minestone and Hydraulic Applications

The Dutch have considered minestone to be a worthy material for hydraulic construction for a number of years (Laan *et al.* 1984). The desperate and urgent need for reconstruction following the disastrous flooding of 1953 brought about a heavy reliance upon minestone and its use in hydraulic applications has increased ever since. A compelling incentive for the Dutch to use minestone for these large scale hydraulic projects is the scarcity of the necessary quantities of traditional construction material together with the ease of access to the Ruhr coal mining region of northern Germany with its abundant stockpiles of cheap minestone. In contrast, in Britain, with the relative abundance of traditional construction material and less precarious geography there has been little or no incentive until recently to utilize minestone for hydraulic purposes. With the increase in environmental awareness and in the cost of more traditional materials, as the more accessible resources are exhausted, together with the interest in large scale hydraulic schemes such as the Severn Estuary Barrage, the opportunity will increase to take advantage of the abundant stockpiles of minestone.

The appeal of minestone as a hydraulic material lies in its greater resistance to current scour and wave attack as compared to dredged sands, the traditional bulk hydraulic construction material (Laan *et al.* 1984). This resistance to hydraulic forces is partly determined by specific gravity and grain size (*see Section 5.4*). The minestone used by the Dutch had a D_{50}

ranging from 15mm unscreened, to 60mm when screened. British minestone has an unscreened D_{50} ranging from 0.425mm to 40mm. By screening the minestone the D_{50} values can be greatly increased thereby improving the hydraulic capabilities of the minestone. The Dutch minestone had a specific gravity of approximately 2.5, and most British minestones have similar values. In comparison a typical marine sand (Oosterschelde sand) has a specific gravity of 2.65 and a D_{50} between 1.5mm and 2.5mm. The following sections discuss actual examples where minestone has been put to effective use as a construction material in a hydraulic related environment.

4.6.1 Dutch Case Studies

Three case studies were cited by Laan and his colleagues (1984) as examples of the use of minestone for hydraulic applications. They include retaining bunds and slope protection for a sand fill dam, a minestone sill used in preparation for a dam closure and scour protection banks along a heavily used shipping canal.

4.6.1.1 Retaining Bunds and Slope Protection

Minestone was used to construct small dams (bunds) at either side of the main structure of the Brouwersdam which separates the Grevelingenmeer from the North Sea (*see Figure 4.1*). Initially the minestone served as protection and provided retainment structures for the main body of the dam during the hydraulic filling phase of the construction. At the completion of this phase the minestone bunds were incorporated into the final structure as protection for the slopes of the dam. Adequate wave resistance was provided by the coarser particles of the minestone. The finer particles especially on the seaward side were winnowed out of the outer layers of the bunds but did not affect the overall effectiveness or defensive capabilities of the minestone bunds. Although the slopes were quite steep after deposition, wave action on the seaward side levelled the slope to a 1:7 gradient.

Final slope protection on the North Sea face was made of graded quarrystone together with rip rap and an asphalt layer to complete the impervious slope on the seaward side. The landward slope was protected by a permeable layer of copper slag blocks incorporating minestone as a filter layer to prevent the sand being washed out between the blocks.

4.6.1.2 Minestone Sill

The 'Markiezaat' comprises the eastern reaches of the Oosterschelde in the delta region of the southwestern Netherlands. In 1983 a dam was used to sever it from the main body of the Oosterschelde. Before closure of the dam could be attained, a 250m section of the seabed with a depth between 7m to 15m, had to be raised. A minestone sill, constructed from fine unscreened minestone from an old spoil heap, was used to fill this deeper section of the dam site (*see Figure 4.2* and *Figure 4.3*).

The minestone was deposited using split barges and slopes averaging 1:2 to 1:2.5 were realised without the help of retaining bunds. In eight weeks a total of 180,000 tonnes of minestone was placed. This minestone had a D_{50} between 7mm and 15mm and a fines content (< 63mm) between 5 per cent and 10 per cent. It was estimated that there was a 25 per cent loss due to scouring and wave action. Despite the fine grade of the minestone, the specified standards were met and the use of minestone was regarded as a 'remarkable' success.

4.6.1.3 Bank Protection for a Shipping Canal

Figure 4.4 shows a cross section of a shipping canal in the Twente area of the eastern Netherlands and illustrates the use of minestone as a scour fill material. Concern was expressed following years of successive scour action at the toe of the canal bank. To remedy the situation 150,000 tonnes of minestone were used along a 8km stretch of the canal. Two zones of minestone were used, a lower zone composed of fine minestone and an upper zone composed of coarse minestone. This method succeeded in shoring up the wooden sheet piling of the canal bank and also protected the canal bed from further scour.

4.6.2 German Case Studies

In Germany minestone is frequently used as a general or additional fill for mounds, earth dams, dikes and embankments for waterway engineering projects (Hauss and Mayer 1990, Hauss and Heibaum, 1990). German interest in this use of minestone dates back to the early fifties when post-war reconstruction was beginning and economy was essential. Today minestone is frequently used to heighten existing canal banks following mining subsidence. *Figure 4.5* illustrates the use of minestone for canal 'dams' and shows how improved compaction techniques have created a rather heterogeneous fill with a soft rather weakly compacted core surrounded by shells of better compacted minestone.

Worry has often been expressed suggesting that minestone would continuously break down, eventually producing an amorphous mass of plastic clay. However, the Federal Waterway Engineering and Research Institute (B.A.W.) found that this was not so when they investigated 22 year old canal dams. It was found that minestone did breakdown with time, though only to gravel and sand size particles. Whereas the original mine waste used in the canal structure consisted of 60 per cent gravel, 30 per cent sand and approximately 10 per cent silt and clay, test pits indicated that 22 years of placement had not had a dramatic effect on the physical characteristics of the minestone, with the resultant grading being 58 per cent gravel, 27 per cent sand, 12 per cent silt and 3 per cent clay. By performing shear strength, permeability, seepage flow, chemical analysis and settlement tests the Federal Waterway and Engineering and Research Institute concluded that, if correctly applied, with adequate quality control, minestone can be considered as a fill material comparable to more traditional materials used for hydraulic applications (Hauss and Heibaum 1990).

4.6.3 Slurry and Tailing Lagoons

Until ten years ago the British experience in applying minestone to hydraulic applications was restricted to 'in house' lagoon embankments for the dewatering and the storage of ash slurry and tailings. The choice of minestone for lagoon confinement was a natural choice because of its abundance and obvious proximity to power stations and colliery sites. The use of minestone as ash lagoon embankments is a world-wide occurrence, with several examples being reported (Bros, 1990; Dunbaven, 1990). Rainbow and Skarzynska (1987) cite several examples where minestone has been used alone or in conjunction with fly ash. A 61m high dam at Moundville, West Virginia has been built using fresh coarse minestone while, at Przezchlebie in Poland, a 20m high, 940m long ash and minestone dam has been constructed for ash storage. In Britain, the biggest structure of this type is the complex of ash lagoons at Gale Common, operated by what was then the CEGB in north Yorkshire. Minestone was used as a low permeability material and provided a wave resistant layer to protect the bulk of the structure which was constructed of rolled ash (*see Figure 4.6*). The scale of such a complex is clear when it is realised that 18 x 10^6 m^3 of freshly wrought minestone were used in the first of three phases of development (Rainbow and Nutting, 1986). Minestone is thought to perform admirably when used for lagoon embankments especially if due consideration is given to the geotechnical properties of the available minestone and 'judicious selection' is practiced (Rainbow and Skarzynska, 1987).

4.6.4 British Case Studies

Although major examples of minestone in hydraulic structures are scarce in civil engineering in Britain, those schemes that have utilized the product have had an important effect on their surrounding environment. Hamilton (1984) reported on the construction of a sea wall at Deal, Kent, where 200,000 tonnes of minestone were used and Kluth (1984) reported on the use of 340,000m^3 of minestone to protect 2000 residential properties from overspill flooding of the River Ouse, Yorkshire. These two instances are considered in more detail to demonstrate how minestone can be put to beneficial use in a hydraulic situation.

4.6.4.1 Deal Sea Wall

During January 1978, the existing shingle/sand sea defence at Deal failed following a tidal surge and wave damage caused by storm conditions. As a consequence 300ha of agricultural land were inundated by 2 x 10^6 m^3 of water. Two mechanisms were thought to be responsible for the failure, the shingle bank crest was drawn down by approximately 2m allowing overtopping to occur, and by seepage through the shingle causing massive piping. The choice of replacement materials for the embankment was primarily governed by what was available at the time. Minestone from Betteshanger Colliery was readily available, being located only 2 km away from the failed sea defences. The minestone was used to provide a partial replacement for the limited shingle supply, support for the existing shingle bank, a firm sea wall crest and crest road foundation and a positive seepage control body. *Figure 4.7* illustrates a typical cross-section of the minestone modified sea defence at Deal.

The design was intended to have a high degree of reliability in resisting wave action, with an extreme design event of 250 years and a significant design event every 5 to 10 years. Details of the construction and logistics of the scheme are given by Hamilton (1984) in some detail and hence will not be reiterated here. Hamilton concluded that the shale compared favourably with the other potential fill materials and showed adequate strength and impermeability for this specified project. Furthermore he concluded that the exposed face of the minestone embankment showed a greater resistance to erosion, both by public wear and tear and water, in respect to other fill materials. Although not important for this project, reservations were expressed as to the interactions of the minestone with adjacent steel and concrete structures.

4.6.4.2 The Selby-Wistow-Carwood Barrier Bank

The choice of minestone for flood protection embankments along the river Ouse in Yorkshire, followed extensive tests upon a test embankment, proving that it compared favourably with the other available materials. The main embankment has an average height of 4m, a maximum of 6m and a length of 5.5km. The purpose of the embankment is to connect riverside flood defences between the village of Carwood and Selby in an attempt to alleviate flooding on the lower reaches of the Ouse, which would affect 2000 residential, commercial, industrial and agricultural properties. Specifically, the embankment has been designed to pond any floods more severe than a 1 in 10 year event (the design criterion for the existing flood protection defences) by containing the water between the embankment and the river channel. Though the area between the channel and the embankment is primarily agricultural land, it represents a greatly diminished area compared to that which would have been at risk prior to its construction. *Figure 4.8* shows a typical cross-section of the flood embankment built along the Ouse. Minestone was supplied from the Gascoigne Wood drift mine which originates from the Selby superpit.

The test bank indicated that the compacted material had a permeability between $2x10^{-6}$ and $1x10^{-6}$ m/s and a density of approximately $1.92Mg/m^3$. The shear strength implied by the slope angle was found to be greater than the average strength of more traditional earthfill materials, even in adverse conditions. The rate of weathering was considered to be high, yet its affect upon strength was thought to be minimal. Basic erosion tests at the site proved that the minestone could withstand the effects of sustained scour without detrimental to the overall structure. The overall results gained from the trial embankment led to the conclusion that minestone compared favourably with the more traditional earthfill materials and, as a local source was readily available, it made economic, logistic and technical sense to utilize it.

4.6.4.3 Other British Sites Employing Dark Shales

There have been instances where cyclothemic and other dark shales have been used to construct earthfill dams in Britain and, though the material was not minestone, their properties can be considered to closely resemble those of colliery waste. Following the construction of the Balderhead Dam, Yorkshire, Kennard *et al.* (1967) concluded that the Carboniferous cyclothemic material used was highly satisfactory as fill, but stressed that their conclusions should not encompass and be applicable to all Carboniferous mudrocks. The Carsington Reservoir Dam, Derbyshire provides an example of the use of a black Carboniferous shale to construct an earthfill dam that subsequently failed. *Section 2.5* discusses the role that pyritic decomposition played in the dam's eventual failure and a full account is given by Pye and Miller (1990). The Carsington Dam was constructed from dark Namurian marine shales (i.e. Coal cyclothems are usually of fresh water or brackish water origin) excavated from a local borrow pit. The embankment consisted of a rolled clay core, the clay being derived from highly weathered local mudstones and periglacial deposits. The shoulders of the embankment were constructed in two zones (*see Figure 4.9*), an inner zone of weathered mudstone and an outer zone of harder essentially unweathered shale. The shales were placed in successive stages over a three year period and were considered to be open to severe weathering during the periods when work was at a standstill. It is thought that the alteration of the geotechnical properties of the Carsington Dam fill, by prolonged exposure to weathering during construction, contributed to the eventual failure of the embankment (Pye and Miller, 1990).

Minestone has also been considered for several other possible hydraulic schemes. British Coal were actively involved in consultations with the East Yorkshire Borough Council concerning the Humberside Coastal Protection Scheme. The highly sensitive and vulnerable eastern coastline was considered for coastal protection works which involved the employment of minestone reefs to damp down the energy of the wave regime. Although environmental considerations can favour the use of colliery waste for construction, environmental concerns can also be raised to discourage its use. This was borne out when East Yorkshire Borough Council showed concern for the stability of minestone and pulled out of the Humberside Project, citing fears of the risk to Bridlington's 'award winning' beaches and the worry of the local fishing fleet if colliery waste was to be used (Bridlington Free Press, 1988). Both minestone and oil shale waste have been assessed as a fill material for a maritime village development involving the partial infilling of a marine harbour (Couper and Montgomery, 1990). However, concern relating to the durability of oil shale waste and minestone in a tidal setting and the proximity of cheaper reserves of dredged sands may bias any decision against the use of a waste material.

4.6.5 Minestone and Tidal Barrage Schemes

A number of the postulated barrage schemes proposed for various sites along the British coastline could become users of substantial quantities of minestone. The biggest proposed scheme in Britain is the Severn Estuary Tidal Power Barrage. The potential of this scheme for employing vast quantities of minestone is great, and the proximity of the South Wales and Somerset coalfields enhances this potential.

4.6.5.1 Minestone and the Severn Barrage Scheme

The Severn Barrage Tidal Power Scheme is expected to supply 17 TWh, or 7 per cent of the present energy consumption in England and Wales (Department of Energy, 1989). The proposed location of the barrage will span across the estuary from Lavernock Point on the Welsh side to Brean Down on the English side (*see Figure 4.10*).

Recent reports (Severn Tidal Power Group, 1986; Department of Energy, 1989) have emphasised the use of hydraulic sand fill for embankment construction, yet early appraisals and reports (Shaw, 1980; Department of Energy; 1981) contemplated the use of waste materials such as minestone and waste sands from the china clay industry in Cornwall. An estimated 19 x10^6 m^3 of bulk fill, with an annual requirement of 4.73 x 10^6 m^3, are thought necessary to build the embankments (Department of Energy; 1981). When minestone has been considered for use in a barrage scheme, it has invariably been regarded as either a permanent peripheral fill to protect the main bulk of hydraulic sand embankments (*see Figure 4.11*) or for the construction of bunds to control the sandfill and provide temporary protection (*see Figure 4.12*). As well as being considered for use in the main body of the barrage, minestone could be used for temporary construction works and the reclamation of poor quality land for peripheral barrage activities (Behrman and Thurlow, 1980).

The embankments will comprise between 10 and 15 per cent of the overall cost of the barrage scheme across the Severn Estuary (Department of Energy, 1981). If minestone is adopted as the main bulk fill, the South Wales coalfield would benefit environmentally following the removal of old, obtrusive colliery tips and the release of land for more beneficial activities. However, recent studies (Severn Tidal Power Group, 1986; Department of Energy, 1989) have emphasised the preference for the use of hydraulic sands primarily on the grounds of economics. Estimates suggest that the unit cost of minestone would be five times that of hydraulic sand. Furthermore, the presence of huge reserves of sand within the Severn Estuary, especially in the Welsh, Middle and Cardiff grounds, places increased competitive pressures upon the possible use of minestone. The argument that by using minestone, valuable resources would be preserved is very tenuous in the case of the Severn Barrage for even the largest sand requirement of the order of $100 \times 10^6 m^3$ represents less than 5 per cent of the existing deposits in the estuary.

4.6.5.2 Design Requirements For Minestone Fill

Although, as we have seen in previous sections, examples exist of minestone being used or considered for hydraulic applications, knowledge of its performance in an aqueous environment is still relatively limited. Whereas certain aspects of the stability of minestone have been addressed, other important areas of research have had scant attention, such an area is the performance when subjected to erosive forces. Furthermore, no research has been carried out to determine the effects of stabilization upon the erosion resistance of the material. Other properties such as strength, permeability, and seepage flow have been shown to compare favourably with the more traditional hydraulic construction materials. Yet, before minestone can be fully accepted as a viable hydraulic material, knowledge must be gained of the resistance to erosion caused by both unidirectional flow and cyclic flow associated with wave and tidal action. This present study is targetted at this particular area and includes erosion tests performed on both stabilized and unstabilized minestone so that the knowledge of this behaviour will be increased from the present sparse level.

As with any fill which has a fine component, minestone used as a bulk material for a hydraulic structure would require appropriate filters to be incorporated into the design to prevent the finer particles from migrating into the outer layers of the structure. Piping may occur where a hydraulic gradient exists within a structure as a result of differential water levels set up by tidal conditions. Wave action may also give rise to piping in a highly voided structure for a zone of compressed air advances ahead of the wave front while, in retreat, the situation is reversed producing a sluicing action which pumps fines out of the structure if precautions are not implemented (Fookes and Poole, 1981). These include graded filters specifically designed for the particular fill (Terzaghi and Peck, 1967) and provision for the compressed air to be dissipated into the atmosphere without any damaging effects. Alternatively, piping can be prevented by the use of geotextiles such as marine filter fabrics or all-in composite filters.

Good compaction of the core material of a hydraulic structure is not only necessary to prevent piping, but has a strong bearing on the porosity, permeability and the shear strength of the core and its ability to support the external armour layers of the structure (Poole, 1991). Actual in-service degradation processes of the core material are considered far less important than those of the outer armour layers and, with proper design, can be limited to the slow removal of any soluble materials. For this reason evaluation of such materials is usually confined to the traditional geotechnical tests and grading limits to ensure that the fill material meets design specifications (Poole, 1991).

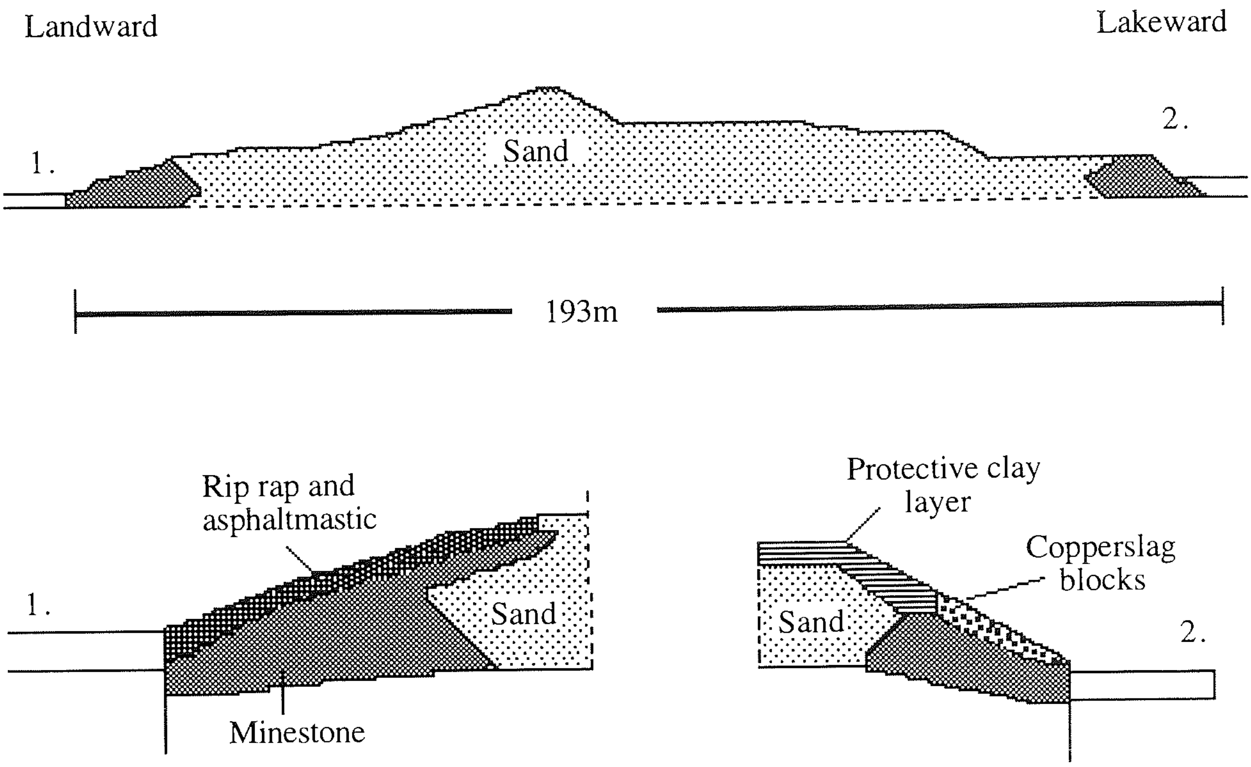


Figure 4.1 The Brouwersdam, part of the Delta Project, incorporating minestone as an integral component (After Laan *et al.* 1984).

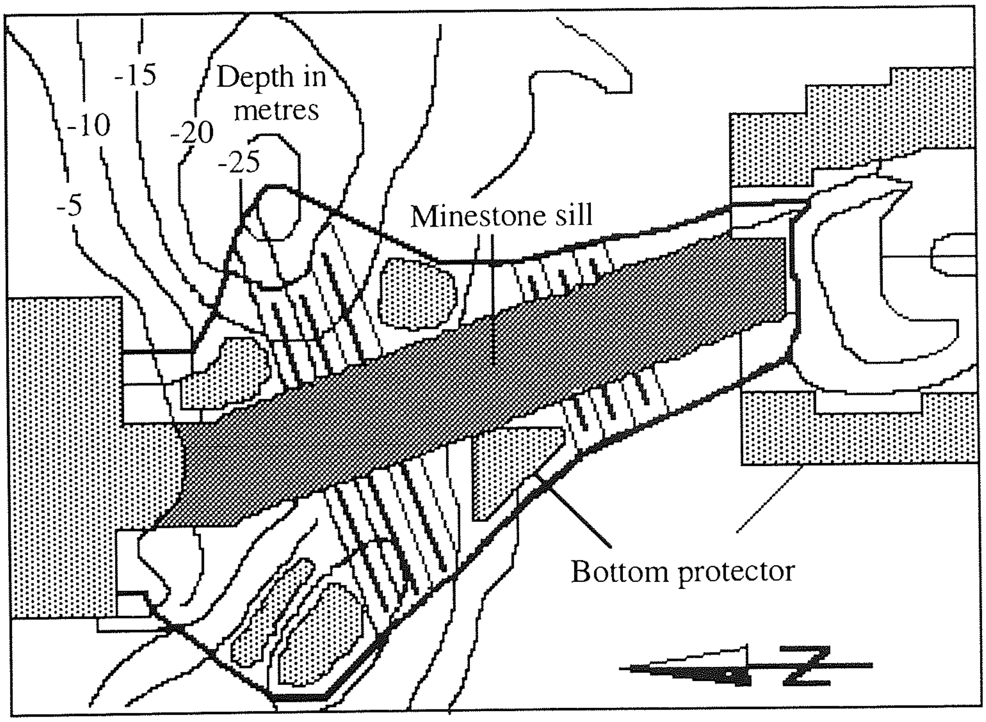


Figure 4.2 Plan view of the minestone sill used as a closure (Oosterschelde, Netherlands) (After Laan *et al.* 1984).

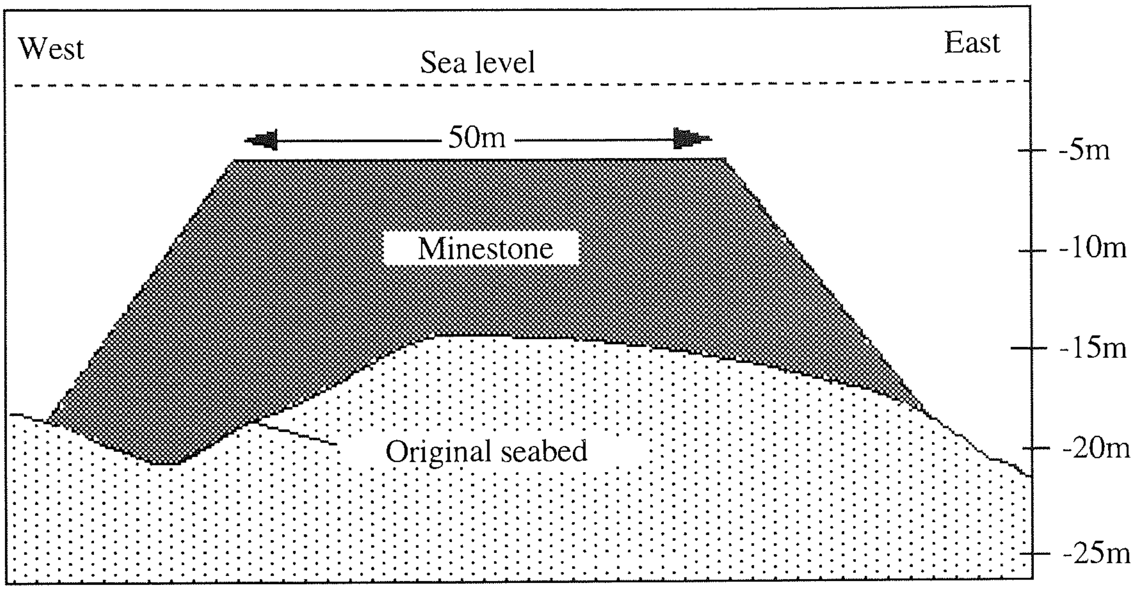


Figure 4.3 Cross-section of the minestone sill used to level out the seabed in preparation for dam closure (Oosterschelde Netherlands).

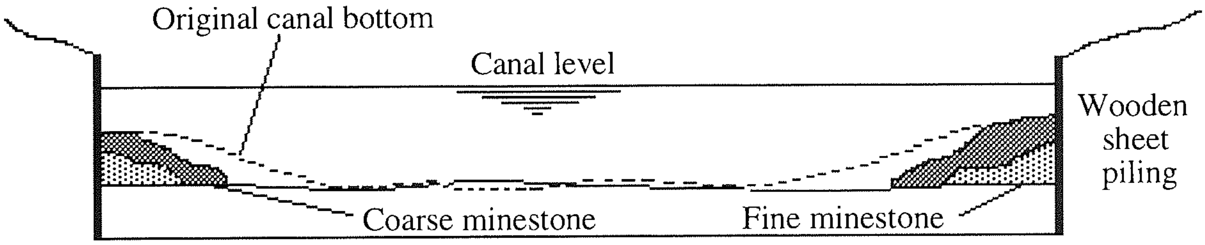


Figure 4.4 Minestone used to protect shipping canal banks from the action of scour (Twente, Netherlands).

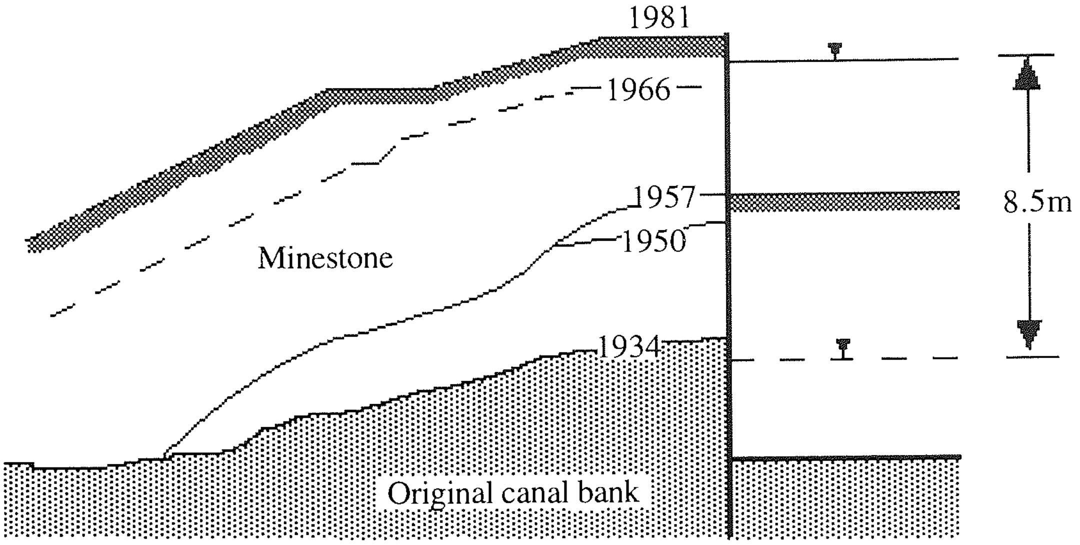


Figure 4.5 The use of minestone to counteract mining subsidence along a shipping canal, Northern Germany.(After Laan *et al.* 1984)

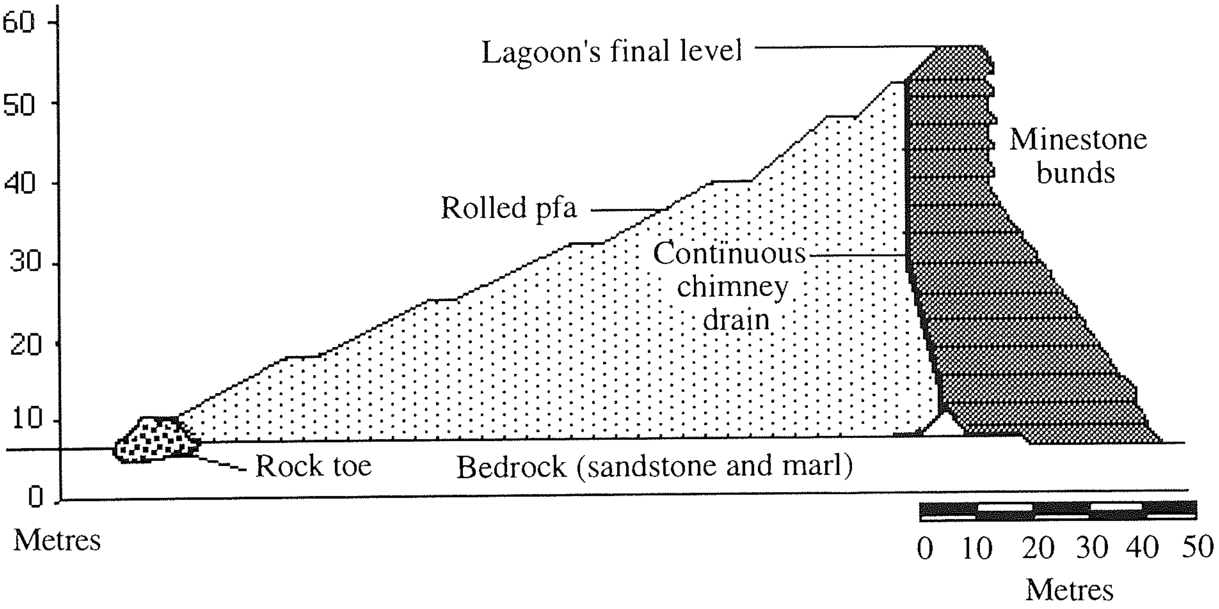


Figure 4.6 Section of the Gale Common ash lagoon (After Rainbow 1984).

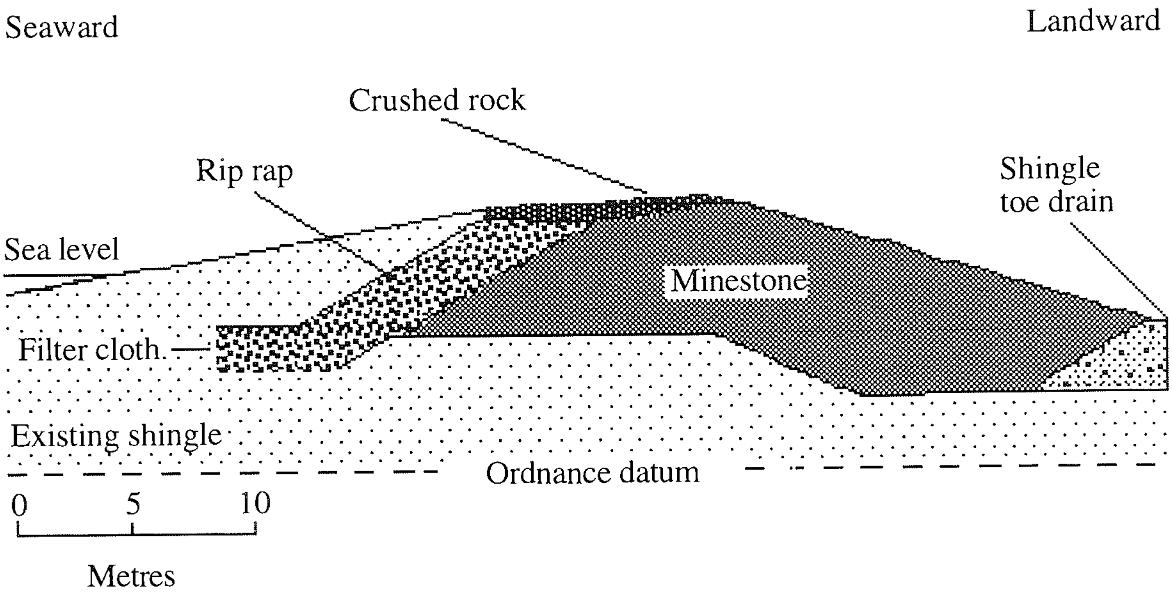


Figure 4.7 Minestone incorporated into sea defences at Deal, Kent.

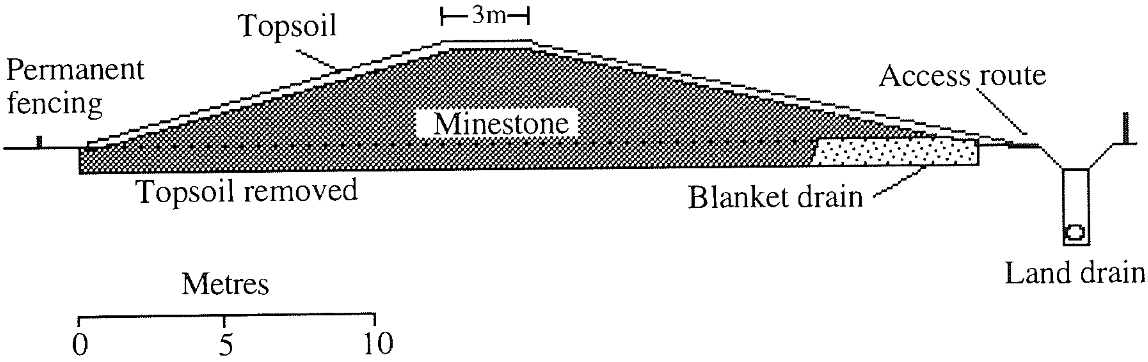


Figure 4.8 Flood embankment constructed out of fresh arisings from the Selby superpit.

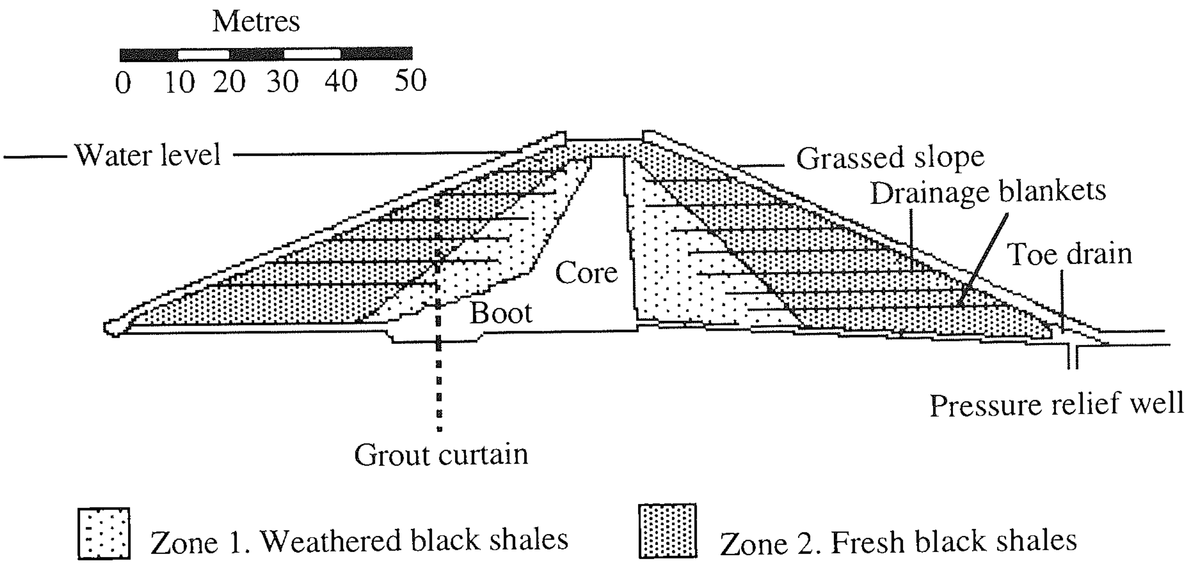


Figure 4.9 A section of the Carsington dam illustrating the differing zones of construction.

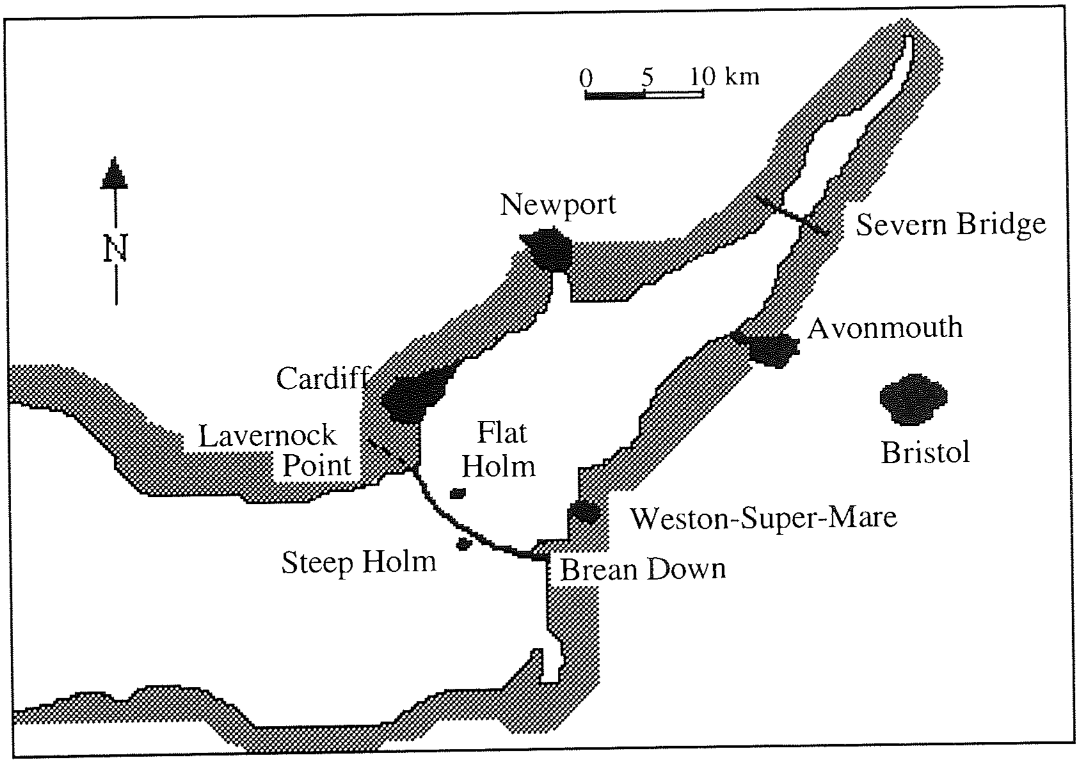


Figure 4.10 The proposed location of the Severn Estuary Barrage.

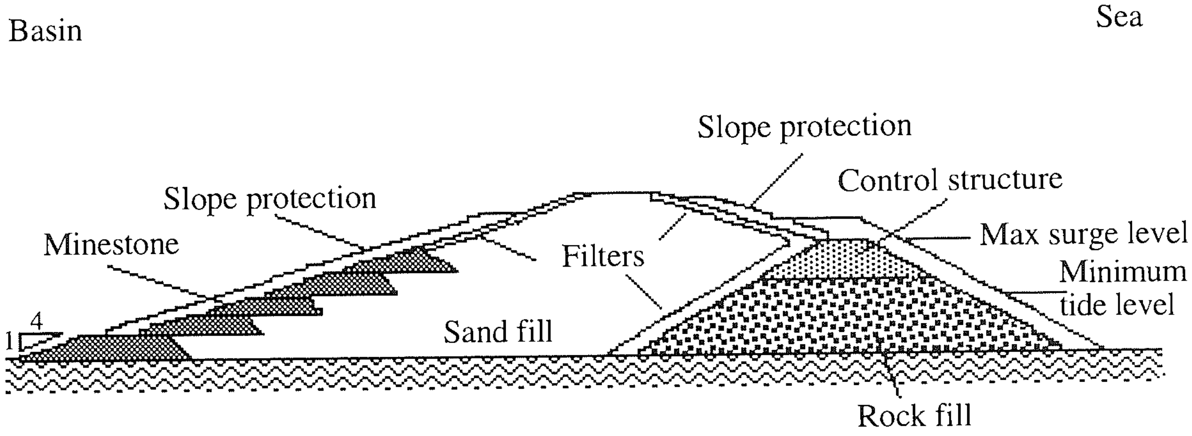


Figure 4.11 A possible design for the main embankment of the Severn Barrage Scheme (After Department of Energy 1981).

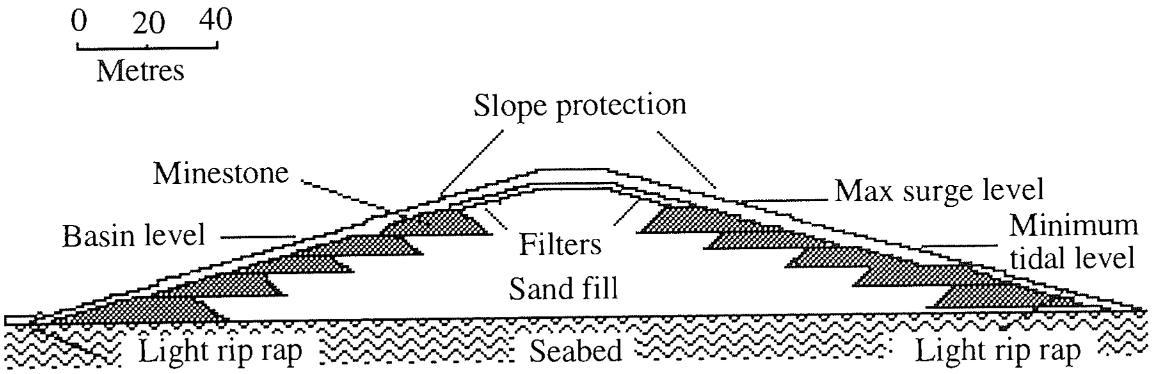


Figure 4.12 The proposed use of minestone for secondary embankments for the Severn Barrage Scheme (After Department of Energy 1981).

5. THE EROSION ENVIRONMENT

5.1 Hydrological Zonation

The prevalence the hydrosphere has on the weathering processes is highlighted by the vital role water plays in the degradation processes mentioned in *Sections 2.5* and *4.2*. The durability of a material within an engineering structure will partly depend upon its relative position in relation to the hydrological zones (Fookes, 1991). Three zones, based on the hydrological divisions of Keller (1957) are recognised, these being the subaerial, sub-surface and sub-aqueous zones. The hydrological processes that may affect exposed geo-material in each zone are illustrated in *Figure 5.1*. In the case of any hydraulic structure, irrespective of its particular role, it will be exposed to degradation in all three zones. A single hydraulic structure will thus be subjected and influenced by all the processes associated with these hydrological zones.

5.2 Nussbaum and Colley's (1971) Exposure zones

Zones of exposure were suggested by Nussbaum and Colley (1971) to define the vertical sections of a dam affected by different weathering and erosional forces.Three exposure zones were recognised :

Zone 1 This is generally a sub-aqueous zone, i.e. that part of a dam that lies below the minimum pool elevation.

Zone 2 This section of the dam lies between the minimum pool elevation and the maximum level of the splash zone and is exposed to the most severe environmental changes.

Zone 3 This section of the dam is generally in a dry state, being situated above the maximum water level and normal splash zones. Climatic affects are important factors influencing a structure or part of a structure situated in this zone.

These exposure zones of were assigned to a channel structure by Shen and Akky (1974) so that Zone 1 corresponded to the channel bottom and that portion of the bank that is constantly below the minimum water level. Zone 2 composed that part of the bank where water level fluctuates with time (seasonal and tidal flows), whilst Zone 3 corresponded to the subaerial area of a structure which is generally in a dry state though it may be affected by rare flooding events. By zoning the face of a soil-cement dam, a more economical design may be obtained, for less cement can be used in those zones subjected to less severe environmental exposure with a greater cement content being necessary only in those zones exposed to the most severe agents of weathering and erosion.

5.3 Zones of Weathering and Erosion of Fookes and Poole (1981)

Fookes and Poole (1981) also recognised that the extent to which a hydraulic structure would be influenced by natural forces depended on the position the individual sections occupied in relation to the water body. They divided a coastal marine hydraulic structure into four weathering zones (*see Figure 5.2*):

Zone 1.

This is situated high above the water level, so the only direct influence the water body would have upon the structure is through splash and spray effects. Subaerial weathering agencies are the predominant factors operating in this zone. Salt may encrust the surface and fill pores and joints. Wind blown sands may lead to degradation by contributing an abrasive component, as on occasions, can wave thrown shingle. By far the most important factor influencing this zone are the climatic changes, although biological factors may be of some importance with plant growth, if present, providing an important insulation and protective cover to the underlying structure.

Zone 2

This zone is situated above high water level, however it may be affected by wave upwash. Abrasion is brought about by wave driven sands and shingle. Although subaerial weathering is still influential, the forces related to the intermittent wetting and drying cycles are considered to be the dominant factor in structural attack.

Zone 3.

The intertidal zone can be regarded as the most aggressive zone for promoting material degradation. The predominant forces operating in this zone are those developed by cyclic wetting and drying. Wave erosion is another significant process causing structural damage within this zone, while biological action such as boring and burrowing may have some local significance. Subaerial weathering together with cyclic wetting and drying, become less important as the low water level mark is approached.

Zone 4.

Within this zone the structure is permanently submerged and is unaffected by subaerial weathering. Although no wave associated abrasion is experienced within this zone, current scour may be highly significant in structural damage. The environmental conditions prevailing here are regarded as being the least aggressive (to a hydraulic structure).

There is clearly considerable similarity between the zone systems presented by Nussbaum and Colley (1971) and Fookes and Poole (1981), with the major difference being that the Zone 2 of Nussbaum and Colley is equivalent to Zones 2 and 3 as described by Fookes and Poole.

5.4 Major Erosion Types

The erosion of a concrete hydraulic structure can be attributed to three major mechanisms namely cavitation, abrasion and chemical attack (ACI Committee, 1987). Chemical attack (discussed in *Section 4.4*) is believed to weaken the cementitious fabric, thereby making it more prone to damage from the other mechanisms, rather than being a true erosion mechanism. At low cement contents, a stabilized soil is exposed to a further mechanism of erosion due to the stresses set up within the boundary layer of the flow. At the critical shear stress, the shear exerted by the flow upon the bed exceeds the inter-particle bonding of the material, that is the natural cohesion of the soil and the bonding properties of the cement. The following section discusses the erosional mechanisms of erosion by flow shear, abrasion and cavitation.

5.4.1 Erosion by Flow Shear

The fluid forces acting upon bed grains are illustrated in *Figure 5.3*. The lift force is the result of the Bernoulli effect. The fluid stream lines over a projecting particle will converge so that the fluid velocity will increase and, as a result of maintaining pressure equilibrium, the pressure above the grain will decrease. Each grain experiences a drag force as a result of the fluid velocity gradient and pressure on the exposed surface. The mean drag on a particle is given as :

$$F_0 = \tau_0 / N \qquad (\textit{Equation } 5.1)$$

where :

F_0 = Mean drag component - force per particle.
τ_0 = Mean bed shear stress - force per unit area.
N = Number of particles per unit area.

The parameter (C_c) governing the condition of critical particle motion can be expected to be dependant upon gravity (g), grain size (d), immersed weight ($\sigma - \rho$), fluid kinematic viscosity (υ) and the bed shear stress (τ_o), thus :

$$C_c = f\,(\, d, g, (\sigma - \rho), v, \tau_o) \qquad \textit{(Equation 5.2)}$$

Shield's rearrangement in Henderson, 1966 and Hemphill and Bramley, 1989 of the above equation gives the function:

$$\frac{C_c}{\left(\sigma - \rho\right) d} = f\left(\frac{\left(\tau_o / \rho\right)^{0.2} d}{\upsilon}\right)$$

(*Equation 5.3*)

For particles larger than 5mm the function in *Equation 5.3* has values from about 0.04 at the onset of instability to 0.06 for full particle motion.

When considering the critical tractive force for a particle situated upon a sloping surface (such as an embankment surface or sides of a channel), the weight of the particle (those larger than 5mm) must be taken into consideration allowing a slope correction factor to be applied (Lane, 1955) :

$$\frac{\tau_o}{\tau_{cr}} = \left(1 - \frac{\sin\alpha}{\sin\phi}\right)^{0.2} = \omega$$

(*Equation 5.4*)

where :

ω = Slope correction factor.

ϕ = Angle of repose of bank material.

α = Slope angle in relation to the horizontal.

The platey nature of minestone particles exposes a greater surface area to flow leading to an increase in bed friction compared to more equidimensional particles such as sand. However, the lift component may also be enhanced by the high surface area as will the drag component. The presence of a cohesive component in minestone material also leads to an increase in the critical condition for particle motion.

The proper selection of an erosion resistant lining material for a channel involves an accurate appraisal of the shear stresses expected (to be encountered) within that channel. Once this information has been gathered, the required objective can be attained by either controlling the hydraulic variables so that the induced shear will always be less than the critical shear value of the lining material or choosing a lining material that has a greater critical shear value than the expected (induced) flow shear.

The boundary shear stress for a plane stationary bed, with a constant cross-sectional area throughout a given distance, and a constant steady-state flow is given by Lane (1955):

$$\tau_o = \gamma R_h S$$ (*Equation 5.5*)

Where :- τ_o = The boundary shear stress.
γ = Unit weight of water.
R_h = The hydraulic radius.
S = Slope of the energy grade line.

The shear stress deduced using *Equation 5.5* represents the average value of shear force per unit wetted area. However, in reality, shear stress is not a uniformly distributed force. One method of determining the true values of shear stress at different locations in a channel is to assume a power law for velocity distribution (Lane, 1955) and by incorporating a function value into the equation that equates to the ratio of bed width to flow depth. Thus:

$$\tau_o = f \gamma R_h S$$

(*Equation 5.6*)

Where f is a coefficient ≤ 1 and relates to a function of the ratio of bed width to flow depth.

Shear stress can be related to flow velocity using Manning's formula:

$$v = \frac{1}{n} R_h^{0.67} S^{0.5}$$ (*Equation 5.7*)

Where v is the flow speed and n is Manning's coefficient (roughness coefficient). If S (gradient) is substituted from *Equation 5.6* into *Equation 5.7*, the following can be derived:

$$v = \frac{\tau_0^{0.5} R_h^{0.16}}{\gamma^{0.5} n f^{0.5}}$$ (*Equation 5.8*)

A stable channel can be designed using *Equation 5.9* either by choosing a hydraulic radius so that :

$$v < \frac{\tau_{cr}^{0.5} \, R_h^{0.16}}{y^{0.5} \, n f^{0.5}}$$ (*Equation 5.9*)

or by choosing the lining material such that:

$$\tau_{cr} > \frac{v_{max}^{2} \, \gamma \, n^{2} f}{R_h^{0.67}}$$ (*Equation 5.10*)

Where τ_{cr} is the critical shear stress measured by experimental procedure.

For cohesive materials whose shear strength depends upon cohesive bonding, Graf (1970) suggested:

$$\frac{\tau_{cr}}{(\gamma_s - \gamma)d} = C_o$$ (*Equation 5.11*)

Where d = The particle diameter.
γ_s = Unit weight of the lining material.
C_o = Cohesion coefficient.

The above equations are only valid for a homogeneous material which maintains mechanical and hydraulic integrity and has not been influenced by external environmental agencies. In more natural conditions, the critical shear stress of a soil or a stabilized soil will be affected with time by the effects of weathering.

As the cement content is increased, the critical shear stress of a stabilized material is increased, until a point is reached when the binding forces of the cementitious compounds are greater than the shear forces associated with the flow. Once this condition is reached, erosion of the material can only continue by abrasive matter being introduced into the flow or by the process of cavitation.

5.4.2 Erosion by Abrasion

The abrasional effects of water-borne solids, whether silts, sands, gravels or ice, can severely damage the fabric of a hydraulic structure. Abrasional erosion can be distinguished from cavitation damage by the smooth worn appearance of the eroded surface rather than the rough pitted appearance of the latter. The rate of erosion is dependant upon a number of factors which include the particle load properties, such as shape, size, quantity and hardness, the velocity of flow and the quality of the lining material.

Most geo-materials that are used for hydraulic structures will be susceptible to abrasion to some degree (ACI Committee, 1987) and, though improving the quality of the material may help to limit damage, economic considerations must be taken into account. Even though soil-cement can be considered inferior to traditional concrete in protecting a structure from abrasive action, it is often the accepted solution to an erosional problem due to its relative low cost. However, it will be necessary to give careful consideration to the material in the design of the structure to minimise or eliminate the hydraulic regime responsible for the abrasive action.

5.4.3 Erosion by Cavitation

Cavitation results from the formation of bubbles or cavities within a flowing fluid. These cavities develop when a pressure drop is experienced within the fluid, caused by surface irregularities altering the streamline configuration temporarily increasing the flow velocity. If the local pressure drops to a critical value, water will vaporize abruptly at its ambient temperature. The types of irregularities that may cause a decrease in pressure include protruding joints, roughened surfaces, abrupt curvatures away from flow, offsets into flow and offsets away from flow (ACI Committee, 1987). Cavitation often begins near curves or offsets in a flow boundary, or at the centre of a vortex.

Cavitation bubbles will increase in size as they flow downstream until they enter an area of flow where the pressure field causes the them to implode. The instantaneous pressures created by implosion are of an extreme magnitude. Those bubbles collapsing close to or upon a surface will exert this extreme pressure over an infinitesimal area for a fleeting period. Successive impact of imploding bubbles can damage virtually any solid material. Once cavitation pitting has been initiated, severe damage can develop rapidly as this initial damage provides further opportunities for cavitation bubble generation (Johnson, 1963).

Microfissures within a material are thought to contribute to the effects of cavitation by setting up tensile stresses within the fissures through the action of compression waves in the water. The simultaneous collapse of a cavitation 'cloud' may exert pressures in excess of 100 bars upon an area of tens of square centimetres. The subsequent elastic rebound of the material may propagate further damage and a weakening of the fabric. Once the surface of the hydraulic structure has been roughened by cavitation damage, further erosional mechanisms such as high velocity flow removing protruding particles, may take effect, and so accelerate the failure of a material. Cavitation pitting usually cuts around the harder coarser aggregate producing an irregular roughened surface, as opposed to the smooth regular surface produced by abrasive action. Cavitation may be limited by implementing fundamental hydraulic design consideration or by a process of flow aeration which has been shown to limit the effects (Wei and De Fazio, 1982). Though cavitation may undoubtably contribute to the erosional attack upon a stabilized soil in a hydraulic regime, the extent of such a contribution lies outside the scope of this study.

Figure 5.1 The principal hydrological zones and associated processes (Keller 1957).

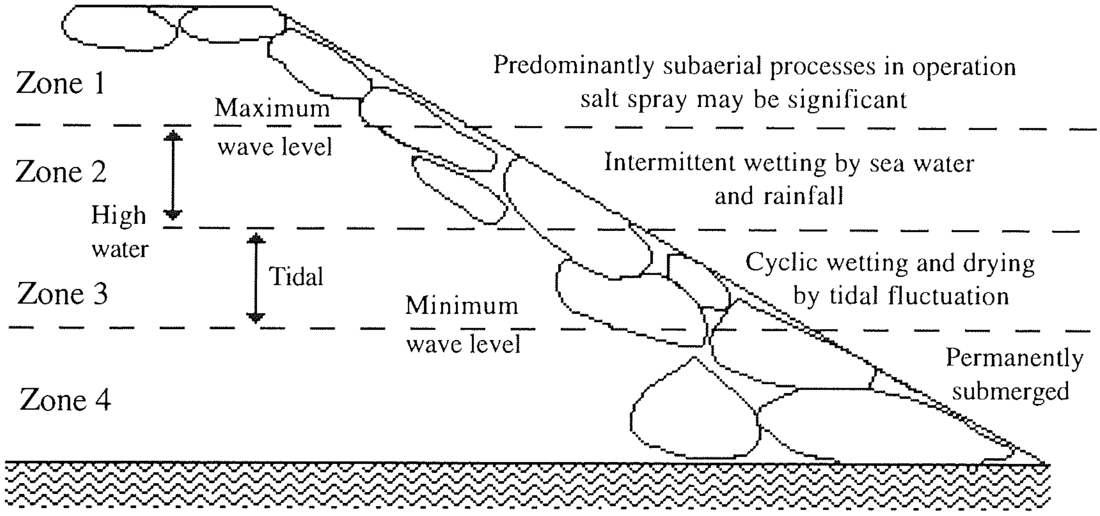


Figure 5.2 The four main weathering zones in a coastal environment (After Fookes and Poole 1981).

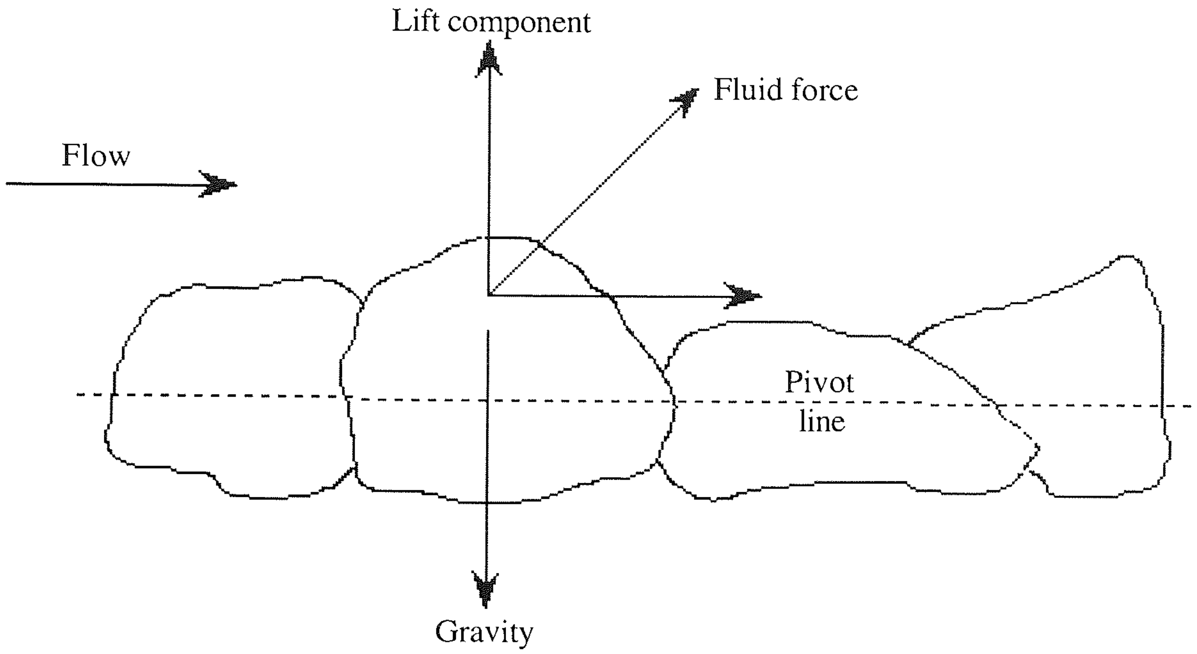


Figure 5.3 The forces acting upon a grain resting upon a bed of similar grains and subjected to a flow above it.

6. MATERIAL CLASSIFICATION AND TEST DESCRIPTIONS

6.1 Introduction

The physical, mineralogical and chemical characteristics of a particular minestone will determine its behaviour when used for engineering purposes. The intention of this chapter is to determine the basic physical, mineralogical and chemical characteristics of the selected minestones to provide a strong foundation for the subsequent research into their performance in a hydraulic environment. These fundamental properties include particle size distribution, the Atterberg limits, specific gravity, the moisture-density relationships, lithology and the chemical properties of cation exchange, pH, and sulphate content.

6.2 Material Choice

Minestones from the following sources were used for this study (*see Figure 6.1*) :

1. Hatfield
2. Lady Windsor
3. Point of Ayr (Above high tide) POAA
4. Point of Ayr (Below high tide) POAB
5. Sutton Manor
6. Wardley
7. Wearmouth
8. Westoe

These sampling locations were chosen following consultation with British Coal. The decision to use minestone from the Wearmouth and Westoe pits was prompted by political and environmental pressures. An E.C. ruling preventing British Coal from dumping minestone into the North Sea, has placed the onus upon the Minestone Executive to find new means of disposal for the solid waste arising from the northeastern collieries. At present this waste together with accompanying liquid 'fines' are the cause of severe pollution along a 40km stretch of the Durham coastline (Rowland, 1988). The minestone from the northeastern region, including the wastes from the Wardley, Wearmouth and Westoe collieries could possibly be supplied as a construction material to build offshore coastal defence reefs that would protect the fragile Humberside coastline from the ravages of the sea. The type and degree of pollution that has been experienced along the Durham coast as a result of indiscriminate dumping may be limited if controlled usage, stabilization and careful seabed management are implemented.

The Lady Windsor site is a possible source of embankment material for the construction of any future tidal barrage scheme across the Severn Estuary, whilst minestone from Sutton Manor is a possible source of material for any barrage scheme that may be proposed for the River Mersey. The choice of minestone from Sutton Manor for inclusion in the present research was also governed by its known resilient nature and hence could be used for comparative purposes. Likewise, minestone from the Hatfield site was chosen for its susceptibility to environmental changes and thus could be used to represent minestone at the opposite end of the scale to the minestone from Sutton Manor. Hatfield colliery, together with its sister pit Thorne, can be regarded as another potential source of material for coastal defence work along the sensitive East Yorkshire coastline. It also has the potential for hydraulic applications within the low lying region surrounding the colliery where flood defence and land reclamation are thought necessary. Minestone from the Point of Ayr site was chosen because it had already been utilized in a hydraulic setting to reclaim land from the sea, upon which a British Coal experimental station has subsequently been built.

6.2.1 Sampling Procedure

Minestone was obtained following the standard sampling procedures specified by British Coal (NCB, 1971) and was sealed and stored in heavy duty polythene bags which were placed within sealed metal storage canisters until required for testing. Those samples taken from spoil heaps were excavated from a depth of at least a metre to avoid using the outer weathered crust. Sampling was carried out at one location per colliery in order to restrict variations in physical properties. There is a risk of biased sampling using this procedure, however if careful inspection of the spoil material is carried out prior to sampling, a representative sample may be assumed to have been taken. This sample is not representative of the spoil heap as a whole, but can be regarded as representative of material expected within that spoil heap. To obtain a representative sample of the whole spoil heap would be an impossibility without the use of specialized excavation and drilling equipment.

A conscious effort was made to use fresh sample material where possible. Minestone collected from the Hatfield, Sutton Manor, Wearmouth and Westoe collieries was of fresh current arisings, at most only a few work shifts old. Two samples were collected from the Point of Ayr site. Point of Ayr (Below) (POAB) was sampled below the high tide level and has been subjected to a period of wave action and submergence in sea water, whilst Point of Ayr (Above) (POAA) samples were collected well above the high tide level and were largely free from the influences of direct sea contact. However, it is possible that the POAA samples have been subjected to marine spray action or storm driven splashing, but it is thought that the effect of this action would be at a minimum as the samples were collected from beneath the surface of relatively fresh material. Both Point of Ayr samples were taken from the same area of the spoil heap and are considered to be of the same age, that is not more than a year old.

Access to the most recent spoil at Lady Windsor was prevented due to pit closure and ongoing dismantlement. The minestone sample taken from this site was taken from a spoil heap which was at least several decades old and, judging from its proximity to the pithead could possibly be up to 150 years old. The Wardley sample was collected after the closure of the colliery and can be considered to be at least twenty years old if not older. With the exception of the Wardley material, the provenance of each minestone can be guaranteed to be that of the named pit, the Wardley spoil heap however, has been used by several collieries during its operational lifetime and therefore the samples cannot be guaranteed to have originated from the Wardley colliery.

6.3 Classification Tests

Although minestone is regarded as a problematic material (Rainbow, 1986), classification of the material may be achieved using British Standard methods in conjunction with modifications specified by British Coal (NCB, 1971). These modifications are intended to minimise the degree of pre-treatment a sample may experience prior to testing. The main problem associated with minestone is that, once allowed to dry out, it will be highly susceptible to slaking if subsequently reintroduced to moisture. Unnecessary slaking is avoided if the NCB modifications are implemented.

6.3.1 Grain Size Distribution.

The grain size distribution of the samples was ascertained using the standard wet sieving method (Test 7(A) BS 1377:1975) modified to British Coal specifications (NCB, 1971). No attempt was made to subdivide that fraction of the sample which passed the 63μm sieve. In a study into the performance of cement stabilized minestone (Thomas, 1987), a grading index devised by Hale (1979) was found to be of some benefit for comparing the grain size distribution of minestones. This index is a numerical representation of the complete grading curve and is calculated by sub-dividing the grading slope into ten subgroups. The percentage of material retained within each subgroup is multiplied by the mean mesh size of that range and the resultant values are summed to give a grading value which, for presentation purposes is divided by 100 to give the grading index. A skewed distribution towards the coarser end of the scale would give a higher grading index than one that is skewed towards the finer end of the grain size scale.

6.3.2 The Atterberg Limits

Although consistency limits are no longer required by the Department of Transport specification (1986) on cement stabilized materials, it is still important to know these basic engineering properties as they can be used to predict the performance of a material. This has been borne out in the work of Thomas (1987) who used them to predict the capabilities of a particular minestone when stabilized with ordinary portland cement. The consistency properties of the clay faction may also give an indication of clay mineral surface activity. Samples with a high plasticity index and liquid limit may suggest the presence of a high percentage of hydrophilic minerals such as the expansive clays. The higher the liquid limit and plasticity index of a soil the greater the affinity it will have for water and correspondingly, a greater potential for swelling. Soils with high plasticity indices have also proven more difficult to stabilize (Ransom, 1963). All the minestone samples were tested to determine their consistency limits even though colliery spoil with less than 10 per cent 'fines' (< 63μm) is normally exempt from such testing (NCB, 1971). The liquid, plastic and plasticity limits, together with the linear shrinkage values were determined according to BS 1377:1975 Tests 2(A), 3, 4, and 5 respectively.

6.3.3 Moisture Content.

During sampling a quantity of minestone was placed in a sealed container. On arrival back at the laboratory, this sample was immediately tested for water content, the results representing the *in situ* moisture content of spoil heap material at the depth of sampling. The stable air dried moisture content of each minestone sample was also determined using the same standard method (Test 1 BS 1377:1975).

6.3.4 Specific Gravity.

The specific gravities of the minestone samples were determined using standard gas jar method for fine, medium and coarse grained soils as specified by Test 6 (A) in BS 1377:1975.

6.3.5 Loss on Ignition

A direct relationship exists between the specific gravity of minestone and the loss experienced when the material is ignited at 815°C. This relationship is associated with the coal content of the minestone. The higher the coal content, the lower the expected specific gravity values would be, which in turn may lead to lower maximum compacted densities.

6.3.6 Moisture Content -Density Relationship

The moisture-density relationship for each sample was determined according to the vibrating hammer compaction test (BS 1924:1975). The tests were carried out on material that had been crushed to pass a 20mm sieve with a 10 per cent cement content. The cement content was chosen following the completion of a preliminary study indicating that at least 10 per cent cement was necessary to gain the strength specified by the DoT (1986). For convenience and economy of time, the optimum moisture content determined for the 10 per cent cement mixtures was used for all subsequent samples in the research programme.

The air dried material was allowed to equilibriate at approximately two thirds of the estimated optimum moisture content for several days before the remaining water and the cement were thoroughly blended into the mixture and the compaction tests carried out (in a similar manner to that used by McNulty, 1985).

6.3.7 Lithology

The lithology of the particles that make up a stabilized soil will to a certain extent determine the resistance of that material to erosion. Generally, the shales, mudstones and claystones will exhibit a greater susceptibility to erosion than the siltstones and the well cemented sandstones associated with the Coal Measures. Coal, if present in significant quantities may render a stabilized material less resistant to abrasion as a result of its weak brittle and friable nature. Whereas a large quantity of pyritic material can leave stabilized materials open to chemical attack if decomposition proceeds, from an attritional view point, a high percentage of pyritic and iron rich clasts would benefit the stabilized material through their highly resistant nature when considering abrasive and erosive attack.

Each sample was examined lithologically and particles were placed into five main categories, namely shale (including true shales, mudstones and claystones), siltstone (both fine and coarse grained), sandstone, (generally well cemented), coal ('small' coals, pyritious coals and low density, highly carbonaceous mudstones) and ironstone (including the iron carbonates, crystal pyrite, vein pyrite, pyritic coal and pyritic rich mudstone). The lithological nature of several grain size categories were individually determined i.e. > 50mm 20-50mm, 10-20mm and 5-10mm together with the lithological nature of the sample as a whole. The sampling procedure for lithological determination followed methodology similar to that used in particle size distribution determination.

6.3.8 Sulphate Content

Acid and water soluble sulphate contents were determined using the methods suggested by Sherwood and Ryley (1970) based upon BS 1377 Test 9(A) and 9(B) but modified to suit colliery shale (NCB, 1971).

6.3.9 pH Determination.

The pH value of each minestone sample was determined using the standard electrometric method (BS 1377:1975 Test 11A). The test was carried out upon a sample with a water:soil ratio of 2.5 as recommended by the standard.

6.3.10 Cation Exchange Capacity

Three size categories (1200-475μm, 475-63μm and sub 63μm) were prepared following the method used by McNulty (1985). Each category was treated with barium nitrate before being acid stripped to produce a filtrate. The filtrate was analysed using atomic absorption spectrophotometry to determine the cation exchange capacity (CEC) of the test material. The cation exchange capacity of a material is normally expressed in milliequivalents per 100 grams (meq/100g). The method used for cation exchange determination is given in Appendix A1.

6.4 X-Ray Diffraction Analysis

X-ray powder diffraction was used to give a general indication of the mineralogy of the whole sample as well to give a specific insight into the clay minerals of the individual minestones. A monochromatic, copper based X-ray diffractometer was used to produce a diffractogram which ranged between 2 to 20Å. A full description of the equipment, methodology and theory behind X-ray diffraction is discussed elsewhere (Wilson, 1987).

6.4.1 Whole Sample Analysis

A representative sub sample was crushed to pass 75μm before being blended with distilled water and smeared evenly onto a glass slide. Care was taken to dry the specimen slowly to avoid peeling or cracking. A diffractogram of the samples is then produced using a copper based X-ray diffractormeter.

6.4.2 Clay Mineral Analysis

Realistically, true quantitative analysis upon an unknown clay sample is a practical impossibility largely because of the problem of clay mineral variability. Such complicating factors as interstratification and intercalation of non-exchangeable clay species allow only a semi-quantitative analysis based on assumptions and estimations (Griffin, 1971; Wilson, 1987). A relatively quick and uncomplicated method, discussed by Griffin (1971), was used to determine the relative proportions of kaolinite, illite, chlorite and expansive mixed-layer clays (generally illite based). This method relies upon three assumptions:

1. The sample is made up entirely of clay minerals.
2. The quality of refraction shown by the clays is consistent.
3. The ratio between the 3.5 Å kaolinite peak and the 3.54Å. chlorite peak is a 1:1 linear relationship.

In the case of most soils and minestone, these assumptions will not hold. Rainbow (1984) and Collins (1976) have indicated that minestone shales and mudstones are composed of only between 50 per cent and 70 per cent clay minerals. The consistency of refraction will depend upon the composition polytype and the degree of crystallinity of the clays and hence consistent refractive quality is not guaranteed. However, despite these contradictions of the basic assumptions, this method is thought to be adequate for comparative purposes (Griffin, 1971) and has been previously used to analyse minestones (Thomas, 1987). Appendix A2 gives details of the procedures and formulae used to calculate the proportions of the salient clay minerals present within a minestone.

6.5 Microbial Analysis

The important role that bacteria have in the decomposition of pyritic mudstones has been stressed in *Section 2.5.3*. The biochemical reactions associated with the metabolisms of various microbes can have a devastating affect upon an engineering structure (Pye and Miller, 1990), hence it is vitally important to determine whether microbial action is taking place within a material. Sulphates and sulphuric acid can be considered the most important deleterious compounds produced by bacterial action. These compounds are the direct or indirect products of the metabolic action of the autotrophic sulphophile aerobes such as Thiobacillus thiooxidans and Thiobacillus ferrooxidans.

At the colliery site, samples were collected using sterile, disposable spatulae and sealed in sterile glass sample tubes. The solid growth medium suggested by Leathen *et al.* (1956) was used to cultivate the presence of any autotrophic sulphurphiles. The propagation of cultured Thiobacillus species takes the form of small raised colonies with irregular margins. These colonies are cream to glistening tan when young but become granular and brown with age. Underneath and surrounding the colonies a brown stained region of oxidised iron may exist. The tests were carried out in the Department of Pharmaceutical Sciences, Aston University.

6.6 Porosimetry

Hudec (1989) suggested that the durability of a rock is a function of grain size, pore size and the rate of capillary absorption of water. Fine grained rocks exhibit faster absorption rates and deeper penetration of both water and salt solution than intermediate grained rocks. The smaller the capillary radius, the greater the capillary suction pressure, which will consequently lead to a higher rate of saturation. Those rocks that have high saturation rates will have high rates of pore pressure increase which in turn, leads to both decrease durability and early compressive strength failure.

Mercury intrusion porosimetry was originally developed by Washburn (1921) and has been subsequently applied successfully by numerous workers to the study of many porous materials. The experimental technique employed in mercury intrusion porosimetry involves evacuation of the volume in which the sample is contained. Mercury is then introduced into the sample cell and a pressure is applied forcing the mercury into the interparticle voids and pores. By progressively increasing the pressure and measuring the volume of mercury penetrating the sample per increment, the pore size distribution may be calculated using the relationship :

$$P = \frac{-2\,\sigma\cos\theta}{r} \qquad \textit{(Equation 6.1)}$$

where :- P = Applied pressure.
r = Capillary radius (Circular).
σ = Surface tension of the fluid.
θ = Fluid contact angle.

The diameter of the pore may be derived from *Equation 6.2* :

$$D = -(1/P)\,4\,\sigma\cos\theta \qquad \textit{(Equation 6.2)}$$

Equation 6.2 represents the ideal situation in which all the pores are cylindrically shaped. Such a model may not best represent pores in actual materials, but generally it is accepted practice to employ this model for treating what, otherwise, would be a most complex problem.

The experimental technique of mercury porosimetry uses several variables to determine the nature of the pore structure of a material. One of these is the wetting or contact angle between the mercury and the pore wall (Lowell and Shields, 1981). The contact angle will vary somewhat according to the composition of the material. It has been recommended that in the absence of specific information concerning the material, a value of 130^{o} is adopted (Micromeritics, 1988), however the contact angle of most substances is thought to be close to 140^{o} (Lowell and Shields, 1981). Another important variable is the surface tension of the mercury, the recommended value is 484 dyne/cm^{-1}, but it may vary with the purity of the mercury.

The pore surface area may be calculated from the PV work expended in forcing mercury into the pores. The work (dw) required to immerse an area (dA) of pore wall is expressed by :-

$$dw = \sigma \cos \theta \, dA = -Pdv \qquad \textit{(Equation 6.3)}$$

Where : Pdv = Pore volume distribution

The total pore area being calculated as :

$$A = \frac{-1}{\sigma \cos \theta} \int_{v_{min}}^{v_{max}} Pdv \qquad \textit{(Equation 6.4)}$$

Where :-

A = Total pore surface area

v_{max} = Volume of maximum pore size

v_{min} = Volume of minimum pore size

Representative samples of shale from each minestone were analysed using the fully computerised Poresizer 9310, manufactured by MICROMERITICS. The same values for each variable were used for each sample.

6.7 Water Stability Analysis

As coal mining became more industrial and automation became more widely used, the property of slaking which is shared by the majority of argillaceous rocks became of increasing concern to the coal mining industry. Early attempts to determine the breakdown characteristics of the 'run of mine' shale passing through coal washing plants included the end-over-end jar slaking test (Berkovitch *et al.* 1959; Horton *et al.* 1964). This test was the principal forerunner of the slake durability test which today is the suggested and most favoured test for determining the slaking characteristics of a rock.

6.7.1 The Slake Durability Test

The slake durability test was developed to give a reliable but simple method for quantifying the disintegration of argillaceous rocks when immersed in water (Franklin and Chandra, 1972). The test apparatus consists of a drum that has a screen opening of 2mm (No. 10 mesh). This drum is rotated by an electric motor at a constant speed of 20rpm within a bath of slaking fluid (Franklin and Chandra, 1972) (*see Figure 6.2*).

The standard test method (Franklin and Chandra, 1972; Brown, 1981) makes use of 10 equidimensional shale pieces (50g apiece), oven dried at 110°C. These pieces are placed into the mesh drum, which in turn is immersed in the slaking fluid and rotated for 200 revolutions (10 minutes). The material remaining within the drum is dried and reweighed before a second cycle of slaking is initiated. The standard slake durability index is calculated as follows :

$$SDI = \frac{\text{Oven dried material retained at the end of the 2nd cycle}}{\text{Oven dried weight of the original sample}} \times 100$$

(*Equation 6.5*)

Thomas (1987) found that the standard slake durability test was unable to distinguish between 16 test samples on the basis of slake susceptibility. The size of the individual sample particles was thought to be the cause of this classification failure. It was considered that clay rich 50g particles would be poorly represented in highly weathered samples, especially in minestone of low rank. This underrepresentation would lead to an overestimation of the slaking resistance of a minestone sample. The modified slaking test (Thomas, 1987) makes use of the standard apparatus, but reduces the particle size range to 5-10mm, increases the test period to 1 hour to maximise the distinction between the minestone samples and avoids thermal alteration of the mineral structure (Hopkins and Deen, 1984; Masatomo, 1989) by using air dried samples and accounting for the initial moisture content in the index formula :

$$\text{MSI} = \frac{M_1 - M_2 \dfrac{100 + w_1}{100}}{M_1} \times 100\% \qquad \textit{(Equation 6.6)}$$

Where :- MSI = Modified slaking index.
M_1 = Mass of air dried sample before test.
M_2 = Mass of oven dried sample after.
w^1 = Moisture content of the original sample.

The present study into the durability of minestone in sea water has made use of the modified slake durability test using both distilled and sea water as the slaking fluids. A further series of tests was carried out using the modified version, but retaining the 10 x 50g sample requirement of the original test.

6.7.2 Laan's Rate of Fineness Test.

The Dutch Road Engineering Division of the Ministry of Transport and Public Works studied the long term disintegration of minestone in terms of grain size distribution (Laan *et al.* 1984). In conjunction with these long term experiments a quick weathering test was devised to compare the resistance of minestone from different sources. A 5kg dried minestone sample, grade 45-31.5mm was placed in water for 3 days, then air dried and screened, the rate of fineness being taken as a measure of a minestone's resistance to weathering. The rate is calculated using the cumulative mass percentages retained on 9 standard sieves ranging from 31.5mm down to 125µm, this value being divided by 100 for presentation purposes. The following relationship was found :-

$$S = \frac{t - a}{326\text{-}48fr} \qquad \textit{(Equation 6.7)}$$

Where :- S = Residue on screen 31.5mm (percentage of the original grade 45 - 31.5mm)
t = Time in weeks.
fr = Fineness rate (as established by Laan's rate of fineness test).
a = a constant

Once fr has been established, the value of 'a' follows from the boundary condition that when t = 0 then S will equal 100. When the fineness rate is 7.5 (as determined using Laan's Rate of fineness test) the relation implies that after 10 weeks the residue remaining on the 31.5mm sieve will be 75 per cent of the original quantity whilst after 100 weeks only 5 per cent will remain (Laan *et al.* 1984). A close correspondence exists between the rate of fineness and the rate of weathering experienced in the long term experiments carried out by the Dutch Ministry. Following experimentation, a basic classification was proposed in which a fineness rate below 7.5 indicated extreme sensitivity to weathering, a value between 7.5 and 8.25 implied high sensitivity and materials with a fineness rate between 8.25 and 9.00 have a moderate sensitivity to weathering.

Slight modifications were made to the test procedure for the present study. All materials to be tested were air dried as opposed to oven drying at 60°C to ensure that heat modification did not occur. British Standard sieves were used for the test, whereas the Dutch experiments used sieves designed to the Netherlands Standard NEN 2560. As a result a slight difference in the initial sample range exists. The Dutch had an initial grading of between 45mm and 31.5mm, in this study the initial grading ranged between 50mm and 28mm. For the same reasons a slight difference exists between the 9 sieves used for the two studies. Both distilled water and sea water were used as the slaking fluids. Tests were also carried out on material at its original moisture content.

6.7.3 Tidal Simulation Testing

This test was designed to determine the effects of continual wetting and draining cycles upon a body of minestone, in a manner that simulates tidal action and conditions of Zone 3 in the Fookes and Poole zonation scheme (1981). A 60kg sample of uncompacted minestone was placed into a container which possessed a drainage outlet at the base (*see Figure 6.3*). Sea water was carefully poured into the container to cover the top of the sample to a depth of 200mm. The minestone samples were not allowed to desiccate prior to testing. The samples were kept immersed for six hours before being drained by opening the base tap. The samples were left to freely drain and stand for 4 hours before being reimmersed for a further 4 hours. Following the second period of immersion the samples were again allowed to drain for 10 hours (overnight). This daily cycle was repeated over a set period of time and following every fourth daily cycle (8 wetting and draining cycles) a grain size analysis was undertaken to determine the extent of degradation by slaking. As the sub sample used to determine grain size distribution had a different moisture content to the main sample it was disregarded and played no further part in the cyclic test. The careful manner in which the container was filled and drained ensured that no impact or attritional component was imposed upon the minestone sample and any change noted in the grain size distribution can be attributed to the cumulative action of cyclic slaking.

6.7.4 Cyclic Wetting and Drying Tests

This test is similar to the tidal simulation tests in so far as the samples were cyclically immersed and drained, however the difference between the two is that the sample in this test was allowed to air dry over a number of days before repeating immersion. This test is designed to simulate conditions expected in Zones 1 and 2 of Fookes and Poole's zonation scheme for weathering (*Section 5.3*). These conditions are associated with only occasional immersion such as high spring tides or during storm events. This test also simulates the conditions associated with intermittent wetting by rainfall and situations where the water table rises on occasion but falls back for a relatively lengthy period allowing any structure or body of minestone to drain. This situation represents the drawdown conditions to which a hydraulic structure could be exposed.

The test involved immersing 20kg of minestone in a bath of sea water for 24 hours, before draining the water off and allowing the sample to air dry at room temperature over a period, usually 5 to 7 days, until the air dried moisture content was stable. A grain size distribution analysis was undertaken at the beginning of each new cycle of test, before the sample was reimmersed. Unlike the tidal simulation test, the sub-sample used for the grain size analysis has a similar grain size to the main body of the sample, and so it was remixed with the whole, for inclusion in further test cycles.

6.8 Cement Stabilized Minestone (CSM) : Preparation and General Test Procedures

6.8.1 Initial Preparation

The bulk samples collected from each colliery were allowed to air dry at ambient room temperature for several days until the moisture content remained stable (Test 1(A) BS 1377:1975) and this value was recorded. Once dry, the materials were sieved and all particles above 20mm were crushed using a vibrating hammer and subsequently returned to the bulk sample. The maximum particle size of 20mm was chosen in consideration of the mould dimensions, with the ratio of mould dimension to maximum particle size falling comfortably within the specifications recommended by British Standards (British Standards Institute, 1975a). By crushing the oversize particles all lithologies of the bulk sample could be represented within the cement stabilized samples without having to include oversize clasts (Thomas, 1987) which could easily jam during compaction affecting the density values of the CSM samples. Once dried and crushed the material was thoroughly blended before being placed into polythene bags and sealed in metal canisters. The same size samples were used for the erosion/abrasion test and all the strength and immersion tests.

6.8.2 Mix Design

The bulk of the samples had a cement content of 10 per cent, although some series of tests used samples with cement contents of 0 per cent, 5 per cent, and 7.5 per cent. A moisture content of 2 per cent above optimum was chosen for mix design. This was based upon the conclusions of McNulty (1985) which suggested that durability and resistance to immersion were marginally to significantly improved when the initial moisture content was raised above the optimum moisture content. Several days before final mixing, each batch was pre-treated with approximately 75 per cent of the required moisture content and stored in sealed plastic containers until required. By pretreating the minestone, the hygroscopic requirement of the shale was satisfied and the potential for post stabilization slaking limited. The absorption of liquid during the pre-treatment period effectively lowered the surface moisture content available at the time of the final mix and hence compaction was achieved nearer to the optimum moisture content than the true moisture content would suggest. A mix design batch sufficient to produce 4 x 100mm cube samples was chosen. The number of samples per mix was deliberately low enabling the compaction of all the samples to be achieved within 20 minutes of the mixing. Compaction of all four samples within a batch was normally achieved within 15 minutes of the end of mixing.

Originally it was intended to mix the cement and the minestone in a Winkworth twin blade pug-mill mixer, but the conditions that prevailed when using the Winkworth were thought to be too aggressive to control the physical properties of the minestone even though the twin blade pugmill mixer has been advocated in several other studies into the stabilization of minestone (Kettle, 1973; McNulty, 1985; Thomas, 1987). Even with an extended mixing time to ensure thorough mixing took place, crushing and degradation of the shale was found to be minimised using an open pan mixer rather than the twin blade pug-mill mixer. Though the mixing action of the Winkworth may resemble that of the pug-mill mixers used in full scale mix-in-plant operations (National Coal Board, 1983; McNulty, 1985), it was decided that the open pan mixer would be more appropriate for controlled laboratory work particularly in these studies.

Compaction was achieved using a vibrating hammer with a constant weight attached. The whole compaction apparatus weighed between 25 and 30kg. The samples were compacted in three layers, each layer being subjected to 1 minute of vibration, a period thought sufficient to compact a sample to refusal. To prevent the formation of compaction planes, each layer was scarified before the placement of the subsequent layer. The compacted samples were covered with a damp cloth and polythene and left to harden overnight. Following the removal of the moulds, the samples destined for the erosion/abrasion test were coated in wax to assist volume determination by water displacement. Each sample was wrapped in industrial cellophane and placed in a polythene bag. All the samples were stored in a curing room set at 25^{o}C and a high humidity until required for testing.

6.8.3 Compressive Strength Determination

The compressive strength test was carried out in accordance with Test 11 of BS 1924 (British Standards Institute, 1975b). For each curing period, four tests per minestone were performed and the mean compressive strength value calculated from the results.

6.8.4 Immersion Test

The immersion test evaluates the strength loss experienced by a sample following a period immersed in water compared to a similar sample cured in a 'dry' environment. The test is designed to reveal any adverse effects associated with deleterious compounds, such as sulphates and the expansive clay minerals, when stabilized materials experience contact with excess water (Rainbow and Sleeman, 1984). Both distilled and sea water were used as immersing liquids.

6.8.5. Dimensional Change

Samples used for the immersion test were also monitored for dimensional changes using 'Demec' spots and a length comparator with a dial gauge accurate to 0.002mm. *Figure 6.4* illustrates the apparatus used to monitor any dimensional changes following immersion. The 'Demec' spots were arranged on the samples in a similar pattern to that used by McNulty (1985), with one pair on the upper and lower surfaces and two pairs on the side surfaces away from any possible compaction planes.

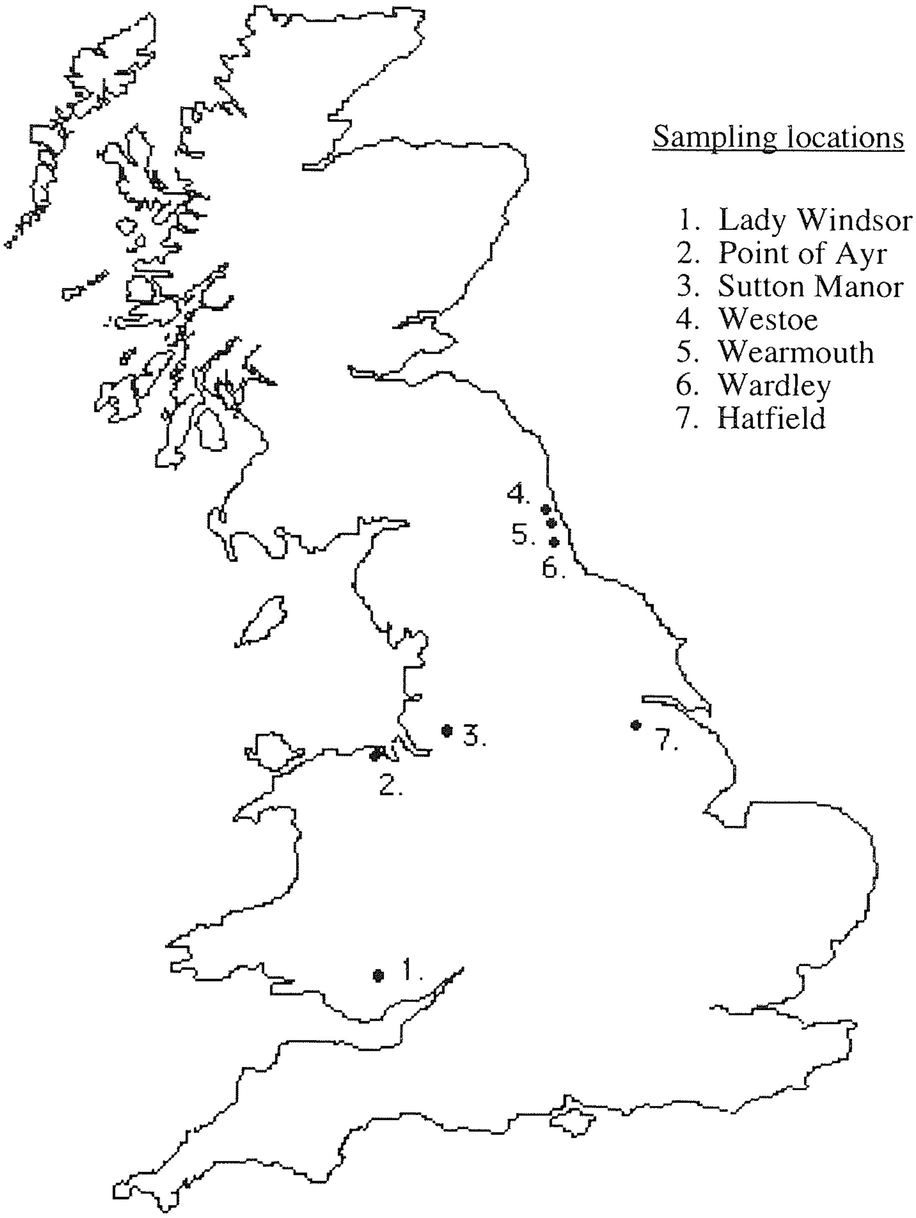


Figure 6.1 The location of the minestone sampling sites.

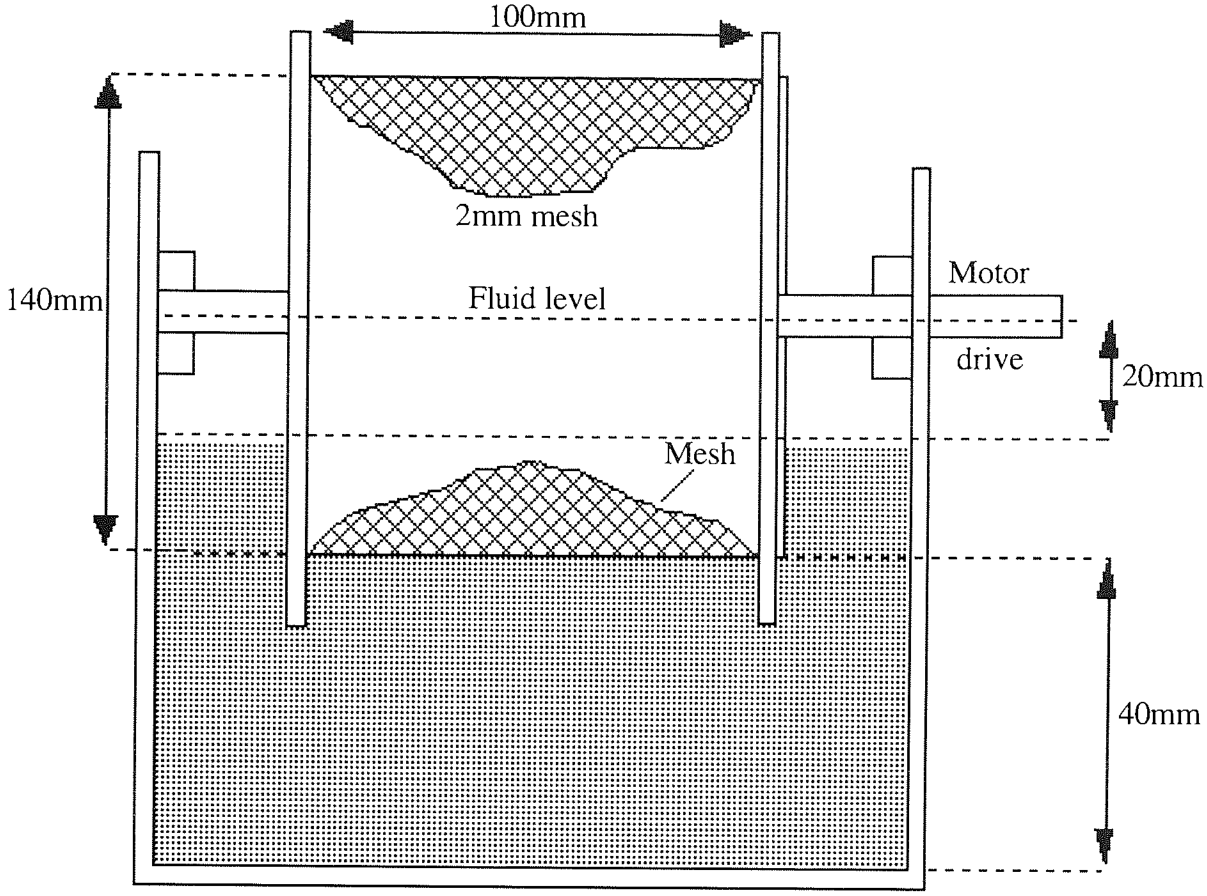


Figure 6.2 The slake durability apparatus used to determine the standard and modified slaking indices.

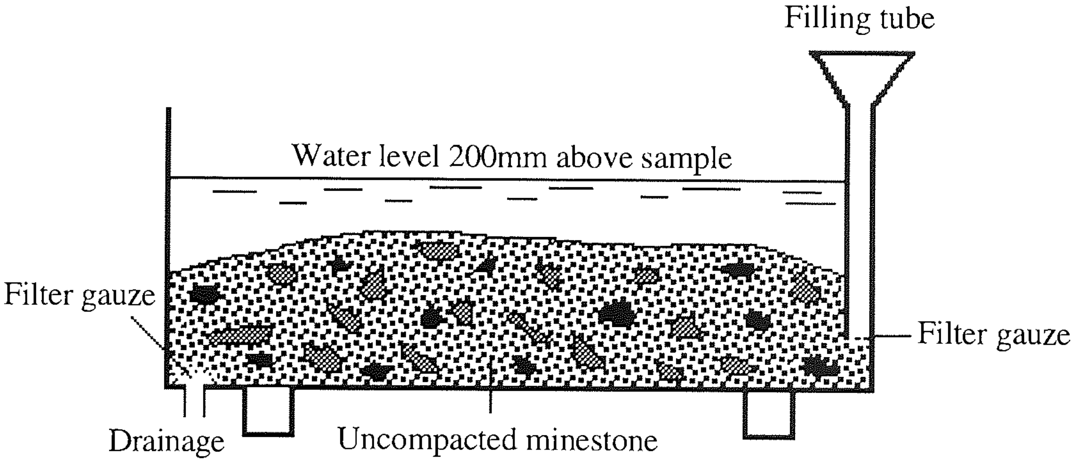


Figure 6.3 Apparatus used for tidal simulation testing.

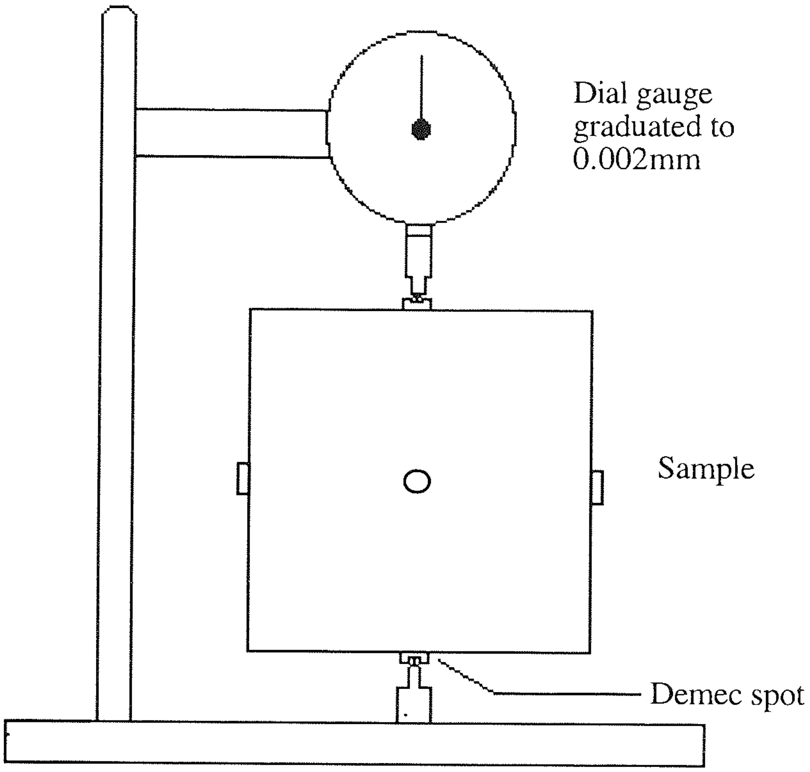


Figure 6.4 Apparatus used to determine length changes as a result of immersion.

7. EROSION TESTING APPARATUS AND TEST PROGRAMME

7.1 Introduction

Erosion testing has been carried out in one form or another for over 50 years. Initially, concern was centred on stream channel and irrigation ditch degradation. Both cohesive and non-cohesive soils were investigated, to determine the most resistant soil type using simple experimental systems. As understanding grew and technology progressed, more advanced test apparatus were developed. The discovery that unstable soils could be stabilized initiated a plethora of new research into the erosion resistance of such material and spurred the development of new test apparatus together with more advanced versions of existing equipment capable of assessing such stabilized soils. In recent years, with increasing concern about the effects of marine erosion, more complex apparatus have been designed to simulate the detrimental effects of wave action.

A preceding study (Hope, 1988) associated with the present research was undertaken to select appropriate test apparatus for assessing the performance of minestone as a hydraulic construction material. This included a thorough literature review outlining the currently available erosion tests and presented the advantages and disadvantages of the various systems. A summary of the review, and conclusions from Hope (1988) is presented in the following section.

7.2 Current Erosion Testing Apparatus

7.2.1 Flumes

The popularity of flumes for simulating hydraulic conditions stems from the representative channel shape of the flume apparatus. A number of variables have been investigated using flumes, including the velocity of stream flow at which serious erosion occurs or commences, the role of stabilization, the influence of different water velocities on erosion rates and the determination of critical bed shear stress of a material (Moore and Masch, 1962). Although flumes have the advantage of structural similarity with stream channels and are suitable for the introduction of a tractive bed load into the test, thus simulating the abrasion process, a number of disadvantages weigh heavily against flumes. The major drawback of the flume is that the critical shear stress cannot be accurately determined and other disadvantages include the space requirements, capital cost and the difficulty in simulating extreme events. Consequently flumes were considered unsuitable for this particular research investigation.

7.2.2 The Water Jet Erosion Test

Like flumes, water jet tests have had a long history of use for erosion testing, work having been being reported by Moore and Masch (1962), Nussbaum and Colley (1971) and Litton and Lohnes (1984). The general principle of this type of test apparatus is shown in *Figure 7.1*, with a vertical water jet impinging on a horizontal test surface.

Whereas the flume can be regarded as a simulative test, the water jet erosion test is considered to be a comparative test and is limited to assessing the relative durabilities of different materials. The water jet erosion test has the advantage of simplicity, ease of use and a good correlation to wave height. The disadvantages lie in the inability to provide deterministic results and the failure to incorporate an abrasive component.

7.2.3 Drill Hole Test

The drill hole test was developed to test the erosion resistance of intact, undisturbed clay samples (Rohan *et al.* 1986) and as a result of its recent origin, its applicability to the testing of stabilized materials has yet to be determined. A cylindrical specimen is placed into the specially designed steel tube before a 6.5mm diameter hole is drilled through its centre and a conical chamfer drilled at its proximal end (*see Figure 7.2*). *Figure 7.3* illustrates the basic concept of the apparatus which consists of a closed hydraulic system containing a reservoir at constant level. Flow passes from the reservoir downwards until it passes through the specimen. The eroding fluid together with the eroded particles pass into a sedimentation tank from where the eroded mass can be recovered. Close control on the circulating fluid is maintained by regular electro-conductivity tests and chemical analysis, while pressure loss due to the friction within the sample is measured by differential manometers. The discharge rate of the reservoir is controlled by an outflow valve and is measured using a rotameter. Erodability is assessed by the measurement of the dried mass of eroded particles expressed as a function of velocity or shear stress.

From a technological point of view, this test can be considered state of the art. Strict control can be kept on the hydraulic conditions and the chemistry of the eroding fluid. Facilities allow for the examination of the rate and mode of erosion and the critical shear stress can be determined accurately and with ease. Despite such advantages, it was felt that the lack of knowledge concerning its ability to cope with stabilized materials weighed heavily against its use for research into stabilized minestone.

7.2.4 Simulated Free Overfall Erosion Test

This erosion test was developed to simulate the velocities and forces experienced during a free overfall of water (Litton and Lohnes, 1982; 1984). *Figure 7.4* illustrates the basic design of the apparatus. Water is supplied at a constant rate from a reservoir to a holding tank. The curved walls of the holding tank funnel the water into the discharge slit, from where it impinges upon the sample beneath. Following set periods of time the sample is removed and weighed to determine the loss of material resulting from erosion. The degree of erodability can then be expressed as a function of flow rate and shear stress. This test was thought to have reasonable flexibility since it can simulate both impact and parallel flow erosion, further support for the test came from the good correlation established between the performance in this test and that in the freeze/thaw test. However, the major disadvantage of the apparatus is the inability to simulate the abrasive aspect of wave action.

7.2.5 Erosion/Abrasion Test

This test was specifically designed to simulate coastal erosion, incorporating both a water impingement element and an abrasive element employing impacting debris. The test apparatus consists of a regulated water jet and a perforated 'plexiglass' tube with a wire cage fitted comfortably inside the tube. During the test, granular abrasive material is placed within the wire cage and tube assembly. The operation of the device can be explained as follows: the impinging water jet hits both the specimen directly and a number of small stones contained within the wire cage. The stones, activated by the water, then impact against the sample causing abrasion. After impact, the stones fall to the base of the tube where they again encounter the continuous water jet stream and are easily reactivated. The water, after impacting upon the specimen, quickly drains through the perforations in the tube. The water jet produces erosional forces on the specimen and also removes the material loosened by the impacting debris. This apparatus and test was subsequently chosen for the current research programme and hence is considered in greater detail in *Section 7.5*. Overall a more accurate and realistic simulation of field conditions is offered by the use of this apparatus. Further advantages include its compactness and ease of operation. The disadvantages are its comparative rather than deterministic nature and the lack of research data due to its recent development.

7.2.6 Rotating Cylinder Test

Of all the erosion tests that were reviewed this was undoubtedly the most popular for testing cohesive and cement stabilized soils. A wealth of data concerning the apparatus, its operation, its performance and results is available. The test was designed using the rotating cylinder principal common to some types of viscometers. A cylindrical sample of material is mounted coaxially inside a larger transparent cylinder which can be rotated at high velocities. The annular space between the stationary sample and the outer cylinder is filled with water. When the outer cylinder is rotated it imparts a shearing action upon the water which in turn transfers a shear force to the sample. The degree of erosion is calculated (by measuring the weight loss of the samples) and plotted against the applied shear stress and the critical shear stress is also determined. As with the erosion/abrasion test, this test was subsequently chosen for the current research and is discussed in detail in *Section 7.4*. The test was thought to closely simulate stream channel flow and to allow the critical shear stress to be determined with a certain degree of accuracy. The simplicity and compactness of the apparatus makes it an ideal laboratory test. The chief misgiving concerning the test is that it does not include an abrasive aspect, however, as the test is specifically designed to determine critical shear stress, abrasive action would only render shear stress determination less accurate.

7.3 Choice Of Research Apparatus

Hope (1988) came to the conclusion that a combination of two particular apparatus would provide the most comprehensive test programme for research into the hydraulic properties of stabilized and unstabilized minestone. The rotating cylinder test was chosen for its ability to accurately determine shear stress, essential information for designing stable channels. The decision to include the erosion/abrasion test in the research programme was based upon the ability of the test to simulate the abrasive action of suspended matter associated with a wave dominated regime. British Coal concurred with the conclusions reached by Hope and consequently the apparatus were duly fabricated by the Civil Engineering Department at Aston University. *Section 7.4* and *7.5* give a detailed account of the rotating cylinder test and the erosion/abrasion test respectively.

7.4 The Rotating Cylinder Test

7.4.1 Test Rationale and Historical Development

The Rotating Cylinder test has been a popular erosion test since its conception by Moore and Masch (1962). The foundation of this popularity was based primarily on the ability of the apparatus to accurately record the value of the critical shear stress at which pronounced erosion commenced, thereby allowing determination of the relationship between sample properties and the critical shear stress.

Moore and Masch's original design (1962) and subsequent variants were principally based upon existing coaxial viscometers. A stationary, cylindrical sample is mounted coaxially within a transparent outer cylinder which is able to rotate at high velocities (up to 3000rpm) under the power of an electric or air powered motor. The annular space between the sample and the outer cylinder is filled with a fluid which, when rotated, imparts a shear stress upon the surface of the sample. As the annular space is constant and the surface of the sample is prepared smooth, the stress experienced by the sample is considered to be uniform over the whole surface area. Axial torque is used to calculate the applied shear stress exerted upon the surface of the sample. In order to determine the critical shear stress, the applied shear is plotted against the erosion that the sample experiences during the test.

Originally the device was designed to investigate the erodability of cohesive soils (Moore and Masch, 1962; Espey, 1963), but by the early seventies, it was being incorporated into studies investigating the relationship between clay chemistry and erosion (Arulanandan *et al.* 1973; 1975). Akky and Shen (1973) broadened the repertoire of the device by testing the resistance of cement stabilized soils to erosion (Akky and Shen, 1973; Akky, 1974). During the early and mid eighties a vastly improved design was employed by Chapuis to study undisturbed, natural clay soils (Chapuis and Gatien, 1986; Chapuis, 1986). Van Wijk (1985) employed the rotational device to investigate the erosional forces associated with rigid pavement pumping, while Fan (1988) used a similar device for research into the prediction of the erosion of cut and fill slopes.

Although the basic principle of the rotating cylinder has remained the same since its conception, continual technical modifications to the design have helped to improve reliability and the accuracy of the data gained from its use. The apparatus used in the present study is based on the improved device of Chapuis (1986) whilst including modifications suggested by Van Wijk (1985) and Fan (1988), although this design still reflects the concepts of Moore and Masch's original model.

7.4.2 The Original Design

Figure 7.5 illustrates the original rotational device used by Moore and Masch (1962) and Espey (1963). The cylindrical sample was mounted coaxially within a slightly larger perspex cylinder. Passing centrally through the sample was a torsion tube which has a yoke, brass rod and torque indicator attached to the distal end. The annular space between the sample and the outer cylinder was filled with eroding fluid (usually water, though water with glycerin has also been used). When the outer cylinder was rotated, the water imparted a shear stress upon the surface of the freely mounted sample. This shear stress was transmitted through the torsion tube to the calibrated torque indicator and, by determining the relationship between rotational speed and the torque, the desired shear stress could be obtained by adjusting the applied rotational speed.

7.4.2.1 Original Calibration Method

Direct real time determination of the shear stress that acted on a sample during testing was not possible using the original device as designed by Moore and Masch (1962). The shear stress was determined by producing a calibration chart prior to full scale testing. Calibration of the original model was accomplished by applying a series of known weights at a tangent to the periphery of a standard sample by means of a pulley system (*see Figure 7.6*) and noting the rotational speed required to initiate movement of the torque indicator. The relationship between rotational velocity and the applied shear stress was calculated by dividing the moment produced by each known load by the exposed surface area of the sample multiplied by the sample radius. Once a calibration chart had been produced, the required shear stress could supposedly be obtained by using the corresponding rotational velocity that was indicated upon the calibration chart (*see Figure 7.7*). Moore and Masch relied upon Schlichting's equation (1979) which incorrectly ignored the relationship between surface roughness and the shear stress. This reliance led to the assumption that a constant relationship exists between rotational velocity, torque and shear stress. This relationship does not exist, for torque and shear stress may vary whilst set at a fixed speed, depending upon the surface roughness of the sample. Normally the torque required to maintain the sample stationary will increase when particles are detached from the surface and will decrease as soon as the eroded zone has once more become smooth.

7.4.2.2 Original Test Procedure

Prior to testing, the soil sample was immersed in the eroding fluid to allow equilibrium to occur and saturation to be attained. Subsequently, each sample was weighed and placed in the rotational device to be subjected to a given shear stress for a preset period of time (1 minute). The sample was then removed, wiped, weighed and remounted in the device in readiness for further testing. The same eroding fluid was used for several successive tests. The measurement of eroded mass using Moore and Masch's technique is awkward, cumbersome and fraught with unscientific assumptions which render the results unreliable. Immediately before each test stage, water uptake/expulsion measurements were taken by immersing the sample in eroding fluid for a few minutes before it was re-weighed and placed into the rotating cylinder. At the completion of the preset test period, the sample was wiped and weighed. The difference between the initial weight and the post test weight, corrected for water absorbtion or expulsion, was considered to be the eroded mass. This procedure was repeated for each substage of the test, in which time the repeated handling of the sample could have led to significant re-moulding of the surface and could have led to accidental dislodgement of material. The wiping of the sample prior to weighing could only be regarded as subjective, rendering the recorded weights as approximations. At low levels of erosion, these approximations could lead to severe inaccuracies in the test results. Chapuis and Gatien (1986) stated that for shear stress values lower than critical, negative erosion rates were frequently calculated in the range 0 to -30g/(m^2 min) and down to values of -60g/(m^2 min).

7.4.3 The Monterval Rotational Device (Chapuis and Gatien 1986)

One of the major limitations of Moore and Masch's original device was its inability to accept undisturbed soil samples. This led Chapuis (1986; Chapuis and Gatien, 1986) to carry out extensive modifications to the original design allowing it to accept such samples and, as a consequence, vastly improving the accuracy, ability and technique of the test. *Figure 7.8* illustrates the rotational device designed by Chapuis in which the soil sample is mounted between an upper brass plate and a lower metallic cylinder of the same diameter. The sample was allowed to rotate freely, being independent of the external cylinder, and guided by ball bearings. Consequently, no central shaft was required to pass through the sample (as the torque shaft is attached directly to the upper brass plate) thus allowing undisturbed samples to be tested. The lower metallic cylinder and brass plate together with the bearing system also help to lower the inherent friction of the apparatus.

7.4.3.1 Calibration and Torque Measurement

To measure the torque experienced by the sample, a pulley and weight system was used with masses ranging from 1 to 40g. It had previously been assumed that a constant relationship was maintained between rotational speed and the torque experienced by the sample, but Arulanandan *et al.* (1975) observed that surface roughness did not remain constant during testing and, as surface roughness affects the torque, a constant relationship between cylinder velocity and torque would not exist. With the pulley and weight system calibration curves are no longer required as the torque is directly measured during the test.

7.4.3.2 Test Procedure

Intact samples were kept at as close to the original moisture content as sampled until testing began. The samples were mounted in the device and the annular space filled with distilled water. The outer cylinder was rotated at a preselected velocity for 10 minutes and the shear stress exerted upon the sample by the fluid was measured, using the pulley system, at three minutes and ten minutes. The annular space was then drained and the eroded particles collected, dried and weighed. The chamber was refilled with fresh eroding fluid and the test repeated at a higher velocity. Critical shear stress was deemed to have been reached once there was a dramatic increase in the rate of applied shear stress or when the eroding fluid showed marked clouding. The inherent friction of the apparatus was determined by plotting torque against rotational velocity. By subtracting this value from the measured shear stress, the true applied stress values were obtained.

7.4.4 Advantages of Chapuis' Device over the Original Design

Chapuis' later design has several major improvements and advantages over the original apparatus designed by Moore and Masch

1. Although not relevant to the present research project, Chapuis' device is capable of accepting undisturbed natural soils, whereas the original design was restricted to using laboratory prepared specimens.

2. By discarding the need for a central shaft passing through the sample and mounting it on bearing-guided base and head cylinders, Chapuis has improved upon the rotational guidance, and alignment of the sample, with a consequent decrease in the inherent friction of the apparatus.

3. The use of the bearing-mounted head and base plates and cylinders for sample mounting has improved the annular flow regime by dampening down the edge and end effects which can occur around the extremities of the sample in Moore and Masch's device.

4. The torque measurement system of the two experimental rigs differs fundamentally.The Chapuis system takes into account the findings of Arulanandan *et al.* (1975) who pointed out that surface roughness changes continually as erosion occurs. This subsequently leads to changes in the shear stress acting upon the surface and thus to the amount of torque experienced and measured. Moore and Masch (1962) did not have the benefit of the research carried out by Arulanandan *et al.* (1975) and hence the calibration chart system used to determine shear stress can be considered flawed. Using Chapuis's torque measurement system, accurate instantaneous torque measurements can be obtained, Moore and Masch's system is incapable of providing instantaneous readings as it is based upon a fixed and flawed calibration chart system.

5. Moore and Masch calculated the eroded mass by weighing the sample before and after testing. This required constant manipulation of the sample which made the measurements prone to inaccuracies. These inaccuracies may have been compounded by the arbitrary way in which the samples were wiped prior to weighing. The sample need never be removed during a complete multistage test using the system developed by Chapuis. After each substage the eroding fluid is drained off and the chamber cleaned using an aspiration system. All eroded particles are collected, oven dried and weighed, thus giving the actual amount eroded during the test.

6. Non-manipulation of the sample during multistage testing is also vital for accurate determination of the applied shear stress. The total shear stress measured during testing is the sum of the variable torque values experienced by the sample added to the inherent friction value of the apparatus. If the sample is undisturbed during a complete multistage test, the inherent friction can be considered constant and therefore may be ignored. The inherent friction of the apparatus can be deduced by extrapolating the shear stress versus rotational velocity curve back to the ordinate axis (*see Figure 7.7*). Chapuis' design does not require sample manipulation, so the inherent friction remains constant and can be ignored. With Moore and Masch's design manipulation is frequent resulting in variable inherent friction and consequently inaccurate shear stress values.

7. The original test regime required the same eroding fluid to be used for a complete multistage test. Chapuis' apparatus allows the eroding fluid to be changed after each sub-stage in a multistage test, so that a tight control can be kept on the ionic and sediment concentrations which are both considered to affect the rate of erosion.

7.4.5 Hydrodynamic Aspects of the Rotating Cylinder Device

The behaviour of the water within the annulus of the rotational device was likened to the behaviour of water in a viscometer (Moore and Masch, 1962). Masch *et al.* (1963) assumed that the flow in the annular space between two concentric cylinders, with one rotating at steady speed, would be parallel (only the tangential component to velocity not being equal to zero) and thus could be analyzed using the Navier-Stokes equations. The moment transmitted by an outer rotating cylinder to an annular fluid can be expressed as (Schlichting, 1979) :

$$M = 4\pi\mu h \frac{r^2R^2}{R^2 - r^2}\omega \qquad \textit{(Equation 7.1)}$$

where
M = moment
τ = shear stress
T = Torque
μ = dynamic viscosity.
r = static inner radius
R = rotating outer radius
ω = angular velocity of rotation
h = height of cylinder

The shear stress (τ) upon the sample surface is given as :

$$\tau = 2\mu \frac{R^2}{R^2-r^2}\omega \qquad \textit{(Equation 7.2)}$$

and the torque (T) transmitted from the annular fluid to the the sample can be expressed as :

$$T = 2\pi r^2 h\tau \qquad \textit{(Equation 7.3)}$$

The applicability of *Equations 7.2* and *7.3* is dependant upon the annular flow between the static inner cylinder and rotating outer cylinder being of stable, Couette type. Moore and Masch generally thought that inertial forces helped maintain stable Couette flow.

The point at which stable flow becomes unstable (turbulent) is governed by the critical Reynolds number (CRN) which can be related to a critical rotational velocity. The CRN is directly affected by the eccentricity of the coaxially mounted cylinders, the sample roughness and the width of the annular space. As a result of the number of variables governing the CRN, a wide span of critical rotational speeds have been experienced. Chapuis (1986) reported critical speeds of the order of 200rpm, Moore and Masch (1962) experienced a critical velocity of 680rpm as did Van Wijk (1985) whilst Akky (1974) reported a critical velocity of 800rpm. Masch *et al.* (1963) believed that for most cohesive soils the rotating cylinder operated normally in the transition zone between Couette flow and turbulent flow and suggested that full, turbulent conditions probably failed to span the full width of the annulus.

7.4.6 Apparatus Used in the Present Research

Following Hope's (1988) appraisal of the erosion apparatus, a rotating cylinder device was built based upon Moore and Masch's original design. In the light of the Monterval publications, a new device was fabricated using the same innovations that Chapuis introduced into the design. Following test trials to determine the feasibility of testing minestone within the rotational device, further modifications were made, similar to those proposed by Van Wijk (1985) and Fan (1988). The basic design of apparatus is illustrated in *Figure 7.8* and shown in *Plate 7.1*.

The outer cylinder is driven by a permanent magnet electric motor (370 watts) and has a maximum velocity of 2500rpm. The rotational speed was measured using a tachogenerator coupled to a multimeter. The tachogenerator generates a voltage proportional to the rate of revolution. The electrical analogue signal produced is converted by the multimeter into the corresponding revolutions per minute. An important consideration in the design of the device was the width of the annular space. The smaller

the annular space, the larger the shear stress exerted upon the sample for a given rotational velocity. It was decided that an annular space of 9mm would give a sufficient range of shear stress values for the output capacity (2500rpm) of the electric motor. The size of the annular space also has control upon the maximum grain size of the sample material. Any eroded particles must be small enough to move without hindrance within the annular space. The maximum grain size used for the present research was 6.3mm, small enough to allow free movement of the eroded material at all times and to permit flushing out following each substage of the test.

Three major modifications to Chapuis design were proposed by Fan (1988) and these were subsequently incorporated into the apparatus used in the present study. Chapuis' original torque measurement system, using pulley and weights, was cumbersome and time consuming and open to experimental error as a result of vibration in the apparatus. To avoid such problems, a digital force gauge was used to measure the torque. This is a quick, accurate and reliable way of measuring the torque value during testing. Its ability to measure torque continuously means that the average value obtained has a far greater accuracy than the limited number of measurements provided by Chapuis' device. The experimental errors arising from apparatus vibration are also kept to a minimum because of the high number of measurements involved in obtaining the mean torque value.

The torsion shaft through which torque is conveyed to the digital force gauge is a hollow tube in Chapuis' design and is attached to the upper brass plate by a small screw. This tube was sensitive to the over tightening of the screw and could be deformed easily, altering the friction value of the apparatus for individual tests. By substituting a solid stainless steel rod for a hollow torsion shaft, Fan's design avoids the possibility of shaft deformation and the associated friction variations. In this apparatus the guidance shaft was connected to the upper brass plate by way of a tightly fitting hexagonal socket (*see Figure 7.9*).

Originally the interface between the guidance shaft and the device had no bearings separating the two and relied upon lubricating grease to reduce friction between them. Any error in the alignment of the sample, the head cylinder and the shaft would result in significant friction errors which were further exacerbated when the sample was under test. To aid alignment, bearings were inserted at the interface between the shaft and the device,

the bearings also help to reduce friction to a minimum. Whereas two sets of bearings were used in Fan's design, only one set was found to be necessary for the present research apparatus.

7.4.7 Test Programme

A feasibility programme was initially carried out to determine whether minestone would be a suitable material for testing within the rotating cylinder apparatus. This programme involved testing both unstabilized and stabilized samples. The stabilizing agent used was ordinary portland cement. Five sets of samples were tested:

1. Unstabilized minestone.
2. Minestone plus 10 per cent cement.
3. Minestone plus 5 per cent cement.
4. Minestone plus 3.5 per cent cement.
5. Minestone plus 1 per cent cement.

The choice of materials for the full test programme depended upon the results of this feasibility study.

7.4.8 Initial Preparation of Test Material

As previously mentioned, the specifications for the test apparatus place limitations upon maximum grain size. The drainage channels and aperture have diameters of 10mm and to allow unhindered drainage, the maximum grain size should not exceed 6.3mm (i.e. all particles should pass the 6.3mm sieve). Furthermore, the shear stress exerted upon the sample can only be uniform if the surface smoothness of the sample is also uniform. If large gravel size particles are present, the surface smoothness would not be maintained and severe turbulence could result within the annular fluid, possibly leading to cavitation and major damage to the sample surface. If this occurs erosion could not be attributed to shear stress alone, cancelling the device's ability to determine the critical shear stress of a sample. Therefore, an air dried sub-sample was initially crushed within a motorised donkey mill, to pass the 6.3mm sieve. Care was taken to prevent excess crushing and powdering of the test material.

7.4.9 Preparation of Samples

7.4.9.1 Unstabilized Samples

The material to be tested was weighed out so that a known sample batch quantity could be produced. The material was mixed with the eroding fluid (either distilled or sea water) at 1 per cent above the optimum moisture content using a Hobart commercial mixer. The mixing period of 10 minutes allowed thorough wetting and moisture adsorption to occur. The resulting mixture was placed in polythene bags, sealed in plastic containers and allowed to equilibriate for two days before being compacted as below.

7.4.9.2 Stabilized Samples

A given percentage by weight of cement was added to the sub 6.3mm minestone and mixed dry for 10 minutes, with the cement being evenly sprinkled and uniformly distributed throughout the minestone. At the lower end of the cement content range, dry mixing was continued for a further 5 minutes to ensure even distribution of cement. The mixture was then mixed wet for a further 5 minutes, at a moisture content of 2 per cent above the optimum and compacted immediately.

For convenience, the samples were compacted using the same procedures and equipment used in the standard soil compaction tests (Test 13. BS 1377:1975). A 4.5kg ram and a 1000cm^3 volume mould were used to make the 105mm diameter and 116.5mm high samples. Originally a vibrating hammer was used to compact the specimens, but the frequent development of prominent compaction planes rendered them susceptible to concentrated erosion along these zones of weakness. These compaction planes formed despite deep and extensive scarifying of the surface of each compacted layer.

Once compacted, three plugs were pushed into the top surface of each sample and then removed producing three sockets, so that the upper brass plate could be firmly attached to the sample when required (*see Figure 7.9*). Moulds, containing unstabilized samples, were immersed in the appropriate eroding fluid for two days to allow full saturation to be attained. Porous plates were placed at either end of the sample to allow the eroding fluid to enter freely. The cement stabilized samples were extruded immediately and carefully wrapped in polythene and left to cure for five days, before being immersed in eroding

fluid and allowed to equilibriate for two days. Once the curing and saturation period was over the samples were removed and prepared for testing.

7.4.10 Test Procedure

The following section of test procedures provides a detailed account of how the samples are mounted within the device, the installation of the device and torque measurement system, running procedures during test and the procedures for eroded mass recovery.

1. Following the removal of the sample from the curing bath, the upper brass plate, with its three pins was firmly attached to the upper surface (*see Figure 7.9*). This sample was then placed flush upon the lower brass plate. Both upper and lower brass plates were of the same diameter as the sample (105mm).

2. Waterproof adhesive tape was used at the interface between the sample and the two brass plates to protect the sample extremities from edge effect erosion resulting from shear stress concentration.

3. A stainless steel ball bearing measuring 6.35mm (1/4") in diameter was mounted upon the central hub of the lower section of the rotational device.

4. The bottom metallic cylinder was placed upon the ball bearing and a second ball bearing of the same diameter placed at the centre of its upper surface. The bottom metallic cylinder has the same diameter as the lower brass plate and the sample.

5. The sample with attached brass plates was mounted centrally upon the stainless steel ball bearing on the upper surface of the metallic cylinder.

6. The stainless steel torsion shaft with an angled end piece was fitted into the upper brass plate (*see Figure 7.9*). The tight fit and the angled nature of the socket prevented any movement of the shaft once it was fitted to the upper brass plate and sample.

7. A perspex cylinder, with an outer diameter of 140mm and an inner diameter of 134mm was placed around the sample, slotting into a bottom groove which had the appropriate sized rubber 'O' ring fitted.

8. Once the perspex cylinder was seated upon the lower section of the rotational device, the upper section of the device was lowered so that the perspex cylinder inserted itself into a groove upon its under surface. An appropriately sized rubber 'O' ring ensured that a water tight seal was maintained.

9. Once a tight and even fit had been achieved between the upper section of the device and the perspex cylinder, the upper section was fastened securely to the lower section of the device using four stainless steel rods. Care was taken throughout to insure that a good seal is maintained and that the perspex cylinder remains perfectly square with the rest of the rotational device.

10. The drainage holes were securely sealed with PTFA threaded nuts before the annular space was filled with eroding fluid to a level 25mm above the top surface of the upper brass plate. The filling apertures were then closed using knurled brass bolts.

11. A force arm was firmly attached to the top of the torsion shaft and centred to exert a force upon the sensor arm of the force gauge mounted upon the frame (*see Figure 7.8*).

12. Prior to testing, a heavy duty perspex safety shield was placed around the rotational device and clipped firmly into place. The test can now be started, the force gauge was switched on and reset to zero. The multimeter, programmed to show rotational speed, is switched on and the speed control for the motor turned to the required speed.

13. Each substage of the test consisted of 5 minutes of rotation with the maximum shear stress being read at 1 minute intervals from the force gauge. On average seven substages per sample were carried out to determine the critical shear stress.

14. Although Fan (1988) and Chapuis (1986) found that the clouding of the eroding fluid occurred when the critical shear stress had been reached, when testing minestone, the eroding fluid clouded up well before the critical shear stress had been reached.

15. Following each substage the annular cell was drained and flushed out, making sure that all the eroded particles were removed. The eroded particles were filtered out of the eroding fluid using a funnel and filter paper, before being oven dried and weighed to determine the eroded mass.

16. Following a full multistage test the apparatus was disassembled and the moisture content of the sample determined.

7.5 The Erosion/Abrasion Test

This test was developed as part of a research programme into the effectiveness of soil cement as a shoreline protection material for artificial islands in the Beaufort Sea, Canada (Oswell and Joshi, 1986). It combines a water jet action with the abrading action of particulate matter. The basic design concept is illustrated in *Figure 7.10*. It consists of a regulated water jet connected to a perforated 'plexiglass' tube inside which a wire cage is closely fitted. Granular material is placed loosely within the wire cage and 'plexiglass' tube assembly, which is rigidly fixed flush against the cube sample. The whole apparatus is inclined at 30^{o} to the horizontal, with the sample situated at the top of the incline (*see Plate 7.2*).

7.5.1 Test Rationale

The rationale of the test is that an impinging water jet hits both the sample directly and the granular material which is retained within the 'plexiglass' tube. The granular material, once activated by the water jet, acts as abrasive projectiles that abrade the surface of the sample. Retention of the projectiles allows for continuous abrasion, with the product of this process being removed by the water jet. The water is allowed to drain away rapidly through perforations in the 'plexiglass' tube. The 30^{o} inclination of the apparatus allows the abrasive projectiles to fall back into the jet stream to be reactivated (Oswell and Joshi, 1986).

The type, amount and rate of sample erosion is a function of several variables. Composition, grading, compressive strength, water jet pressure and the nature of the impacting material all contribute to the resultant erosion. By increasing the water jet pressure more energy is conveyed to the abrading material which in turn increases the energy of impact between the projectile and the sample surface. The number, mass, composition and shape of the abrading particles all affect the rate and the type of erosion.

If the abrading stones are weaker than the sample material, the stones would be expected to suffer a far greater degree of erosion than the sample. Angular shaped stones would be expected to cause a greater level of erosion than would smooth rounded stones.

The number of stones used as abrading material is important, with an optimum number existing, too many and interference will take place between the individual particles resulting in a lower erosion rate and if too few are used a lower rate of erosion will be the outcome, making it necessary for an increase in the test period so that a measurable amount of erosion can be achieved.

7.5.2 Hydrodynamic Aspects of the Erosion/Abrasion Test

Erosion is partly controlled by the water impact pressures. It is possible to relate the impact pressures generated during testing to pressures generated by breaking waves in natural conditions. The impact pressures of a breaking deep water wave can be calculated using a semi-empirical formula originally developed by Gaillard (Bascom, 1980) :

$$P = 1.31(C + V_m)^2 \rho / 2g \qquad \text{(\textit{Equation 7.4})}$$

Where

P = Total exerted pressure.
C = Celerity of the wave.
V_m = Maximum orbital velocity at the wave crest.
ρ = Mass density of water.
g = Acceleration due to gravity.

The celerity of a wave is its velocity in shallow water and the maximum orbital wave velocity can be defined as :

$$V_m = \frac{\pi H}{T} \qquad \text{(\textit{Equation 7.5})}$$

Where

H = Wave height.
T = Wave period.

The impact pressures chosen by Oswell and Joshi (1986) (namely 70, 140, 210 and 280kPa) in their original test programme simulated impact pressures generated by breaking waves of between 1.5m and 6.5m in height (5ft and 21ft) with varying wave periods of between 5 and 10 seconds. The same series of test pressures was selected for the present research, initially for comparative purposes but also because the test pressures equate to the impact pressures expected in areas where minestone could be considered for coastal works (*see Section 10.1*).

7.5.3 Original Test Procedure

Soil-cement specimens were prepared in prismatic moulds measuring 76.2mm x 76.2mm x 100mm (3 x 3 x 4 inches). A masonry sand with a D_{50} of 0.5mm, artificial sea water and cement or cement/fly ash combined was mixed to produce the soil-cement samples. The moulds were filled in three layers, each layer was rodded before the subsequent layer was poured into the mould, the mould and enclosed sample was then dropped several times from a small height (10 to 20 mm) to facilitate proper consolidation of the mix. The samples were then cured for 1, 3 and 7 days to obtain various degrees of compressive strength.

Before each test, the sample volume was measured by immersing it in water and the density calculated. The sample was inserted into the support frame and tightly secured. The 'plexiglass' tube and cage, which held five rounded stones weighing 4g in total, was fitted flush against the face of the sample and fixed in place by a support clamp. The whole apparatus was inclined at 30^o to the horizontal. The water jet was started and set, using the regulator, to the predetermined test pressure. After a test period ranging from 15 to 105 minutes the specimens were removed from the apparatus, surface dried and weighed. The same procedure was then repeated using a new sample face and new abrading stones. The eroded mass was calculated by subtracting the post-test weight from the pre-test sample weight. The erosion was calculated as the average volume loss (weight loss divided by initial density) per unit time over a number of repeated tests. The test pressures used in the original test programme were 70, 140, 210 and 280kPa (10, 20, 30 and 40psi).

7.5.4 Test Modifications for the Present Research Programme

Several modifications were carried out to the original apparatus to improve the reliability and accuracy of the test. The wire cage which was used to hold the abrasive granular material was omitted as it was found to interfere with the drainage during the test.

The greater the amount of residual water within the 'plexiglass' tube during the test, the more likelihood there is for energy absorption by the projectile and a decrease in the impact force of that projectile. The wire cage could be omitted as the 'plexiglass' tube was sufficiently resilient to withstand the impact of the abrading stones and was sufficiently pervious to allow adequate drainage but, at the same time, able to retain the abrading material. Although the original design specified a 4.5mm internal diameter nozzle to produce the water jet, it was found that a 2.5mm nozzle was required to produce the necessary test pressures for the present research programme. By using a smaller diameter nozzle a constant test pressure could be maintained during the test, with fewer adjustments being needed and hence less monitoring of the apparatus whilst testing was under way.

A major difference between the original test and the test used in the current research is the way eroded mass was determined. In the original test, the eroded mass was obtained by weighing the sample prior to testing and weighing it after the test and subtracting the two values. Where large quantities of material are being eroded this may be adequate for determining the eroded mass, however where small quantities are concerned, this method is open to error. During testing the sample may absorb water or may even, through evaporation, lose water. These inconsistencies arising from changes in water content can lead to discrepancies in the eroded mass results. Further scope for error lies in the way the samples were surfaced dried before weighing at the close of the test. The use of an air hose to surface dry the sample can only be regarded as a subjective practice and could result in errors in the calculation of the eroded mass. To prevent these errors from influencing the weight loss measurement the apparatus used in the present research had the capacity to collect the eroded material by means of a sieve basket attached to the tray beneath the sample and water jet assemblage (*see Figure 7.11*).

The size of the sieve mesh used for the basket was 63μm, small enough to capture the bulk of any unstabilized minestone and virtually all the eroded material emanating from a cement stabilized sample. If necessary a 'fines factor' could be introduced when calculating the eroded mass to take into account the percentage of minestone passing the 63μm sieve. Any particles eroded from the abrading stones and captured in the sieve basket can be taken into account by weighing the oven dried projectiles before and after testing and subtracting the resultant mass from the overall total collected in the basket.

Splash screens surrounding the drainage slot in the tray help to funnel the eroding fluid together with the eroded particles into the detachable collection basket (*see Figure 7.12*).

7.5.5 Preparation of the Samples

The samples were prepared using the same procedures as those used for the unconfined compressive strength test (see *Section 6.8*). Once compacted and removed from the mould, the samples were coated in paraffin wax, wrapped in polythene and left to cure for seven days. The unstabilized samples were allowed to dry out for four days following demoulding before being coated in wax. The wax was used to facilitate the handling of the unstabilized samples and also to prevent the absorption of water whilst the volume of the sample was being ascertained. The high sensitivity of the unstabilized samples to immersion, warranted waxing before obtaining accurate volume measurement by water displacement.

7.5.6 Test Procedure

Prior to testing, each sample was weighed and the volume determined by the water displacement method (BS 1377: 1975 Test 15f), so that the bulk density of the samples could be established. Five equidimensional stones were carefully selected with diameters between 5mm and 8mm, their total weight being approximately 4g. The same type of abrading material was used for every test in the research programme. Limestone chippings of standard aggregate grade were initially chosen out of convenience, but as they proved to be highly resilient, they were subsequently chosen as the abrading material for the entire research programme. Most granular material encountered along coastlines shows a high degree of roundness. It was therefore decided that only stones showing a high degree of roundness and an equidimensional nature would be chosen for testing. Separate tests were carried out to determine how rapidly the stones would become smooth and rounded under test conditions. The selected stones were placed in the 'plexiglass' tube which was fitted flush against the sample and held in place by a clamp. The water jet was started and the required test pressure was obtained by adjustment of the pressure regulator.

After being exposed to the water jet for a predetermined time (15, 30, 60 or 90 minutes) the sample was removed and the depth of the cavity determined using a vernier gauge. The abrading stones were dried using an hair drier and weighed. Unlike Oswell and Joshi

(1986), who used five new stones and a fresh surface for each test period, it was decided that the same cavity and the same stones would be used for the complete set of consecutive test periods i.e 15, 30, 60 and 90 minutes, so that a complete set of readings

related to a single cavity. By using a new face for each test period the Oswell and Joshi test procedure (1986) may lead to highly variable results, with erosion rates for 15 minutes occasionally being greater than those for 30 minutes exposure. The heterogeneous nature of the minestone samples, compared to the homogeneous nature of the sand-cement mixes tested by Oswell and Joshi (1986), may account for such inconsistencies. By using the same cavity for each test period and accumulating weight loss and erosion rate readings, such variability was reduced.

At the end of the test, the apparatus was carefully hosed down to ensure that all the eroded particles were washed into the sieve basket. The sieve basket was removed and the eroded material transferred to a drying dish to be oven dried and weighed. For each impact pressure three tests were preformed, each on a fresh surface to determine the average erosion rate.

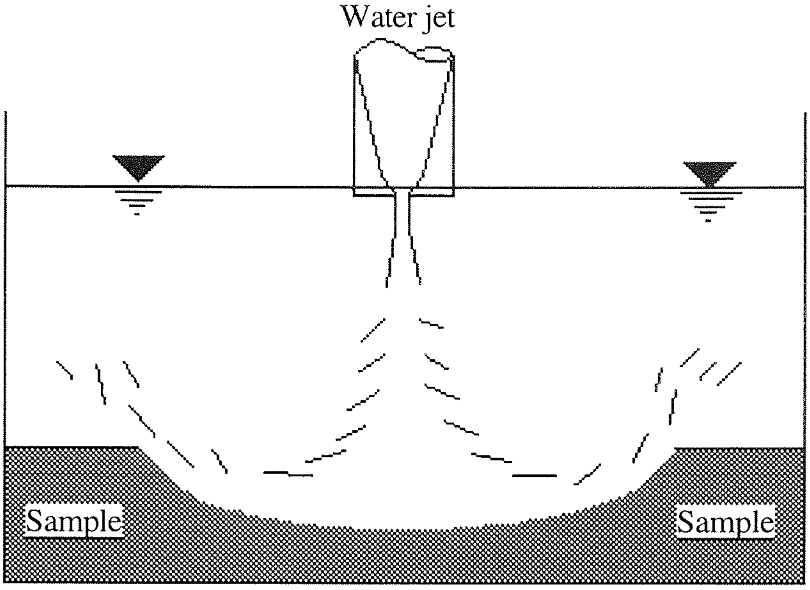


Figure 7.1 Arrangement for the vertical jet erosion test showing the scour effect upon a cohesive sample.

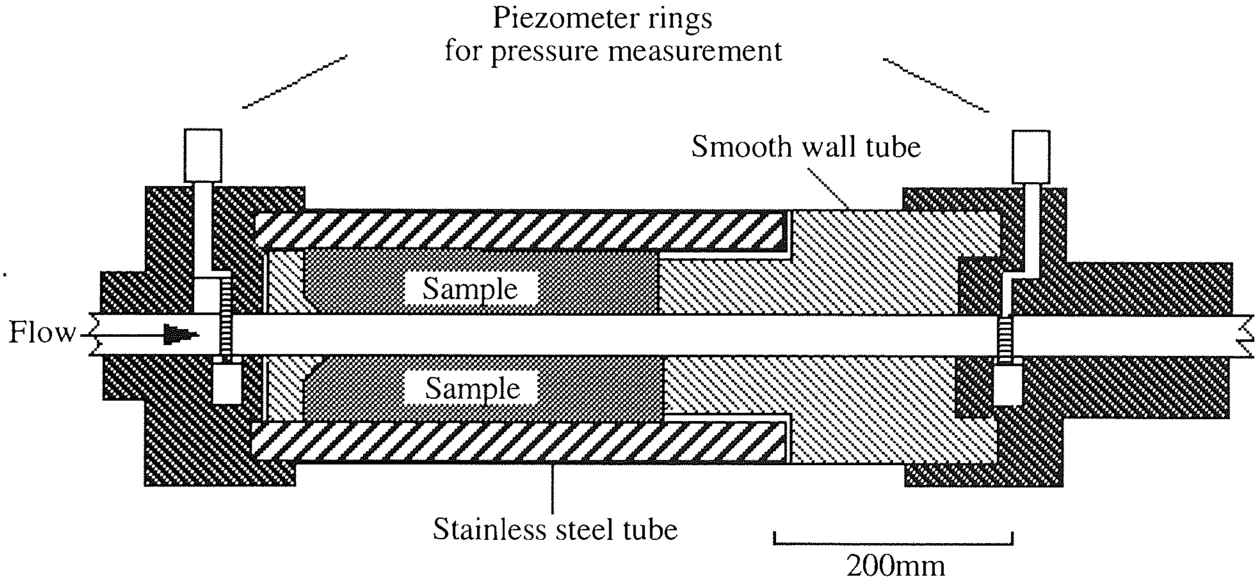


Figure 7.2 Sample mounting equipment for insertion in the drill hole test.

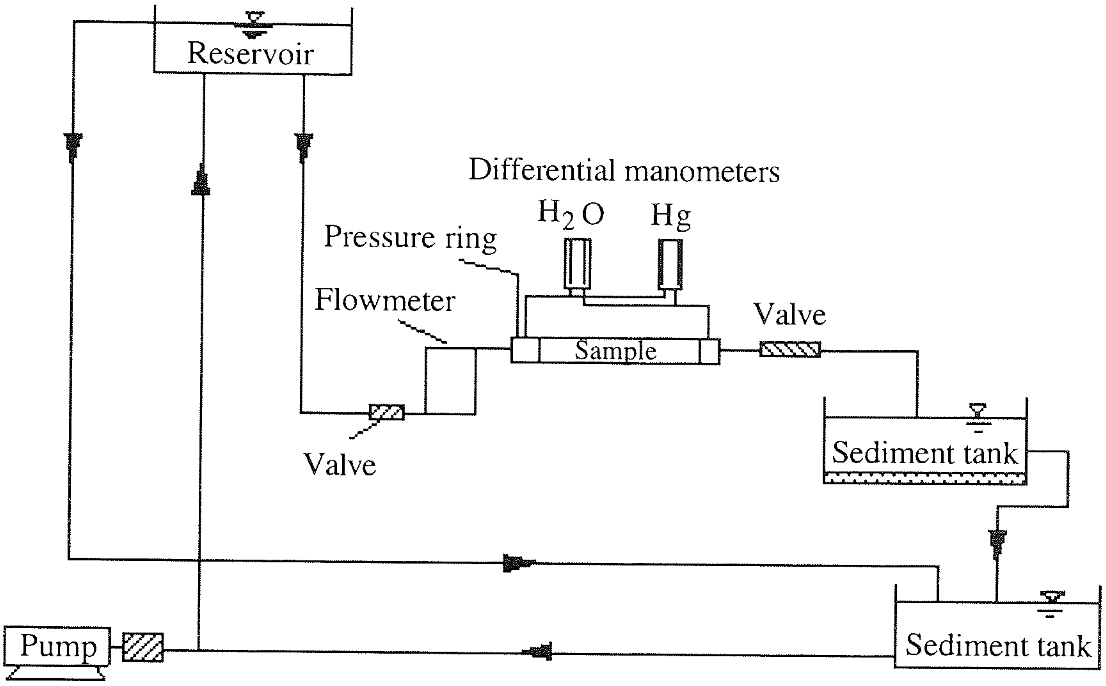


Figure 7.3 Hydraulic and monitoring system for the drill hole test.

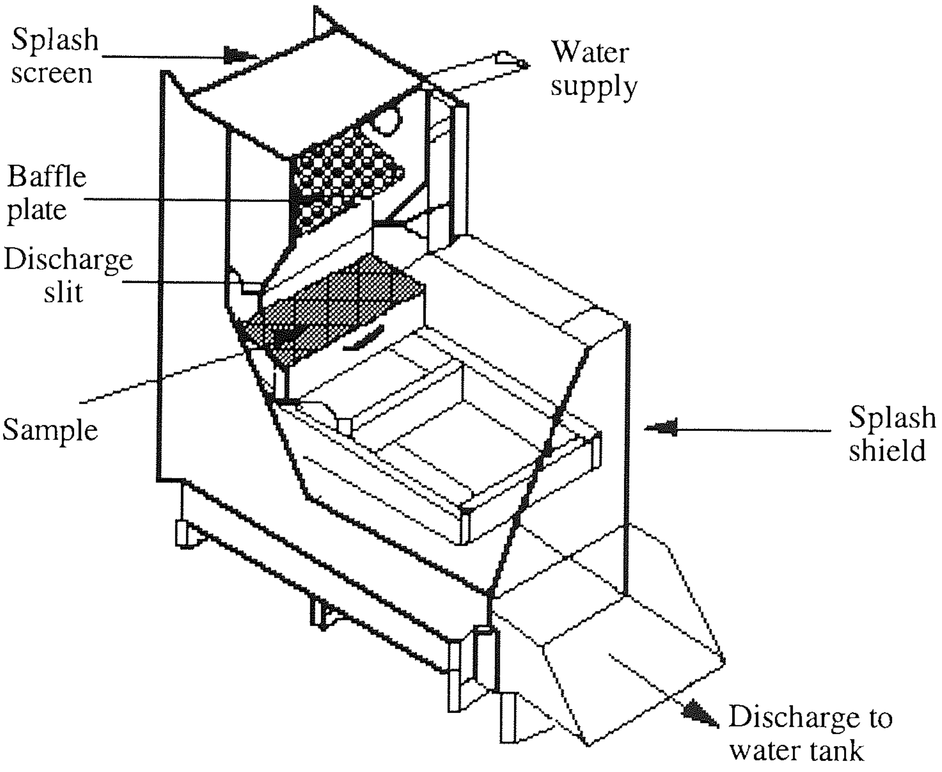


Figure 7.4 The basic design of the simulated free overfall erosion test.

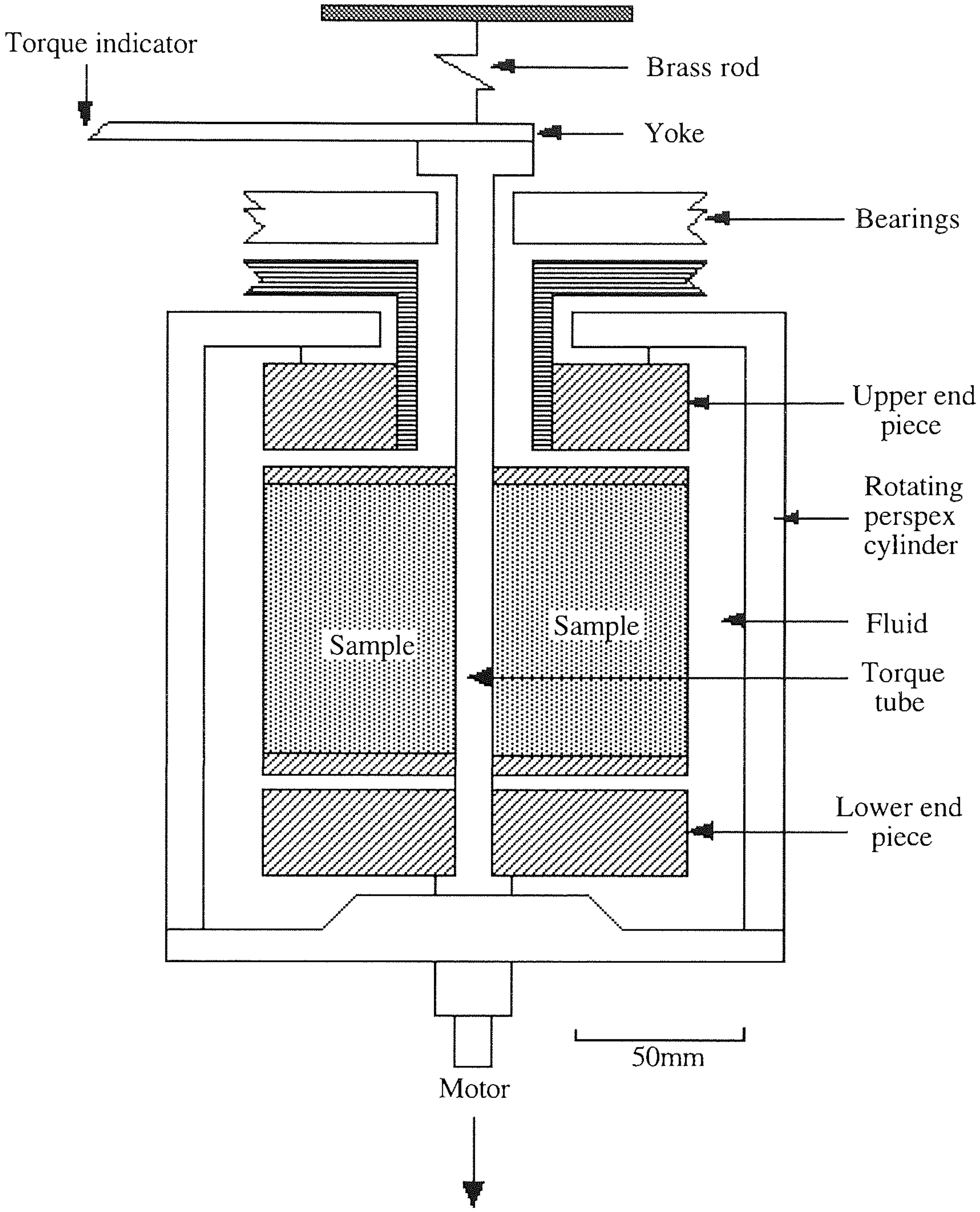


Figure 7.5 Moore and Masch's (1962) original rotating cylinder apparatus.

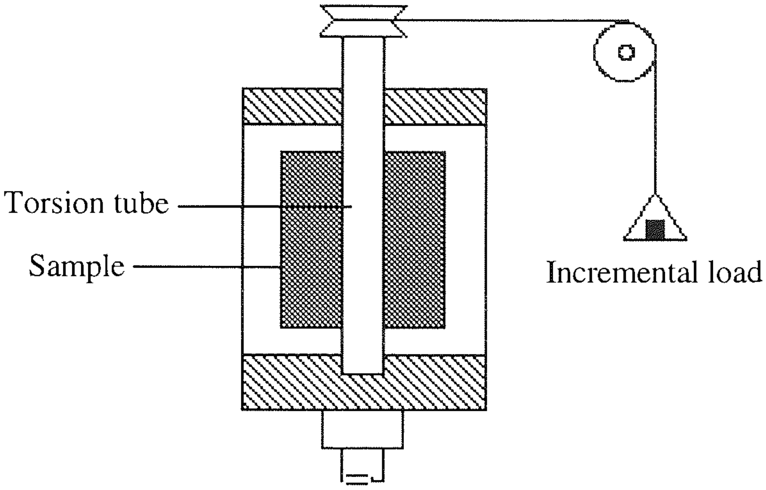


Figure 7.6 Pulley and weight system used by Moore and Masch (1962) to produce a shear stress versus rotational velocity calibration chart.

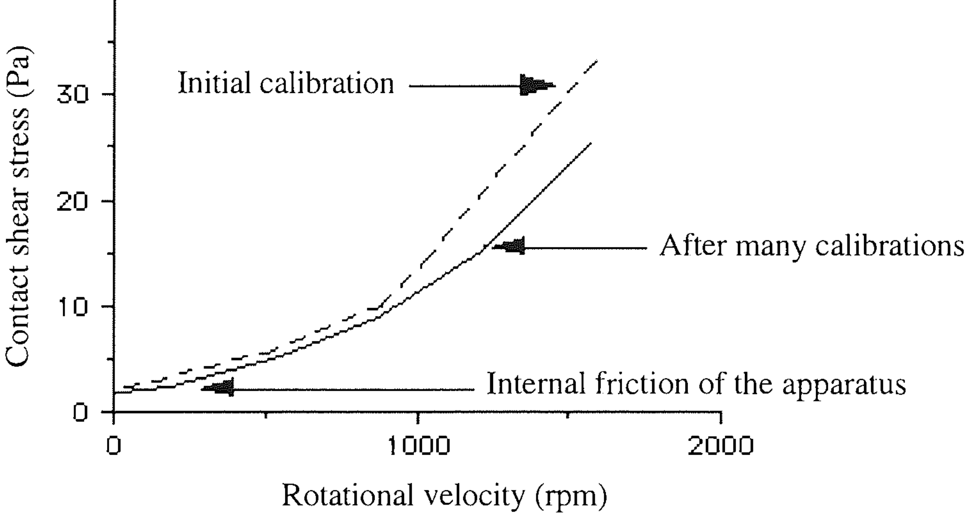


Figure 7.7 Calibration chart needed with the original rotating cylinder apparatus to calculate contact shear stress.

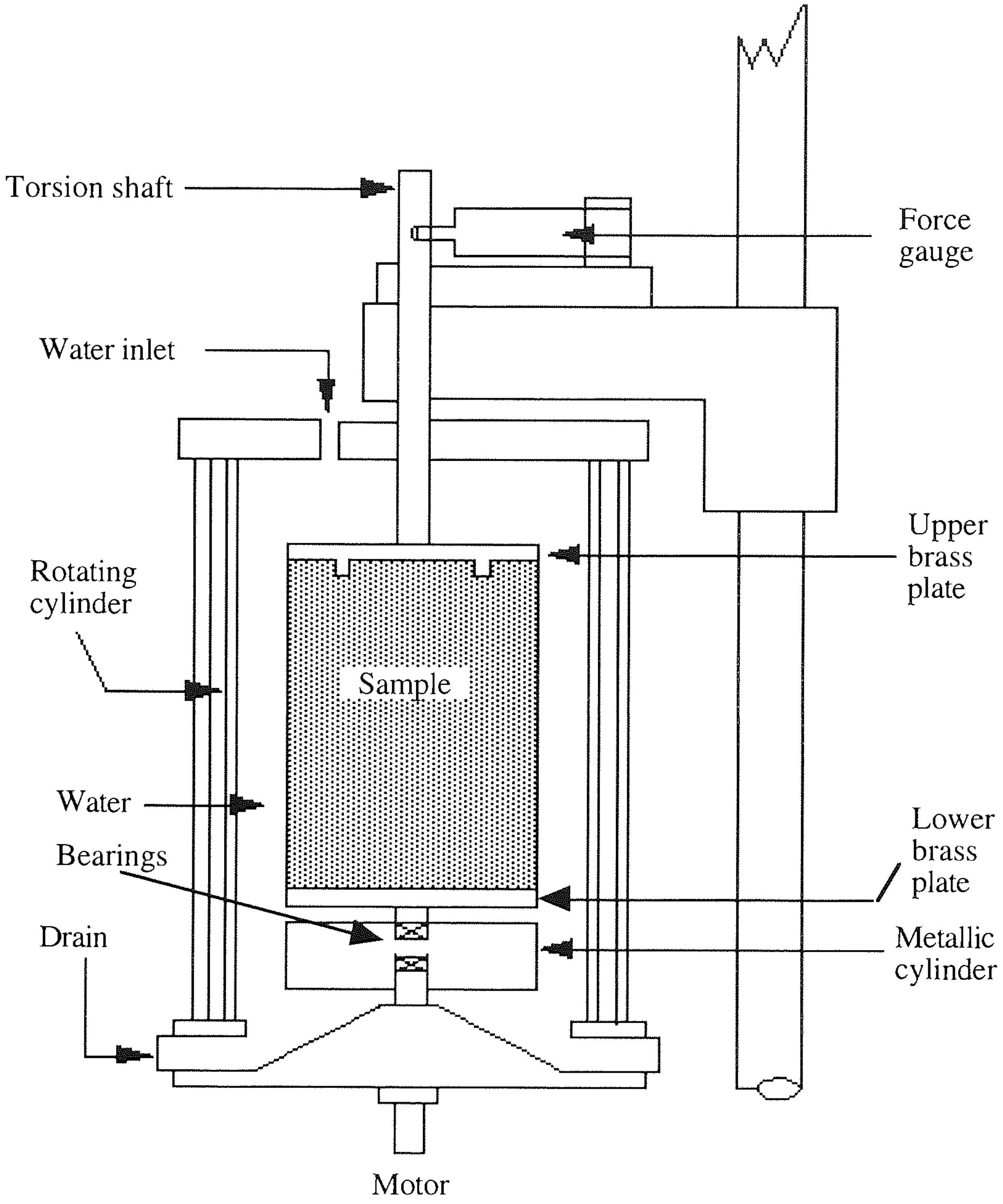


Figure 7.8 The modified rotational device incorporating the improved design features of Chapuis (1986) and used in the present research.

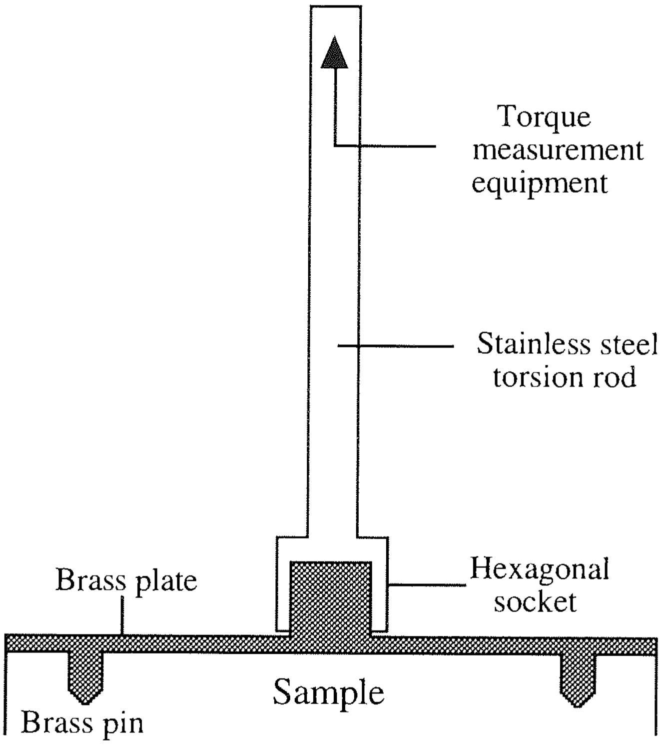


Figure 7.9 The arrangement of the torsion rod in relation to the sample.

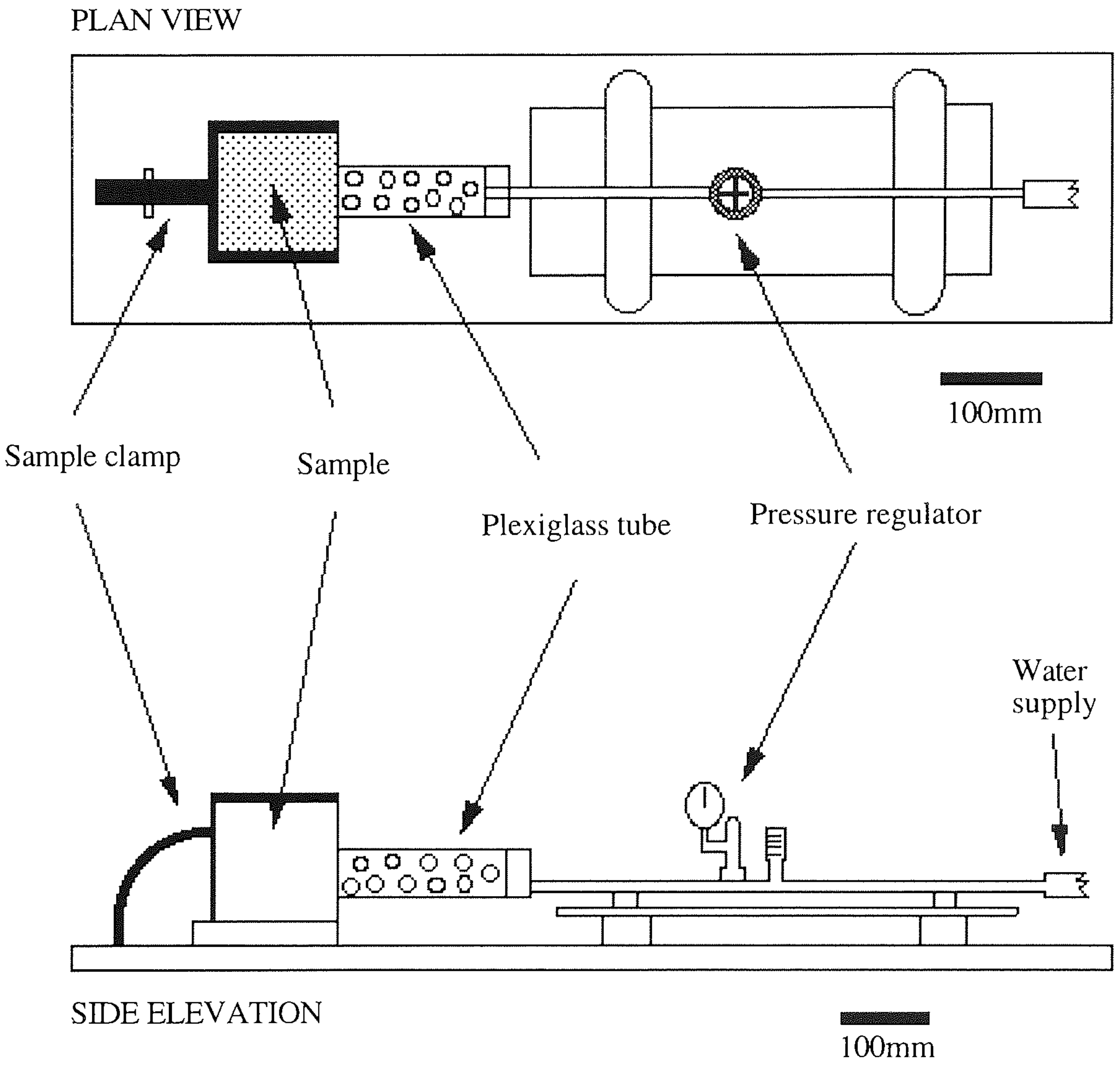


Figure 7.10 Diagrammatic representation of the erosion/abrasion test.

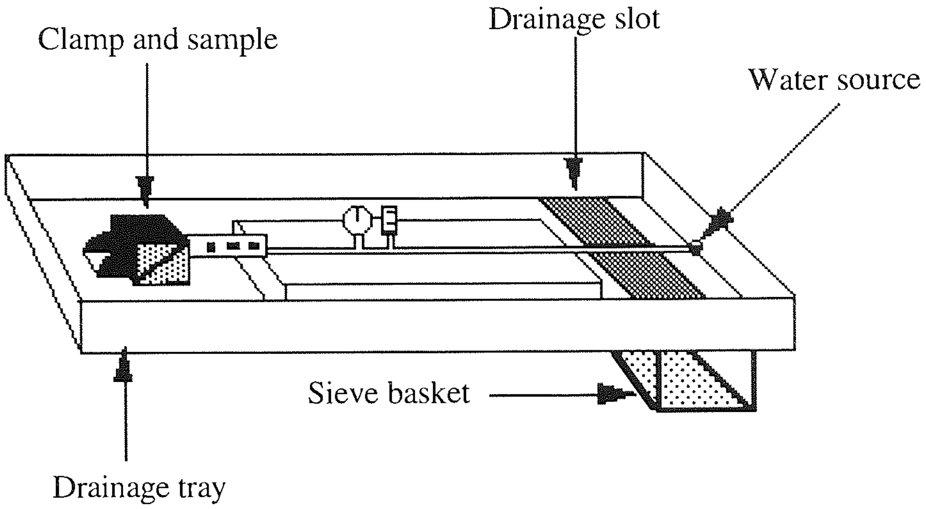


Figure 7.11 Modified erosion//abrasion apparatus showing sieve basket and drainage slot.

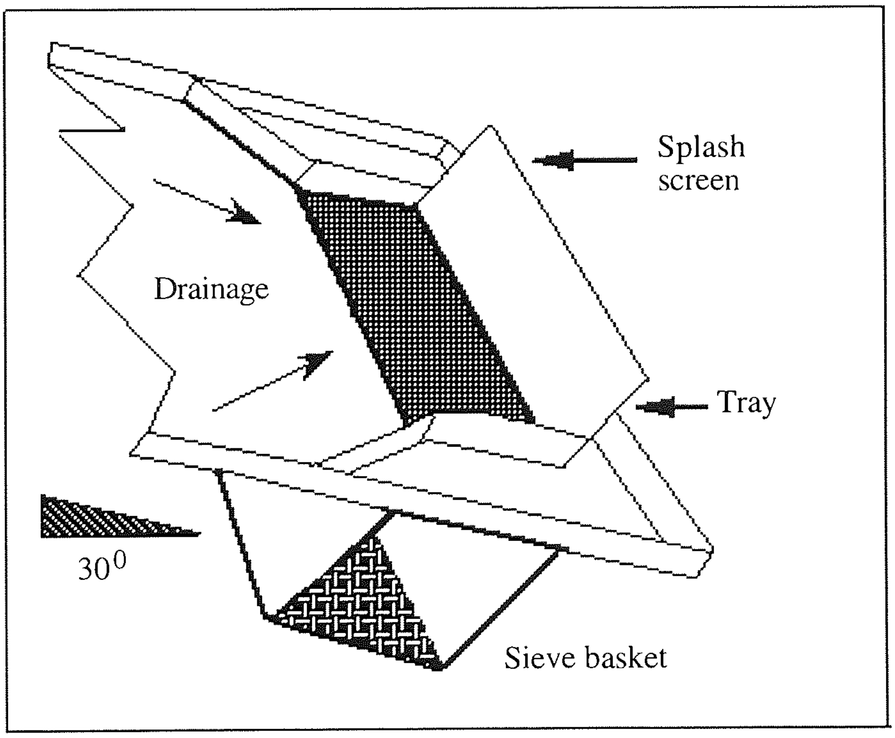


Figure 7.12 Detail of the modified erosion/abrasion apparatus illustrating the splash screens, drainage slot and sieve basket.

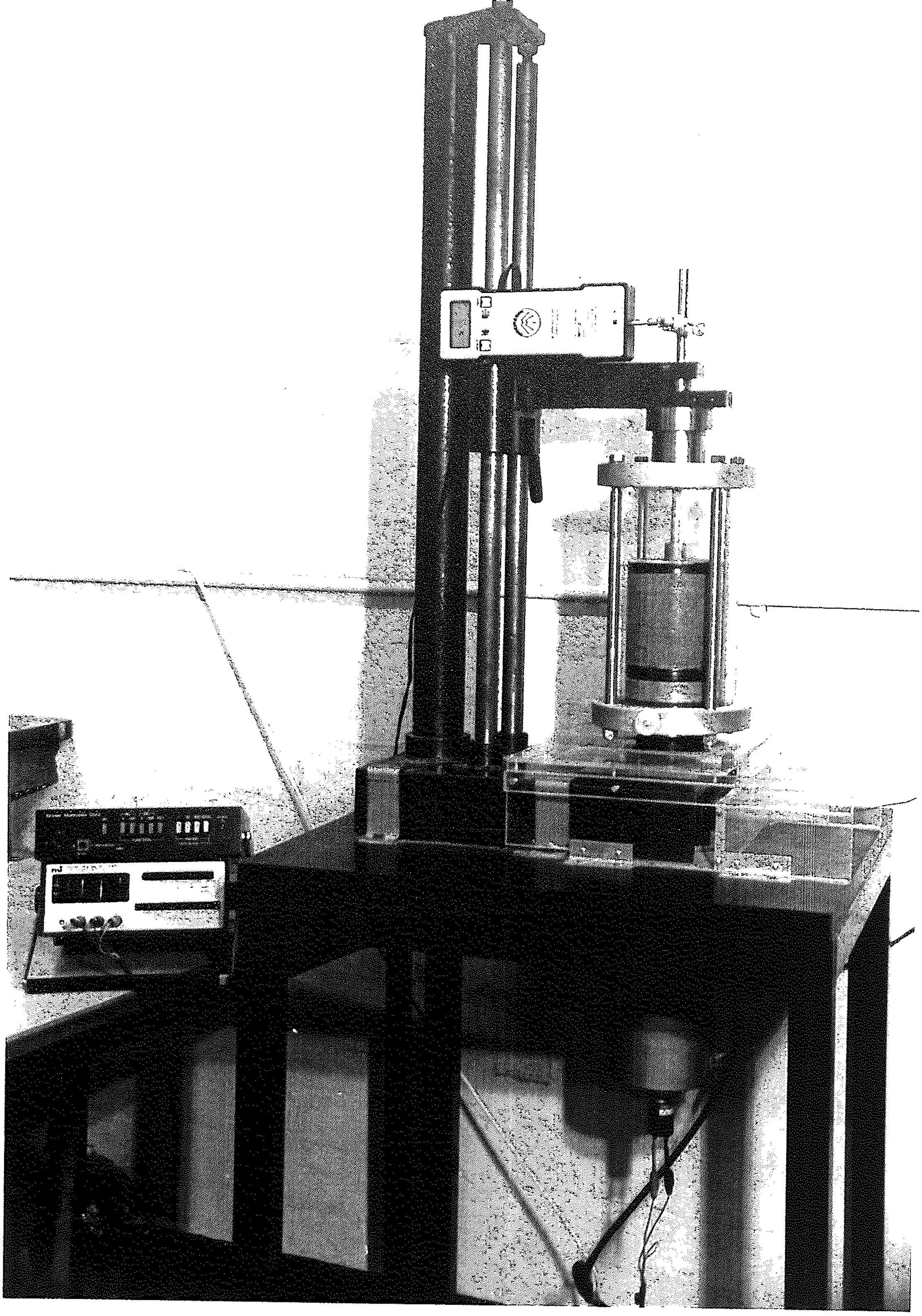


Plate.7.1 The Rotating Cylinder Device.

Plate 7.2 The Erosion/Abrasion Test Apparatus.

8. ANALYSIS AND DISCUSSION OF THE CLASSIFICATION AND WATER STABILITY RESULTS OF MINESTONE

8.1 Introduction

A complete knowledge of the basic physical, lithological and mineralogical properties of a minestone is essential when considering it for engineering applications. Previous studies into the durability of minestone have shown that these basic properties may be correlated with the performance of stabilized minestone used for engineering applications (McNulty, 1985; Thomas, 1987). Hence a detailed knowledge of the basic properties may be used to predict the performance of a particular minestone and may subsequently be incorporated into a system of quality control and assurance.

8.2 Visual Description

A preliminary visual inspection can reveal a great deal about the condition of a minestone sample. Such information may include the degree of weathering, an indication of the slaking characteristics and the relative amounts of carbonaceous and fine material.

The Sutton Manor sample was charcoal grey to black in colour and this may be attributed to the high carbon content within the shale fraction. The majority of the larger particles within the sample were amorphous in form, with no obvious preferred orientation of the fabric. When dry, the Hatfield sample was light grey primarily resulting from a thin dust of fine powdered clay coating the surface. When a fresh surface was exposed the true colour was a medium grey with a greyish-blue tinge. The fracture of the Hatfield shale produced a regular, elongated orientation so that lath and needle like particles were predominant within the sample. The shale within the Hatfield sample was characterised by the presence of abundant plant debris in the form of carbon film fossils.

Despite sampling from the same location and minestone batch there was a marked difference in appearance between the two Point of Ayr samples (i.e. POAB and POAA) due to the influence of marine action on the POAB sample. The samples taken from above high tide level had a muddy texture with the finer fractions showing a marked agglutinated form, while the samples taken from below high tide had a characteristically clean appearance with a small, but significant number of grains showing abraded surfaces. An abundance of flat, cleaved surfaces to the particles suggested that the Point of Ayr shales have a marked degree of fissility. The POAA samples were similar in colour to the POAB samples, both being dark grey to black however, as a result of pyritic decomposition, minestone taken from above high tide level exhibited extensive areas of yellowish-green discolouration. Such discolouration was absent from the POAB samples suggesting that the sulphur compounds had been removed by sea action. Despite a limited exposure period, the extensive presence of sulphur compounds suggests that the minestone from the Point of Ayr site is highly susceptible to weathering and probably at an accelerated rate considering the limited exposure.

Even though the sample taken from the Lady Windsor site was taken from a depth of at least a metre, the initial impression gained from its appearance suggested that weathering was at an advanced stage, both physically and chemically. Pyrite degradation was well advanced, indicated by a large release of sulphur products that discoloured the minestone to a light greenish-brown, whilst the muddy texture suggested that the percentage of fines was significantly high. The mode of fracture exhibited by the Lady Windsor shale particles show some fissility but on the whole the particles were mainly massive and equidimensional.

The samples from the Wardley site were visually similar to those from Lady Windsor, in that they showed similar signs of advanced weathering. The Wardley samples however, exhibited a more regular and platey fracture suggesting a higher degree of fissility and/or laminations present within the rock. The Wearmouth and Westoe samples were very similar in appearance, being medium grey in colour with a massive, bulky fracture and showing no external signs of weathering.

When tested using a streak plate (a more reliable indicator of colour), the minestones under investigation fit into a series of increasing darkness which may represent an increase in carbon content together with increasing iron content. The Hatfield sample had the lightest streak (silver grey) followed by Wardley (light brown) Wearmouth,Westoe and Lady Windsor (grey to greyish brown). At the darker end of the scale lay Point of Ayr (chocolate brown) and Sutton Manor which, when powdered, was charcoal to very dark brown.

8.3 Physical Characteristics of the Raw Minestone.

The particle size distributions of the samples are given in *Table 8.1* while *Figure 8.1* illustrates the grading envelope for all the test samples. All the samples had continuous gradings and were classified as coarse grained soils (BS 1377:1975). Furthermore, they all satisfied the limits set down by Clause 805 of the DoT specification (1986). These minestones can be described as a mixtures of gravel, sand and silt fractions with the finer fractions showing low to medium plasticity. *Figure 8.1* indicates that the unweathered samples contained between 70 per cent and 95 per cent gravel size particles. The samples from Hatfield, POAB, Sutton Manor, Wardley, Wearmouth and Westoe contained a fines content (the percentage of material below 63μm) below 5 per cent, only POAA and Lady Windsor exhibited a higher percentage of fines, with 19 per cent and 9 per cent respectively. *Figure 8.2* illustrates the difference in the grading between the POAA sample taken from above high tide and the POAB sample taken from below the high tide level where it had been influenced by marine conditions. These two samples were considered to belong to the same batch of waste material being excavated at approximately the same time. However, the POAB sample had a coarser nature than the POAA sample and it is suggested that this can be attributed to the removal, by marine action, of the finer components of the POAB sample. The finer nature of the POAA sample is peculiar considering its recent excavation. However, chemical weathering of this sample was fairly extensive which may suggest that physical weathering was also at an advanced stage. Another explanation for the fine nature was that tailings may have been incorporated into the coarser material or that the sample material may belong to a faulted section which had physically weakened the fabric of the rock.

Although the Wardley material was collected from a spoil heap and is considered to be at least 25 years old, only a limited percentage of fines was present which suggests that physical weathering may have been limited especially in the generation of fine material. That the Wardley sample has been subjected to physical weathering is not in doubt as the portions containing the coarser particles have diminished and the larger particles exhibit cracking and water mark features along fracture planes. The Lady Windsor samples are at least 100 years old and physical and chemical weathering was much in evidence though, again, the percentage of fines present is still fairly low suggesting that physical weathering was slow at generating the clay fraction of a spoil heap. The samples from Hatfield, Sutton Manor, Wearmouth and Westoe all characteristically exhibit low fines values, typical of minestone that has passed through modern washery plant.

Without exception, the sub 63μm content of all the samples satisfy the 20 per cent criterion suggested by Thomas (1987) for successful stabilization. The grading index was found to be highly sensitive to differences in particle sizes in the coarser range and rather insensitive to the finer categories. As it is usually the finer fraction of a material that influences the engineering properties (Rainbow and Sleeman, 1984), the grading index is of limited use for classification and selection purposes though it has been used successfully for comparative purposes in another section of the research.

Table 8.2 presents the consistency characteristics together with moisture content data for the selected minestones. Although the present DoT specification (1986) concerning cement stabilized materials does not specify any consistency limits, it is still important to know these basic engineering properties as they can be used to predict the performance of a material. This has been borne out in the work of Thomas (1987) who suggested that for a stabilized minestone to gain the recommended strength and durability, the maximum values for the plastic limit and plasticity index are 22 and 15 per cent respectively. With the exception of the Wearmouth and POAA samples the minestones possess plastic limits at or below the limiting value of 22 per cent. Wearmouth has a plastic limit of 24 per cent while the plastic limit of the POAA sample is 23 per cent, neither of which greatly exceeds the limit. All the samples satisfy the plasticity index limit.

The linear shrinkage values of the samples are all very similar, averaging 5 per cent, though Hatfield has a shrinkage value of 8.6 per cent. All the test minestones fall on or below the 8.5 per cent value suggested by Ransom (1963) to delimit the potential of a soil for successful stabilization. The consistency limits together with the linear shrinkage values may reflect the type of clay present within a soil. Generally high shrinkage and plasticity values suggest the presence of significant amounts of active clays such as montmorillonites. The shrinkage values exhibited by the test samples all lie at the lower end of the scale.

With one exception, the specific gravity of each of the minestone samples falls close to the mean value for minestones from all areas of the United Kingdom (British Coal, 1989), the exception being the POAA sample (see *Table 8.3*). This sample has a low specific gravity of 2.15. This could be explained by a high coal content, though it only experienced a low loss on ignition. Another explanation could be the leaching of the iron content from pyrites. It was obviously evident that the pyrites within the POAA sample had undergone extensive decomposition. The specific gravity of 2.29 for the Wardley minestone is somewhat lower than the majority of the test samples, although still close to the national mean value. Again iron leaching may be evoked to explain the lower specific gravity of this minestone.

8.4 Chemical Properties of the Raw Minestone

Table 8.4 presents the pH values of minestone when immersed in either distilled or sea water. These results are presented for comparison in *Figure 8.3* and it is clear that the samples tested in sea water are affected by the strong buffering properties of sea water. Certain compounds in sea water exert a strong influence against any variation from the 7.8 to 8.3 range for normal, surface, oceanic sea water. Within this normal pH range, sea water contains both HCO_3^{-1} and a plentiful supply of unionised H_2CO_3, in addition to a small amount of of CO_3^{-2}. The first two compounds are the main buffering components of sea water.

Attempts to increase acidity are countered by the forward reactions of *Equation 8.1* and *Equation 8.2* :

$$H^+ + CO_3^{-2} \longrightarrow HCO_3^- \qquad (\textit{Equation 8.1})$$

$$HCO_3^- + H^+ \longrightarrow H_2CO_3 \qquad (\textit{Equation 8.2})$$

In addition, the vast quantity of $CaCO_3$ present within sea water acts as a buffering agent :

$$CaCO_3 + H^+ \longrightarrow Ca^{+2} + CO_2 + HCO \qquad (\textit{Equation 8.3})$$

Attempts to increase alkalinity (as in the case of freshly excavated minestone) is equally thwarted by the reaction of *Equation 8.4* :

$$HCO_3^- + OH^- \longrightarrow CO_3^{-2} + H_2O \qquad (\textit{Equation 8.4})$$

Precipitation of $CaCO_3$ will also act to keep the alkalinity within the buffered zone :

$$Ca^{+2} + HCO_3^{-1} \longrightarrow CaCO_3 + OH^- \qquad (\textit{Equation 8.5})$$

As a result of the above reactions the pH value is lowered in cases where the alkalinity is above 7.9 (natural pH of sea water) and, in the case of the POAA sample, the pH was increased when acidic conditions were encountered. Unweathered minestone is usually alkaline when excavated but as chemical weathering of the pyrite proceeds the pH value drops as the result of sulphuric acid production. The slightly acidic nature of the POAA sample further indicates that chemical weathering has had a marked influence upon this material. The pH value of minestone plus 10 per cent cement was naturally high (12.4 in distilled water) but the effect of buffering was still evident when minestone plus 10 per cent cement was tested in sea water, however the effectiveness of the buffering system was limited due to the extreme alkalinity of cement.

A further consequence of the maintenance of alkaline conditions by the buffering action of sea water would be to severely curtail the oxidation of pyrite by the catalytic actions of the obligate acidophile bacteria. Sulphate production as a result would be slow especially if the permeability of the stabilized minestone was kept low.

The results of the acid soluble and water soluble sulphate tests are given in *Table 8.5* and *Figure 8.4*. The DoT specification (1986) governing the use of cement stabilized soils limits the acid soluble sulphate content for granular soils to 1 per cent and for cohesive soils to 0.25 per cent. With the exception of the POAA sample which has an acid soluble sulphate content of 1.2 per cent, all the samples tested fall well within the specified limits with sulphate contents ranging from 0.05 per cent for Wearmouth and 0.43 per cent for Lady Windsor. Even when using the more stringent specification of 0.8 per cent as suggested by Thomas (1987). All the samples fall within the limits, with the exception of POAA. If 0.25 per cent is used as the limit as suggested for all soils containing clay minerals (Sherwood and Ryley, 1970) the Lady Windsor, POAA and POAB samples would fall outside the specification. However, it is appropriate to question whether this specification can be used for non-cohesive minestones with limited 'fines' contents. The water soluble sulphate content of the each test minestone is understandably lower than the corresponding acid soluble content. The highest water soluble sulphate content is exhibited by POAA with 0.42 per cent, the other test samples have water soluble contents ranging from 0.01 per cent for Wearmouth to 0.28 per cent for Lady Windsor.

8.4.1 Lithology of the Test Minestones

The lithology of the particles in different size categories was determined with five major lithologies being recognised. These are mudstone (shale, claystone and mudstone) sandstone (fine to coarse sandstone) siltstone (fine to coarse siltstone) pyrite/ironstone (iron rich particles having a high specific gravity) and coal (particles with a carbonaceous nature and exhibiting a low specific gravity). The percentages of these lithologies in the different size categories are given in *Appendix A3*, the lithology for each whole sample is presented in *Table 8.6* and illustrated in *Figures 8.5* and *8.6*.

In a general sense the lithological nature of the various test samples is very similar with some 70 per cent to 80 per cent of each sample consisting of mudstone and siltstone, the siltstone commonly being fine grained. The Lady Windsor sample has the lowest percentage of finer lithologies with just over 60 per cent, though 25 per cent of this sample consisted of dense, iron-rich, mudstone nodules that have been classed within the ironstone category. If these nodules were classified as mudstones, the Lady Windsor sample would fall within the range exhibited by the other test samples.

The next biggest category was the sandstone group which generally represented 15 per cent of the whole sample, though Wearmouth contained approximately 25 per cent sandstone with both Lady Windsor and POAA containing less than 10 per cent sandstone. Generally the coal content of the samples was 7 per cent or less but the POAA sample had a coal content of approximately 15 per cent. With the exception of the Lady Windsor and the

POAA samples, the percentage of pyrite rich particles and ironstone within the whole sample was usually less than 4 per cent. The 25 per cent iron-rich nodules within the Lady Windsor sample has already been mentioned. The long exposure to the actions of physical weathering experienced by these samples may account for the increased percentage of the more resilient nodules and the accompanying decrease in the clay and siltstone based clasts.

In the above discussion the POAB sample has been deliberately left out of the comparison as it had probably been altered and modified by marine action. *Figure 8.7* compares the lithology of the two Point of Ayr samples, the POAA sample having been uninfluenced by marine action unlike the POAB sample which has been subjected to wave conditions and tidal pumping. It is believed that both samples belong to the same excavated batch and, originally, the POAB sample would have resembled the POAA sample in its lithological characteristics. The wave modified POAB sample has a relatively higher percentage of sandstone, siltstone and ironstone/pyrite, the more resistant lithologies. Similarly the lower percentage of coal in this sample is probably due to the relatively low specific gravity of coal so that it was more susceptible to the winnowing erosion produced by wave action.

Lithology has a strong bearing upon the abrasion resistance of a material. Coal has a brittle and soft nature and would be expected to be susceptible to abrasion. Sandstone and the coarser forms of siltstone would be expected to have more resistance as would iron and pyrite rich particles. Generally mudstone will be the most vulnerable to abrasion though this may not always be the case, as some mudstones may have an indurated nature, which is certainly true for the mudstones in the Sutton Manor minestone.

8.4.2 Mineralogy of the Test Minestone

8.4.2.1 Whole Sample

As expected, a qualitative analysis of the diffractograms of the whole sample for each minestone indicated that the samples were dominated by the clay minerals (7.1 Å and 10 Å) and quartz (3.34 Å and 4.26 Å). Generally a whole sample analysis carried out upon a smear specimen provides little information of intrinsic value (Thomas, 1987). However, some useful information may be gleaned from these results appertaining to the state of the minestone sample. The diffraction peaks associated with gypsum (7.61 Å and 7.56 Å), alunite (3.04 Å) and jarosite (3.08 Å) were absent from the Sutton Manor, Westoe, Wearmouth, Hatfield and POAB samples. In contrast, gypsum peaks were in evidence in the Lady Windsor, Wardley and POAA samples. The relatively prominent gypsum peak associated with the POAA sample was further highlighted by the absence of a gypsum peak in the POAB diffractogam. The presence of these sulphate minerals indicates that the samples have undergone a certain degree of weathering. To what extent this weathering has affected the minestone is uncertain though the pronounced nature of the POAA gypsum peak suggests that at least this sample had suffered substantial pyrite degradation. The absence of a gypsum peak in the POAB diffractogram may reflect the enhanced solubility of gypsum in sea water, as this sample has undergone many periods of sea water immersion.

The source for these secondary sulphate minerals is the decomposition of iron pyrites and its sister mineral, marcasite. Pyrite was detected in all the minestone samples with the exception of Hatfield. In addition to the pyrite peaks (1.63 Å and 2.71 Å), a marcasite peak (3.44 Å) was readily discernible in both the Point of Ayr samples.

The random nature of the whole sample diffractogram places a restraint upon its reliability for mineral identification, especially when background 'noise' interfered with the peaks of individual minerals. A wide variety of minerals in subordinate quantities may be found interspersed with the clay minerals (Wilson, 1987). Definite calcite peaks (3.03 Å) were detected for the Lady Windsor and Point of Ayr samples, whilst the Wardley sample gave a probable but ill defined calcite peak.

The iron carbonate minerals ankerite (2.90 Å) and siderite (2.79 Å) though poorly defined were present in each sample. Goethite and limonite indicated by intense reflections at 4.18 Å with lesser peaks at 2.69 Å and 2.45 Å were shown to be present in the Lady Windsor and Wardley samples.

8.4.2.2 Specific Clay Mineral Analysis

For engineering purposes the presence of the smectite clays must be regarded as the most worrying characteristic of any clay assemblage. Smectites (such as montmorillonite) are characterised by broad and diffuse non basal reflections or bands reflecting random layer stacking. Generally at room temperature and normal humidity the basal spacing for the smectites ranges from 11 Å to 15.5 Å. No evidence of a separate smectite phase was found for any of the minestones tested.

Figure 8.8 illustrates the ratio of the four main clay mineral groups, namely kaolinite, illite, chlorite and the mixed-layer clays. The clay mineralogy of a material will certainly influence its performance when subjected to a differing environmental regime. However, the contrasting performance of the Hatfield and Sutton Manor minestones, when immersed in water, cannot be explained by any significant difference in their clay mineralogy as both are dominated by illite and mixed-layer clays (70 per cent of the total clay content) with a similar content of the subordinate kaolinite which represents some 20 per cent of the total clay content.

All the samples are dominated by illite and illite-based mixed layer clays. Kaolinite mainly comprises between 10 per cent and 20 per cent of the total clay content, however, with the northeastern minestones (Wardley, Wearmouth and Westoe), it represents up to a third of the overall clay content. No major difference exists between the POAA sample and the POAB material, despite the fact that the POAB sample may have been subjected to intermittent sea water submersion for a period of up to a year. A cursory assessment of the samples would suggest that basically all are mineralogically similar when considering the clay mineral content.

8.4.3 Cation Exchange

The cation exchange capacities of each test sample are given in *Table 8.5* and additionally, *Table 8.7* provides the exchange capacities, in terms of calcium hydroxide, for each sample. The cation exchange capacity (CEC) of clay minerals increases in the order of kaolinite, illite and montmorillonite (Hillel, 1971; Wilson, 1987). The CEC of montmorillonite is usually well in excess of 100 meq/100g. All the samples have CEC's below 50 meq/100g in all the size categories tested. Lady Windsor has the highest CEC at 42 meq/100g while Wearmouth has the lowest at 10.5 meq/100g (the average value of the three size categories tested). Generally the CEC value is shown to increase as the grain size decreases. This may be due to the increase in the exposed surface area associated with a decrease in the particle size but, though the CEC is partly controlled by particle size, the main influence upon the value is the clay mineralogy. The CEC values associated with the test minestones suggest that significant quantities of active clay minerals were absent. This was subsequently borne out by the X-ray diffraction results discussed above.

Active clays have the potential to react strongly with lime ($Ca(OH)_2$) thereby lowering the pH level of a stabilized soil which subsequently affects the primary hydration of cement (Croft, 1967). As McNulty (1985) pointed out, the potential of a stabilized minestone to suffer from lime depletion is not solely governed by the CEC value but also depends upon the actual size of the finer fractions of the soil. This is taken into account in the equation used to determine the exchange capacity :

$$EC = \frac{P \times W \times CEC}{100}$$ (*Equation 8.6*)

where:
EC = Exchange capacity (mg $Ca(OH)_2$/100g).
P = Fraction size (%).
W = Equivalent weight of Ca $(OH)_2$.
CEC = Cation exchange capacity (meq/100g).

The exchange capacity is expressed in milligrams of calcium hydroxide per 100g of minestone. A fully hydrated cement will liberate up to 30 per cent of its own weight as $Ca(OH)_2$ (Neville, 1981), but it must be noted that, under normal conditions cement is never fully hydrated and so the lime released will be less than 30 per cent (of the cement weight).*Table 8.7* presents the exchange capacities (in terms of $Ca(OH)_2$/100g) for individual size fractions. It also presents the exchange capacities for the samples as a whole as a percentage of $Ca(OH)_2$. Bearing in mind that a fully hydrated cement will liberate up to 30 per cent $Ca(OH)_2$, a 10 per cent cement content would release a maximum of 3 per cent $Ca(OH)_2$ while 5 per cent cement would release 1.5 per cent. From Table *8.7* (column 8) it is shown that the lime demand for each of the minestones does not exceed the lime released by either a 10 per cent or 5 per cent cement content.

8.5 Porosimetry

Hudec (1989) has shown that a relationship exists between capillary radius (pore diameter) and the rate of failure, and the degree of slaking susceptibility, exhibited by a rock. As will be discussed in *Section 8.7,* of all the samples tested, the material from Hatfield has the greatest propensity to slake. All the other samples exhibit a certain degree of similarity in resisting slaking degradation following immersion. No lithological or mineralogical characteristic of the Hatfield sample suggests that it should be any less resistant to slaking than the other test samples. However when pore size and pore size distribution characteristics are examined, it is clear that the Hatfield sample is dissimilar to the other test materials.

An excellent indicator of the nature of the fabric of a material is the cumulative pore surface area which is measured in m^2/g. The relationship between the pore diameter and the total pore surface area is illustrated in *Appendix A4*. If the smaller pores are present in vast quantities, their combined surface area will be high and so the more the pore size distribution is skewed towards the smaller pore diameters, the greater the value of the total pore surface area. *Figure 8.10* illustrates and compares the total pore surface area of each of the test samples. With one exception, all have total pore surface areas in the range of 3 to $5 m^2/g$. The exception is Hatfield which exhibits almost twice the total pore surface of the other samples, $9.5 m^2/g$. This suggests that the pore size distribution was similar for the majority of the samples, but markedly different for the Hatfield material. The greater total pore surface area exhibited by the Hatfield sample suggests that the sample possessed a far greater number of finer pores than the majority of the test samples.

Figure 8.9 compares the cumulative pore volume increase versus pore size for each of the test samples against that of the Hatfield material. It can be seen from these graphs that the Hatfield distribution is skewed towards the left of each of the samples tested, which reinforces the suggestion that the pore sizes within the Hatfield sample were generally finer than those in the other samples. The pronounced rightward skew shown by the Point of Ayr sample is probably explained by the fissile nature of its fabric creating microfractures that were represented as relatively large pores in the porosimetry data.

Figure 8.11 illustrates the percentage of pores finer than 0.008 and 0.006μm (the finest measured). Only Hatfield had pores smaller than 0.006μm (using a contact angle of 130° to calculate pore size). Ten per cent of the pores within this sample are below this diameter. The smallest pore diameters present within the other samples are between 0.008 and 0.006μm. With the exception of Wearmouth and Hatfield, all the test samples had 11 per cent or less of pores within this size range. The Wearmouth sample has 21 per cent of its pores within this size range whilst Hatfield has 30 per cent .

The capillary pressure generated by immersion is inversely proportional to the capillary diameter, and if the capillary pressure generated exceeds the fabric strength of the material, slaking will occur. It can be expected that the capillary pressures developed in the shale from Hatfield will be far greater than from the other sites and, judging by the degree of slaking suffered by this shale, these capillary pressures are far greater than the bonding forces within the material. Although the extent of this porosimetry study is limited, a relationship has been demonstrated between the slaking potential and the total pore surface area.

8.6 Bacterial Action

All the samples were tested for the presence of sulphophile bacteria. Those samples that represent recent arisings, i.e. Sutton Manor, Hatfield, Westoe and Wearmouth, gave negative results. The Wardley and the Lady Windsor material, both of which have spent a protracted period within a spoil heap, also gave a negative response when used to inoculate a culture medium. The sample taken from above the high tide level at the Point of Ayr site (POAA), gave a positive indication that bacteria with a sulphur-based metabolism were present. Microscopy revealed that the bacterium involved belonged to the Thiobacillus family. The presence of this bacterium may explain why this sample showed signs of advanced decomposition of its pyrite content, despite its relatively recent excavation. The sample taken from below high tide (POAB) gave a negative result despite being sampled within 10 metres of the infested POAA sampling site. The dynamic nature of the environment that existed below high tide, together with the alkalinity of a marine environment clearly prohibited the colonization of the minestone by the acidophillic, obligate aerobic microbes. A marine environment would most probably represent too harsh a habitat to allow successful or significant development of Thiobacillus colonies or related bacteria.

8.7 Immersion Stability Results

8.7.1 Tidal Simulation Results

Table 8.8 shows the original grading indices for each of the test samples together with the grading indices following 8, 16, and 24 cycles of immersion and draining in sea water. For each material the original grain size distribution curve and that following 24 cycles of immersion and draining are shown graphically in *Figure 8.12*. It can clearly be seen from these graphs that there has been very little change in the grain size distribution of any of the minestone samples following 24 cycles of immersion and drainage to simulate tidal conditions. The grading indices were more erratic in nature but, again, they indicate only a low level of degradation following 24 cycles. The slightly erratic nature in the grading indices is a probable result of the sensitivity towards the coarser end of the distribution scale. Limited variations in the size and mass of the larger particles within a minestone can lead to a marked variation in the percentage of particles passing a given sieve size.

Figure 8.12 shows the effects of the 24 cycles on Wardley minestone which exhibited a marked change in the distribution over these 24 cycles. Unlike the other samples, this material was air dried prior to the start of the test but, apart from its initial state, it was subjected to the same cycling procedures as the other samples. It can be seen from the grading indices in *Table 8.8* that the Wardley samples continued to breakdown throughout the test, suggesting that the initial state of the minestone had a bearing upon its reaction following immersion.

It can be concluded from these tidal simulations that, even though the samples experienced periodic drainage at temperatures of between 25°C and 30°C, the particles did not dry out sufficiently to allow significant slaking to occur, for slaking tends not to occur in particles that are saturated. However, if the particles were air dried prior to immersion, slaking continued for an extended period, despite being in a relatively constant environment with respect to saturation levels. Eventually a moisture equilibrium would be achieved within the samples so that slaking would slow and eventually cease, thereby enabling the sample to behave in the same manner to those that which had not been previously dried out.

8.7.2 Cyclic Wetting and Drying Test Results

The cyclic wetting and drying test simulated the conditions experienced by a structure above the high tide mark, namely intermittent wetting by storm action, sea spraying, or meteoric wetting (Zone 1 and 2 of Fookes and Poole's scheme (1981) and Zone 1 of Nussbaum and Colley's erosion scheme (1971)). The results of these tests, expressed in terms of the grading index, are presented in *Table 8.9*, while *Figure 8.13* gives a graphical representation of the behaviour of the material in the form of grain size distribution charts. It can be seen from the these charts that there is a distinct movement of the grain size distribution curve towards the left (finer) for all minestone tested. This leftward drift represents a fining of the sample as a result of slaking damage. As the main lithology of the minestones was mudstone and shale, it is not surprising that slaking plays a major part in the weathering and degradation of the samples.

The POAA and Hatfield samples suffer the greatest decrease in particle size, with a 67 per cent and 65 per cent fall in the grading index respectively followed by Westoe (58 per cent), Lady Windsor (54 per cent) and Sutton Manor with a 50 per cent reduction in its grading index. The samples showing the least reductions are Wearmouth (38 per cent), POAB (40 per cent) and Wardley with a 45 per cent decrease (*see Figure 8.14*). It was considered that the most recently excavated materials would have the greatest potential for slaking and this is supported by the behaviour of the minestones from Hatfield, Westoe and POAA. However, similarly recent samples from Wearmouth and Sutton Manor shows more resistance leading to a lower percentage drop in their grading indices following cyclic testing. The resistance of the Wearmouth sample can be attributed to its lithology as a large percentage of the sample is composed primarily of sandstone and siltstone, both of which are fairly resistant lithologies. The resistance of the Sutton Manor sample cannot be explained by this means as it has a similar lithological composition to the least resistant samples such as Hatfield. This suggests that the actual shale and mudstone that makes up the bulk of the Sutton Manor material has a far more resistant nature than that present in the other samples.

Although the POAB sample is recent material, its exposure to marine action, especially the continuous changes associated with a tidal regime, has probably modified the material to the extent that the particles present within the sample represent the more resistant fraction of the original material. The greater resistance exhibited by the Wardley sample can be explained in a similar manner but, with the understanding that the modification has occurred whilst the material had been within the spoil heap where natural atmospheric weathering would have been responsible for the modification of the original minestone.

As *Figure 8.13* suggests, though an overall fining of the samples has taken place, generally the finest fractions of each sample show very little increase as a consequence of cyclic testing. The low quantity of fines produced from the cyclic exposure limits the risk that a minestone structure, so exposed, would fail due to the production of large quantities of particulate matter. The production of minimal quantities of fines suggest that, once particles have slaked to a particular size, they gain stability and play no further part in minestone degradation. It can be surmised from this that the rate of slaking will slow down as the percentage of stable particles increases. The Wardley results may be evidence of this behaviour with a decreasing trend in the effects of slaking with each subsequent cycle as the percentage of stabler particles increase.

8.7.3 Laan's Rate of Fineness Results

Laan's rate of fineness test was specifically devised to test the sensitivity of a minestone to weathering. The basic classification proposed by Laan *et al.* (1984) suggests that a fineness rate below 7.5 indicates an extreme sensitivity to weathering, a value between 7.5 and 8.25 implies high sensitivity and material with a fineness rate between 8.25 and the maximum 9.0 has a moderate sensitivity to weathering processes.

The results of the tests, using both sea water and distilled water as the immersing fluid, are presented in *Table 8.10*. Although none of the samples has a fineness value below the extreme sensitivity limit of 7.5, the Hatfield samples, tested both in distilled and sea water, produce values very close to this limit. Thus the Hatfield minestone borders the arbitrary divide between extreme sensitivity and high sensitivity to weathering. When tested with distilled water the POAA sample is classified as highly sensitive, but it only exhibits moderate sensitivity when placed in sea water. The Lady Windsor samples shows high sensitivity in both fluid media, while the Sutton Manor, POAB, Wardley, Wearmouth and Westoe samples indicate relatively high resistance to weathering by producing values within the moderate (8.25-9.0) sensitivity group with both immersing fluids. *Figure 8.15* compares the effects of sea water and distilled water upon air dried minestone during Laan's test. It is clear that all the minestones when immersed in sea water perform better than when immersed in distilled water. Most of the samples showed an improvement of between 0.5 per cent and 2 per cent when tested in sea water over performance in distilled, however POAA improved by 8 per cent when sea water is used as the immersing fluid.

When the samples were tested at their 'as received' moisture contents (Wardley not being tested) a different picture emerged. Without air drying prior to testing, all the samples exhibited a moderate sensitivity bordering on 9.0, this being the maximum in Laan's test. Slaking damage within these samples was negligible with a maximum of only 1.09 per cent dislodged material. This small percentage of dislodged material may be due wholly or partly to the effects of handling during the test and to initial flaws and weaknesses within the particles. Slaking may account for only a small proportion of this dislodged material and could even be absent altogether. The absence of slaking in the samples tested at their natural moisture content suggests that the mechanism of slaking was predominantly due to the compression of the air within pores and microfractures by water imbibed during immersion. The samples that retained their natural moisture content can be considered to have had a high degree of saturation with very little air filling the microcavities of the rock so that slaking as a result of air breakage was limited or even absent.

8.7.4 Modified Slake Durability Test Results

Both Badger *et al.* (1956) and Raybould (1966) have reported a link between low coal rank and an increased susceptibility to weathering. Point of Ayr, Sutton Manor, Wardley, Wearmouth and Westoe are all at the lower end of the medium coal rank codes (600-500), which would suggest a reasonable degree of resistance to weathering and slaking. This moderate degree of slaking resistance is confirmed by all the test results given in *Tables 8.11* and *8.12*. Hatfield with a low coal rank code of 700 confirms the rank/susceptibility relationship by also having poor resistance to slaking. The Lady Windsor sample has a high coal rank (301) and a relatively high resistance to slaking though not as high as the medium ranked samples with the exception of Point of Ayr.

Thomas (1987) suggested that a minimum Modified Slaking Index (MSI) of 60 per cent was necessary if the specified durability and strength criteria were to be achieved for stabilized minestone. All the samples, with the exception of Hatfield with an MSI of 52 per cent, clearly exceeded the 60 per cent value when tested in distilled water. All the samples passed the 60 per cent criterion when tested in sea water, the Hatfield sample again having the lowest MSI value, 70 per cent, compared to Sutton Manor which had the highest MSI value of 92 per cent. Like the results of Laan's test, a consistent pattern exists within the test results suggesting that these minestones are more stable when immersed in sea water than when immersed in distilled water (*see Figure 8.16*). The Hatfield sample showed an improvement of 25 per cent when tested in sea water and the POAB sample improved by 11 per cent, the other samples having improvements between 2 per cent and 6 per cent. Sea water, with its far greater ionic strength, may not disperse the adsorbed cations which surround clay particles to the same extent as distilled water. This would result in the clay minerals having a slightly lower negative charge which produced a weaker repulsive force than was experienced when the immersing fluid was distilled water. The electrolyte rich nature of sea water may have also affected the surface tension of the shales resulting in reduced capillarity. A weaker capillary action is expected to decrease 'air breakage' and also lower the amount of water available for ionic adsorption and interparticle swelling. Osmotic pressures should also be lower when the immersing fluid is sea water and hence osmotic swelling should also be lower.

A direct comparison between the results of the modified slake durability test (MST) and the standard slake durability test (SST) is limited by the disparity between the exposure periods. A comparison can be made however, if the test periods are equalised. The standard test time for the SST is 10 minutes, but for the purpose of comparison this was increased to 60 minutes. As the data in *Table 8.12* and *Figure 8.17* indicate the MST, using grains between 5mm and 10mm in size, gave constantly lower values for slaking resistance than the SST which used larger lumps of material.

A much larger surface area is present if smaller grains are used. This presents a far greater absorbent surface through which slaking fluids can enter the shale, thereby leading to an increase in the slaking rate as demonstrated by the results of the modified slaking test. However, larger lumps of rock can be cleaned and prepared more satisfactorily than can smaller clasts. The lower standard of preparation for the smaller particles may also have contributed to the lower MSI values. The concerns over the misinterpretation of lithologies and a lack of larger shale clasts in well weathered minestone (Thomas, 1987) is a valid point. The modified slake durability test is probably the more appropriate slaking test to use with weathered spoil but the standard test, extended to 60 minutes and controlled by careful selection of shale clasts, would probably give more representative results. The extended standard test may well be adequate enough to determine the susceptibility of current arisings and carefully sampled spoil.

The study into the pore structure for the minestones was limited. However, the results of the tests were plotted against the standard slaking index (*see Figure 8.18*) where it was observed that all but one of the results are clustered. The Hatfield result appears as an outlier compared to the majority of results. In order to establish whether this was a true representation, additional tests were carried out on a selection of samples with known slaking properties.These further samples included a minestone from Thorne (sister pit to Hatfield), a minestone from a second site at Sutton Manor and an unknown minestone from a previous research study. The results of these tests together with the original results are plotted in *Figure 8.19* where it is observed that Hatfield is not an outlier, and that a sensible relationship exists between pore surface area and the standard slaking index. Hatfield and Thorne were shown to have the greatest propensity to slake and it was these minestones that had the greatest pore surface area per gram. The high pore surface area of the Hatfield sample is caused by the presence of pores that are less than 0.006μm in diameter. None of the other samples had pores that were less than 0.006μm which suggests that the severe slaking suffered by the Hatfield sample was a direct result of the presence of these micropores. Research by Hudec (1989) suggested that a direct relationship existed between pore diameter and rock failure. The present study into the pore structure of minestone, though limited, bears out Hudec's conclusions.

Minestone	Percentage finer than Millimetres				Percentage finer than Microns			Grading Index
	50	**37.5**	**20**	**5**	**600**	**150**	**63**	
Hatfield	95	91	57	17	3	1	0.5	13.94
L.Windsor	90	83	69	33	14	10	9	14.48
POAA	94	87	66	41	28	23	19	14.16
POAB	82	80	59	17	6	4	3.5	18.48
Sutton Manor	88	78	55	16	6	3	2	19.17
Wardley	100	94	70	37	10	3.7	2	10.38
Wearmouth	100	100	74	15	5	4	2.5	15.00
Westoe	72	64	50	23	11	4	1	12.93

Table 8.1 Grading characteristics of the sampled minestone.

Minestone	As Sampled M/C (%)	Air Dried M/C (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)
Hatfield	13.0	7.5	33.0	19.0	14.0	8.6
Lady Windsor	7.0	3.0	27.9	20.2	7.7	4.3
POAA	14.0	11.5	33.4	23.4	10.0	6.3
POAB	7.2	3.0	27.5	17.9	9.6	4.3
Sutton Manor	7.5	2.2	30.4	22.2	8.3	5.0
Wardley	-----	2.7	36.0	22.0	14.0	5.7
Wearmouth	4.8	2.2	32.2	24.1	8.1	5.0
Westoe	7.6	3.5	31.6	21.0	10.6	6.4

Table 8.2 Consistency characteristics of the sampled minestones.

Minestone	Loss on Ignition (815^{0}C) (%)	Specific Gravity	Optimum M/C (%)	Maximum Dry Density (Mg/m^3)
Hatfield	13	2.57	13	2.16
L. Windsor	25	2.41	8	2.12
POAA	17	2.15	14	1.80
POAB	16	2.53	10	2.08
Sutton Manor	23	2.40	8	2.12
Wardley	16	2.29	9.5	2.06
Wearmouth	15	2.48	8	2.16
Westoe	16	2.53	10	2.07

Table 8.3 Some physical properties of the sampled minestones.

Minestone	pH Value (Distilled)	pH Value (Sea water)	Sample + 10% Cement (Distilled)	Sample + 10% Cement (Sea Water)
Hatfield	8.7	7.9	12.4	12.0
L. Windsor	7.1	7.5	12.3	11.9
POAA	6.5	7.2	11.7	11.2
POAB	8.8	7.7	12.4	12.0
Sutton Manor	9.1	8.5	12.4	11.9
Wardley	9.4	8.4	12.4	11.9
Wearmouth	10.1	8.3	12.4	11.9
Westoe	8.5	7.8	12.4	12.0

Table 8.4 The pH of minestone under different environmental conditions.

Minestone	Cation Exchange Capacity (meq/100g)			Sulphate Content (% SO_3)	
	1.2mm-425μm	425μm-63μm	Sub 63μm	Acid soluble	Water soluble
Hatfield	28.3	29.4	32.7	0.21	0.14
L. Windsor	36.4	41.3	47.4	0.43	0.28
POAA	19.1	29.4	27.4	1.20	0.42
POAB	22.5	26.2	31.5	0.36	0.15
Sutton Manor	22.2	18.4	24.2	0.07	0.02
Wardley	10.3	17.5	16.4	0.25	0.11
Wearmouth	6.2	12.4	12.9	0.05	0.01
Westoe	12.5	19.9	15.6	0.09	0.04

Table 8.5 Some chemical aspects of the test minestones.

Minestone	Lithology (% weight)				
	Mudstone	Sandstone	Siltstone	Pyrite/ Ironstone	Coal
Hatfield	61	18	15	4	2
L.Windsor	61	7	1	24	7
POAA	73	3	2	7	15
POAB	28	16	36	16	4
Sutton Manor	72	8	12	1	7
Wardley	66	15	12	---	7
Wearmouth	34	27	36	1	2
Westoe	54	18	15	4	2

Table 8.6 The lithological composition of the test minestones.

Minestone	Cation Exchange Capacity of Sample Fractions							
	1.2mm-425μm		425-63μm		Sub 63μm		Exchange Capacity	
	% Retained	$Ca(OH)_2$ mg /100g	% Retained	$Ca(OH)_2$ mg /100g	% Retained	$Ca(OH)_2$ mg/100g	$Ca(OH)_2$ mg/100g	$Ca(OH)_2$ (%)
Hatfield	5.8	60.7	1.9	20.5	0.4	4.7	85.9	0.09
L.Windsor	9.8	131.3	4.2	64.7	8.6	150.5	346.5	0.35
POAA	7.9	55.5	7.5	81.9	19.3	195.3	332.7	0.33
POAB	6.1	51.0	1.5	14.3	3.5	40.8	106.1	0.11
S. Manor	5.5	45.1	4.1	27.9	2.4	21.3	94.3	0.09
Wardley	21.1	80.3	21.0	135.7	2.4	14.7	230.7	0.23
W'mouth	9.8	22.9	3.2	14.7	4.2	20.1	57.7	0.06
Westoe	10.6	49.2	3.4	25.0	4.0	22.8	97.0	0.10

Table 8.7 Cation exchange capacity in terms of equivalent calcium hydroxide content.

Minestone	Grading Index			
	Original	8th cycle	16th cycle	24th cycle
Hatfield	13.94	14.29	14.80	14.92
Lady Windsor	14.48	13.92	15.92	15.40
POAA	14.16	14.06	18.39	13.55
POAB	18.48	16.59	20.55	19.01
Sutton Manor	19.17	14.66	16.47	17.11
Wardley	10.38	8.15	7.62	7.03
Wearmouth	15.00	17.68	14.70	17.57
Westoe	12.93	11.60	12.92	11.64

Table 8.8 Tidal simulation results.

Minestone	Grading Index				
	Original	1st cycle	2nd cycle	3rd cycle	4th cycle
Hatfield	13.94	9.28	8.29	5.87	4.82
Lady Windsor	14.48	13.15	12.79	7.63	6.62
POAA	14.16	7.60	7.02	5.36	4.68
POAB	18.48	14.92	12.48	12.13	11.18
Sutton Manor	19.17	16.03	10.31	11.36	9.65
Wardley	10.38	7.35	5.32	5.37	4.67
Wearmouth	10.38	9.73	7.43	6.78	6.42
Westoe	12.93	10.12	8.25	8.14	5.40

Table 8.9 Cyclic wetting and air drying test results.

Minestone	Immersing Fluid			
	Distilled		Sea Water	
	Air dried	As dug	Air dried	As dug
Hatfield	7.52	9.00	7.57	9.00
L. Windsor	8.00	9.00	8.17	9.00
POAA	7.86	9.00	8.57	9.00
POAB	8.70	9.00	8.88	9.00
Sutton Manor	8.81	9.00	8.94	9.00
Wardley	8.74	---	8.83	---
Wearmouth	8.86	9.00	8.91	9.00
Westoe	8.40	9.00	8.71	9.00

Table 8.10 The results from Laan's rate of fineness test for air dried minestone and minestone that has retained its 'as dug' moisture content.

Minestone	Sea Water			Distilled Water		
	MSI (%)	Air Dried m/c %	Post Test m/c %	MSI (%)	Air Dried m/c %	Post Test m/c %
Hatfield	77	4.0	6.9	52	4.0	8.4
L. Windsor	86	1.2	4.5	84	1.2	4.0
POAA	82	2.7	6.9	77	2.7	6.3
POAB	93	1.7	5.6	82	1.7	6.6
Sutton Manor	94	1.2	3.6	92	1.2	4.0
Wardley	87	2.6	3.3	85	2.2	4.5
Wearmouth	91	1.0	6.3	86	3.1	6.8
Westoe	86	1.0	5.4	84	2.5	5.4

Table 8.11 The results of the modified slake durability test.

Minestone	Slake Durability Test	
	Modified Slake Test	Standard Slake Test
Hatfield	52	58
L. Windsor	84	88
POAA	77	82
POAB	82	87
Sutton Manor	92	94
Wardley	85	92
Wearmouth	86	91
Westoe	84	91

Table 8.12 A comparison between the modified slake durability test and the standard slake durability test run for 60 minutes.

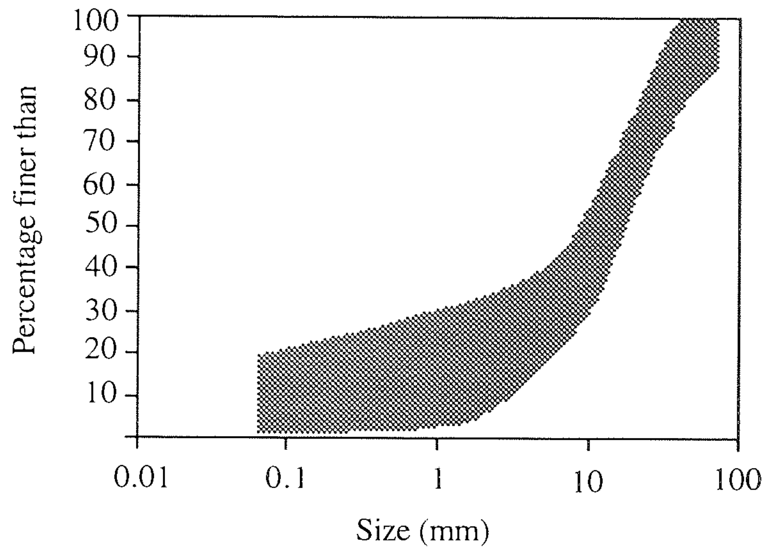


Figure 8.1 Grading envelope for the samples used in the present research.

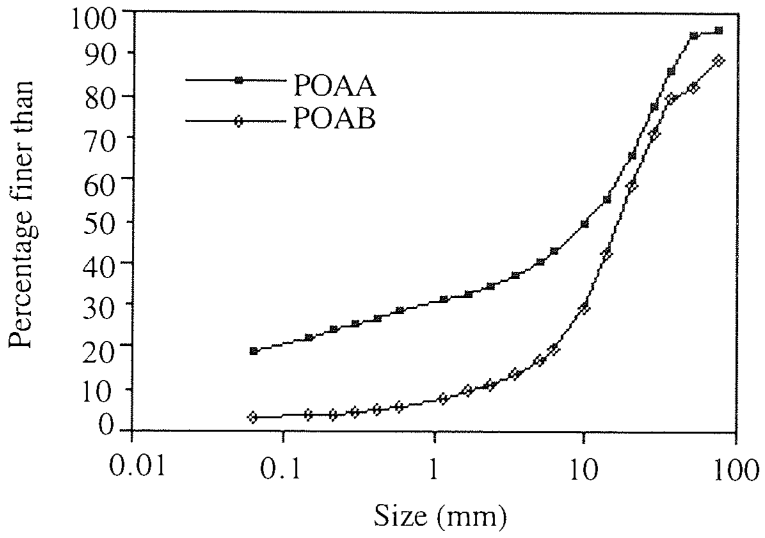


Figure 8.2 Grading curves for the two Point of Ayr samples.

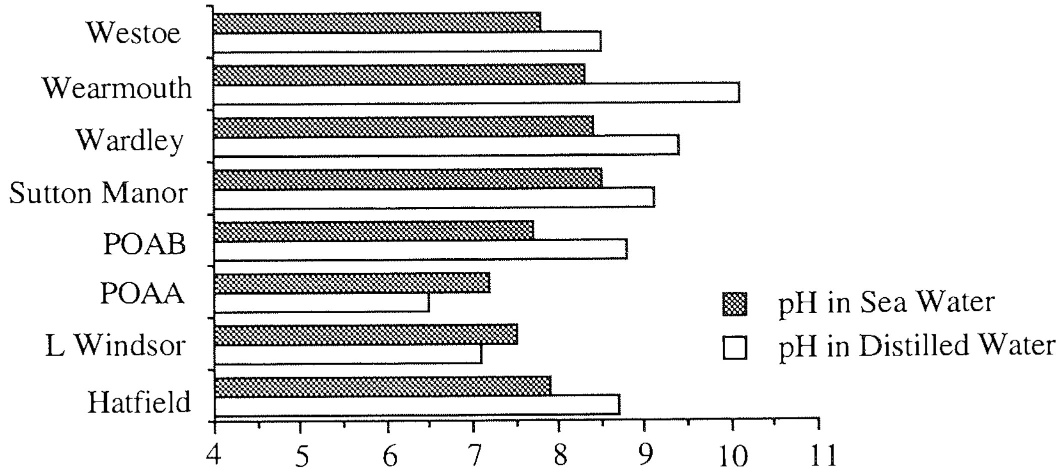


Figure 8.3 A comparison between the pH of minestone immersed in distilled water and in sea water.

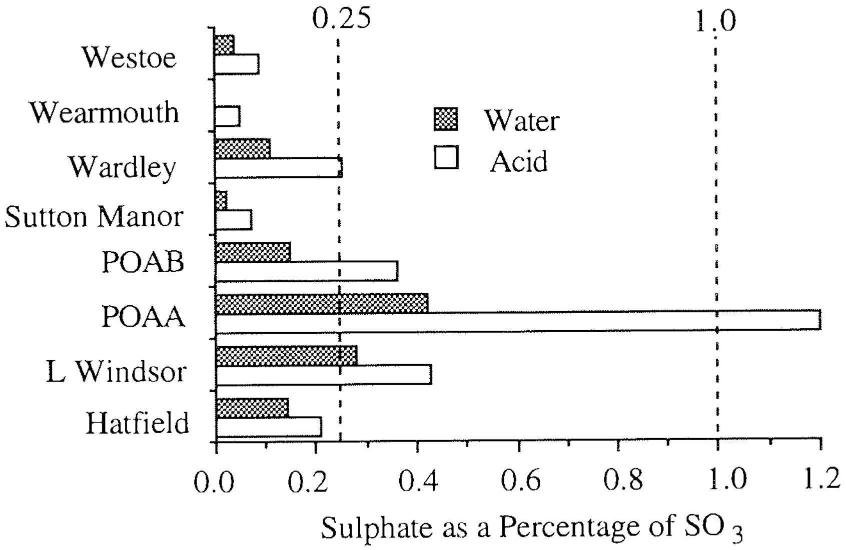


Figure 8.4 Acid and water soluble sulphate content of the test minestones.

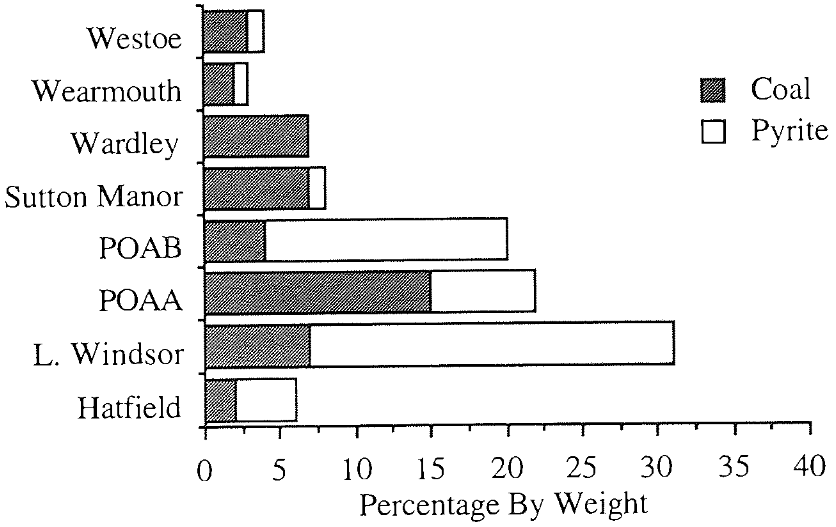


Figure 8.5 The percentage of coal and ironstone/pyrite in the test minestone.

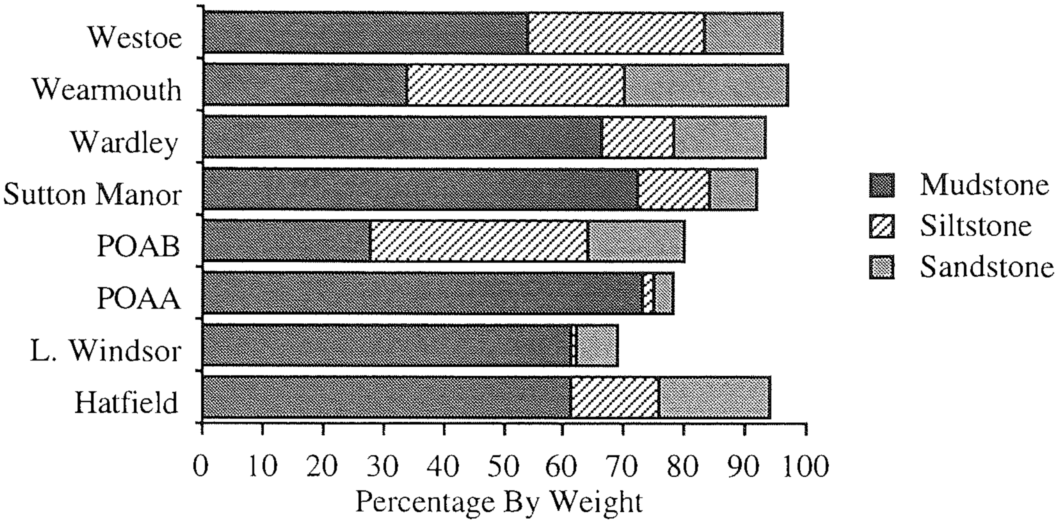


Figure 8.6 The proportion of the main lithological groups in the test minestones.

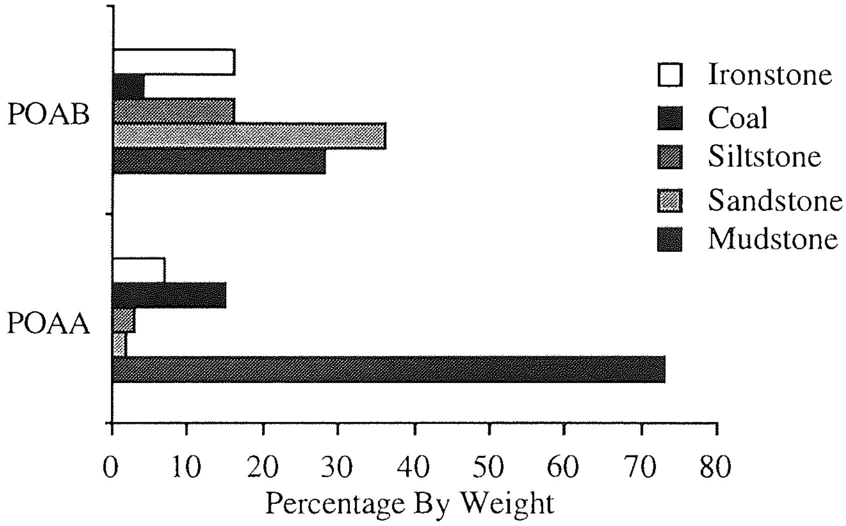


Figure 8.7 A comparison of the lithological composition of the Point of Ayr samples.

Figure 8.8 The relative proportions of the main clay mineral groups in the test minestones.

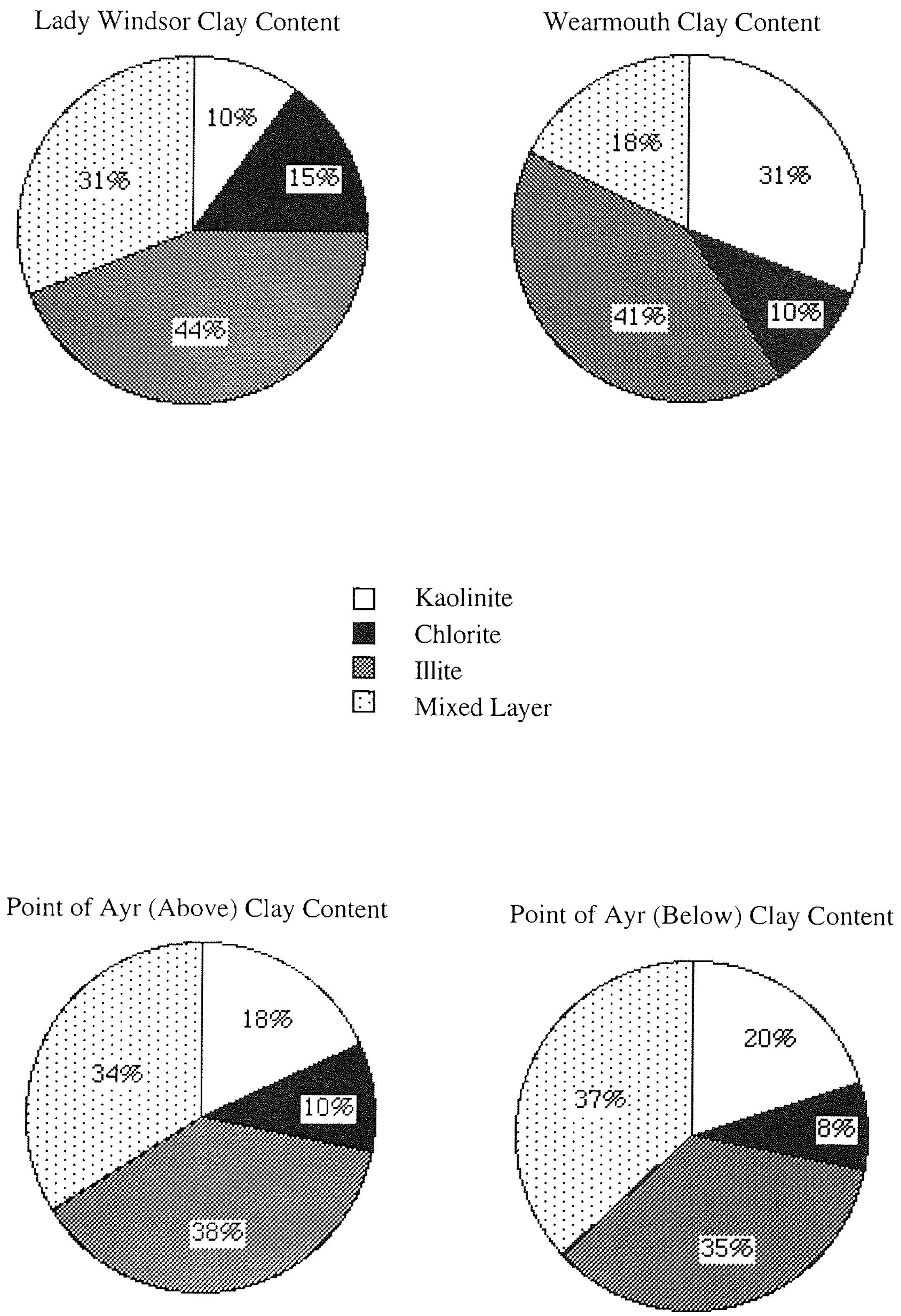


Figure 8.8 (continued)

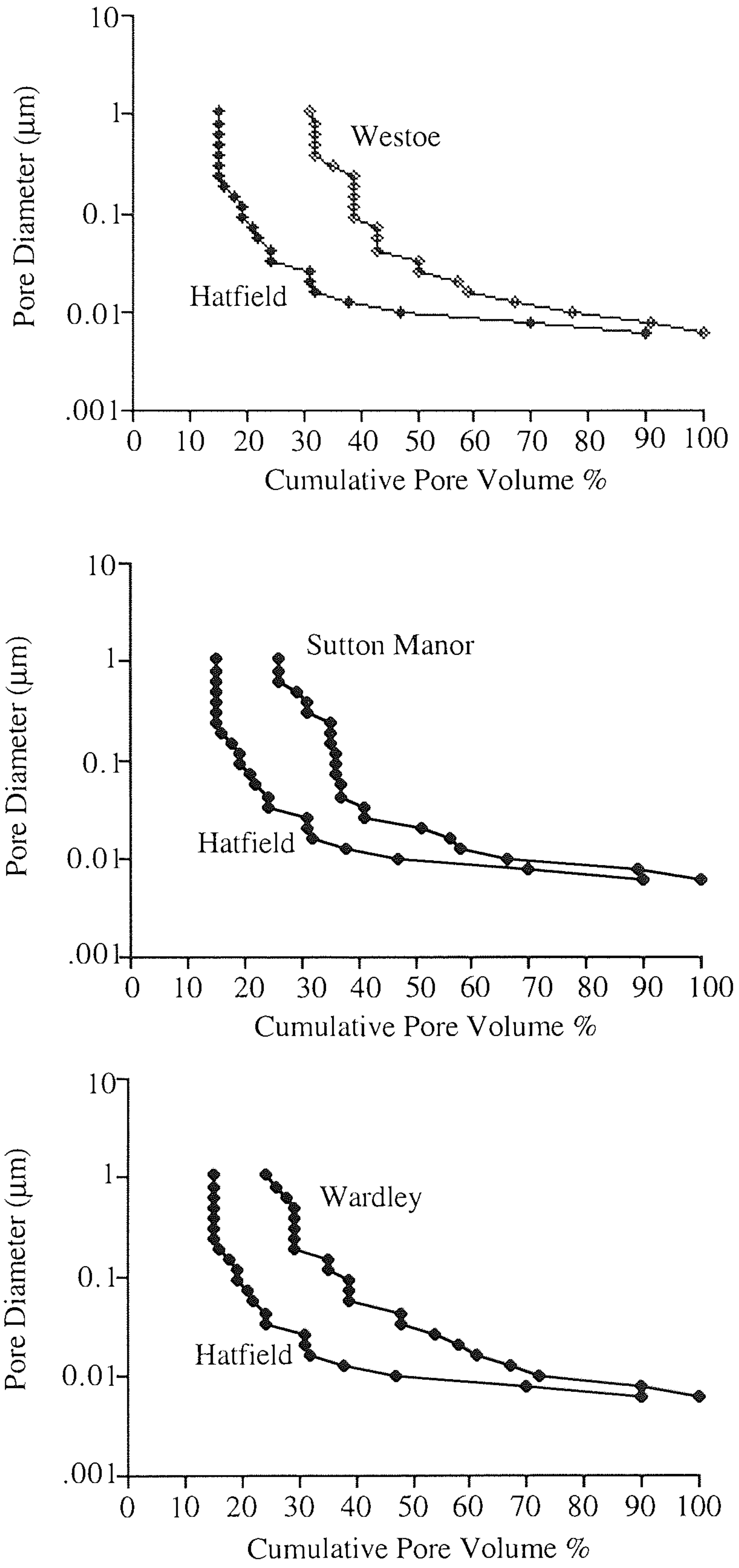
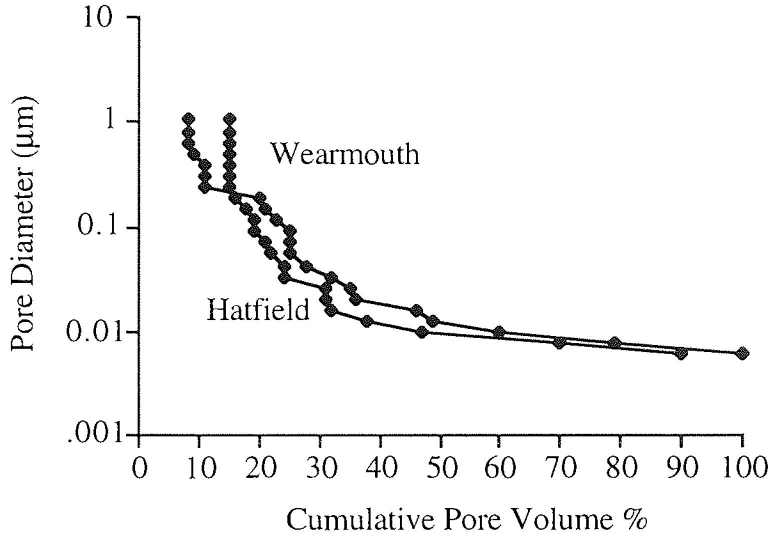
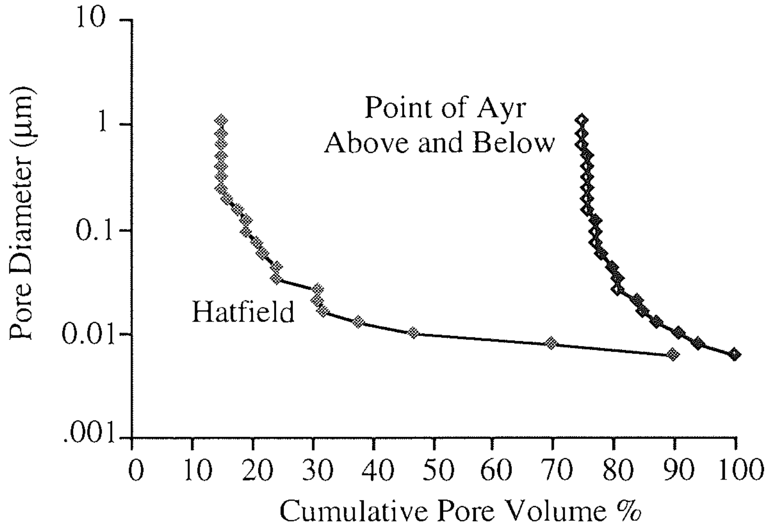


Figure 8.9 A comparison between the cumulative pore volume curve for Hatfield and the other test minestones.





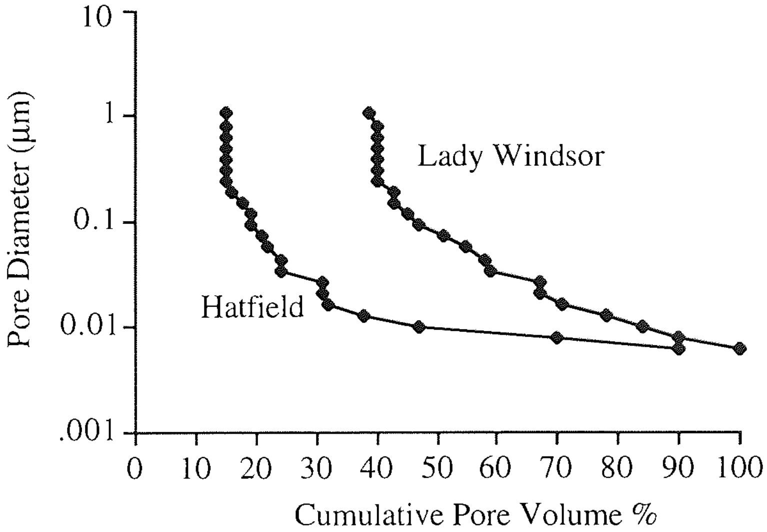


Figure 8.9 (continued)

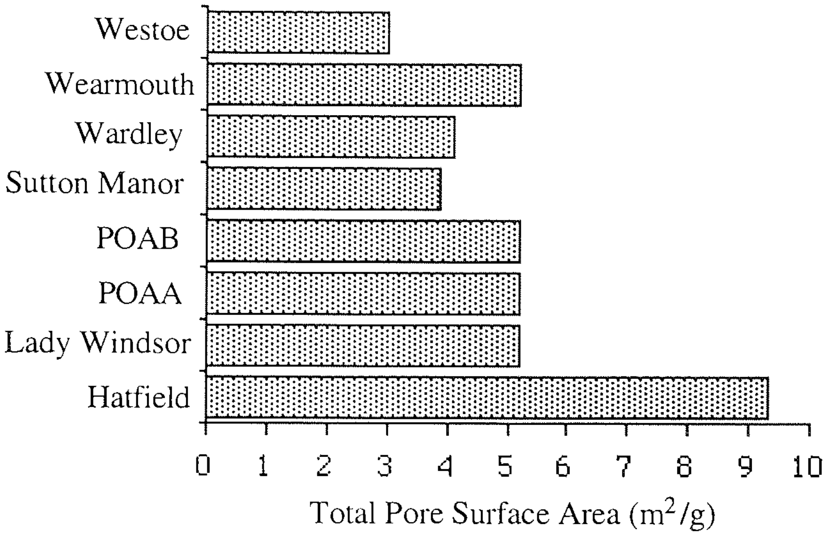


Figure 8.10 Total pore surface area as revealed by porosimetry analysis.

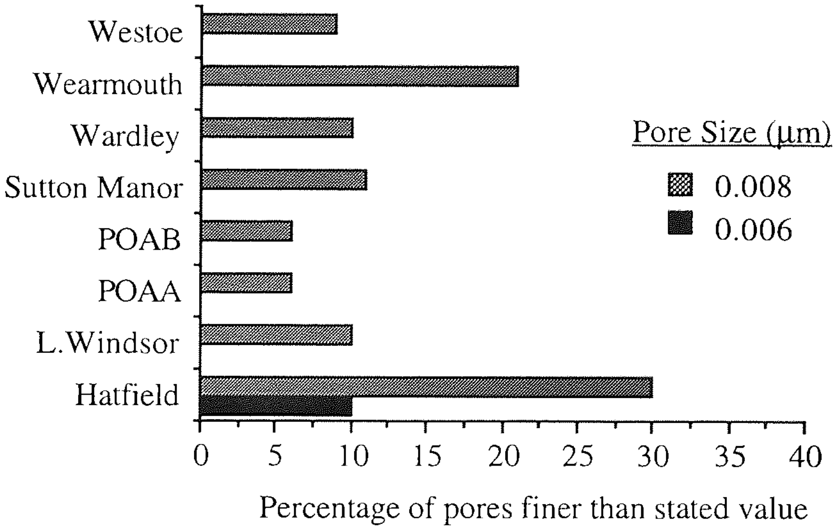


Figure 8.11 Distribution of micropores within the sample minestones.

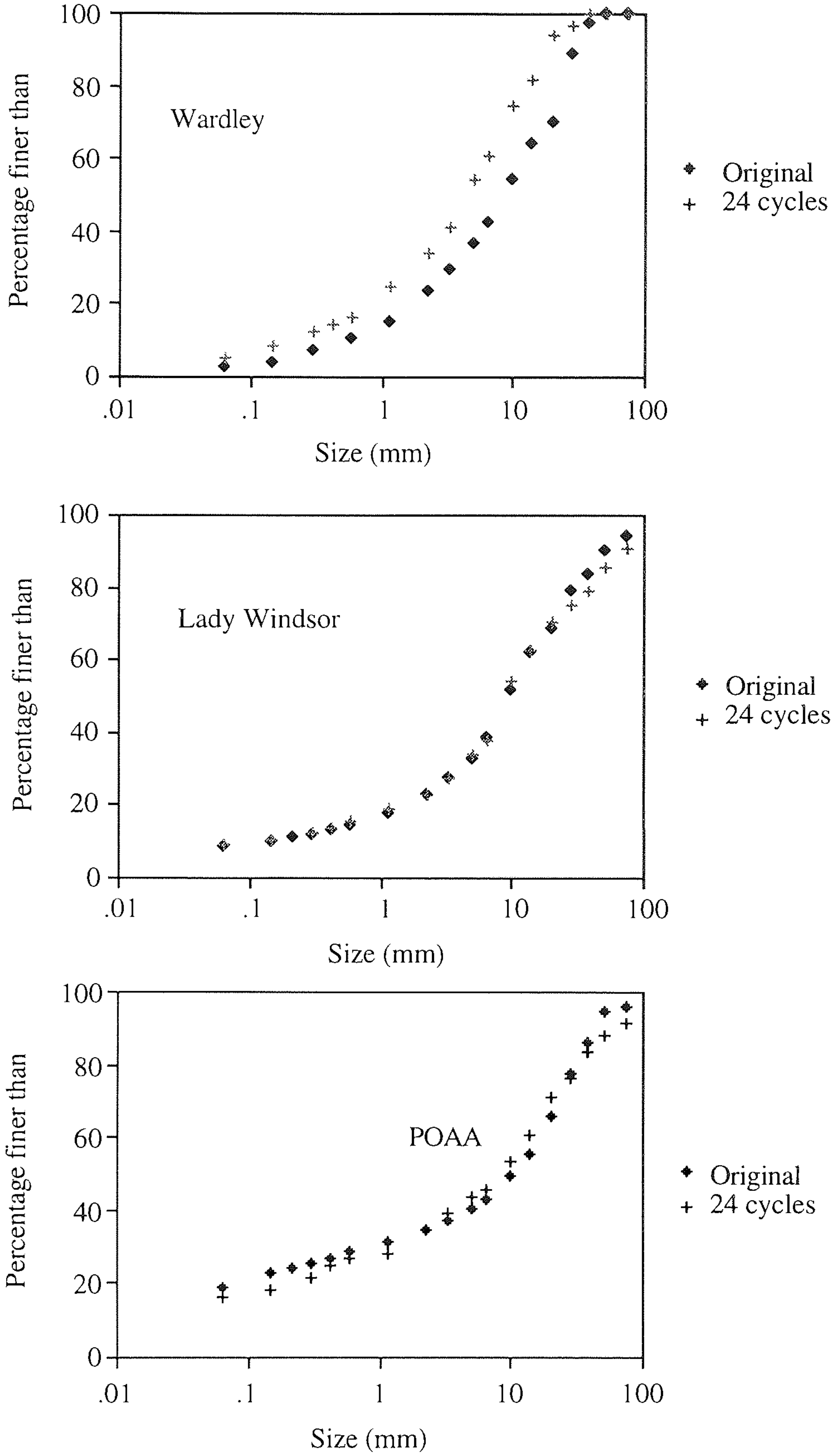


Figure 8.12 Tidal simulation upon the test minestones

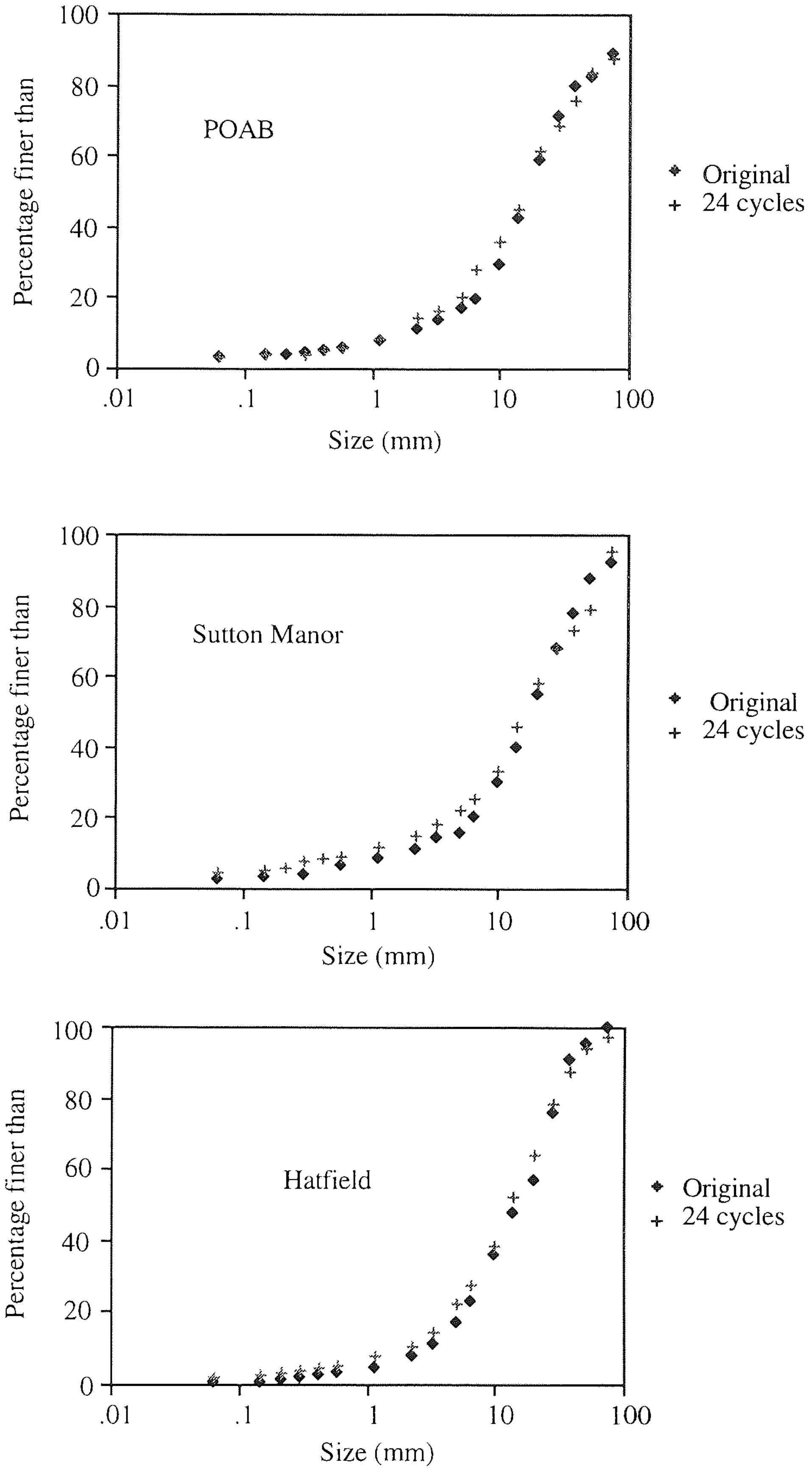
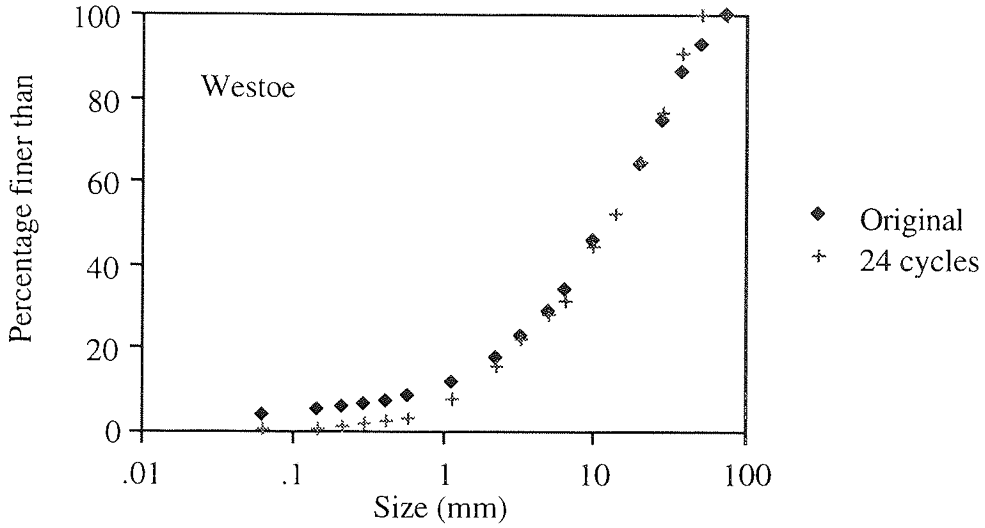


Figure 8.12 (continued)



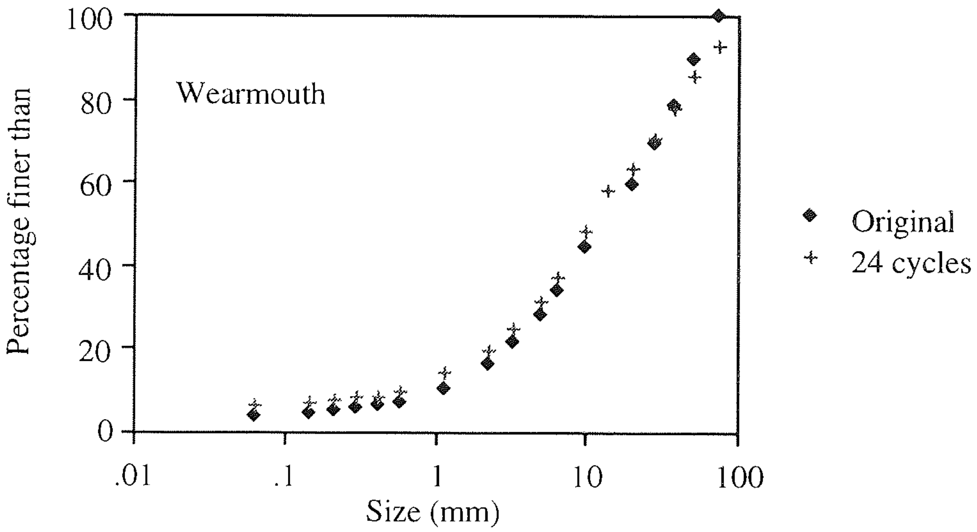


Figure 8.12 (continued)

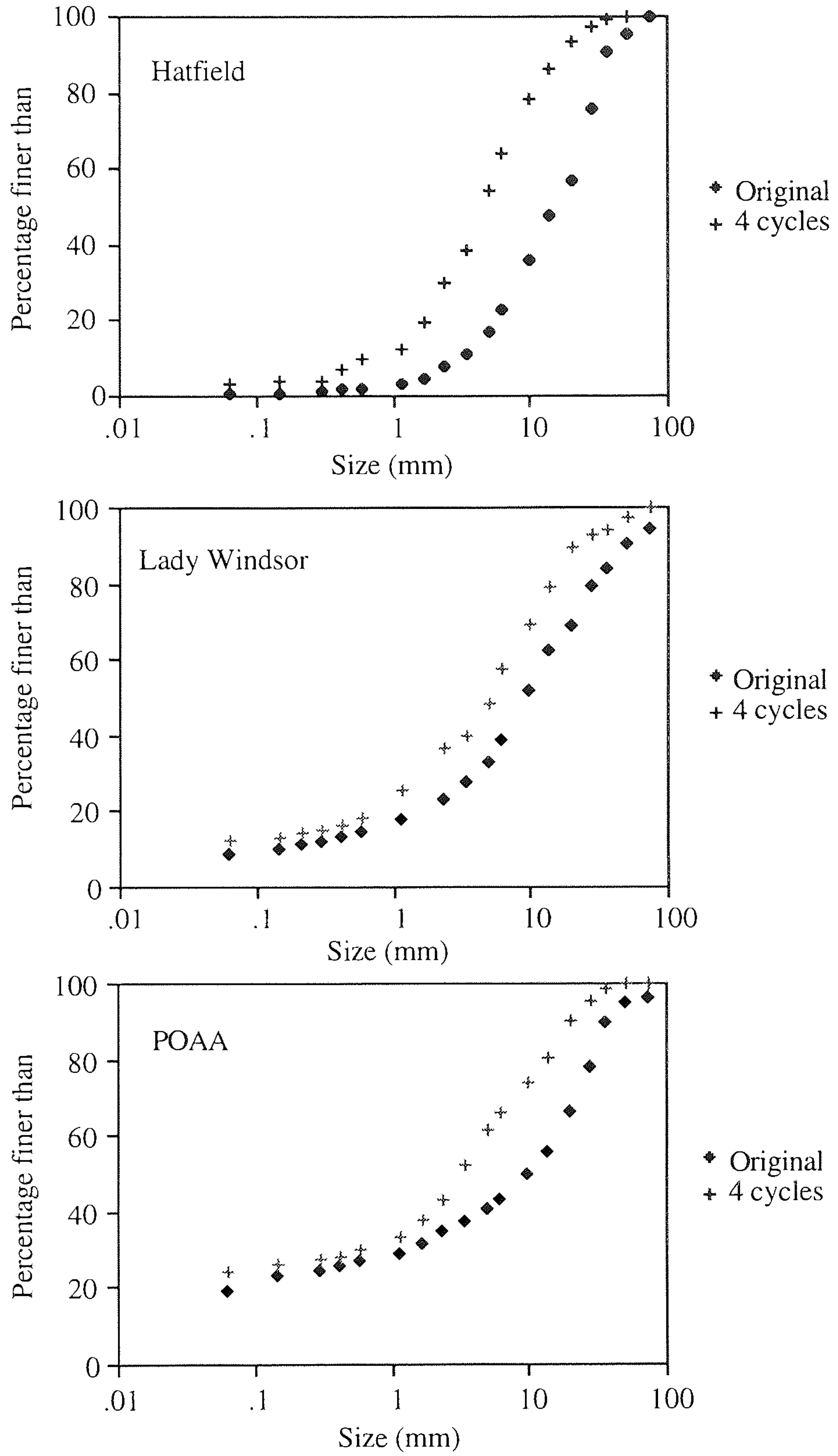


Figure 8.13 The effects of wetting/air drying cycles on grain size distribution.

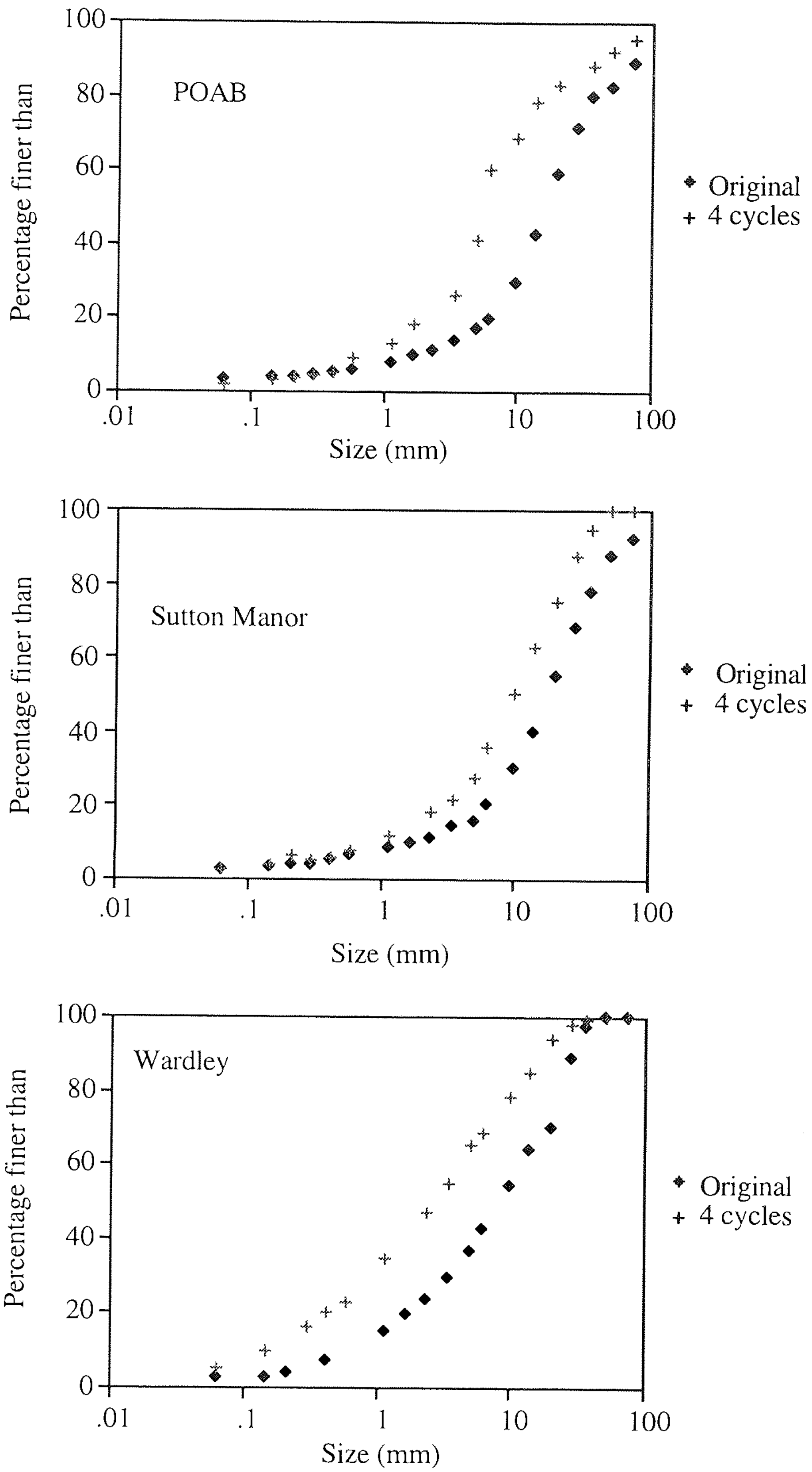
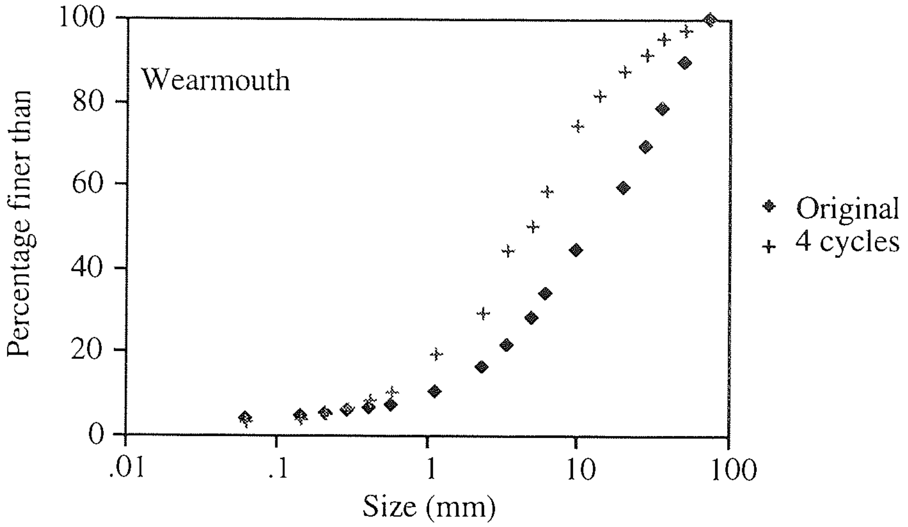


Figure 8.13 (continued)



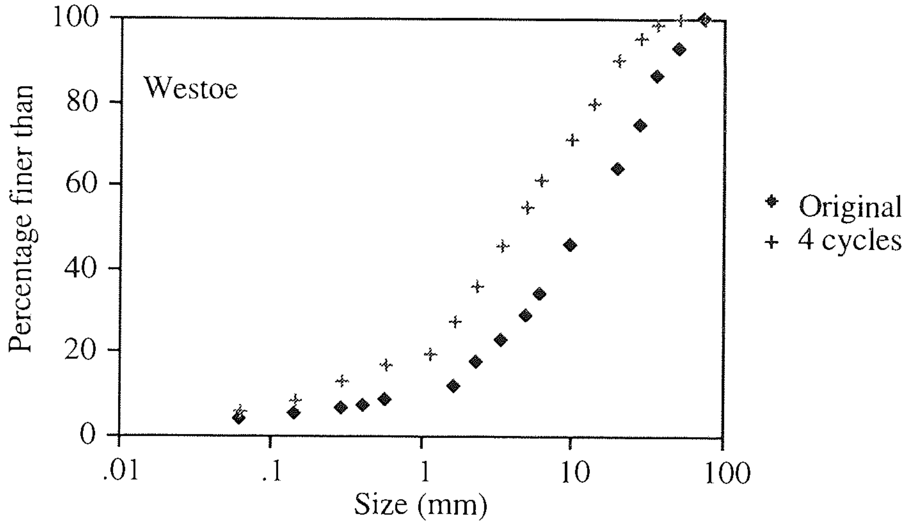


Figure 8.13 (continued)

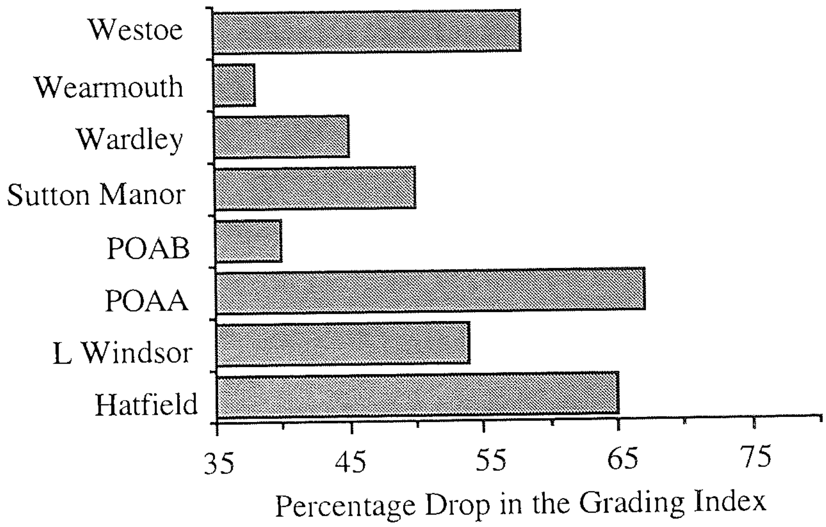


Figure 8.14 The percentage drop in the grading index following four cycles of wetting and air drying.

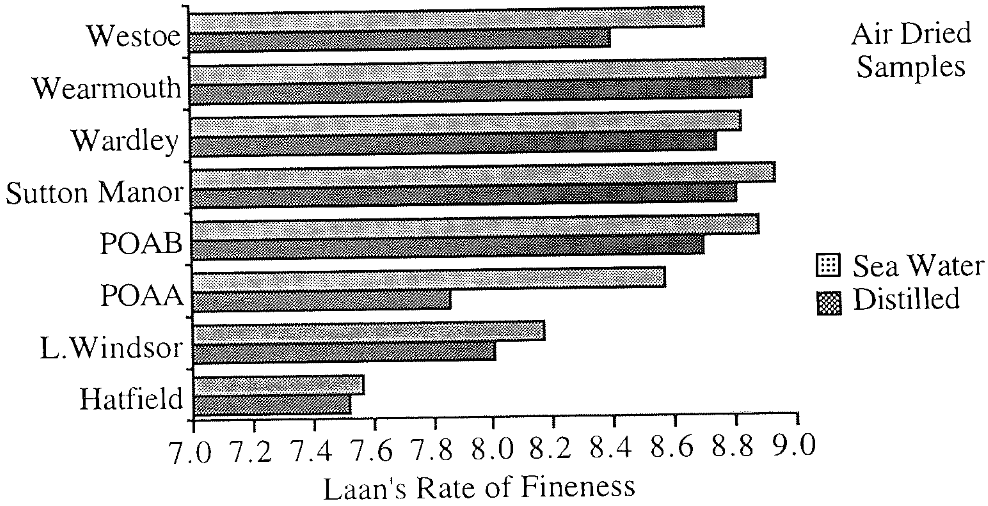


Figure 8.15 Laan's rate of fineness results.

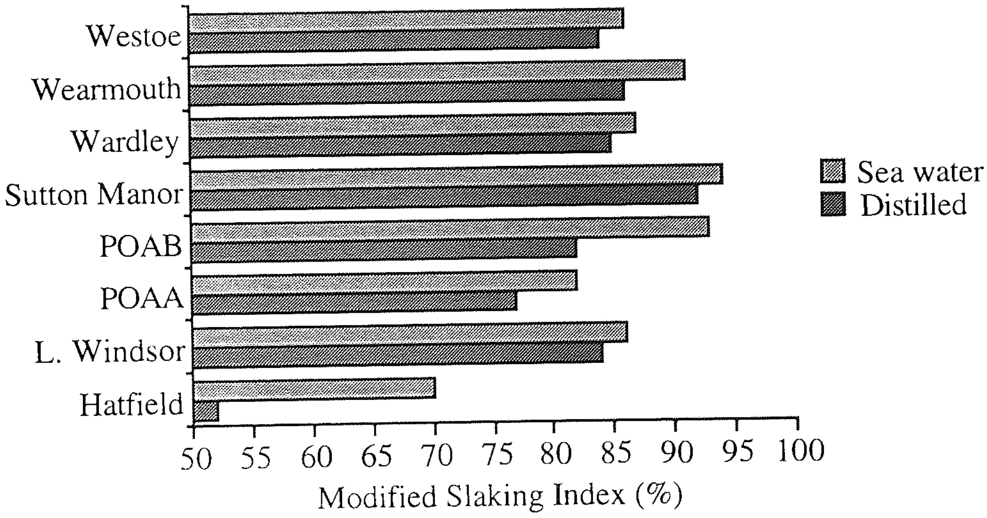


Figure 8.16 Modified slake durability test results.

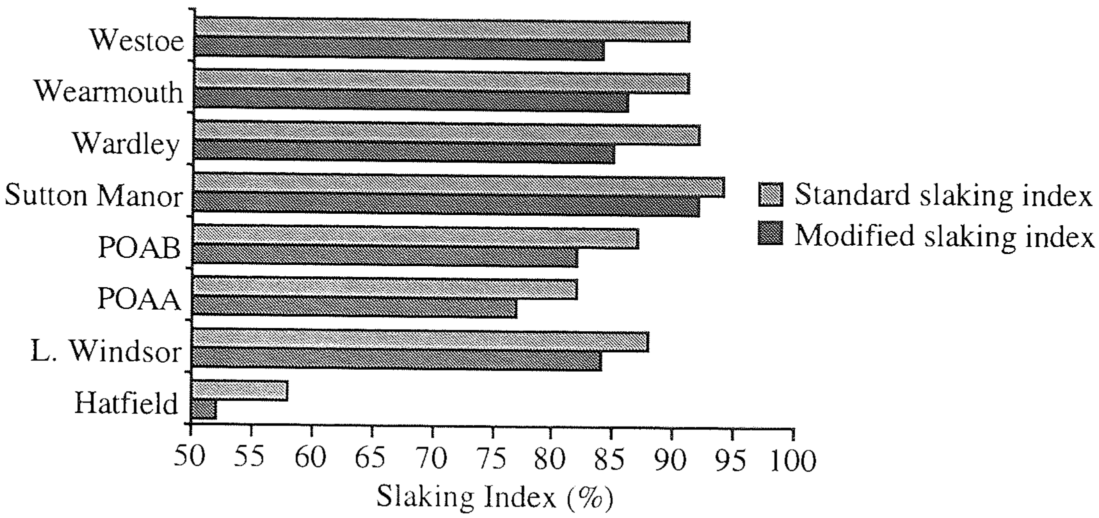


Figure 8.17 A comparison between the modified slake durability test and the standard slake durability test run for 60 minutes.

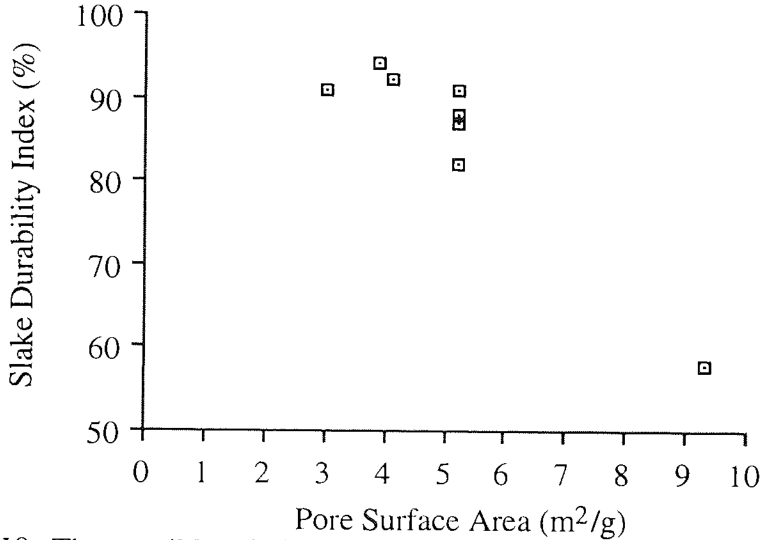


Figure 8.18 The possible relationship between the slake durability index and the total pore surface area based on limited results.

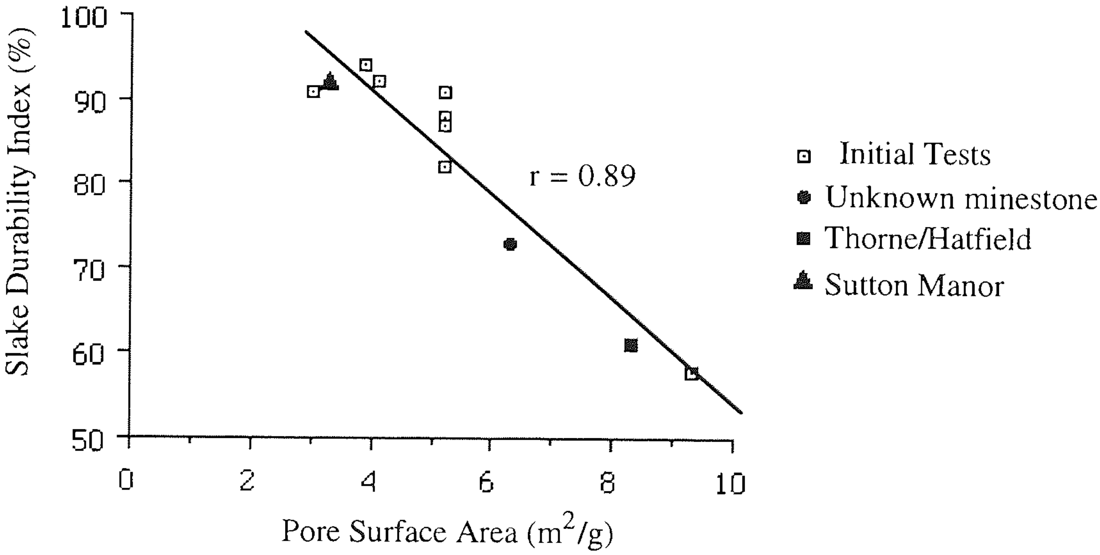


Figure 8.19 The relationship between the slake durability index and the total pore surface area based on an extended series of results.

9. ANALYSIS AND DISCUSSION OF THE STRENGTH AND IMMERSION DURABILITY RESULTS FOR CSM

9.1 Strength Requirements

The requirements relating to cement stabilized materials, as laid down by the Department of Transport (Clause 1036, 1986), stress that two particular strength criteria must be complied with. These properties are the 7 day unconfined compressive strength, with a minimum average strength requirement of 4.5MPa, and minimum 7 day strength for each cube of 2.5MPa. In addition it is essential that the material retains a minimum of 80 per cent of its air cured strength, following a 7 day period of immersion in water, this criterion is usually expressed as an Immersion Ratio.

9.2 Compaction Characteristics

The compaction properties of the cement stabilized minestone samples were given in *Table 8.3*. No clear relationship was shown to exist between the Atterberg limits and the optimum moisture content of the samples, though relationships between these parameters have been established for soils (Lo and Lovell, 1983) and have also been verified for minestone during a study involving a more variable range of materials than has been used in the present study (Thomas, 1987). The relationship between air dry moisture content and the optimum moisture content, established by Thomas (1987), has been verified in this study. When these two moisture properties are plotted against one another a positive correlation of 0.87 is shown to exist (*see Figure 9.1*).

The high optimum moisture content for the POAA sample can be attributed to its fines content (sub 63μm) of 19 per cent, but the relatively high optimum moisture content of the Hatfield sample does not correlate with a high fines content, as the sub 63μm content of that material is less than 1 per cent. The high optimum moisture and air dried moisture contents can be explained by the nature of the pore structure.The mercury intrusion porosimetry data demonstrate that it has a higher percentage of finer pores than the other samples. These pores act in a similar way to the high 'fines' content of the POAA sample, in that the large surface area they provide gives a higher potential for water retention. These pores initially absorb a high percentage of mix moisture to achieve saturation before sufficient water is able to coat the exterior of the minestone grains. As *Figure 9.2* indicates, a relationship also exists between the 'as dug' moisture content and the optimum moisture content. Again the POAA and Hatfield samples have the potential for absorbing a far greater percentage of water as a result of the higher surface areas associated with the finer particles (POAA sample) and the pore structure and distribution (Hatfield mudstone).

Figure 9.3 illustrates the relationship between the optimum moisture content and the maximum dry density of the samples. However, this plot is only significant if the results of the Hatfield sample are not included in the analysis for, although the optimum moisture content is high, the maximum dry density is much higher than would normally be expected. The internal hygroscopic requirement of the Hatfield material is high as a result of the pore structure. This internal water will not affect the compaction characteristics of the minestone as it is effectively bound within the internal structure. It is the free moisture that affects the compaction properties of the material. Though the apparent optimum moisture is high for Hatfield, the free moisture content is low, hence allowing a relatively high maximum dry density to be achieved.

In previous studies it has been shown that higher optimum moisture contents are generally associated with lower maximum dry density values (McNulty, 1985; Rainbow, 1986, Thomas, 1987). This has also been found to be true in this study, with the POAA sample possessing a high optimum moisture content and a low dry density (1.80Mg/m^3). It is also apparent that the coarser minestones have lower optimum moisture contents and higher dry densities compared to those associated with the finer minestones.

9.3 Short Term Strength and Durability Results

9.3.1 Unconfined Compressive Strength

A preliminary strength study was conducted prior to the erosion/abrasion programme to determine the quantity of cement needed to achieve the specified strength and durability values. Three cement contents were chosen, these being 5, 7.5 and 10 per cent ordinary portland cement. Previously it has been stated that a 10 per cent cement content would be the maximum quantity for economic viability (Kettle and Williams, 1977), though larger quantities have been used for stabilized soil structures for hydraulic projects in the United States (ACI, 1990). As can be seen from *Table 9.1*, none of the samples passed the 7 day compressive strength specification of 4.5MPa when stabilized with 5 per cent OPC, the highest strength being exhibited by the POAB at 3.3MPa. Interestingly the weakest sample was POAA with a 7 day strength of only 1.4MPa, this disparity between the two Point of Ayr samples is of note considering that they were taken from less than 10 metres apart. However, wave modification and tidal flushing of the POAB material can explain the difference in the strength properties of the two samples, for by these processes the bulk of the finer particles would have been removed and the percentage of weaker particles would have decreased as a result of the abrasive and slaking action of the sea.

Only the POAB and Sutton Manor samples pass the 7 day strength specification when stabilized with 7.5 per cent OPC. Again the POAA sample is the weakest attaining a strength of 2.1MPa after 7 days. At a cement content of 10 per cent, only the POAA (4.04MPa) and Westoe (4.17MPa) samples fail to gain the specified 7 day strength of 4.5MPa though, when sea water is used for the mix, the Westoe sample achieves a strength of 4.82MPa (*see Table 9.2 and Table 9.3*). It has been suggested that an average 7 day compressive strength of 5.2MPa is required to ensure that the specified minimum strength is satisfactorily exceeded under field conditions (Williams, 1986). This was suggested when the specified 7 day strength was 3.5MPa, so the suggested value of 5.2MPa could be considered to be inadequate to ensure a minimum value of 4.5 MPa.

For hydraulic applications, strength is thought to be subsidiary to durability (Litton and Lohnes, 1982) though it is accepted that the two are closely related. The use of cement stabilized soils for hydraulic purposes has been limited in the UK and so very little guidance has been issued. The United States, however, has extensively used soil-cements for hydraulic applications and have specified accordingly. The United States Bureau of Reclamation has specified a 7 day strength for soil-cement dam facings and channel linings of 4.1MPa and a 28 day strength of 6.03MPa. With the exception of the POAA sample, which has a borderline value of 4.03MPa, the USBR 7 day strength criterion has been attained by all the samples when stabilized with 10 per cent OPC (*see Figure 9.5*) and the 28 day USBR strength specification is achieved by all the samples (*see Table 9.2*). Only 7 day strength values were determined for minestone stabilized with 5 and 7.5 per cent OPC. With 7.5 per cent cement, only the POAB and the Sutton Manor samples achieve the specified USBR strength values. When stabilized with 5 per cent OPC, none of the samples gained sufficient strength to pass this 7 day strength criterion.

The usual practice of the USBR requires the field cement content to be increased by 2 per cent above the minimum amount required to achieve the specified strength to account for variations at the site of operation. If this was to be applied to the samples used in this study, the POAB and Sutton Manor would require cement contents of 9.5 per cent whilst the remaining samples would need 12 per cent to satisfy the soil-cement specifications and the current practices of the USBR.

Vast experience has been gained in the USA relating to the use of soil-cement in hydraulic applications, for in addition to the activities of the PCA, USBR and USACE, individual county authorities have had a considerable influence on the specific quality of soil-cement mixes for such purposes. An example is Pima County, Arizona where a 7 day strength of 5.2MPa is specified and it is frequently assumed that a strength of 6.9MPa can be achieved once the additional 2 per cent cement has been added to the minimum cement content (ACI, 1990). If these values were to be applied to the minestone researched in this study, only the POAB and Sutton Manor material would be permitted for soil-cement mixes.

It must be stressed that generally, the most appropriate way to test the compressive strength of soil-cement mixes is to soak the samples prior to testing (ACI, 1990). This is only a recommendation and not a requirement and it can be assumed that the specifications relate to both unsoaked and soaked materials. The minestone samples used in the present study were not soaked prior to testing so the strength values for these samples are likely to be higher than if they had been soaked prior to testing (*see Section 8.3* on strength loss due to excess moisture). An approximation to the 7 day soaked strength may be obtained by using the immersion ratio as a correction factor. If this is done only the Wearmouth, Sutton Manor and POAB samples pass the USBR minimum 7 day strength specification. It is clear therefore, from this initial assessment that the selected minestones can produce CSM mixes, some of which satisfy the North American specifications while others would be considered unsuitable. Clearly this provides a broad basis on which to judge the overall performance of CSM for hydraulic structures.

The initial study also included an assessment of the effects of using sea water as the mixing fluid compared with the use of distilled water. Only minestones from Wardley, Wearmouth and Westoe were tested, the results are given in *Table 9.3* and illustrated in *Figure 9.4*. Generally the compressive strength is higher for the stabilized minestones prepared with distilled water than sea water, though the differences are usually small. With conventional concrete, the use of sea water has been reported (Neville, 1981) to have had very little affect on the 28 day strength of the material. The principal effect that sodium/magnesium chloride and magnesium sulphate has on concrete in the early stages of hardening is to increase the early strength so that the 7 day strength is achieved within 3 days (Steinour, 1960). Although early strength development is high when sea water is used as the mixing fluid, long term strength is generally lower, though the strength loss is usually no greater than 15 per cent (Neville, 1981). After 35 days, the Wardley samples with sea water exhibited a 4 per cent lower strength than samples made with distilled water. The Wearmouth samples had 12 per cent lower strength values whilst the Westoe samples showed a slightly enhanced strength (2 per cent) with sea water.

9.3.2 Immersion Strength

The results of the immersion tests carried out on minestone samples stabilized with 10 per cent OPC are given in *Table 9.4*. Both sea water and distilled water were used as the immersion medium for the samples. The DoT specification (1986) which requires a minimum immersion index of 80 per cent, is exceeded by Wearmouth, POAB, Sutton Manor and Westoe when immersed in both sea water and distilled water. The Wardley sample attained the specified immersion strength in sea water, whilst in distilled water it achieved an immersion index of only 74 per cent. The results of the immersion tests are illustrated in *Figures 9.6* and *9.7*, while *Figure 9.8* compares the performance of the CSM immersed in sea water and distilled water.

The low immersion index of the POAA sample can be explained by its relatively high fines content (19 per cent) which can be detrimental to stabilization in several ways. It leads to a low dry density which, ultimately, reduces the strength of the cement stabilized minestone. There is a propensity for the clay particles to form aggregations that impair the efficiency of achieving a uniform cement distribution. Finer material will generally have a greater effective surface area which will ultimately require a higher percentage of cement to stabilize the material. Croft (1967) also noted that lime depletion may occur as a result of the pozzolanic reactions that occur between the calcium hydroxide released by cement hydration and the clay fraction of the stabilized material. However, the POAA sample, stabilized with 10 per cent OPC, is unlikely to be affected by lime depletion (*see Section 8.4.3*). The low immersion strength of the POAA sample is mainly due to its low density value combined with the deleterious effects of the characteristic aggregations of agglutinated 'fines' that were apparent in the original sample.

Thomas (1987) demonstrated that a relationship exists between the immersion ratio and the slaking index of the minestone. The higher the slaking index of the raw minestone, the greater the immersion resistance of the stabilized minestone. Indeed, the low immersion ratio of the Hatfield sample can be correlated with the severe slaking properties of this particular minestone. The weakening of the CSM fabric as a consequence of the slaking of the minestone component occurred despite attempts to precondition the minestone by adding moisture several days in advance of mixing with cement. Thomas (1987) suggested that, once the cement was added to the mix, the minestone entered into competition with the cement for the water, thus altering the hygroscopic requirements of the mix system and possibly allowing air breakage to resume (if more water is added to the system). This may explain the persistence of slaking in the Hatfield samples despite attempts to equilibrate the internal conditions within the minestone with its external environment. It is also probable that slaking continued for a period of time after the initial immersion, hence even though the minestone was allowed to equilibrate for several days prior to mixing, it would have failed to gain equilibrium with its external environment within this time period.

The immersion ratio values for the POAB samples are in contrast to those of the POAA samples. The immersion ratio of the POAB samples for both distilled and sea water is 82 per cent which is above the minimum value specified by the DoT (1986).The immersion ratio for the POAA samples however, has a reduced ratio for both sea water and distilled water of 54 per cent and 53 per cent respectively. The removal of the finer particles together with those that are of an unstable nature from the POAB sample as a result of wave action has produced a more resistant cement stabilized minestone.

9.3.3 Short Term Dimensional Change

All the samples exhibit increases in mass and volume when immersed in sea water or distilled water (*see Appendix A6*). Although the immersion strength ratios show a marked disparity (e.g.Wearmouth has a immersion ratio of 94 per cent in distilled water whilst Lady Windsor has a ratio of only 48 per cent), this is not reflected clearly in the short term volume increase experienced by the samples. Thomas (1987) was able to place his cement stabilized samples into two groups according to the expansion experienced following 24 hours of immersion :

1.	Linear expansion less than 0.2%	IR exceeds 40%
2.	Linear expansion greater than 0.4%	IR less than 40%

With the exception of the POAB (sea water) sample, which shows a volume increase of only 0.1 per cent, the remaining samples exhibited increases ranging from 0.3 to 0.7 per cent. Using the above scheme, minestones such as Wearmouth, Sutton Manor, Westoe and Wardley would be expected to have an immersion ratio less than 40 per cent when in fact they have immersion ratios in excess of 70 per cent and in the majority of cases well in excess of 80 per cent. The lack of correlation between the 24 hour expansion and the immersion ratio may be the result of sample insensitivity due to the use of an increased sample size, for Thomas worked with 50mm cylinders whilst the present research used 100mm cubes. The greater dimensions of the samples used in the present study as compared to the samples used in the above scheme would mean that water would take longer to penetrate the whole fabric of the sample.

Following 7 days immersion in distilled water the minestones susceptible to strength loss, i.e. POAA and Lady Windsor, are characterised by greater swelling than the more resistant samples such as POAB, Wearmouth and Sutton Manor, though at most, only one per cent separated the most resilient samples from the least resistant. The POAB sample showed the least expansion after 7 days in distilled water, with a 0.5 per cent increase in volume, whilst the most susceptible sample, POAA, registered an increase of 1.6 per cent. A similar situation is evident for samples immersed in sea water, the POAB sample expanded in volume by only 0.2 per cent following 7 days of immersion, whilst the POAA sample increased in volume by 1.5 per cent.

Generally, between 50 and 60 per cent of the total expansion experienced during 7 days of immersion occurs within 24 hours though, in the case of the Wearmouth samples, 70 per cent of the expansion occurs within the first 24 hours, while for the POAA samples the value is only 33 per cent. Overall, the initial pattern of expansion shown by the samples is similar to that exhibited by the specimens used by Thomas (1987) but the expansion is lower than that reported by McNulty (1985). Thomas (1987) demonstrated that by using lower dry densities, the initial expansion would be accommodated by the higher percentage of air voids. This may be true for the present study, but it is also suggested that the coarser minestones such as Westoe, Wearmouth, Sutton Manor and Hatfield produce samples with relatively high percentages of larger voids as they had insufficient fines to fill the large voids. Again these voids would help to cushion the initial expansion caused by interparticle swelling.

Figure 9.9 depicts the increase in volume experienced by the samples during the early stages of immersion, both in distilled and sea water. Several general points can be made. For the first 14 days, the amount of expansion is approximately the same, regardless of the immersing fluid. Initially the samples undergo a period of rapid swelling but, by the seventh day of immersion, the rate of expansion slows. As the initial 'shock' from immersion recedes, the rate of expansion decreases though swelling does not cease. This early response to immersion occurs regardless of the type of immersing fluid. However, the behaviour of the samples in the two different immersing fluids begin to diverge between the 14th and the 28th day of immersion.Whereas the distilled water samples continue to expand albeit at a reduced rate, the sea water samples exhibit a period of accelerated swelling. *Figure 9.9* indicates this divergence between the volume expansion curves for the two sets of samples.

Regular visual inspections of the two sets of samples confirmed the progressively differing affects of the two types of immersing fluid. Within two days of immersion in sea water the samples show severe surface staining in the form of a white efflorescence. In comparison, even after several weeks of immersion, the samples in distilled water show only very slight discolouration, generally in the form of an extremely thin and patchy film of pale violet efflorescence, though no signs of any major crystallization of leached material is discernible. Only the sea water samples provided sufficient leached material to allow X-ray diffraction analysis to be carried out. As expected, sea salt based minerals are much in evidence, including halite (NaCl), epsomite ($MgSO_4.7H_2O$) and mirabilite ($Na_2SO_4.10H_2O$). Besides these minerals, the diffractograms show significant calcite and gypsum peaks together with a much smaller ettringite peak.

The efflorescence is caused by the leaching of calcium carbonate and gypsum from the cement stabilized material. The presence of magnesium sulphate within sea water almost certainly led to the production of significant quantities of gypsum following its reaction with the cement compounds (*see Sections 4.4 and 4.5*). Generally gypsum and ettringite are rather insoluble in water however, when there is a high presence of chloride ions (as is the case with sea water), gypsum and ettringite leach out from the samples and only crystallize when saturation point is reached.

The production of gypsum within the distilled water samples is much slower. No sulphate compounds are found in the immersing fluid and, with the exception of the POAA sample, sulphate levels can be considered to be within the specified limits denoted by the DoT. The sulphate which was originally present within the samples soon becomes exhausted after the initial reactions and thus further quantities of gypsum and other voluminous minerals can only form if more sulphate is released from the material.

Whereas the samples immersed in distilled water showed little and, in some cases, no external signs of disruption in the early weeks of immersion, the samples immersed in sea water began to show signs of distress within five days of being immersed. The basic pattern of disruption is similar regardless of the source of material, though the rate of disruption and some details differed. The first sign of disruption is a softening of the corners and edges. This occurred within a day for the POAA samples, 5 days for the Lady Windsor, 7 days for Hatfield and Sutton Manor samples and 14 days for the POAB samples. Together with the softening of the corners the Hatfield samples also exhibited distinct slaking damage of individual aggregate clasts which had free surfaces at the edges and corners of the cubes. The next sign of disruption is superficial cracking and flaking of the trowelled surface followed by the blistering and flaking of the other surfaces. By the third week of immersion the corners of all samples, with the exception of those from Sutton Manor and POAB, had deteriorated to the extent that disintegration occurred when the corners were handled. Severe and substantial cracking had developed by the beginning the fourth week in all the samples, though the extent and severity of the disruption in the Sutton Manor and POAB was markedly lower.

The swelling of stabilized materials results from the interaction of several mechanisms involving both the aggregate and the cementitious fabric. In the case of minestone, the particles may expand as a result of a number of intraparticle swelling mechanisms including air breakage, ionic dispersion or ion adsorption (Badger *et al.* 1956; Taylor and Spears, 1970; McNulty, 1985) or by interlaminar swelling in the case of expansive clays. The cementitious fabric may suffer from expansion as a result of sulphate attack on cement hydrates. Here, more voluminous minerals, such as gypsum and ettringite, replace the original products of cement hydration. The initial period of expansion can be equated to the readjustment of the clay minerals and particles to the change in moisture conditions. During this period, swelling and slaking of the clay minerals are the main mechanisms causing the samples to expand. The secondary period of expansion exhibited by the sea water samples is the result of an accumulation of gypsum caused by sulphate attack. The replacement of the cement hydrates by the more voluminous products of sulphate attack eventually exerts a disruptive internal force leading to the disintegration of the cement stabilized fabric. Normally, concrete in sea water suffers no expansion following sulphate attack as a result of the increased solubility of gypsum and ettringite when high chloride levels are present. The constant flushing of the concrete by sea water helps to remove and prevent the build up of these harmful minerals (Lea, 1970). However, in laboratory conditions, the stagnant environment enables the solutions to reach saturation point and the minerals crystallize out and eventually cause disruption. The distilled water samples did not experience the secondary increase in expansion during the early weeks of immersion. The absence of sulphate in the distilled water thwarted or drastically hindered the production of gypsum and ettringite, thus helping to curtail the expansive behaviour of the samples.

Although the POAA minestone, with a sulphate content of 1.2 per cent, marginally exceeds the specified limit for acid soluble sulphate, no secondary increase in the sample volume was discernible. This may be because, as it straddles the specification limit, the expansion caused by sulphate attack was masked by the overall expansion behaviour of the sample. As it is the free sulphate that reacts with the products of cement hydration, it can be assumed that water soluble sulphate would be the most reactive as it is able to circulate freely to sites of reaction. The POAA sample has a water soluble sulphate content of only 0.4 per cent, which is probably low enough to prevent the build-up of excessive and disruptive amounts of gypsum.

9.4 Long Term Strength and Durability Results

9.4.1 Long Term Strength Development

The long term strength development for CSM cured in sea water, distilled water and air is presented in *Appendix A5*. *Figure 9.10* illustrates the compressive strength development for the CSM in the three environments. The strength gained by the air cured samples after 500 days was between 2 and 3 times greater than the 7 day strength values for all the samples.

Generally all the air cured samples show a similar pattern of strength development, that of an initial rapid increase in strength followed by a slower phase of strength development ie. the classic pattern of strength development for cementitious materials. None of the air cured samples exhibit a decrease in the trend of compressive strength development during the test period. As *Table 9.5* indicates the majority of the samples gain over 80 per cent of their 500 day strength within 50 days. The exceptions to this are POAA and Lady Windsor, though it is agreed that the Lady Windsor samples were close to achieving 80 per cent of their 500 day strength within 50 days. Similarly at 100 days the majority of the samples had gained over 85 per cent of their 500 day strength and in several instances virtually 100 per cent of this value (Wearmouth, Wardley, Westoe). For the POAA samples and to a much less extent the Lady Windsor samples (two of the finer samples) the percentage strength at 100 days was lower than for the majority of the samples. This suggests that generally, the coarser samples (Hatfield, POAB, Sutton Manor, Wardley, Wearmouth and Westoe) had developed their approximate maximum strength sooner than the finer samples (POAA, and to less extent Lady Windsor). It is suggested that the finer samples would have had a greater ability to retain moisture than the coarser samples, which would be available for pozzolanic or conventional hydration reactions. The coarser samples also had a lower clay content and would thus be expected to have had a lower pozzolanic potential.

Thomas (1987) found that in an idealized situation the strength development curve of immersed samples could be divided into three phases: (1) an initial strength loss due to exposure, (2) partial recovery of strength (3) a second phase of long term, strength loss. Generally a similar pattern can be seen for the immersed CSM in the present study whether the immersing medium is distilled or sea water though, as with the earlier study, some variation do exist.

Section 9.3 discusses the immediate response of the CSM samples to immersion in sea water and distilled water. With the exception of Sutton Manor, Wearmouth and Westoe, which gained strength following the first 7 days of immersion, all the samples experience an initial strength loss in both environments. The strength loss in distilled water ranges from 6 per cent (Wardley) to 47 per cent (Lady Windsor) while in sea water the loss ranges from 8 per cent (Wardley) to 52 per cent (Lady Windsor). Strength loss following the first 7 days of immersion is generally less in sea water than in distilled water. This may be explained by the presence of chlorides in sea water which act as an accelerator during the early stages of cement hydration (Owens, 1989).

Although the Sutton Manor and Wearmouth samples show an initial increase in compressive strength immediately following immersion in distilled water, after 50 days these samples show a slight decrease in strength before steadily increasing in strength again, though at a slow rate. The POAB samples continued to increase in strength even after 500 days. The Hatfield, Lady Windsor, POAA and Wardley samples all began to exhibit a strength loss, though generally at a slow rate, following 300-400 days. All the samples cured in distilled water for 500 days exhibited higher compressive strengths than samples cured normally for 7 days and immersed in water for 7 days. In sea water only the POAA, POAB and Wardley samples had lower strengths values at 500 days than following normal curing for 7 days and 7 days immersion. Both the POAB and POAA samples had higher strengths at 500 days than after 7 days following immersion. The remainder of the samples all showed higher strength following 500 days of immersion in sea water than at 7 days of normal curing and 7 days immersion.

In the sea water environment, the Hatfield samples continued to lose strength until approximately 200 days before regaining strength up to some 400 days of immersion when strength gain ceased and the strength remained constant for the remainder of the test period. The Lady Windsor samples lost strength during the first 50 days but subsequently recovered for a period before beginning to lose strength again after 400 days. The POAA samples exhibited very little recovery following the initial phase of strength loss, but did steadily recover, albeit slowly, before starting to lose strength again after 400 days. The POAB samples also showed a rather subdued pattern of recovery following the initial strength loss reaching a maximum at 200 days before a relatively slow decline for the remainder of the test period. The Sutton Manor samples increased in strength for the 200 days but thereafter the strength decreased with the rate of decline decreasing after 400 days. The Wearmouth and Westoe samples immersed in sea water showed a slight but steady decline in strength following the initial 50 days of strength gain. The Wardley samples exhibited a pattern of strength development that mirrored that of the idealized situation suggested by Thomas (1987). Generally, the finer samples (POAA and Lady Windsor) exhibited the steepest rate of decline during the second phase of strength loss.

The initial strength loss experienced by the samples was attributed by Thomas (1987) to the expansion following immersion. Expansion is a function of several characteristics of the minestone such as plasticity, fines content, the propensity of the material to slake and the water soluble sulphate content. The initial strength loss correlates well with the rapid first phase of expansion experienced by the CSM (*see Figure 9.9 and Section 9.3*). Further strength loss of the samples immersed in sea water can be attributed to sulphate attack on the cement fabric by sea water sulphate compounds. Sea water cured samples were shown to suffer from surface disruption and cracking within 7-14 days of immersion together with [illegible]ching of gypsum from the samples.

The strength recovery following the first phase of strength loss is associated with a decrease in the expansion rate and water uptake. Saturation of the pore waters with the products of sulphate attack would also slow down the reactions associated with sulphate attack thus eventually reducing the rate of strength loss. Strength gain is only achieved if there are reactions continuing within the samples which would improve bonding. The benefits from these reactions must exceed the detrimental effects of fabric expansion. As expansion decreases following the initial decline in strength, the benefits from cement hydration begin to exceed the detrimental effects and strength gain begins to be recorded again. In the case of Sutton Manor, Wearmouth, and Westoe the benefits gained from cement hydration in the first 200 days of immersion outweighed the detrimental effects of expansion hence these samples do not suffer the initial strength loss.

Despite cement hydration, all the samples show a curtailment of their strength development, and, in some cases (especially POAA and Lady Windsor), a decline following 300-400 days immersion in sea water. This cannot be associated with an increase in water uptake and expansion. Thomas (1987) suggests that the long term strength loss is not a result of physical mechanisms but the effect of chemical mechanisms especially sulphate attack. Extensive evidence of chemical disruption is exhibited by the samples immersed in sea water. The leaching of gypsum and calcite from the samples are much in evidence (*see Plate 9.1*). Both X-ray diffraction and SEM analysis of the crack surfaces revealed extensive gypsum build up and minimal quantities of ettringite. *Plate 9.2* shows the typical build up of gypsum that was common within the cracks of the samples immersed in sea water. Similar analysis of the distilled water samples shows that only very small quantities of gypsum and ettringite are present in the Lady Windsor, POAA and POAB samples though in much reduced quantities than that found in the sea water samples. Evidence of chemical attack on the other distilled samples is not apparent.

Overall, the strength of the samples are greater when immersed in distilled water than when immersed in sea water. The difference in compressive strength ranged between 15 per cent (Hatfield) and 48 per cent (Wearmouth) but generally fell within 30 to 40 per cent. Neville (1981) suggested that when sea water is used for mixing, the long term-strength values can be expected to be 15 per cent lower than if potable water is used. A similar drop would be expected if the immersing fluid is sea water as similar inhibiting conditions would be expected to prevail. If this expected decrease in strength is taken into account, the difference between the distilled water and the sea water samples would range from 0 per cent (Hatfield) to 34 per cent (Wearmouth) but generally between 15 and 25 per cent. This suggests that the 15 to 25 per cent strength loss exhibited by the sea water samples is caused by long term disruptive chemical reactions directly caused by sea water. *Plate 9.3* illustrates the typical state of samples cured normally for 500 days. The pristine condition of these samples is in contrast to the CSM samples cured in sea water (marked SW in *Plates 9.4, 9.5* and *9.6*). The CSM samples cured in distilled water (D in the same plates) showed only very superficial disruption such as hair line surface cracking (*see Plate 9.5*) or corner rounding due mainly to slaking (*see Plate 9.4*). Visual examination and comparison of the sea water and distilled water samples suggests that the strength loss for the sea water samples should have been much greater than was recorded. The sea water samples exhibited severe disruption, cracking, staining and mineral deposition on the surface. The smaller than expected difference between the strength of the distilled water samples and the sea water samples suggests that a good percentage of the external disruption, usually associated with the edges and corners, could be classified as superficial and caused by the weakness associated with the free edges of the sample.

9.4.2 Long Term Dimensional Change

The samples wrapped in polythene and cured in air exhibited only limited volume change (*see appendix A6*). With the exception of the Hatfield samples, all exhibited a volume shrinkage of between 0.06 per cent (POAB) and 0.24 per cent (Lady Windsor). These percentages correspond to an average linear shrinkage of between 20 μm (POAB) and 80 μm (Lady Windsor) across a 100 mm cube. The samples made from the coarser grained minestones (POAB, Sutton Manor and Wearmouth) exhibited less shrinkage (less than 0.1 per cent) than those made from minestones with a higher percentage of fines (0.15 per cent to 0.24 per cent) such as Lady Windsor, POAA and Wardley. No mass change accompanied the dimensional changes. Any shrinkage experienced by the samples had ceased within 35 days of moulding. The Hatfield samples exhibited a volume increase of 1.04 per cent with an average linear expansion of 350μm across a 100mm cube. This expansion had ceased within 50 days of moulding.

All the samples were visually inspected at intervals during the curing period. The Lady Windsor and the POAA samples showed slight hair line cracking mainly on the trowelled surfaces, though some small cracks were seen on other surfaces associated with some aggregate clasts. These cracks were found in the matrix and seemed to form as a result of the matrix pulling away from the aggregate. The Hatfield samples showed slight hair line fractures on the trowelled surfaces, while the remaining samples showed no external signs of distress. All the external signs of distress shown by the samples, seemed to be of a superficial nature and ceased to develop after 30 to 50 days of curing.

Shrinkage may be a result of cement paste contraction (Neville, 1981) or due to moisture removal from the clay minerals as a result of cement hydration (Thomas, 1987) who noted that only the finer more plastic samples exhibited a swelling when cured in air. This is in contrast with the characteristics of the Hatfield samples which exhibited swelling despite being of a relatively coarse and non-plastic nature. Thomas (1987) suggested that expansion was caused by the absorption of the mixing fluid by the shale and mudrock clasts. Although this mechanism may contribute to the expansion of the Hatfield samples, it is more likely, knowing the propensity of this minestone to breakdown in water, to be the result of delayed slaking following mixing. This is partly supported by the observation that some of the aggregate particles at the corners and edges of the cubes showed clean cut cracking and in some cases complete slaking and sloughing of aggregate pieces.

The expansive behaviour of the samples exposed to the sea water and distilled water environments is illustrated in *Figure 9.11*. The volume changes experienced by the samples are listed in *Appendix A6* together with the average change in linear dimension. Following 500 days of immersion in distilled water the magnitude of swelling ranges between 584μm (Wearmouth) and 862μm (POAA). All the samples cured in distilled water had a percentage linear expansion of less than 1 per cent (1mm). In sea water the linear expansion exhibited by the samples ranges between 844μm (Sutton Manor) and 1572μm (POAA) representing a percentage linear increase of less than 2 per cent. However, when some of the sea water samples were measured at the edges and corners the samples were seen to have expanded by over 2mm (2 per cent), though this was not the case for the Sutton Manor or POAB samples. None of the samples in the present research show excessive linear expansion (in excess of 2 per cent) as was shown by some of the samples in research carried out by Thomas (1987). On average the 500 day linear expansion in sea water is a factor of 3.8 times greater than the 7 day expansion (the POAB factor is anomalously high at almost 12 times greater). This is out of character for the POAB samples which tends to be more stable than most of the other test minestones especially the POAA samples, which, having a much higher clay content than the POAB samples would be expected to suffer greater expansion especially within the first stages of immersion. The POAB data are hence omitted from the calculations. The average factor in distilled water is 2.6 times that of the 7 day expansion. The factor for sea water ranges between 3.08 (Sutton Manor) and 11.93 (POAB) or 4.97 (Wardley) if the POAB values are regarded as anomalies. For the distilled water samples the factor ranged between 1.32 (POAA) and 4.87 (POAB) or 2.87 (Hatfield) if again the POAB values are regarded as anomalies.

Appendix A6 presents the volume change experienced by the samples when immersed in distilled and sea water. The magnitude of volume increase in distilled water ranges between 1.70 per cent (Wearmouth) and 2.58 per cent (POAA), while in sea water it ranges between 2.54 per cent (Wearmouth and Sutton Manor) and 4.57 per cent (POAA). It can be seen from *Appendix A6* and from *Figure 9.11* that the samples immersed in sea water suffered a greater degree of swelling than the samples immersed in distilled water.

Thomas (1987) illustrated that a set pattern of expansion exists when samples of stabilized minestone are immersed in water and recognised two swelling phases :- a) an initial swelling which ceased anywhere between 10 and 200 days and b) a prolonged period of secondary swelling but at a slower rate. In the present research a similar pattern is discernible for both the distilled and sea water samples, though the first phase usually ended within 50 days whereas Thomas (1987) suggested that first stage expansion could continue for up to 200 days. Thomas also noted that the rate of mass increase, as a result of water absorption, decreased with time as the pore-water pressure equilibrated. He noted that the samples stopped imbibing water after 150 to 200 days. In the present research, the sea water samples had begun to decompose and shed material within 14 days of immersion and hence a detailed analysis of mass increase from water uptake was not possible. The samples cured in distilled water stopped imbibing water after 100 to 150 days. The coarser samples (POAB, Hatfield, Sutton Manor, Wearmouth and Westoe) imbibed over 70 per cent of the total water uptake within a day of immersion. The samples made from the weathered minestone (Lady Windsor and Wardley) imbibed 60 per cent of the total uptake within a day, while the POAA sample, which are composed of the finest material imbibed only 35 per cent in the first day of immersion.

The cause of the first stage of expansion can be attributed directly to water uptake as the large voids became saturated and the water began to enter the less accessible microvoids. The increased surface area associated with the microvoids and the increase in interparticle contacts increased the likelihood of swelling due to surface effects and intraparticle swelling. However Thomas (1987) demonstrated that the second phase of expansion could not be due to water uptake as this ceased after 200 days although expansion continued. He suggested that chemical mechanisms rather than physical processes were responsible for the second phase of expansion and, in particular, the effect of sulphate attack upon the cement fabric. A chemical mechanism can explain the second phase expansion of the sea water samples (the presence of gypsum was much in evidence) and, possibly, the second phase expansion of the POAA samples cured in distilled water, though SEM analysis suggested only a small degree of secondary mineral formation, the remainder of the samples cured in distilled water showed very little or no development of voluminous secondary minerals. The absence of these minerals makes it difficult to attribute the secondary phase of expansion to sulphate attack. The continuing slow expansion can possibly be explained by the slow uptake of water into the internal structure of the clay minerals which subsequently swell. The amount of water needed to produce the continuation of swelling may be very small and hence imperceptible given the way the mass was monitored during the research. Further, the water needed for swelling may have been drawn from the voids, which were then gradually closed or diminished by the swelling minerals and continued cement hydration, hence negating the need for further water uptake.

Minestone	5 Per Cent OPC		7.5 Per Cent OPC		10 Per Cent OPC	
	Strength (MPa)	Dry Density (Mg/m^3)	Strength (MPa)	Dry Density (Mg/m^3)	Strength (MPa)	Dry Density (Mg/m^3)
Hatfield	1.70	2.12	3.27	2.11	4.81	2.14
L. Windsor	2.16	2.02	3.07	2.02	4.45	2.03
POAA	1.38	1.80	2.07	1.80	4.04	1.73
POAB	3.69	2.05	4.61	2.08	6.71	2.06
S. Manor	3.45	2.07	4.47	2.05	5.93	2.03
Wardley	2.00	2.04	2.60	1.90	5.09	1.95
Wearmouth	3.51	2.08	3.22	2.05	4.99	2.06
Westoe	2.00	2.02	2.51	2.04	4.17	2.05

Table 9.1 7 Day compressive strength results for samples made with a range of cement contents.

Minestone	Curing Period (Days)	Compressive Strength (MPa)	Bulk Density (Mg/m^3)	Dry Density (Mg/m^3)	Moisture Content (%)
Hatfield	7	4.81	2.34	2.14	OMC +2%
	14	5.21	2.35	2.15	
	21	5.33	2.32	2.16	
	28	6.07	2.30	2.10	
L. Windsor	7	4.45	2.20	2.03	OMC+2%
	14	5.76	2.17	2.02	
	21	6.14	2.20	2.06	
	28	6.35	2.24	2.08	
POAA	7	4.04	1.93	1.73	OMC+2%
	14	4.50	1.96	1.79	
	21	5.53	1.95	1.77	
	28	6.76	1.99	1.83	
POAB	7	6.71	2.22	2.06	OMC+2%
	14	7.52	2.17	2.02	
	21	8.94	2.16	2.02	
	28	9.57	2.17	2.02	
S. Manor	7	5.93	2.14	2.03	OMC+2%
	14	9.01	2.22	2.11	
	21	12.58	2.15	2.05	
	28	13.47	2.16	2.08	
Wardley	7	5.09	2.10	1.92	OMC+2%
	14	5.37	2.09	1.92	
	21	6.33	2.12	1.97	
	35	9.11	2.08	1.96	
Wearmouth	7	4.99	2.24	2.09	OMC+2%
	14	5.81	2.25	2.09	
	21	7.73	2.25	2.11	
	35	9.96	2.26	2.11	
Westoe	7	4.17	2.21	2.06	OMC+2%
	14	5.52	2.23	2.05	
	21	6.33	2.22	2.06	
	35	8.07	2.22	2.06	

Table 9.2 Early strength development results for the test samples stabilized with 10 per cent OPC.

Minestone	Distilled				Sea Water			
	7 day	14 day	21 day	35 day	7 day	14 day	21 day	35 day
Wardley	5.09	5.37	6.33	9.11	4.53	5.16	5.60	8.77
Wearmouth	4.99	5.81	7.73	9.96	4.83	5.42	6.48	8.78
Westoe	4.17	5.52	6.33	8.07	4.82	5.05	5.79	8.20

Table 9.3 The effect of using differing mixing fluid for cement stabilized minestone.

Minestone	Immersing Fluid	14 Day Compressive Strength (MPa)	Immersed Strength (MPa)	Immersion Ratio (%)
Hatfield	Distilled	5.21	3.42	65.64
	Sea Water	5.21	3.69	70.83
Lady Windsor	D	5.76	3.10	47.78
	SW	5.76	3.03	52.44
POAA	D	4.50	2.15	53.83
	SW	4.50	2.36	52.60
POAB	D	7.52	6.15	81.64
	SW	7.52	6.19	82.31
Sutton Manor	D	9.01	7.74	85.90
	SW	9.01	8.28	91.89
Wardley	D	5.37	3.97	74.00
	SW	5.37	4.46	83.10
Wearmouth	D	5.81	5.44	93.60
	SW	5.81	5.19	89.30
Westoe	D	5.52	4.92	89.10
	SW	5.52	4.88	88.40

Table 9.4 Results of the immersion test using minestone stabilized with 10 per cent OPC.

Minestone	Percentage Value of 500 Day Compressive Strength	
	50 Days	100 Days
Hatfield	79.2	93.3
L. Windsor	77.3	81.0
POAA	67.5	67.5
POAB	84.8	87.7
Sutton Manor	88.3	88.0
Wardley	95.7	99.5
Wearmouth	83.2	99.4
Westoe	81.9	100.0

Table 9.5 The 50 and 100 day strength results presented as a percentage of the 500 day compressive strength results.

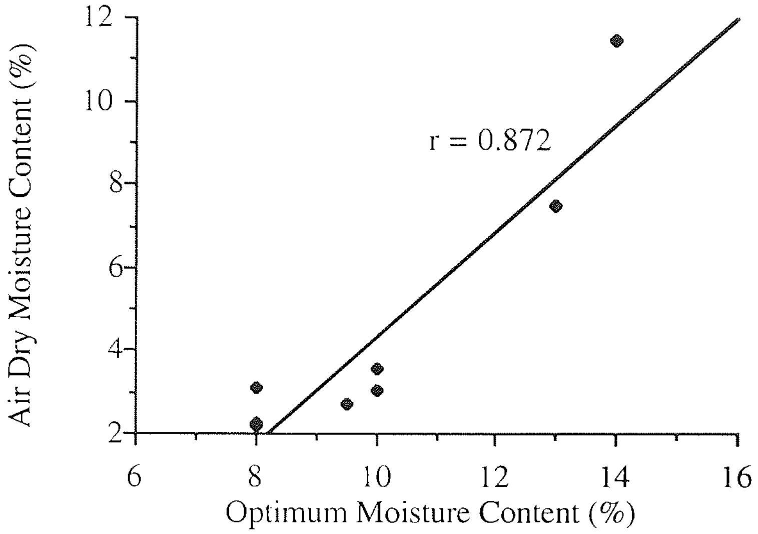


Figure 9.1 The relationship between the air dried moisture content and the optimum moisture content.

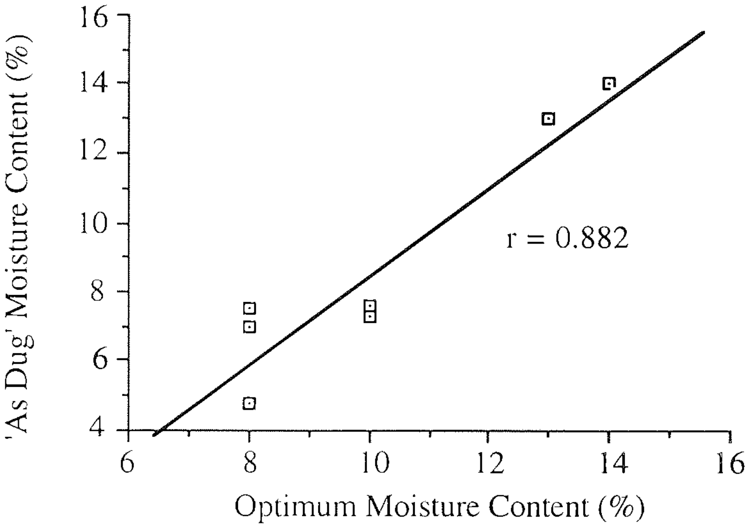


Figure 9.2 The relationship between the 'as dug' moisture content and the optimum moisture content.

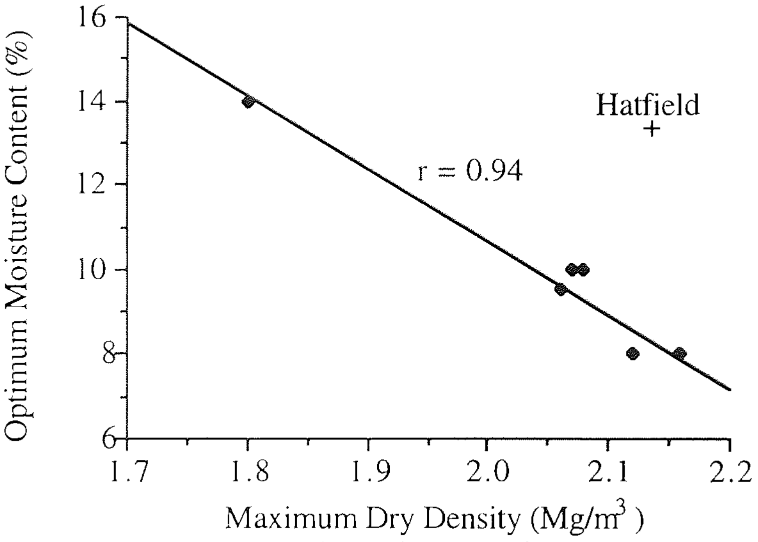


Figure 9.3 The relationship between the optimum moisture content and the maximum dry density.

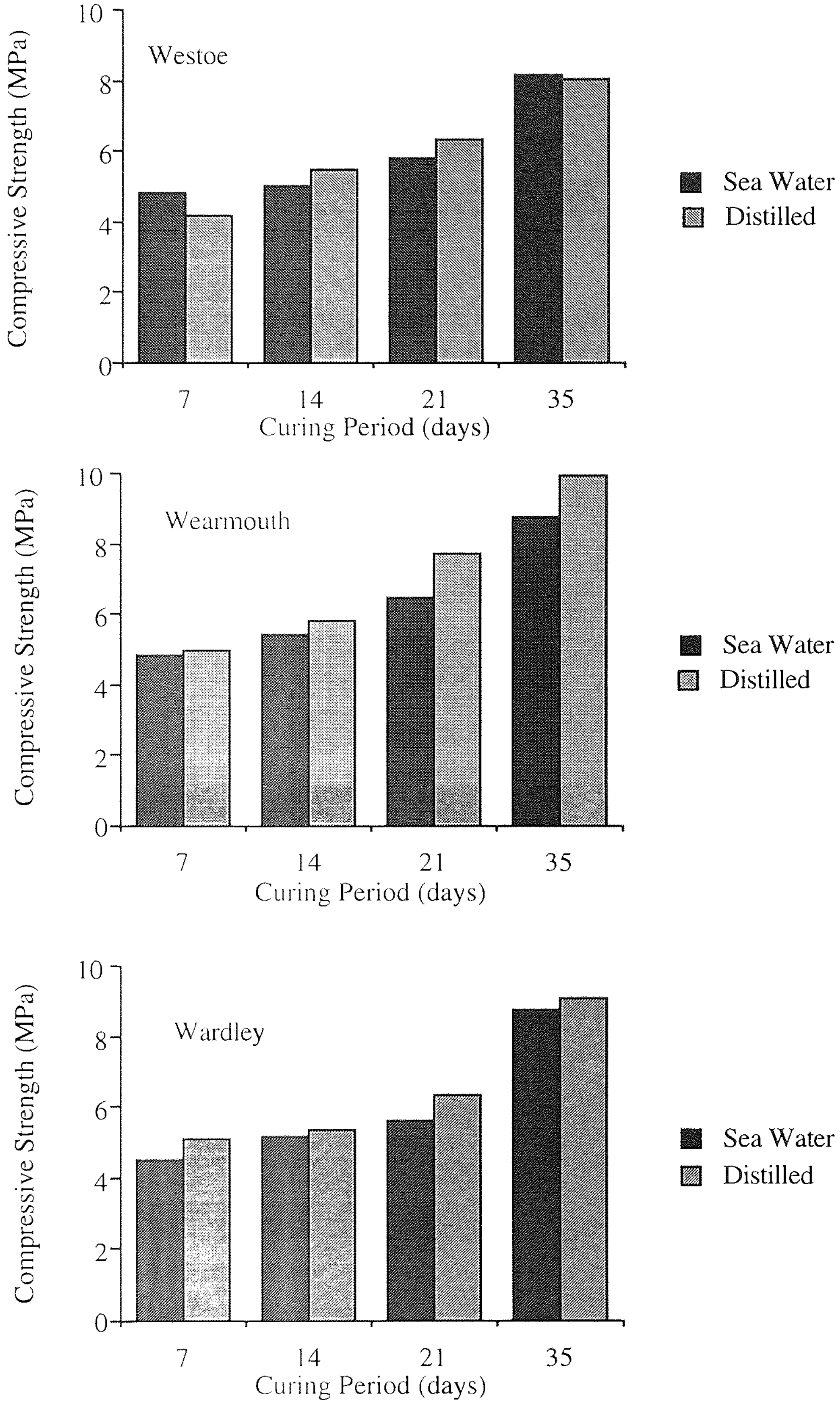


Figure 9.4 The effect of using distilled and sea water as the mixing fluid.

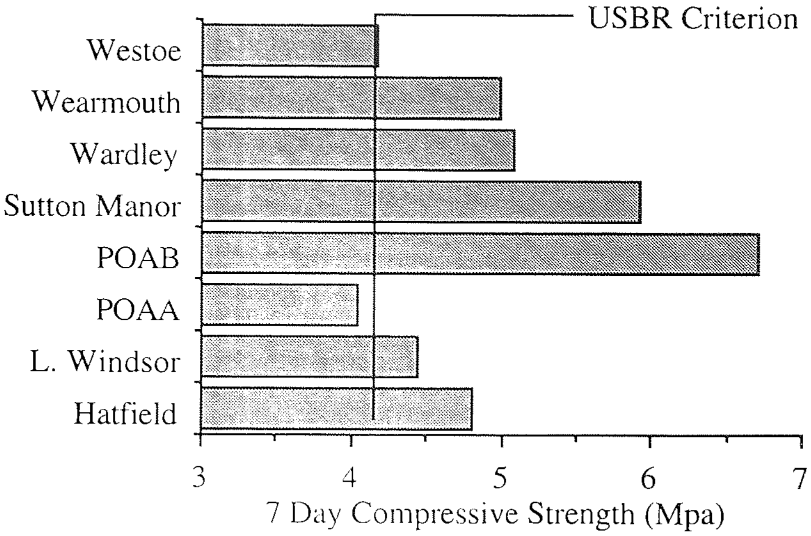


Figure 9.5 A comparison of the 7 day compressive strength of CSM (10 % OPC) cured in air.

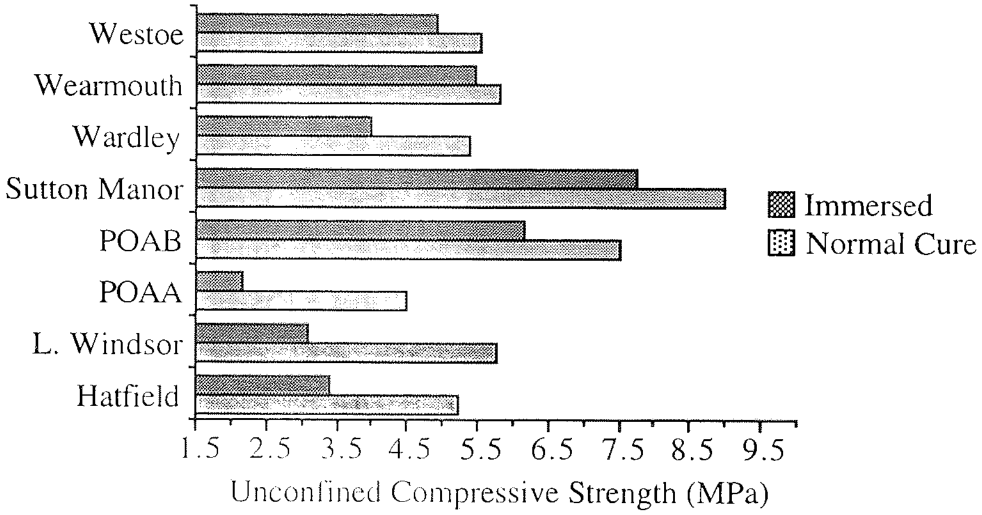


Figure 9.6 Results of the immersion test when samples are immersed in distilled water.

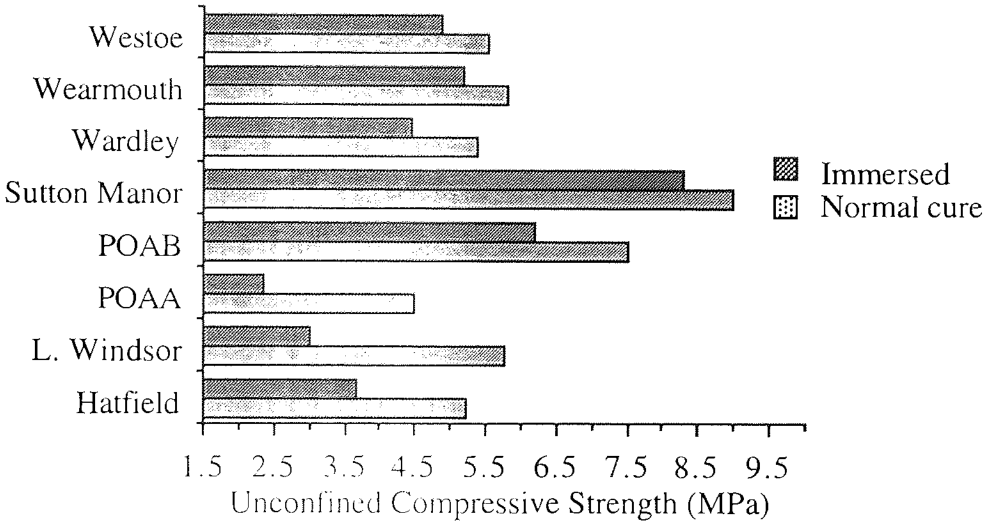


Figure 9.7 Results of the immersion test when samples are immersed in sea water.

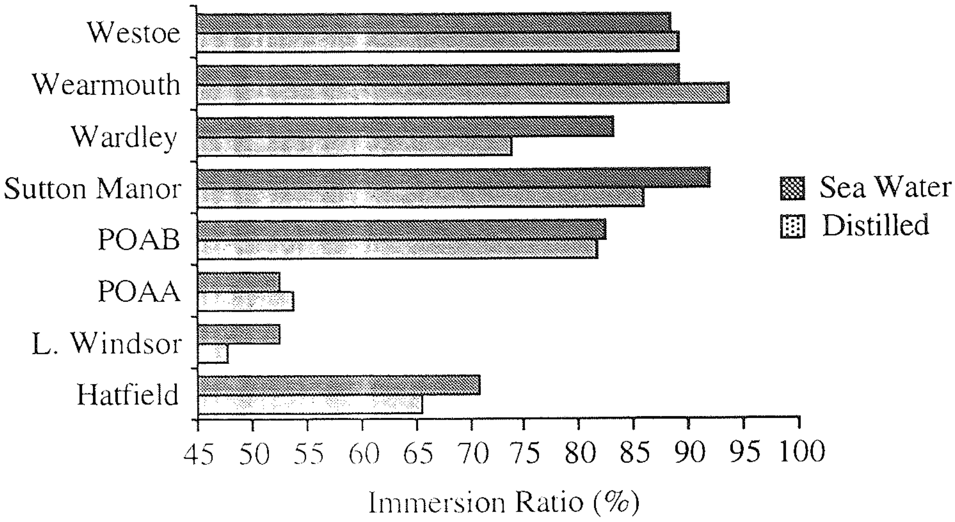


Figure 9.8 A comparison between the immersion ratio of CSM samples.

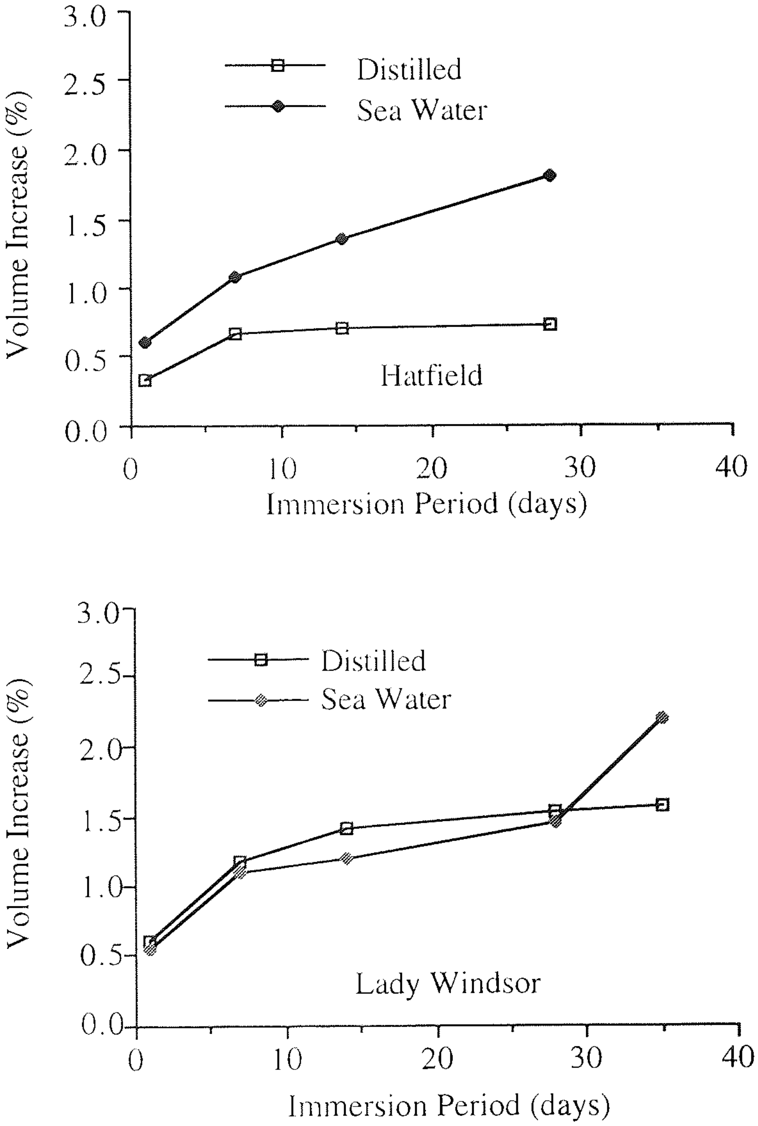


Figure 9.9 Volume changes in CSM (10% OPC) during the early stages of immersion.

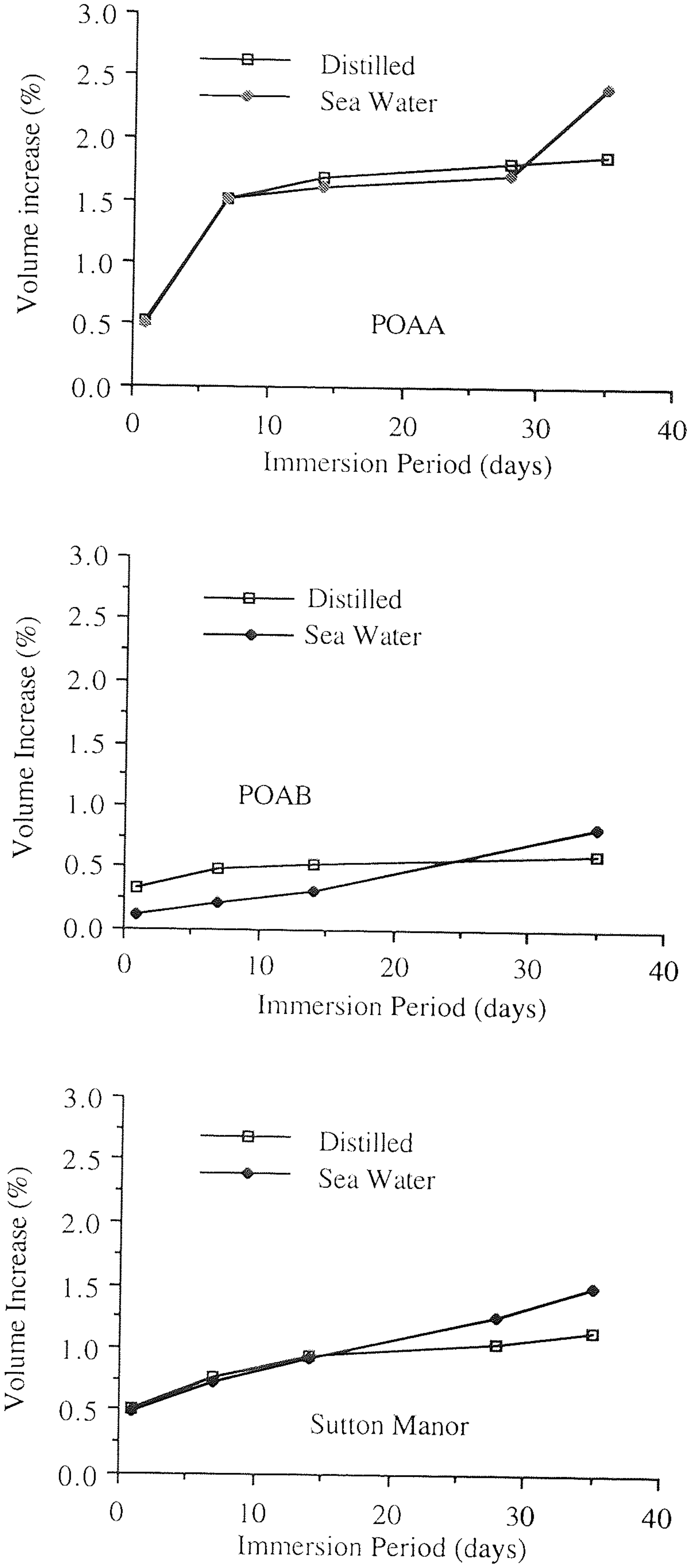
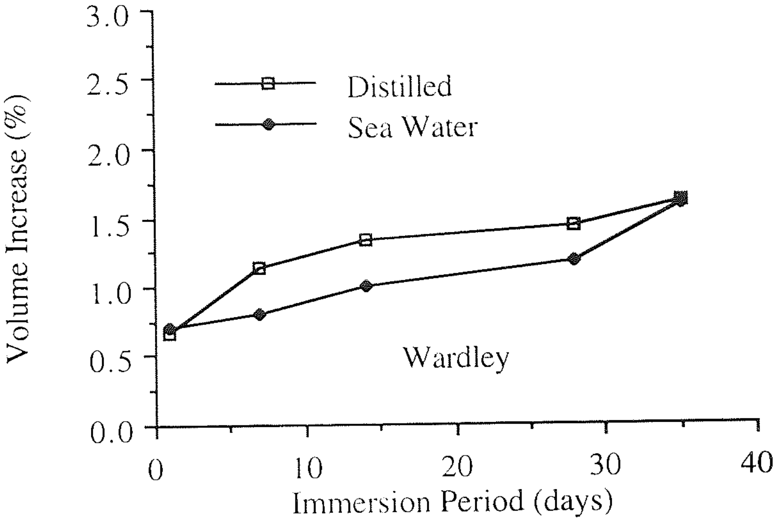


Figure 9.9 (continued).



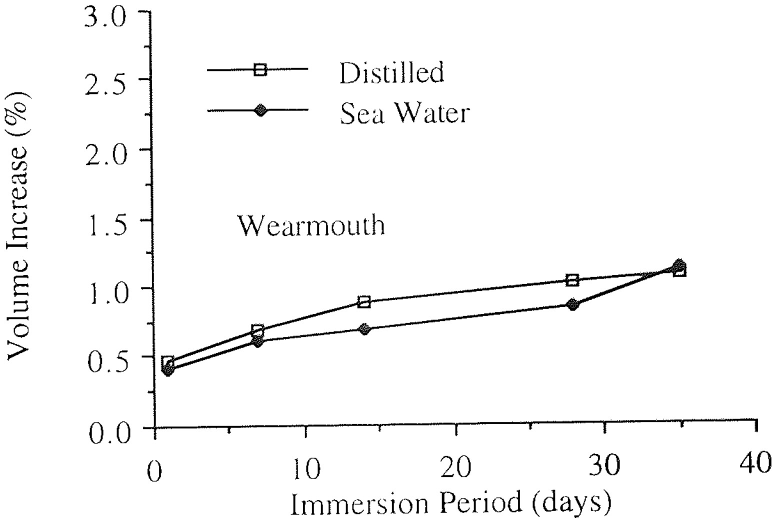


Figure 9.9 (continued)

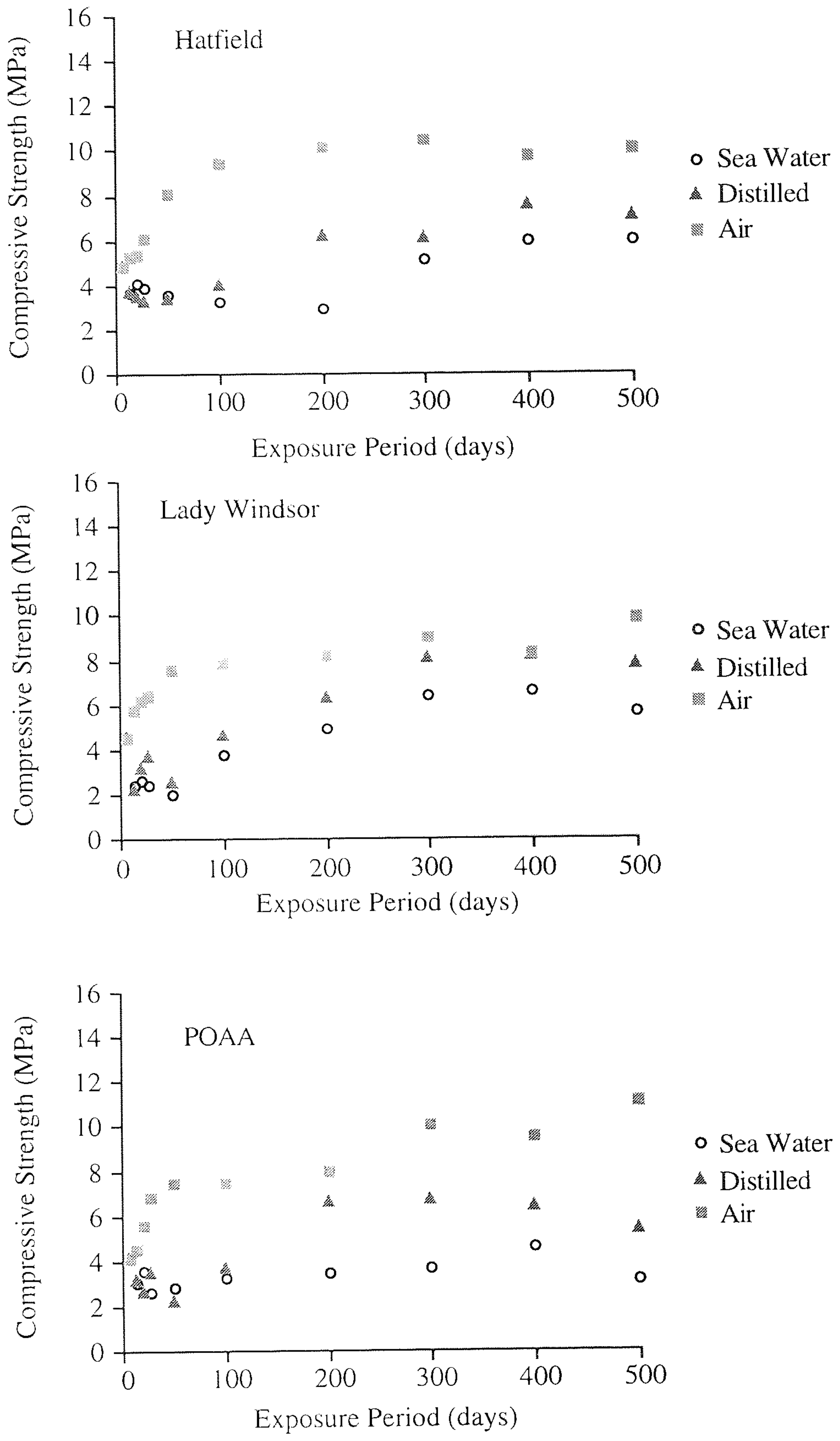
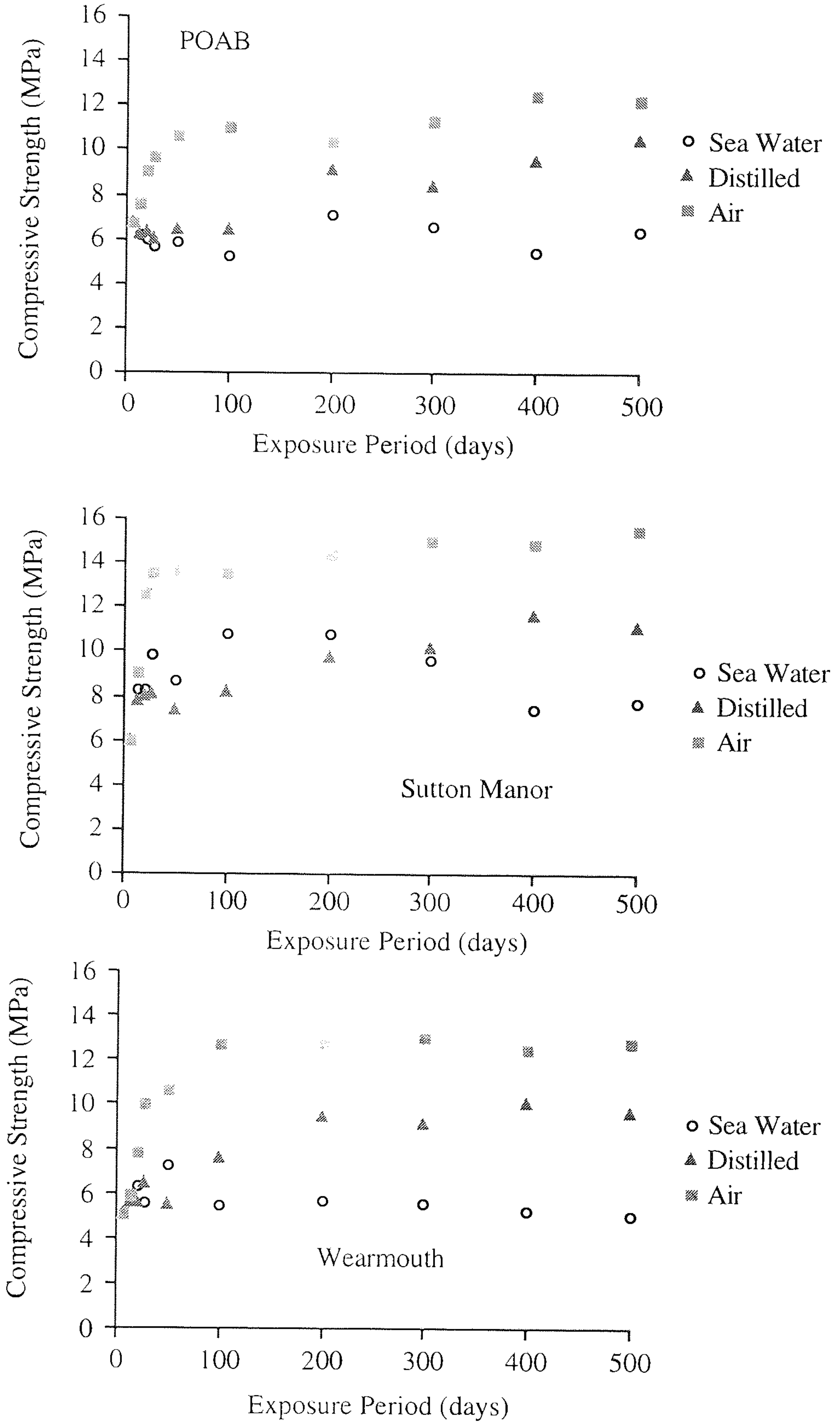


Figure 9.10 Long term strength development for CSM (10% OPC).



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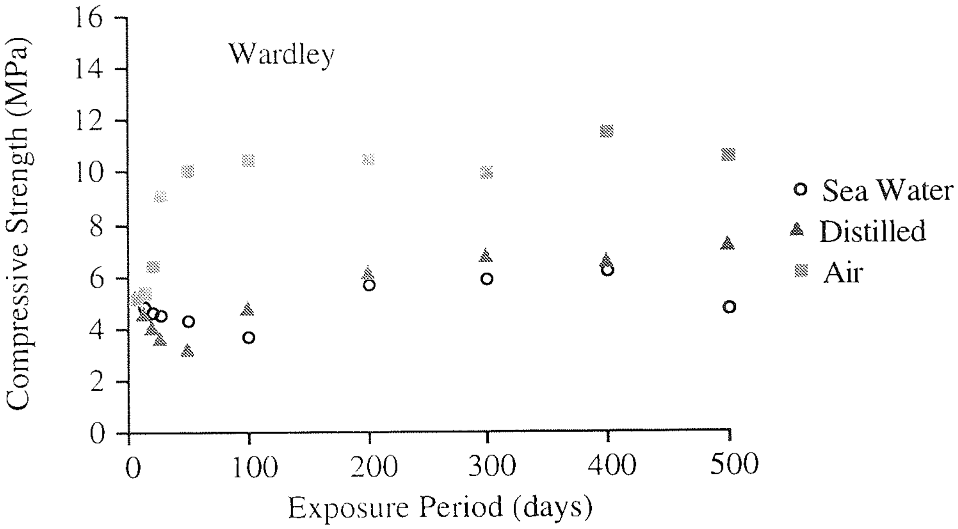


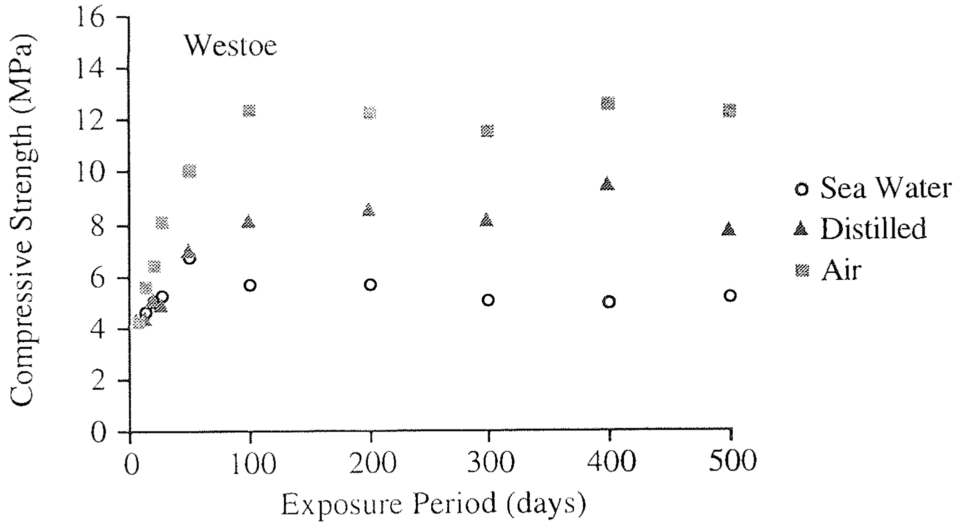


Figure 9.10 (continued).

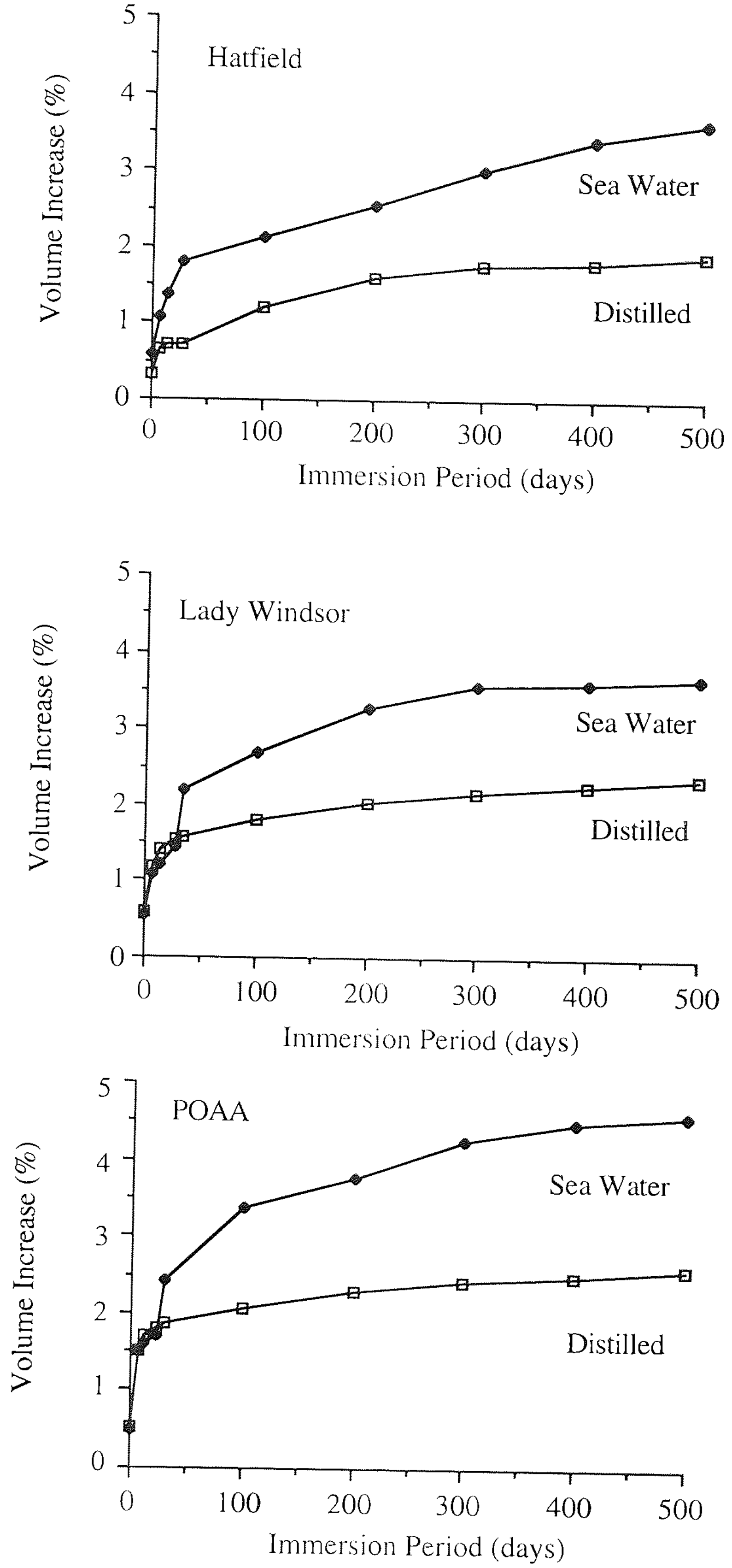
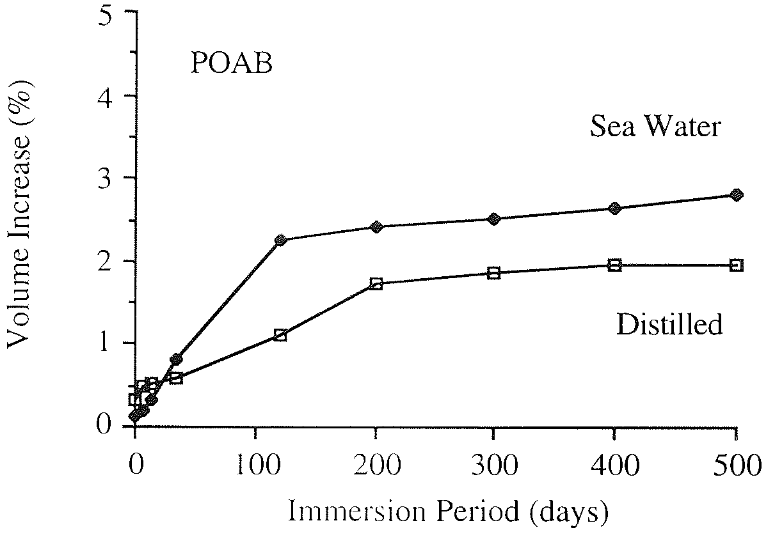


Figure 9.11 Long term changes in volume following immersion.



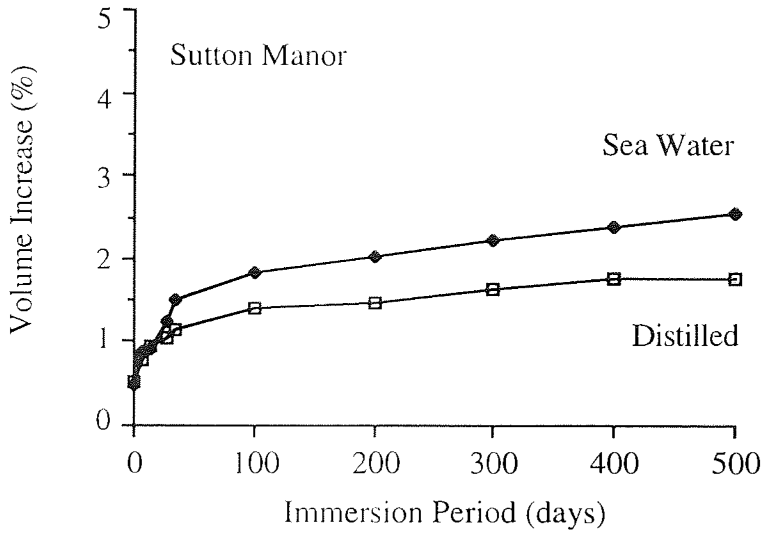
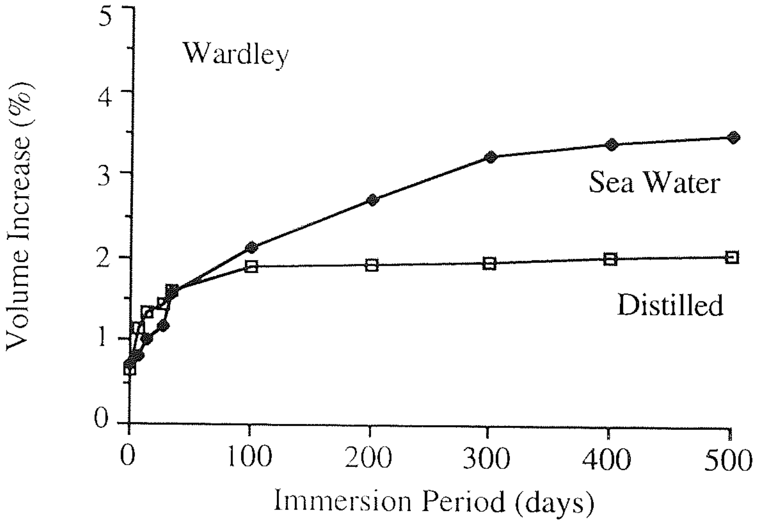


Figure 9.11 (continued).



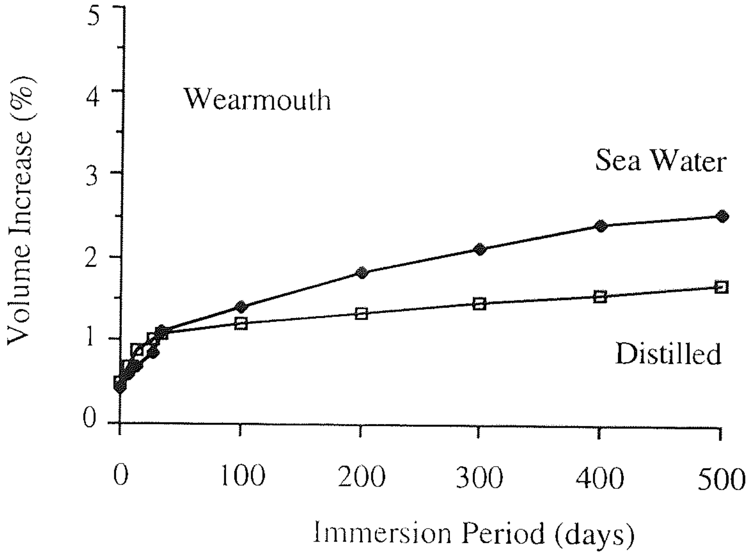


Figure 9.11 (continued).

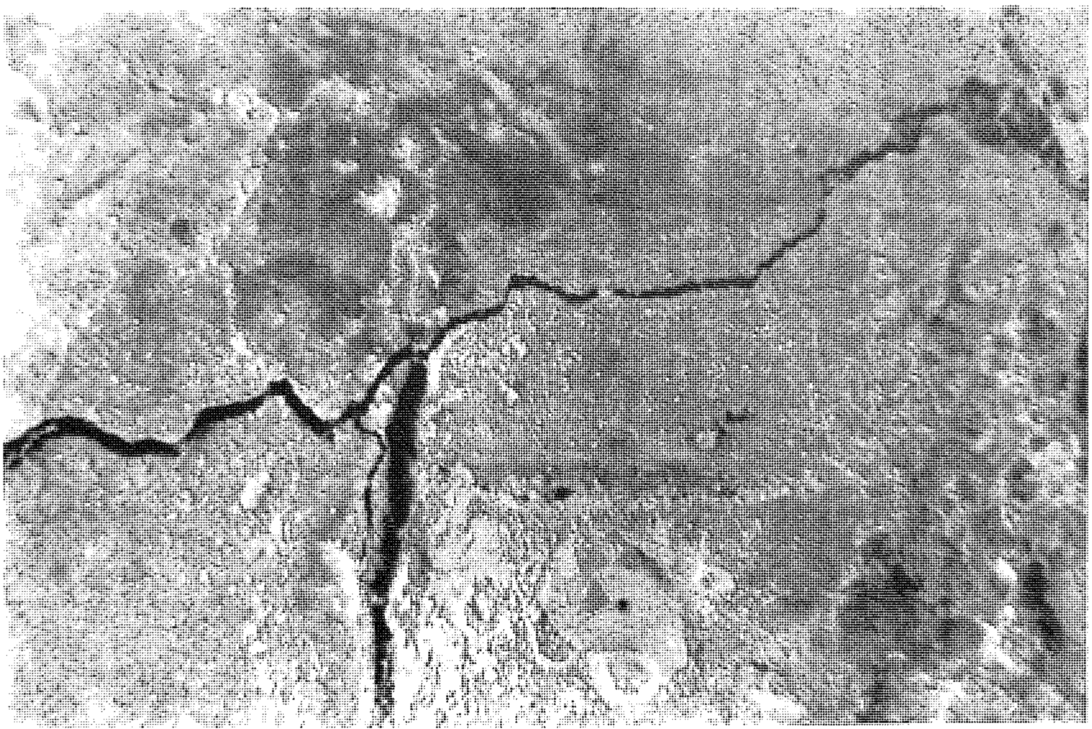


Plate 9.1 Cracks in a CSM sample which has been immersed in sea water for 100 days, showing evidence of calcite and gypsum deposition.

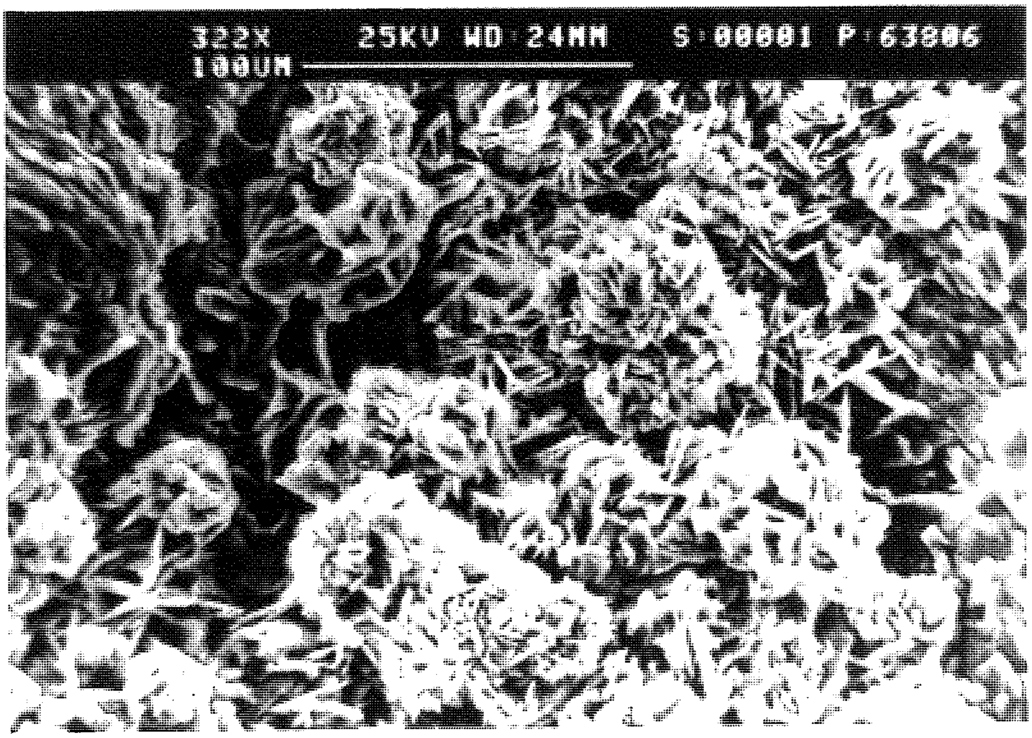


Plate 9.2 Gypsum florettes, common in cracks of CSM samples immersed in sea water.

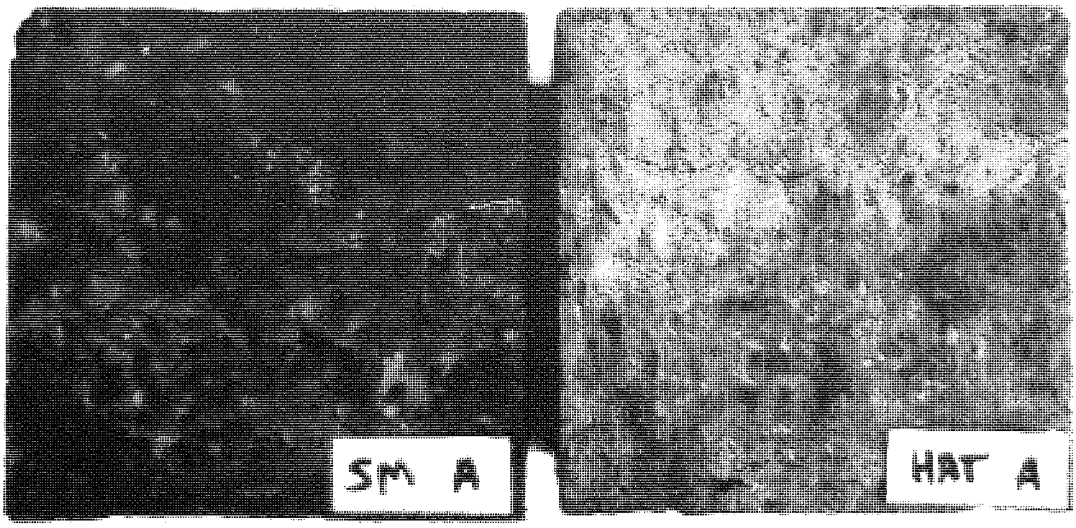


Plate 9.3 Typical state of 100mm CSM cubes cured normally for 500 days.

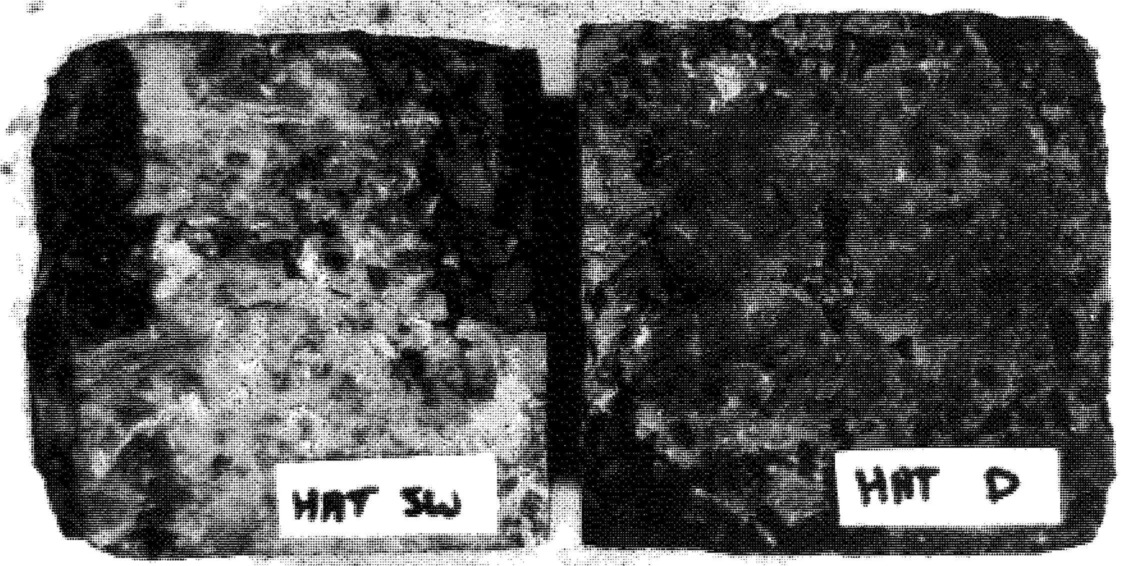


Plate 9.4 The condition of 100mm CSM cubes (Hatfield) following 500 days immersion in sea water (SW) and distilled water (D).

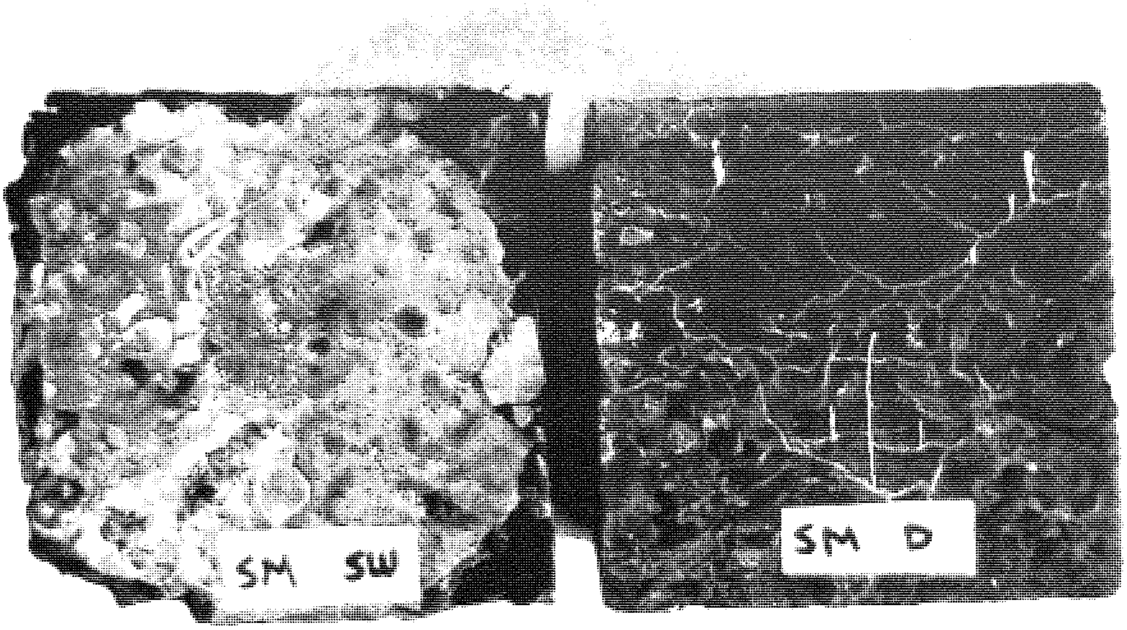


Plate 9.5 The condition of 100mm CSM cubes (Sutton Manor) following 500 days of immersion in sea water (SW) and distilled water (D).

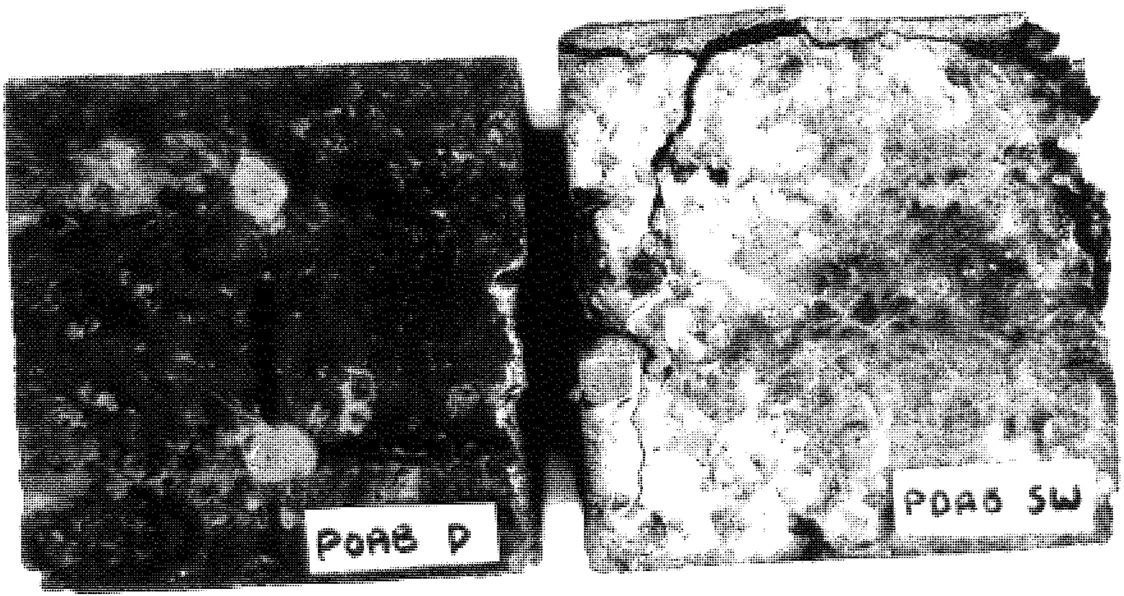


Plate 9.6 The condition of 100mm CSM cubes (POAB) following 500 days of immersion in and distilled water (D) and sea water (SW).

10. ANALYSIS AND DISCUSSION OF THE EROSION/ABRASION TEST RESULTS

10.1 The Applicability of the Erosion/Abrasion Test for Simulating Field Conditions

As explained by Oswell and Joshi (1986) and outlined in *Section 7.5*, it is possible to relate the impact pressures produced during this test to those generated by breaking waves in natural conditions by use of a formula originally developed by Gaillard (Bascom 1980). The test pressures selected by Oswell and Joshi ranged from 70kPa (10psi) to 280kPa (40psi) and represent the impact pressures generated by breaking waves between 1.5m and 6.5m in height with wave periods varying between 5 and 10 seconds. The same range of test pressures was selected for the present study basically for two reasons. Firstly, it enables comparison with the results obtained by Oswell and Joshi (1986) and secondly, it can be shown that these test pressures equate to the impact pressures expected (using Galliard's formula) in areas where minestone could be considered for coastal works around the UK.

The biggest prospective project that could utilize minestone is the Severn Barrage Scheme. Wave data collected as part of the Severn Barrage Study (Severn Tidal Power Group 1986) indicate a maximum wave height of between 2.04m and 6.05m and a wave period of between 3 and 5 seconds, while the maximum significant wave height for the study period ranged from 1.21m and 3.09m. Applying this information to Galliard's formula suggests that these wave conditions would produce wave impact pressures of between 68kPa (9.7psi) and 274kPa (39psi), which are within the range selected for the present study. Another possible coastal project where minestone has been considered is the Port Edgar Maritime Village development, South Queensferry, Scotland. Couper and Montgomery (1990) have stated that the one year, winter storm, wave height would exceed 2.2m with a period of less than 6 seconds so generating impact pressures in excess of 88kPa (13psi) and, even with a doubling of the maximum wave height, the impact pressures expected would be within the study range.

10.2 Results and Discussion

The suitability of minestone as a material for coastal construction works will depend upon its ability to withstand direct wave action and the attendant destructive forces. The erosion/abrasion test was developed to simulate the erosive affects of wave action and can be directly related to actual field conditions. If minestone is to compete with the more traditional hydraulic materials such as dredged sands, it must be shown that its ability to resist marine conditions is at least equal to the properties of these materials. The following section discusses the results obtained from the erosion/abrasion test with regard to the performance of minestone and its equivalence to that of dredged estuarine sand.

10.2.1 Unstabilized Tests

If minestone is to be used in significant quantities for coastal works, it is likely that it will be in the unstabilized state, thereby replacing hydraulic sands as the chief bulk material.*Tables 10.1* to *10.4* give the erosion rates for individual tests together with the average erosion rates for the test minestones and the laboratory sand mixture (0.2mm to 0.4mm) in the unstabilized state, while *Figures 10.1* and *10.2* present these erosion rates graphically. It can be seen from the graphs that in the unstabilized state, all the samples exceed the 2.5cm^3/h limit suggested by Oswell and Joshi (1986) as the division between negligible and significant erosion. Even under the lowest impact pressures (70kPa) the erosion rates are almost four times above this level. However, this division between negligible and significant erosion is a completely arbitrary figure suggested by Oswell and Joshi for their research and, furthermore, it was devised in relation to soil-cements rather than unstabilized material. It would be expected that any unstabilized material, especially non-cohesive material, would have a lower resistance to wave erosion than a stabilized material. It is most likely that any unstabilized bulk fill material considered for use in a semi-permanent or permanent structure would be protected from direct wave action by such features as rip-rap, geo-textiles and/or graded layers.

Figure 10.1 illustrates the relationship between the impact pressure of the water jet and the resultant rate of erosion expressed in cm^3 /h. A general pattern can be seen to exist for the majority of minestones as well as the laboratory sand samples. The impact pressure/erosion rate curve may be divided up into two zones. At the lower test pressures, the increase in erosion rate for each incremental increase in impact pressure is relatively small giving rise to a shallow slope. The second zone, associated with the higher pressures, is characterised by a rapid increase in the erosion rate with each incremental increase in impact pressure. With the majority of the samples including the laboratory sand, the change, from limited distress to a rapidly increasing erosion rate, occurred at approximately 140kPa (20psi). With the Sutton Manor samples the change occurred when the impact pressures reached 210kPa (30psi). Several reasons may explain this sudden change in the erosion rate. It may represent the point when the forces associated with the impacting water exceed the cohesive forces that bind the matrix together. Once the matrix is severely disrupted the aggregate itself is exposed to erosion. It may also represent the point at which the force of the draining water emanating from the cavity becomes sufficiently high to cause secondary erosion, supplementing the primary erosion caused by the initial impact. This is evident from the way wings and grooves were seen to form in the cavity as a result of water escape. The sudden change in the erosion rate may also represent the point at which the force of the water jet remained sufficiently high, following the primary impact, to produce secondary jets or sprays which deflected off the flat angled faces of the aggregate clasts into the surrounding walls of the cavity. This explanation can be supported by the observation that the diameter of the cavity tended to increase at higher impact pressures.

It can be seen from *Tables 10.1* to *10.4* that the individual erosion rates for each unstabilized minestone varies quite markedly even at constant impact pressures. This is not surprising considering that the samples are made from materials with components of varying strengths, densities and a wide range of particle sizes. Furthermore the unstabilized nature of the samples allows uneven quantities to be eroded at different rates such that at one moment very little material would be seen to be removed while at the next a collapse of the cavity wall would occur producing a relatively large quantity of eroded debris.

A comparison between the minestone and the laboratory sand at the various test pressures is also presented in *Figure 10.2*. The most obvious feature exhibited by the graphs is that minestone is more resistant to the erosional forces of the water jet than the laboratory sand throughout the range of test pressures. The difference between the performance of the two types of materials increases as the impact pressure increased. Generally, the laboratory sand erodes 3 to 6 times faster than the minestone over the 70kPa to 280kPa range of impact pressures. The relative superiority of minestone in resisting erosion over the laboratory sand mix is revealed in *Figure 10.3,* which compares the erosion suffered by the sand with the most resistant and one of the weaker minestone samples, namely POAB and POAA respectively.

The difference between the performance of the minestone and that of the sand can be ascribed to several factors and mechanisms. First, as discussed in *Section 5.4*, the susceptibility of a particle to erosion will depend upon its mass and density. Larger, heavier and denser particles will be more able to resist the forces of erosion than smaller, lighter and less dense particles. Although minestone has some light particulate matter in the form of coal, the greater part of its composition is composed of members of the clay family minerals that have higher densities than quartz, the primary constituent of sand (Read, 1976; Morton *et al.* 1984), furthermore the minestone samples have a maximum grain size of 20mm, whereas the grain size for the laboratory sand is of medium grade, no greater than 0.3mm. Secondly, the coarser particles of the minestone interlock together thus increasing the resistance to the water jet action. The larger particles within the minestone also offer large surfaces which helped to dissipate the water jet energy by redirecting it back into the jet or off towards the sides. The cohesive component of minestone also affords protection against the forces of the water jet by binding the sand and small gravel size particles in the matrix which, in turn, protects the larger aggregate clasts from the winnowing action of the water jet. Even if half the matrix is eroded away from an aggregate clast, the remainder of the matrix behind the clast allows it to remain firmly embedded in the overall fabric of the sample. The large aggregate clasts further provide erosion protection to the remainder of the sample.

Normally, a hydraulic sand will be non-cohesive but to aid in the preparation of the laboratory sand samples, up to 3 per cent clay was added to ease handling. Although the clay made the sand mixture more cohesive, the effectively large surface area of the sand reduced the benefits gained from this cohesiveness, in comparison to that mobilized around the larger aggregate particles in the minestone. It is thought that the laboratory sand mixture would be more resistant to erosion than a natural hydraulic sand, which does not have the benefit of a cohesive element in its resistance to erosion. The uniformity, non-cohesiveness and smallness of the particles in the sand samples gave very little protection against the impacting forces of the water jet.

An insight into the mechanism of erosion can be gained by observing the formation of the cavity during testing. Generally, this is similar at all impact pressures, with just the intensity differing. In the early stages of erosion the cavity is hemispherical with only slight removal of the finer material from between the large clasts. When the large clasts achieve interlocking, winnowing by the water jet is kept to a minimum, while the eroding stones are prevented from eroding the matrix as long as the joints between the aggregate clasts are smaller than the stones. Once erosion proceeded sufficiently to open up joints and gaps, usually within twenty minutes of the test starting, the eroding stones become trapped within the cavity. This entrapment helps to protect the matrix from the the winnowing effects of the water jet. As erosion proceeds, the cavity becomes deeper, especially at the base and, as the diameter gradually decreases inwards, the cavity becomes more conical in shape. The type and extent of erosion damage inflicted on unstabilized minestone by the erosion/abrasion test is illustrated in *Plates 10.1 to 10.4.*

[illegible] ssures, or if the interlocking aggregate remains intact, the cavity gradually [illegible] irregular as erosion proceeds along lines of least resistance and water [illegible] the cavity starts to erode channels and grooves within the sides of the cavity (*see Plate 10.4*). Eventually the matrix between the larger clasts is washed away, loosening the aggregate and leading to a smoothing of the cavity wall. Occasionally the water jet pressure is not sufficient to remove the loosened aggregate clasts and these would subsequently lie loose at the base of the cavity, thus affording further protection to the sample from the effects of the water jet.

With the sand samples the cavity remained conical throughout the test. Whereas the eroding stones became entrapped within the cavity when unstabilized minestone was tested, the eroding stones remained in circulation when the unstabilized sand samples were tested which further contributed to the erosion of the material. However, as with the unstabilized minestone, the water jet itself provided the main agent of erosion, washing and winnowing away the sample material. As the test proceeded further, the material surrounding the cavity became wetter, making it more susceptible to erosion. Although there was a small degree of cohesion shown by the sand samples (provided mainly by the clay additive), very little particle support or interaction existed to develop erosion resistance. With the unstabilized minestone, the strong particle interaction and support provided internal resistance to the erosive forces of the water jet.

By studying *Figure 10.2* it is clear for the unstabilized minetone that the samples from POAA, Wardley and Hatfield exhibit the least resistance to erosion, whilst those that show the greatest resistance are POAB, Lady Windsor and Wearmouth. The samples from Westoe and Sutton Manor show an intermediate resistance between these extremes. In a previous unpublished study carried out by Aston University for the Coal Research Establishment, unstabilized minestone from Francis, Hatfield, Lady Windsor, Tilmanstone, Sutton Manor and Point of Ayr were tested using the erosion/abrasion apparatus. The results for these tests are shown in *Appendix A7*. Surprisingly, it was found that the Sutton Manor minestone showed the least resistance to water impact despite being considered to be one of the most stable of minestones, while the Hatfield samples, despite their notoriety (personal communication N. Withers Minestone Services) as an unstable material, exhibited a greater resistance to erosion. However, the samples which showed the least susceptibility again were Lady Windsor and Point of Ayr. The relatively poor performance of the Sutton Manor samples could probably be attributed to a lack of fines within the individual samples. If there was sufficient fine material present it would form a matrix between the larger aggregate clasts to help to hold them in place and protect them to some degree from erosion. The Hatfield, Francis and Point of Ayr samples had a more balanced mixture of matrix and aggregate size particles whose mutual relationship helps develop protection to erosion. Erosion resistance can also be reduced if there is a predominance of fines as is the case with the POAA samples in the present study. In this case the joints between the aggregate clasts are wide so that the matrix is washed away relatively quickly exposing the aggregate to erosion.

Within this study, the Wearmouth and POAB samples possessed a better balance between aggregate and matrix and this is demonstrated by their relatively good erosion resistance. Although the Lady Windsor samples have a higher percentage of finer material than the POAB and Wearmouth samples (*see Table 8.1*), they had a comparable resistance to erosion. This may be attributed to the higher percentage of clay size particles (9 per cent < 63μm) within the material which would impart the property of cohesion to the matrix, thereby increasing its ability to resist the action of the water jet.

The ability of the cohesive fraction to protect the non-cohesive elements of a minestone from wave action was apparent at the Point of Ayr site. Here minestone had been deposited on low lying coastal land and partly used to reclaim land from the sea. Following the initial erosion and denudation of the minestone platform, the leading edge of the colliery waste in contact with the sea had become a shallow, smooth incline. This leading slope consisted of a tough, resistant layer composed of large rounded pieces of minestone firmly embedded in a clay matrix. This layer was extremely difficult to disturb even with the use of hand held digging equipment.

The results of the present research into unstabilized minestone using the erosion/abrasion test indicate that the samples were more susceptible to erosion than those tested for the Coal Research Establishment (*see Tables 10.1 to 4 and Appendix A7*), despite the fact that several minestones used in the two studies were from the same colliery. Although the samples were obtained from the same spoil heaps, the inherent variation in the stockpiled material could result in wide differences in the properties of the individual samples. Furthermore, there were differences in the treatment of the samples before testing. All the test samples were manufactured in a similar way, but the CRE samples were allowed to dry for a longer period than those used in the present study. The drying out of clays acts as a form of stabilization which helps to improve the resistance to erosion of the minestone.

10.2.2 Tests Upon Cement Stabilized Materials

It is well established that by incorporating cement into a soil or soil like material, the strength and durability of that material is greatly enhanced. Minestone is no exception, it has been shown to have improved capabilities when ordinary portland cement is added. The erosion/abrasion tests performed upon unstabilized minestone and the laboratory sand mixture revealed that neither of these materials is sufficiently resistant to erosion to stay within the maximum permissible erosion rate of 2.5cm^3/h (Oswell and Joshi, 1986). A study was therefore undertaken to determine the effects of ordinary portland cement on their erosion resistance. Initially, materials from Wearmouth, Westoe and Wardley were stabilized with 5, 7.5 and 10 per cent cement to determine the cement content necessary to lower erosion to within the suggested limit. The results of this initial programme are given *Table 10.5*. The erosion rate was calculated as the average volume loss (weight loss divided by initial density) per unit time. The results in *Table 10.5* represent the mean of three individual tests on each mixture. A consideration of the data in *Table 10.5*, and shown in *Figure 10.4* reveals that the erosion rates generally decreased with increasing cement content and decreasing impact pressures. *Figure 10.4* also permits comparison between the performance of the cement stabilized samples and the same unstabilized samples. The same data are plotted in *Figure 10.5* but with the unstabilized results omitted to provide greater clarity.

It is obvious from the graphs that even a comparatively low cement content of 5 per cent was very effective in reducing the susceptibility of the minestone to the combined effects of the water jet and the abrasive material. Typically, minestone stabilized with 5 per cent cement had between 5 and 20 times the resistance to erosion of the same unstabilized minestone, the ratio varying with the force of the water jet. For samples stabilized with 7.5 per cent cement the improvement in resistance ranged between 12 and 22 times that of the unstabilized material, while the improvement ranged between 11 and 23 times when 10 per cent cement was used to stabilize the material. *Figure 10.6* presents the erosion results for stabilized minestone as a percentage of the unstabilized erosion rates for the same minestone. From all these figures it is clear that the greatest degree of improvement was shown by the initial addition of 5 per cent cement. A further significant improvement in the erosion resistance is noticeable when the cement content was increased to 7.5 per cent. However, further addition of cement to 10 per cent failed to increase significantly the erosion resistance of the minestone.

Table 10.6 lists those samples of these cement stabilized minestones that achieved a negligible rate of erosion when subjected to different impact pressures and stabilized with a range of cement contents. At an impact pressure of 70kPa, 5 per cent cement was sufficient to reduce the erosion rate of the three minestones to below the suggested maximum permissible rate (Oswell and Joshi, 1986). With the exception of the Westoe samples with 5 per cent cement, all the samples have a negligible erosion rate at a test pressure of 140kPa. Whilst the 7.5 per cent Wardley sample failed to achieve the negligible erosion rate at 140kPa, the margin of failure was in itself negligible and since the 5 per cent Wardley sample managed to satisfy this criterion, it is suggested that this 7.5

per cent result was an anomaly. Of the samples stabilized with 5 per cent cement only Wearmouth was able to resist impact pressures of 210kPa to a sufficient degree.The Wardley samples were unable to satisfy the erosion rate requirement at impact pressures above 210kPa regardless of the quantity of cement used in this study. At impact pressures of 280kPa none of the samples were sufficiently resistant enough when stabilized with 5 per cent cement and only the Wearmouth samples showed sufficient resistance with a 7.5 per cent cement content. At a 10 per cent cement content, both the Westoe and the Wearmouth samples exhibited erosion rates that were below the permissible level as suggested by Oswell and Joshi (1986). As 10 per cent cement content was generally required to reduce the rate of erosion to a negligible level and that this cement content is regarded as the limit for economic viability, the decision was taken that the remainder of the study would be carried out upon samples stabilized with 10 per cent cement.

A study was then carried out to compare the behaviour of minestone crushed to pass a 20mm sieve with that of the same material crushed to pass a 2.36mm sieve to determine the contributions of the matrix material and the aggregate to the erosion resistance of the samples. The test samples were produced from the Wardley, Wearmouth and Westoe minestones stabilized with 10 per cent cement and they were subjected to a range of impact pressures. The results of the comparison are given in *Table 10.7* and illustrated in *Figure 10.7*. It is obvious that the samples with a maximum particle size of 2.36mm were less resistant to the impacting forces and abrading action than similar samples with 20mm maximum aggregate size.

The Wearmouth samples composed of the finer material were, on average, 49 per cent less resistant than the coarser Wearmouth samples, the finer Westoe samples were 42 per cent less resistant while the finer Wardley samples were on average 32 per cent less resistant to erosion than their coarser equivalents. This suggests that the matrix between the larger clasts in the coarser test cubes was less resistant to abrasion than the aggregate clasts themselves. The finer samples were only able to maintain a negligible rate of erosion at the lowest impact pressure (70kPa), at the higher impact pressures the suggested negligible erosion rate was exceeded by all the finer samples.

The results of the erosion/abrasion test performed on all the minestone samples and the laboratory sand mix stabilized with 10 per cent OPC and cured for 7 days in a constant environment room are presented in *Tables 10.8* to *10.11*. In addition, *Figures 10.8* and *10.9* illustrate the relationships between the impact pressure and the resulting erosion rate for these materials. Generally, as the impact pressures increased there was a corresponding increase in the erosion rates. Although a simple linear correlation gives the most significant fit, occasionally with some of the data, extrapolation of that curve suggested that erosion would occur even if the impact pressure was zero, which is obviously impossible. A logarithmic curve is only slightly less significant and is more appropriate as it cuts the impact pressure axis rather than the erosion rate axis. By extrapolating the curve back to the pressure axis, it is possible to estimate at what impact pressures erosion is likely to begin. By employing such a technique it is estimated that no significant erosion would be expected to occur until the impact pressures exceeded 40kPa. The logarithmic curves applied to the data gave significant correlation coefficients ranging from 0.80 for Sutton Manor to 0.99 for the Wardley and POAB samples.

Figure 10.10 compares the erosion rates for the stabilized minestones and the stabilized sand mix for impact pressures from 70 to 280kPa. The most obvious feature of these graphs is the poor performance of the sand mix compared to the minestone samples. The sand mix with a cement content of 10 per cent is on average 5 to 7 times more susceptible to erosion than minestone with an equivalent cement content and curing period. The sand mix fails to meet Oswell and Joshi's (1986) suggested negligible erosion rate at all impact pressures. In comparison, at impact pressures of 70kPa and 140kPa, all the minestone samples exhibit erosion rates below the suggested 2.5cm^3/h limit. Even at the high impact pressures of 210 and 280kPa, the majority of the minestones suffered only negligible erosion, with only Wardley and Hatfield exceeding the suggested limit and only by a small amount. The poorer performance of the Hatfield samples may be explained by its exceptional sensitivity to water immersion. Its propensity to slake and its natural fissility makes it more prone to erosion especially if the abrasion allowed water to penetrate further into the matrix and the individual aggregate pieces. The poorer performance of the Wardley samples is harder to explain, though the fact that the minestone from the Wardley site was subjected to at least 25 years of surface to sub-surface weathering prior to the present research could explain a weakening of particles and hence an increased susceptibility to degradation. Furthermore Wardley minestone contains a higher percentage of the softer lithology, coal, and a lower percentage of the more durable particles such as siltstone/sandstone and ironstone compared to the Wearmouth and Westoe samples, both of which would contribute to the greater susceptibility to erosion of the Wardley minestone.

Oswell and Joshi (1986) concluded that, for the plastic soil-cement mixes used in their study, a compressive strength of 16MPa would be necessary to reduce erosion to a negligible level at an impact pressure of 70kPa. For the higher water pressures of 140, 210 and 280kPa they extrapolated that the limiting compressive strengths would be 25, 32 and 40MPa respectively. To maintain a negligible rate of erosion with stabilized minestone at an impact pressure of 70kPa, a compressive strength of only 2 MPa was required. At 140kPa a strength of 4MPa equated with a negligible rate of erosion ,while at 210 and 280kPa a compressive strength of 6MPa was obtained from samples associated with negligible erosion rates.

Oswell and Joshi (1986) were able to demonstrate that there was exponential decay in erosional losses with increasing compressive strength as well as a linear correlation between the logarithm of erosion rate and the logarithm of compressive strength. Using the strength and erosion data from the Wardley, Wearmouth and Westoe samples stabilized with 0, 5, 7.5 and 10 per cent cement, *Figure 10.11* indicates that a similar exponential decay in erosional losses is apparent with the minestone. The linear relationship that was shown by Oswell and Joshi to exist between the logarithm of erosion rate and the logarithm of compressive strength for their stabilized sands had correlation coefficients of 0.92, 0.86, 0.90 and 0.90 for impact pressures of 70, 140, 210 and 280kPa respectively. The correlations for the Wardley, Wearmouth and Westoe samples cured for 7 days and with a range of cement contents are slightly lower though still significant, with coefficients of 0.87, 0.84, 0.92 and 0.90 for 70, 140, 210 and 280kPa respectively.

Whereas the force of the water jet is the predominant agent in the degradation of the unstabilized samples, with abrasion being an insignificant factor, the reverse is true for the stabilized samples, where particle abrasion became predominant. Although the water jet provides the driving force for the abrading material, it causes very little damage in its own right. *Plates 10.5* and *10.6* illustrate the typical damage suffered by the CSM samples which have been subjected to 60 minutes of erosion/abrasion testing. The form of the erosion is similar for all stabilized samples regardless of the cement content or impact pressure, only the depth of the cavity differs. Typically after 15 minutes of abrasion, a very shallow, smooth, and reniform to hemispherical depression forms, with evidence of pitting both within the matrix and, especially, of the more brittle and softer particles such as coal and soft mudstones. By 30 minutes this cavity had deepened while its cross section had enlarged and become oval. The deepest part of the cavity is at the base, the roof of the cavity is usually shallow and concave in form. Although all the clasts are usually abraded flush with the general surface of the cavity, occasionally durable clasts such as sandstone or those rich in pyrite stand out in relief above the general surface area of the cavity. Following 60 minutes of testing the morphology of the cavity is usually similar to that at 30 minutes, the only difference being the overall deepening of the cavity. At all stages during testing, erosion proceeds in a similar way, the matrix and softer clasts become pitted, while the harder clasts become polished but remaining in relief.

10.2.3 The Effects of Curing and Immersion on the Erosion Rate

10.2.3.1 Long Term Curing in Air

The Wardley, Wearmouth and Westoe samples together with the sand mix samples that had been cured normally for a prolonged period were tested at regular intervals in the erosion/abrasion apparatus. The results of these long term tests are given in *Figures 10.12* to *10.15*. With their of soil-cements Oswell and Joshi (1986) demonstrated that a relationship existed between the erosion resistance and compressive strength, where an increase in strength should be matched with a corresponding decrease in abrasion susceptibility. Although the compressive strength of the Wardley, Wearmouth and Westoe samples doubled or tripled within the study period, a corresponding significant decrease in the erosion rate is plainly not apparent. At impact pressures of 70 and 140kPa very little change is noticeable over the 500 day curing period, although a pattern of decreasing erosion rates with sample age is discernible for the Wardley and Westoe samples tested at 140kPa. At higher impact pressures, the relationship between age, strength and erosion resistance is slightly more apparent though, again, the improvement in erosion resistance is not spectacular despite increases in the compressive strength. However, the improvement is sufficient to lower the erosion rate into the negligible band for the Wardley samples at impact pressures of 210kPa and brings them very close to that limit at impact pressures of 280kPa. A similar situation also occurs with the Westoe samples in that, following a period of extended curing, the erosion rate is below the suggested negligible level. Although a slight decrease in the erosion rate is discernible as the age and compressive strength of the samples increases, a strong and direct relationship between compressive strength and erosion resistance is shown not to exist for stabilized minestone. This is illustrated in *Figure 10.16* which presents the relationship between strength and erosion rate for Wardley minestone.

Figure 10.15 illustrates the erosion rates for the laboratory sand mix cured for a prolonged period in a constant environment room and tested at regular intervals. From these graphs it can be seen that the erosion resistance of the sand mix increases as the curing period lengthens. *Figure 10.17* presents the long term erosion and compressive strength data for the sand mix samples. A direct relationship is apparent between the erosion rate and the compressive strength of these samples. However, full analysis of the data would be pointless as the range of compressive strengths was low, unlike the range used by Oswell and Joshi in their research. The point to be stressed is that with CSM the relationship between compressive strength and erosion susceptibility is not as apparent as with the sand mix. Although this may be attributed to the limited range in the compressive strength data

(Oswell and Joshi used a range of cement contents, PFA contents and curing periods), the difference between the minestone and the sand samples may be attributed to the fundamental difference in grain size of the two samples. In the case of the sand samples, the abrading material was several orders of magnitude greater in size (diameter) and mass than the sample particles, so that the test situation involved a large mass impacting upon particles that have, in comparison, insignificant masses. It is the strength of the cement bond that would determine the erosion resistance of the sand samples. With the cemented minestone, the abrading material was often smaller and lighter than the aggregate particles within the sample, so that the abrasion involved a small mass impacting against a larger particle and, as such particles often occupied over half the surface area being struck by the water jet and the abrading material, the erosion of cement bonded matrix may more often than not be undertested, which may lead to a significant underestimation of the true susceptibility to erosion. The actual durability of the aggregate sized clasts within the minestone samples would primarily determine the durability of the overall sample.

A further factor that may explain the difference in performance between the sand and the minestone samples could be the nature of the particles. The sand particles, being composed of silica could be regarded as being extremely durable, but the hardness that provided their durability would also increase their ability to transmit shock waves through the structure. These shock waves would be propagated through the sand grains and transmitted from grain to grain and into the matrix weakening the cement bond, leading to the degradation of the sample. However, the aggregate within minestone samples was mainly composed of shale, a lithology rich in the clay minerals. The structure of the clays, with their phyllosilicate units, would probably act as a shock absorber. As stacked matresses would absorb the energy on impact of a falling body, so these phyllosilicate units would absorb the forces generated by the impacting abrasive material. This absorption of the energy helps to sustain the cement bonding and so preserve the fabric of the sample, thereby lowering its susceptibility to erosion. The large grain sizes associated with the the minestone samples would also help to absorb any disruptive energy generated by the impact of the abrasive material and similarly, the contact area between the abrading particles and the minestone grains was larger than that developed by the sand grains. This would help dissipate and dilute the effects of the transmitted shock waves produced by the impact of the abrading material. With the sand grains, the contact between individual grains was usually at a single point and associated with the cement bond so, as the shock waves were transmitted from grain to grain they became concentrated in narrow zones at the point contact of the grains and propagated through the thin and weak cement bonds at the contact.

A further reason for the enhanced performance of the stabilized minestone compared to that of the sand mix, is that there is a strong likelihood that the water jet and abrasive material may, have impinged for prolonged periods of time upon a resistant clast such as one rich in pyrite or a well cemented sandstone. This situation would further reduce the erosion rate exhibited by the samples but would not truly reflect the level of erosion susceptibility of the minestone samples. There was a difference in the way the minestone samples abraded compared to the sand mix samples. With the sand samples, whole sand particles were broken from the cement matrix. Observations under the microscope suggested that the sand grains were seldom broken and that the failure had occurred within the cement bond. With the minestone samples, the greater portion of the eroded debris ranged between silt and sand in size and was generally fresh and angular suggesting that it had been broken off larger clasts.Whereas with the sand samples the weakness was evidently between the particles and the cement bond, with CSM the weakness was equally shared between the interparticle bonding and the minestone particles.

10.2.3.2 The Effects Of Long Term Immersion in Distilled Water on Erosion Resistance

Samples from Wardley, Wearmouth and Westoe collieries were immersed for a prolonged period in distilled water and tested at regular intervals in the erosion/abrasion apparatus. As a comparison, the laboratory sand mix with the same quantity of cement was subjected to the same curing conditions and test regime. The results of these tests are given in *Figures 10.18* to *10.21*. All the samples, regardless of the material, showed an improvement in erosion resistance when immersed in distilled water. *Figures 10.18* to 10.*21* show that over a 500 day curing period the majority of the improvement occurred within the initial 100 days. This improvement depended on the impact pressures to which the samples were subjected. The improvement shown by the Wardley and Westoe samples in the first 100 days of curing are largest at the lower impact pressures. At 70kPa the Westoe samples were 38 per cent more resistant than samples cured in air for 7 days while the Wardley samples exhibited a 21 per cent improvement. At 280kPa the improvement in the erosion resistance was 11 per cent for Wardley and 7 per cent for Westoe. For the laboratory sand mix and the Wearmouth samples the reverse is true, the improvement was lower at the lower impact pressures (27 per cent for Wearmouth and 11 per cent for the sand) but was higher when the samples were subjected to higher impact pressures. At 280kPa the Wearmouth samples exhibited an improvement in erosion resistance of 45 per cent when compared to samples cured for 7 days in air, while the sand samples showed an 18 per cent improvement.

A similar pattern exists following 500 days of immersion in distilled water. The improvement for the sand samples and Wearmouth minestone was higher at the high impact pressures (for Wearmouth an improvement of 46 per cent at 280kPa compared to an improvement of 39 per cent at 70kPa and for the sand samples 32 per cent at 280kPa compared to 17 per cent at 70kPa).The Wardley and Westoe samples showed the greatest improvement at the lowest impact pressures. Westoe improved by 32 per cent following 500 days of curing when tested at 70kPa compared to a 10 per cent improvement at 280kPa, while Wardley improved by 19 per cent at 70kPa but only by 12 per cent at 280kPa. The pattern of improvement in erosion resistance as a result of immersion in distilled water can be explained if the level of erosion resistance of the sand and Wearmouth samples were closer to their maximum at the lower pressures than the Westoe and Wardley samples, hence any improvement to their fabric would instigate a smaller percentage increase in erosion resistance than the Westoe and Wardley samples which had a greater scope for improvement. The improvement in the fabric of the Westoe and Wardley was large enough to improve the erosion resistance at the lower impact pressures but failed to impart the same resistance at the higher impact pressures. With the Wearmouth and sand samples, the improvement was smaller at the lower impact pressures because of the lower scope to improve, while the improvement in the fabric of the samples was sufficient enough to increase erosion resistance at the higher impact pressures of 210 and 280kPa.

Generally the improvement in the erosion resistance of the samples continued until they were 200 days old and the level of erosion resistance remained relatively constant up to the end of the research period (500 days), though in the case of the Wardley samples, there was some deterioration in performance following 300 days of curing, though this deterioration was very slight and did not exceed the original 7 day erosion rate exhibited by the sample.

On the whole, the laboratory sand mix samples exhibited less improvement in erosion resistance over the 500 day curing period than the stabilized minestone samples and the Wardley samples showed the least improvement of the minestones. The enhanced performance of the stabilized minestone compared to the stabilized sand samples may be explained by pozzolanic reactions between the cement and the clay minerals present in the minestone. The sand mix being composed predominantly of inert quartz, with clay minerals being absent or present in only trace quantities, did not have the opportunity for these pozzolanic reactions. The poorer performance of the Wardley samples compared to the other minestones may be explained that, as a result of pre-weathering on the tip, it contained a higher percentage of softer lithologies, weathered products and a lower percentage of durable lithologies.

Figure 10.22 shows the relationship between the erosion resistance and compressive strength of the sand mix samples with 10 per cent cement. As compressive strength increases a corresponding decrease occurs in the erosion susceptibility. *Figure 10.23* shows that a similar relationship exists for the stabilized Wearmouth samples, while *Figure 10.24* shows that though a similar relationship exists for Westoe, the correlation is not as strong. *Figure 10.25* gives the strength/erosion resistance relationships for the Wardley samples. These samples show the lowest correlation between compressive strength and erosion resistance, though a broad pattern is discernible. The correlation between compressive strength and erosion resistance in minestone samples cured in distilled water is in contrast to the poor correlation of these two properties when the samples are cured in sea water. The following section will discuss the long term effects of sea water immersion on the erosion resistance of stabilized minestone.

To summarise, long term immersion in distilled water, which in practice could equate to a fresh water situation, has no detrimental effect upon either a sand or minestone sample's resistance to erosion. For the first 200 days, the erosion resistance increases as the compressive strength increases and then both remain relatively steady for the remainder of the test period of 500 days.

10.2.3.3 The Effects of Long Term Immersion in Sea Water on Erosion Resistance

Wardley, Wearmouth and Westoe and sand mix samples were also subjected to the erosion/abrasion test following long term immersion in sea water. *Figures 10.26* to *10.28* give the erosion rates plotted against the curing period for the three minestones at the various test pressures. Although the erosion susceptibility generally increases with an increase in the immersion period, this being particularly noticeable at the higher impact pressures, it is considered that the erosion damage caused by the water jet and abrasive material did not reflect the true erosion susceptibility of these samples. It has been shown in *Chapter 9* that the minestone samples suffered quite severe strength loss as well as fabric disruption (*see Plates 9.3 to 9.6*) when immersed in sea water for a prolonged period. The fabric disruption rendered the samples prone to disintegration when handled, even under the lightest of touches. This disruption usually occurred at the edges and corners and produced a thin superficial skin on the faces of the samples. The impact of the water jet during the erosion/abrasion test was usually well away from the vulnerable periphery, although it did impact upon the weakened surface skin but this only contributed superficially to the overall disruption of the test material. Once the weakened skin and scale had been removed, the water jet and abrasive material impinged on firmly emplaced and durable aggregate. This is similar to the situation that occurred with the air cured samples, where individual clasts bore the brunt of the impacting jet and the erosion rate mainly reflected the durability of these clasts and not that of the whole sample.

Although testing the edges was difficult, because of their frailty, a limited test programme was carried out upon Wearmouth samples to compare the rates of erosion at the edges and corners with those experienced at the centre of the faces. *Figure 10.29* summarises the erosion rates experienced on the faces following 500 days immersion in sea water and on the edges and corners of the samples following 35 and 500 days immersion in sea water. It can be seen that the edges and corners were far more susceptible to erosion, after only 35 days in sea water they exhibited more damage than did the faces which had been subjected to 500 days of sea water immersion. After 500 days at an impact pressure of 70kPa, the edges and corners were 9 times more susceptible to erosion than the faces of the same sample, while at 140, 210 and 280kPa they were 5, 6 and 4.5 times more susceptible respectively. With only 35 days immersion, the edges showed, on average, 3.5 times greater erosion than the faces which were tested after 500 days immersion. However, even at the corners and edges it was observed that once the disrupted fabric had been removed, and this occurred rapidly, the water jet again impinged on relatively sound material so that the rate of erosion became similar to that experienced when the faces were tested. The disrupted fabric was usually removed in large pieces by the water jet and abrasive material within 15 minutes of the test commencing, erosion would then proceed in a similar way to that experienced on the faces with silt and sand size particles being abraded from the matrix and the aggregate.

The sand-cement mix samples were also tested in the erosion/abrasion apparatus following periods of immersion in sea water and *Figure 10.30* gives the erosion rates for these samples plotted against the curing period. It can be seen from these graphs that a similar pattern exists for the erosion rates at each of the impact pressures. For the first 200 days of immersion the erosion rate decreased before levelling out after 300 days and then increased as the curing period extended beyond 400 days eventually reaching a similar level to that experienced following 7 days of normal curing.

Figures 10.31 illustrates the correlation between the compressive strength of the sand samples and the erosion rates they experienced when tested. Initially as the strength of the samples increased there was a corresponding decrease in the erosion rate, but after 200 days of immersion the compressive strength stopped increasing and this was accompanied by a levelling out of the erosion rate. The later stages of immersion were characterised by a decrease in compressive strength which correlates well with an increase in the erosion rate reflecting the deterioration of the sample fabric. A strong and distinctive relationship between erosion rate and compressive strength could not be said to exist for the minestone samples immersed in sea water. This is illustrated in *Figure 10.32* which gives the relationship between the erosion rate and compressive strength of the Wardley samples as an example.

From the earlier discussion, it is clear that the maximum grain size in the samples plays a major role in determining the erosion rate of the samples. Although it can be concluded that the cement stabilized minestone performed better than the sand samples stabilized with an equivalent amount of cement, it should also be stated that the minestone samples with a maximum particle size of 20mm show a lack of sensitivity to the erosion/abrasion test. Basically, it was found that it was possible for the water jet and abrasive material to act upon a single aggregate piece for the majority of the test period. The erosion resistance of that individual particle may or may not represent the erosion resistance of the sample as a whole.

Whereas the results gained from testing the sand samples may be directly correlated with what happens in a field situation because of its homogeneous nature, the results gained from testing stabilized minestone should be qualified, as the abrasive material only impinged on a small area of a heterogeneous sample and could in some instances have acted solely on an individual aggregate clast. Concern is further heightened by the consideration that minestone may contain clasts several hundreds of millimetres in diameter and as it is very unlikely that minestone would undergo any major modification prior to being used, the results obtained from this study may not truly apply to conditions that would be experienced if minestone was to be used in actual field conditions.

Despite these reservations, the results derived from the present research still suggest that stabilized minestone is more resistant to erosion than a sand stabilized with the same quantity of cement (i.e. 10 per cent). A comparison between the erosion rate of sand samples and minestone with a maximum particle size of 2.4mm (a more relevant comparison than comparing sand with a minestone with a 20mm maximum aggregate size) is illustrated in *Figure 10.33*. The results show that minestone is between 4 and 7 times more resistant to erosion than the stabilized sand at impact pressures of 70kPa, 2 to 3 times at 140kPa and 4 times more resistant at pressures of 210 and 280kPa, further reinforcing the positive conclusion with regard to the erosion resistance of these cement stabilized minestones.

10.3 Summary

10.3.1 Unstabilized Material

All the minestones possess a greater resistance to erosion than the laboratory sand which represents a traditional dredged hydraulic sand. This is true at all the impact pressures used during testing and was particularly obvious at the higher impact pressures. With unstabilized materials, such as minestone and the sand mix, the greatest cause of erosion and damage is the impact of water jet impinging upon the sample and washing away the material, as opposed to any abrasive action due to the presence of the eroding stones or of material derived from the sample itself. Both the minestone and the sand mix samples exceed the 2.5 cm^3/h limit suggested by Oswell and Joshi (1986) as being the acceptable maximum rate of erosion. However, this limit was suggested with reference to soil-cement structures and not unstabilized materials.

The performance of minestone during the erosion/abrasion test depends upon the grading characteristics of the material. Samples with a predominance of gravel-size particles and insufficient matrix are susceptible to the winnowing effects of the water jet and exhibit a lower resistance to erosion than a sample which has sufficient quantities of fines to fill the space between the aggregate clasts. Samples with a high percentage of fine material (< 5 mm) also experience greater susceptibility to erosion. Samples which have sufficient aggregate, matrix and cohesive fines will possess a greater resistance to erosion than would samples which possess just an adequate aggregate and matrix grading only but a deficiency in fines content.

10.3.2 Stabilized Material

Erosion resistance increases dramatically when only a small quantity of cement, as little as 5 per cent, is used to stabilize the minestone. The improvement is further increased when 7.5 per cent cement is used but further improvement is limited with the addition of an extra 2.5 per cent cement. At the lower impact pressures, 5 per cent cement is sufficient to keep the erosion rate to a minimum, but at higher pressures, 7.5 and 10 per cent cement contents are necessary and, in the case of Wardley, even these quantities were inadequate to achieve a negligible rate of erosion, though it only marginally exceeds the suggested 2.5cm^3/h limit. Grading is shown to control erosion resistance, with finer samples being more susceptible to erosion compared to samples made from coarser particles. Although the water supplied the energy to propel the abrading particles, it causes very little damage in its own right.

CSM is less susceptible to erosion than the sand mix stabilized with an equivalent percentage of cement. Higher compressive strengths are required to reduce abrasion to a minimum for the sand mix samples compared to that of CSM. The relationship between compressive strength and abrasion resistance for minestone is shown to be less reliable than that for the stabilized sand mix. The CSM samples increased resistance to abrasion may be due to an increased ability to absorb shock waves produced by impacting projectiles, compared to the structure of the sand-cement fabric. Whereas with the sand samples the weakness is evidently between the particles and the cement bond, with CSM the weakness is equally shared between the bonding and the minestone particles themselves. The sensitivity of test may be in doubt when testing CSM samples with large aggregate pieces, as the test area is often confined to a single aggregate piece which cannot be regarded as representative of the sample as a whole. Erosion resistance of both the sand mix and the CSM samples improves in line with compressive strength when immersed in distilled water. When the samples are immersed in sea water the erosion resistance initially improves but then gradually declines following sulphate attack on the fabric of the samples, again reflecting changes in compressive strength.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.17	49.28	22.71	17.99	21.72	16.60
		42.27	19.48		18.65	
		25.54	11.77		9.43	
L. Windsor	2.00	21.02	10.51	10.29	24.15	16.67
		16.92	8.46		10.76	
		23.78	11.89		15.11	
POAA	1.89	42.94	22.72	18.71	23.45	18.76
		37.55	19.87		17.81	
		25.57	13.53		15.03	
POAB	2.03	16.57	8.16	8.79	10.16	9.76
		20.99	10.34		10.79	
		16.00	7.88		8.33	
S.Manor	2.16	31.45	14.56	14.79	16.54	16.65
		22.01	10.19		10.98	
		42.40	19.63		22.43	
Wardley	1.92	22.00	11.45	14.50	19.40	20.92
		17.42	21.59		23.72	
		41.45	9.07		27.00	
		30.50	15.89		35.00	
Wearmouth	2.04	17.25	8.46	10.94	9.42	12.19
		26.35	12.92		11.53	
		23.36	11.45		15.62	
Westoe	2.17	33.53	15.45	15.74	18.45	17.50
		25.65	11.82		11.62	
		43.27	19.94		25.43	
Lab Sand	2.40	61.01	25.42	23.82	------	-------
		73.49	30.62		------	
		57.03	15.43		------	

Table 10.1 Erosion rates for unstabilized materials at impact pressures of 70kPa.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.20	53.79 38.79 44.84	24.45 17.63 20.38	20.82	27.74 22.38 26.71	25.61
L. Windsor	1.97	22.20 16.76 32.94	11.27 8.71 16.72	12.16	15.99 13.54 17.43	15.65
POAA	1.91	48.07 58.68 37.26	25.17 30.72 19.51	25.13	31.16 33.42 22.21	28.93
POAB	1.99	20.22 30.61 18.75	10.16 15.38 9.42	11.65	16.33 24.29 9.71	16.78
S.Manor	2.15	40.09 36.45 48.49	18.65 16.95 22.55	19.38	25.99 23.74 33.24	27.66
Wardley	1.89	71.01 94.30 74.65	37.57 49.90 39.40	42.29	52.30 65.00 54.38	57.23
Wearmouth	2.04	31.52 21.66 36.50	15.45 10.62 17.89	14.65	17.43 13.52 12.19	14.38
Westoe	2.19	49.09 84.51 86.25	22.41 38.64 39.38	33.48	36.72 38.04 45.00	39.42
Lab Sand	2.35	81.92 95.50 104.47	35.62 41.52 45.42	40.85	------- ------- -------	-------

Table 10.2 Erosion rates for unstabilized materials at impact pressures of 140kPa.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.19	98.92 109.52 87.36	45.17 50.01 39.89	45.02	50.42 54.62 44.17	49.74
L. Windsor	2.01	52.96 40.56 42.89	26.35 20.18 21.34	22.62	34.16 30.54 30.72	31.81
POAA	1.85	84.77 69.09 90.02	45.82 37.33 48.66	43.94	38.45 44.62 45.32	42.80
POAB	2.03	41.31 55.13 37.41	20.35 27.16 18.43	21.98	28.00 28.05 22.16	24.40
S.Manor	2.16	61.58 50.43 45.42	28.51 23.35 21.03	24.36	31.51 35.56 30.47	32.51
Wardley	1.95	104.35 132.67 92.87	55.21 67.01 46.67	56.30	59.32 65.30 40.62	55.08
Wearmouth	2.09	53.21 47.91 32.04	25.46 45.82 15.33	28.87	32.43 35.52 28.17	32.04
Westoe	2.11	57.90 68.42 53.55	27.44 32.43 25.38	28.42	32.43 35.52 42.45	36.80
Lab Sand	2.35	260.52 247.57 230.93	110.86 105.35 98.27	104.82	------ ------ ------	------

Table 10.3 Erosion rates for unstabilized materials at impact pressures of 210kPa.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.20	143.90 152.20 128.19	65.41 69.18 58.27	64.29	59.47 62.57 52.45	58.16
L. Windsor	2.00	71.24 55.24 60.86	35.62 27.62 30.43	31.22	43.17 38.92 41.15	41.08
POAA	1.89	112.68 122.32 100.32	59.62 64.72 53.08	59.14	50.47 55.36 49.14	51.86
POAB	2.00	51.24 52.00 62.86	25.62 26.00 31.43	27.68	30.38 29.89 36.17	32.15
S.Manor	2.16	108.51 92.54 114.11	50.24 42.84 52.83	48.64	51.61 47.43 54.44	51.16
Wardley	1.95	84.86 56.19 112.48	43.30 28.82 57.68	62.23	51.84 53.94 65.92	57.23
Wearmouth	2.05	62.23 54.07 74.71	30.36 26.28 37.45	31.39	37.42 45.39 35.36	39.39
Westoe	2.17	81.22 76.86 92.96	37.43 35.42 42.84	38.56	45.62 47.28 37.45	43.45
Lab Sand	2.35	380.68 355.57 435.01	165.50 154.59 189.13	169.74	------ ------ ------	------

Table 10.4 Erosion rates for unstabilized materials at impact pressures of 280kPa.

Impact pressure kPa	Minestone	Cement Content					
		5%		7.5%		10%	
		Erosion Rate and Maximum Cavity Depth					
		cm^3/h	mm	cm^3/h	mm	cm^3/h	mm
70	Wardley	1.68	7.37	0.91	5.75	0.72	4.23
	Wearmouth	0.58	4.56	0.70	3.67	0.66	4.15
	Westoe	1.65	8.60	0.72	4.59	0.69	3.17
140	Wardley	2.45	9.65	2.71	9.35	2.38	8.68
	Wearmouth	2.18	8.25	1.24	6.77	1.43	8.15
	Westoe	4.79	15.50	1.62	5.29	1.31	6.41
210	Wardley	4.22	13.47	4.13	12.83	3.09	10.41
	Wearmouth	2.48	9.21	1.84	7.65	2.14	6.91
	Westoe	5.35	14.87	2.46	8.99	1.94	7.87
280	Wardley	7.97	21.24	4.89	15.29	3.96	11.04
	Wearmouth	3.55	12.19	2.03	8.03	2.25	9.17
	Westoe	6.69	18.81	2.82	11.22	2.52	9.29

Table 10.5 Effect of cement content on the erosion rate of cement stabilized minestone.

Impact Pressure	Cement Content		
	5 %	7.5%	10%
70kPa	Wardley Wearmouth Westoe	Wardley Wearmouth Westoe	Wardley Wearmouth Westoe
140kPa	Wardley Wearmouth	Wearmouth Westoe	Wardley Wearmouth Westoe
210 kPa	Wearmouth	Wearmouth Westoe	Wearmouth Westoe
280kPa		Wearmouth	Wearmouth Westoe

Table 10.6 Minestone showing negligible rates of erosion (<2.5 cm^3/h) at varying cement contents and test impact pressures.

Impact pressure (kPa)	Minestone grade	Bulk density (Mg/m^3)	Comp. strength (MPa)	M/C (%)	Erosion rate (cm^3/h)	Cavity depth (mm)
70	Wardley <20mm	2.06	4.53	8.12	0.72	4.23
	Wardley < 2.4mm	1.95	2.37	10.25	1.16	5.24
140	Wardley < 20mm	2.06	4.53	8.12	2.38	8.68
	Wardley < 2.4mm	1.95	2.37	10.25	3.41	12.99
210	Wardley < 20mm	2.06	4.53	8.12	3.09	10.41
	Wardley <2.4mm	1.95	2.37	10.25	4.38	13.07
280	Wardley < 20mm	2.05	4.53	8.12	3.96	11.04
	Wardley < 2.4mm	1.95	2.37	10.25	5.65	14.53
70	Wearmouth < 20 mm	2.24	4.83	8.14	0.66	4.15
	Wearmouth < 2.4mm	2.05	2.48	11.25	1.34	5.65
140	Wearmouth < 20mm	2.24	4.83	8.14	1.31	6.41
	Wearmouth < 2.4mm	2.05	2.48	11.25	2.93	10.03
210	Wearmouth < 20mm	2.24	4.83	8.14	2.14	6.91
	Wearmouth < 2.4mm	2.05	2.48	11.25	3.44	10.84
280	Wearmouth < 20mm	2.24	4.83	8.14	2.25	9.17
	Wearmouth <2.4mm	2.05	2.48	11.25	4.53	13.60
70	Westoe < 20mm	2.24	4.82	9.48	0.69	3.17
	Westoe < 2.4mm	2.14	2.51	11.25	1.03	4.91
140	Westoe < 20mm	2.24	4.82	9.48	1.43	8.15
	Westoe < 2.4mm	2.14	2.51	11.25	2.82	9.95
210	Westoe < 20mm	2.24	4.82	9.48	1.94	7.87
	Westoe < 2.4mm	2.14	2.51	11.25	3.28	10.77
280	Westoe < 20mm	2.24	4.82	9.48	2.52	9.29
	Westoe < 2.4mm	2.14	2.51	11.25	4.54	13.30

Table 10.7 Effect of grading on the erosion rate of minestone stabilized with 10 per cent OPC.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.19	1.20 1.78 1.98	0.55 0.81 0.90	0.75	3.20 4.16 3.16	3.51
L. Windsor	2.29	1.78 1.21 1.16	0.78 0.53 0.51	0.61	3.13 2.17 2.89	2.73
POAA	2.11	1.66 1.89 2.13	0.79 0.90 1.01	0.90	2.51 3.27 3.03	2.94
POAB	2.15	1.86 2.09 1.62	0.87 0.97 0.75	0.86	2.89 2.50 3.12	2.84
S.Manor	2.21	1.92 1.53 1.70	0.87 0.69 0.77	0.78	3.60 2.79 3.21	3.20
Wardley	2.06	1.72 1.58 1.14	0.84 0.77 0.55	0.72	4.64 3.88 4.16	4.23
Wearmouth	2.20	1.32 1.20 1.82	0.60 0.54 0.83	0.66	3.32 4.00 5.14	4.15
Westoe	2.24	1.28 1.95 1.50	0.57 0.87 0.62	0.69	3.32 3.96 2.24	3.17
Lab Sand	2.35	10.25 13.77 11.26	4.36 5.86 4.79	5.00	12.94 13.52 11.50	12.65

Table 10.8 Erosion rates for material plus 10 per cent OPC at impact pressures of 70kPa.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.19	4.53	2.32	2.30	9.74	8.57
		5.09	2.23		8.18	
		4.89	2.34		7.79	
L. Windsor	2.29	1.33	0.58	0.79	3.11	3.57
		2.13	0.93		3.98	
		1.98	0.87		3.62	
POAA	2.11	2.09	0.99	1.15	5.05	4.61
		3.21	1.52		4.51	
		2.01	0.95		4.27	
POAB	2.15	2.91	1.35	1.52	3.76	4.01
		3.12	1.45		3.98	
		3.79	1.76		4.29	
S.Manor	2.21	1.84	0.83	0.86	3.70	3.84
		2.26	1.02		4.21	
		1.62	0.73		3.62	
Wardley	2.06	3.72	1.82	2.29	8.88	9.49
		6.10	2.95		10.70	
		4.34	2.11		8.88	
Wearmouth	2.20	1.94	0.86	1.31	5.52	6.41
		3.27	1.49		6.94	
		3.50	1.59		6.78	
Westoe	2.24	1.62	0.72	1.43	----	8.15
		4.14	1.85		6.76	
		3.82	1.71		9.54	
Lab Sand	2.35	15.32	6.51	6.90	15.15	15.21
		17.04	7.25		16.22	
		16.25	6.92		14.25	

Table 10.9 Erosion rates for material plus 10 per cent OPC at impact pressures of 140kPa.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.19	9.54	4.36	3.97	12.04	10.22
		8.16	3.73		10.01	
		8.38	3.83		8.61	
L. Windsor	2.29	5.19	2.26	1.54	5.45	4.30
		3.16	1.38		4.17	
		2.25	0.98		3.27	
POAA	2.11	3.81	1.80	1.65	5.61	5.18
		3.09	1.46		5.03	
		3.57	1.69		4.89	
POAB	2.15	4.16	1.93	1.85	5.13	4.66
		3.86	1.80		4.36	
		3.94	1.83		4.49	
S.Manor	2.21	4.33	1.96	1.88	4.68	4.19
		3.39	1.79		4.00	
		4.16	1.88		3.89	
Wardley	2.06	6.05	2.93	3.09	8.44	10.41
		6.73	3.26		12.34	
		6.20	3.09		10.44	
Wearmouth	2.20	5.85	2.66	2.14	7.20	6.91
		5.15	2.35		7.92	
		3.12	1.41		5.62	
Westoe	2.24	2.47	1.90	1.94	8.34	9.48
		4.62	2.07		8.18	
		4.18	1.86		7.10	
Lab Sand	2.35	24.74	10.53	11.28	18.52	20.09
		25.94	11.04		20.21	
		28.81	12.26		21.53	

Table 10.10 Erosion rates for material plus 10 per cent OPC at impact pressures of 210kPa.

Minestone	Bulk density (Mg/m^3)	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Hatfield	2.19	10.33	4.72	5.03	13.70	13.63
		10.86	4.95		14.22	
		11.89	5.43		12.98	
L. Windsor	2.29	3.36	1.47	1.56	4.90	4.79
		4.25	1.86		5.27	
		3.06	1.34		4.20	
POAA	2.11	4.79	2.27	2.25	6.61	7.02
		4.19	1.99		6.87	
		5.29	2.50		7.59	
POAB	2.15	4.11	1.91	2.11	6.00	5.00
		4.62	2.15		4.75	
		4.86	2.26		4.25	
S.Manor	2.21	4.45	2.01	1.95	4.17	4.16
		3.82	1.73		4.51	
		4.67	2.11		3.79	
Wardley	2.05	9.49	4.63	3.96	11.94	11.04
		6.46	3.16		9.92	
		8.36	4.08		11.26	
Wearmouth	2.20	5.52	2.51	2.25	11.08	9.17
		4.81	2.18		8.08	
		4.56	2.07		8.34	
Westoe	2.24	6.11	2.73	2.52	10.56	9.29
		5.15	2.30		9.08	
		5.69	2.54		8.22	
Lab Sand	2.35	42.93	18.27	17.33	26.42	27.32
		38.66	16.45		27.53	
		40.61	17.28		28.00	

Table 10.11 Erosion rates for material plus 10 per cent OPC at impact pressures of 280kPa.

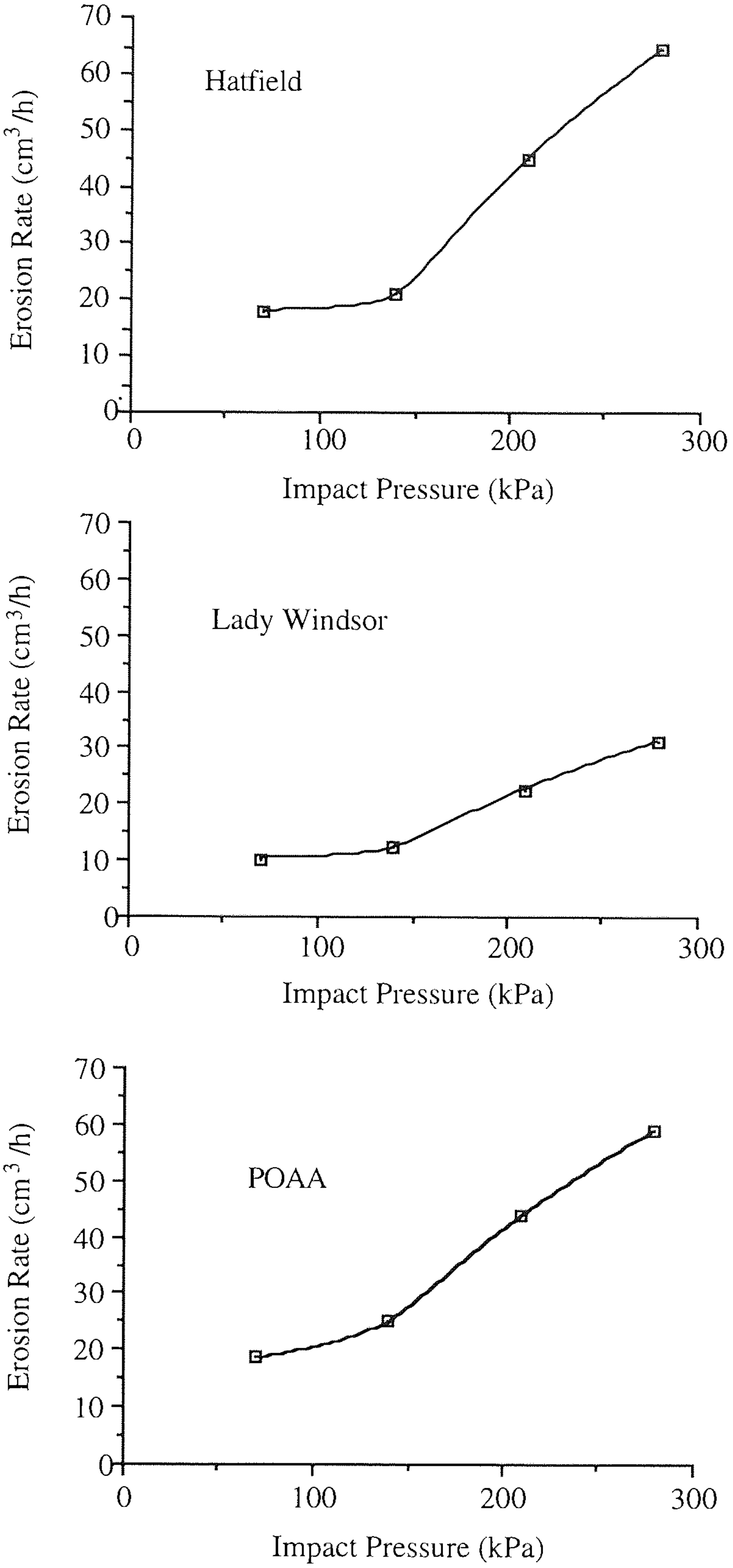


Figure 10.1 The relationship between impact pressure and the erosion rate for unstabilized minestone.

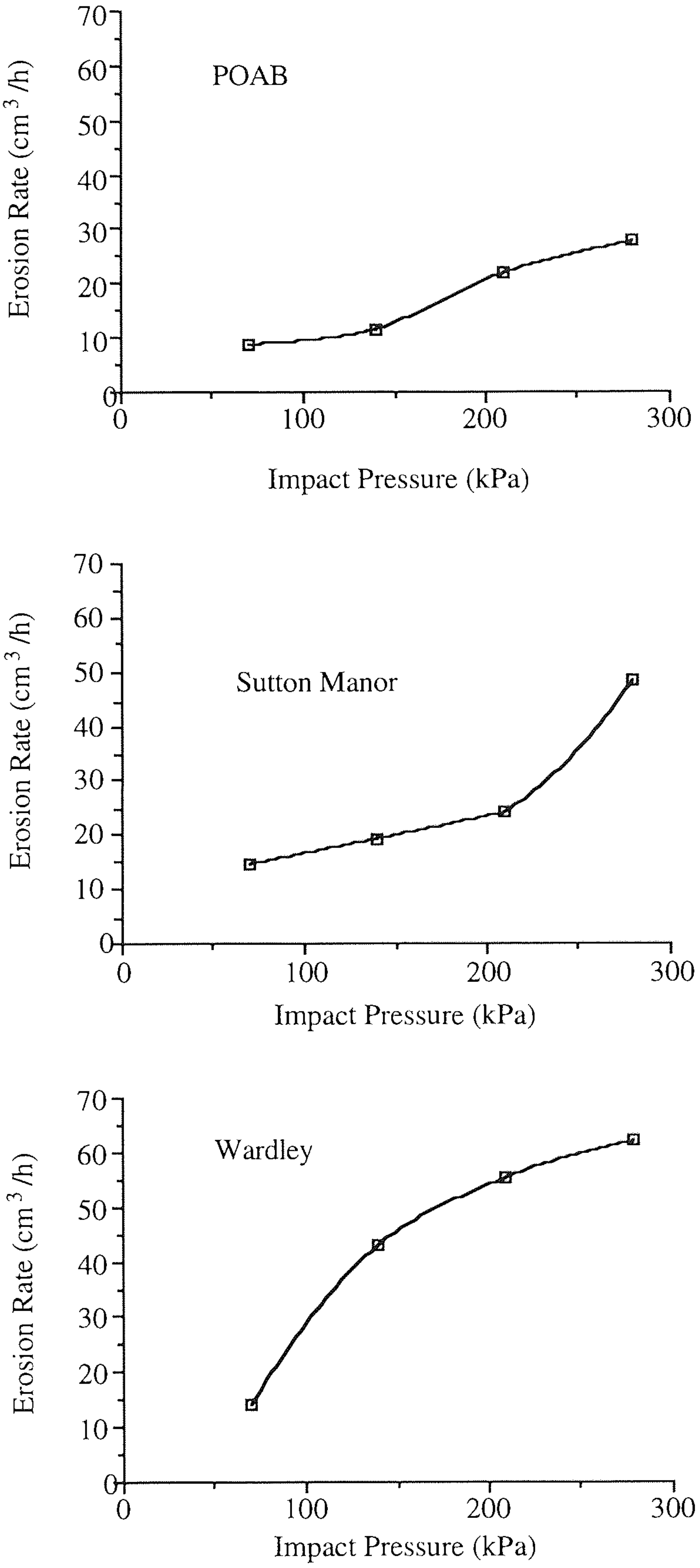


Figure 10.1 (continued).

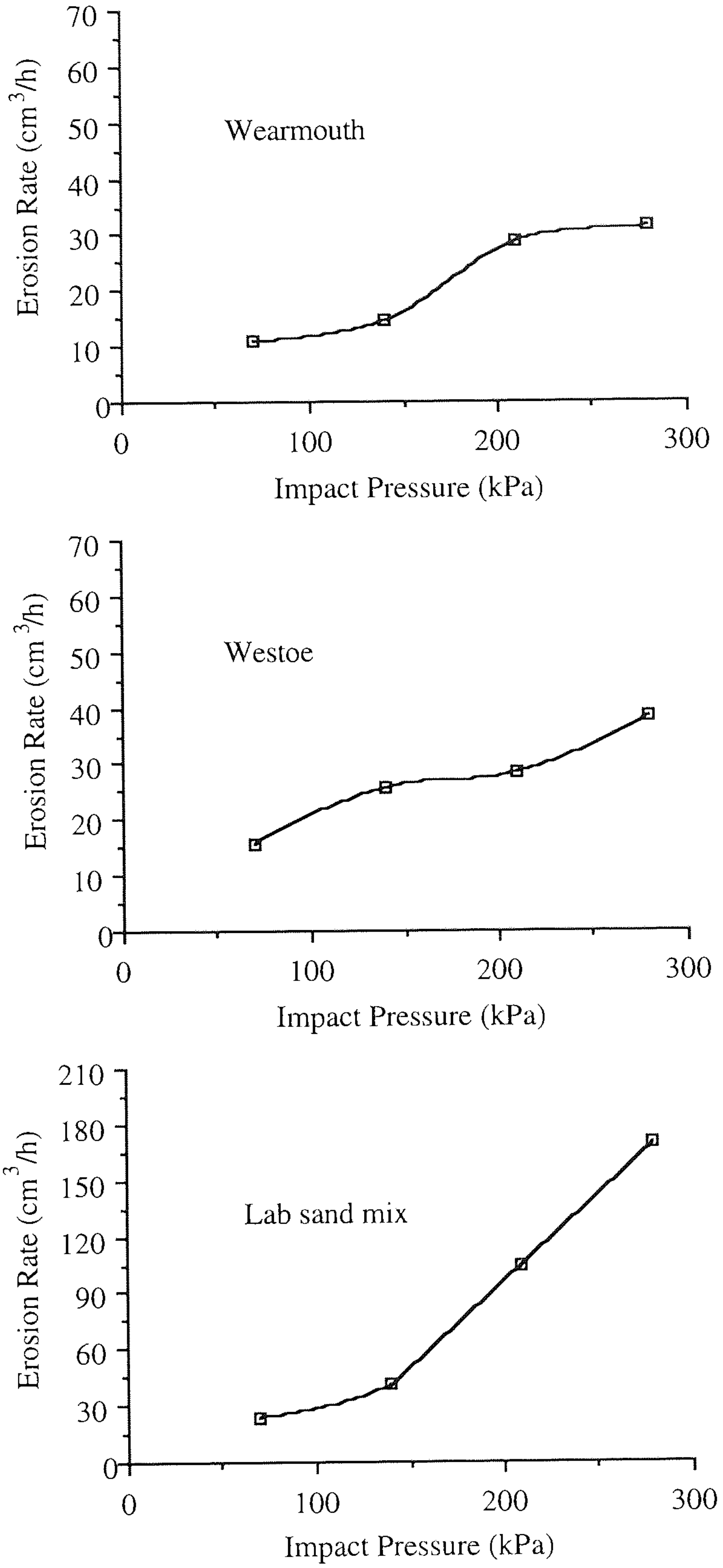
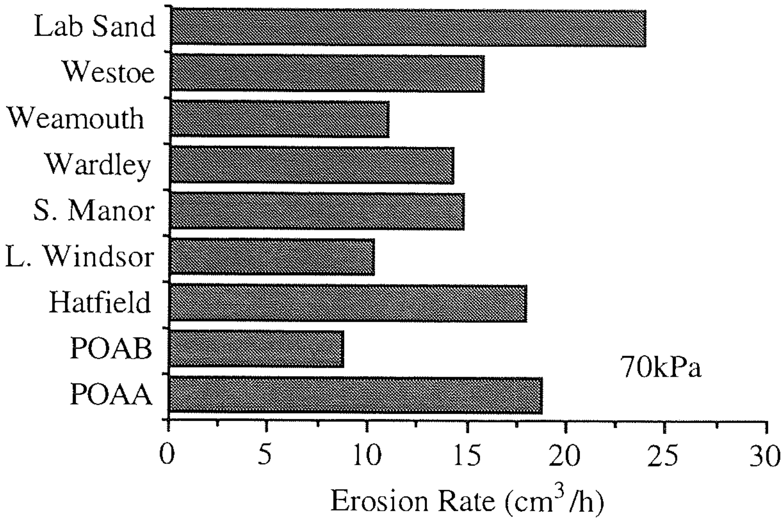


Figure 10.1 (continued).



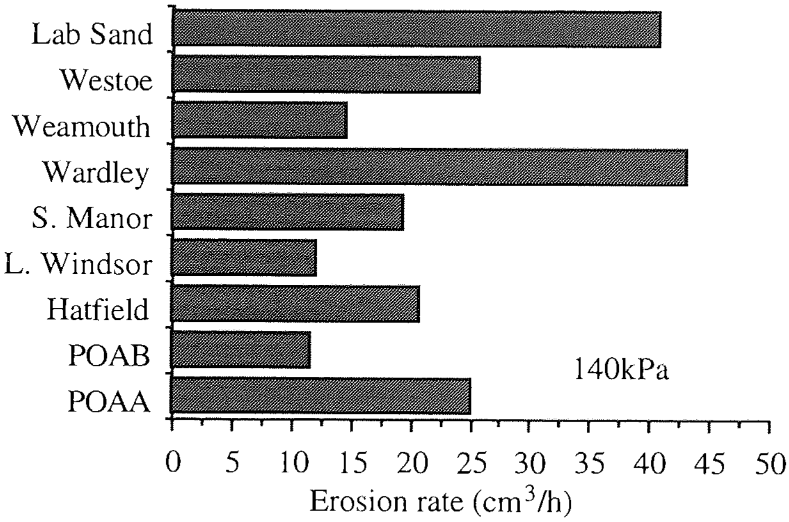


Figure 10.2 Erosion rates for unstabilized materials at various impact pressures.

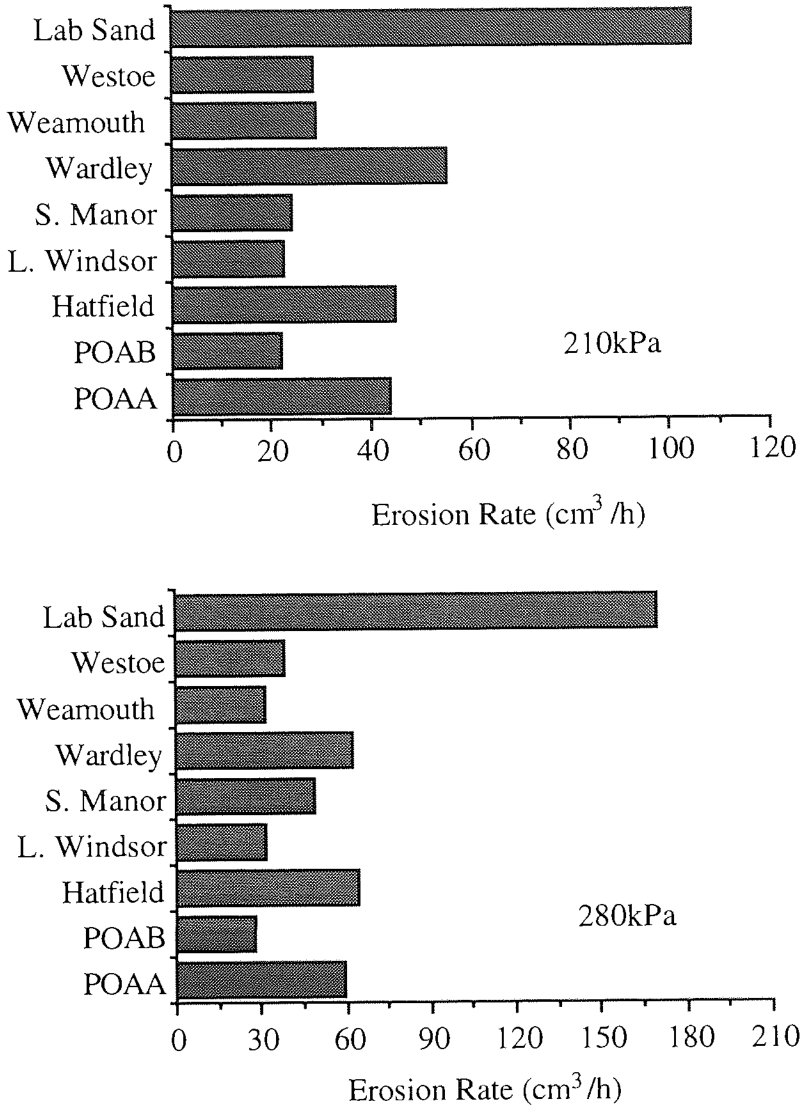
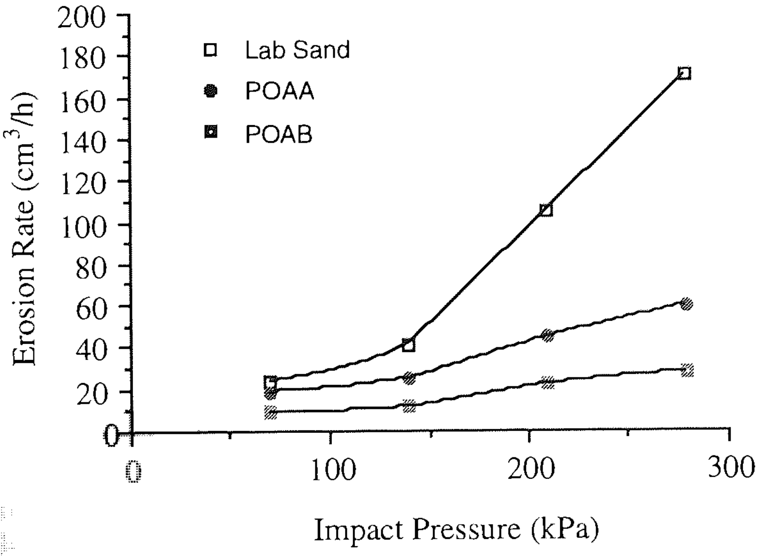


Figure 10.2 (continued).



10.3 A comparison showing the relative superiority of minestone over the sand mix in relation to erosion resistance.

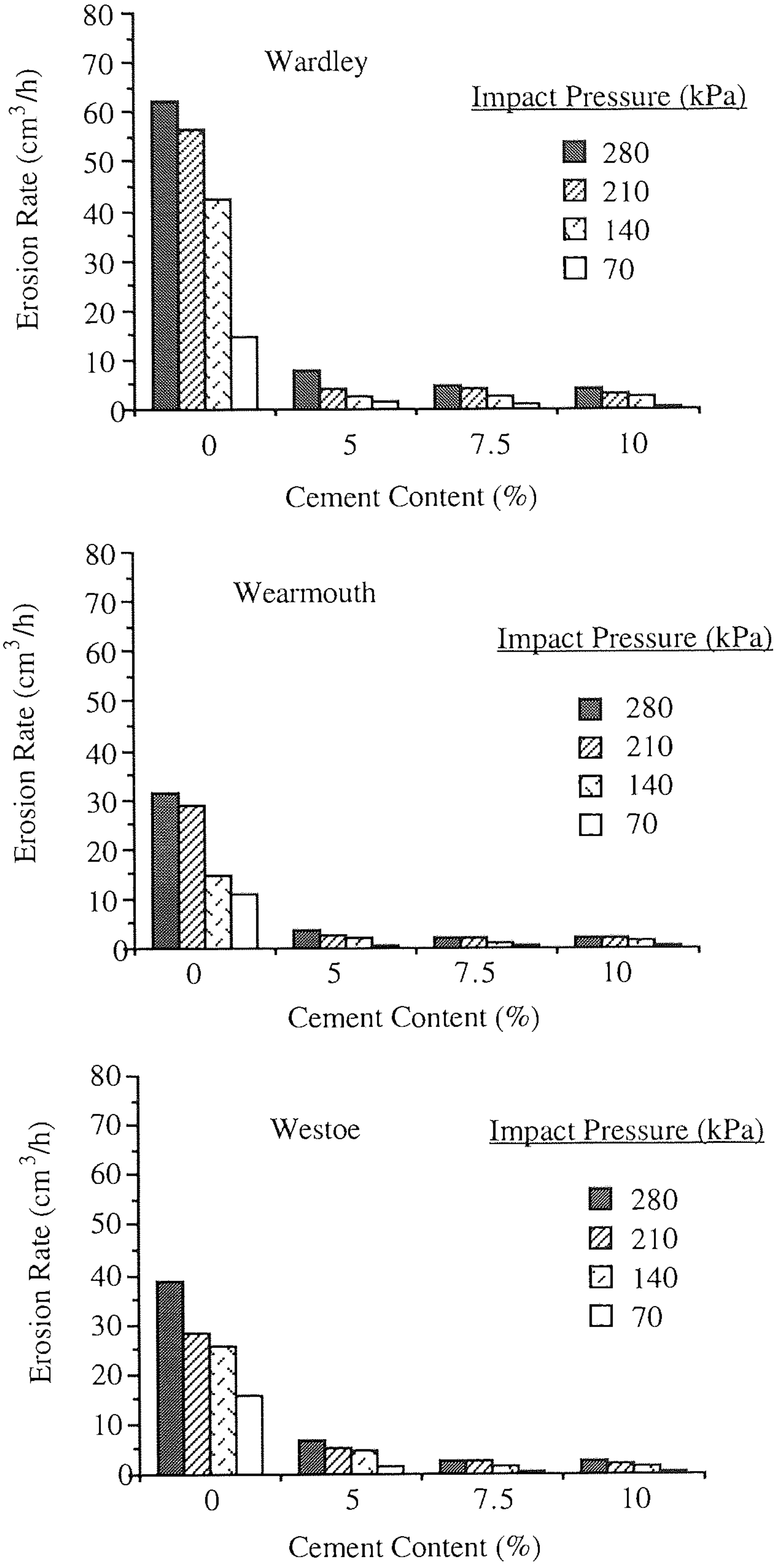


Figure 10.4 The relationship between cement content, impact pressure and erosion rate for minestone stabilized with ordinary portland cement.

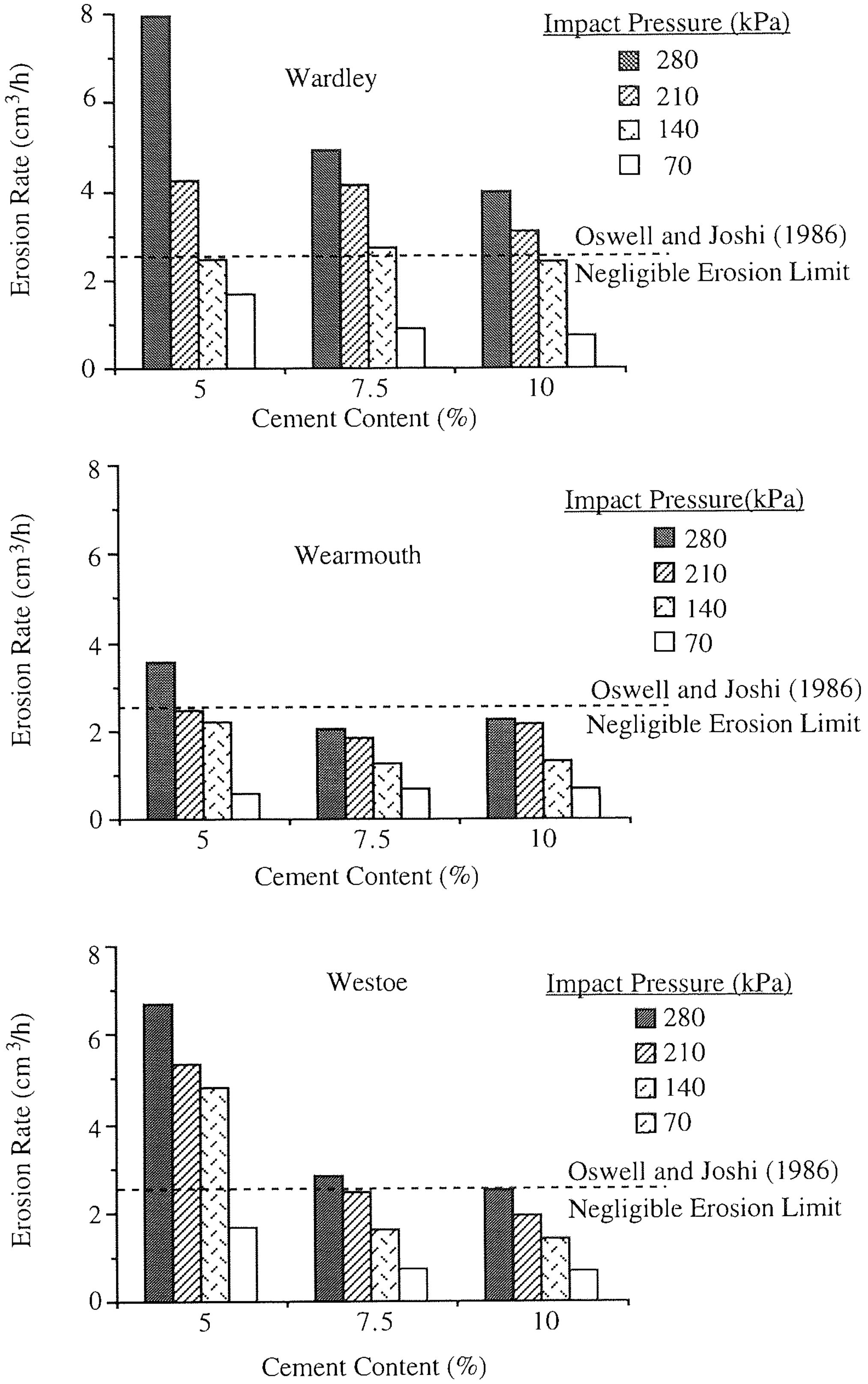


Figure 10.5 Detailed section of *Figure 10.4* illustrating the relationship between cement content and the erosion rate for stabilized minestone.

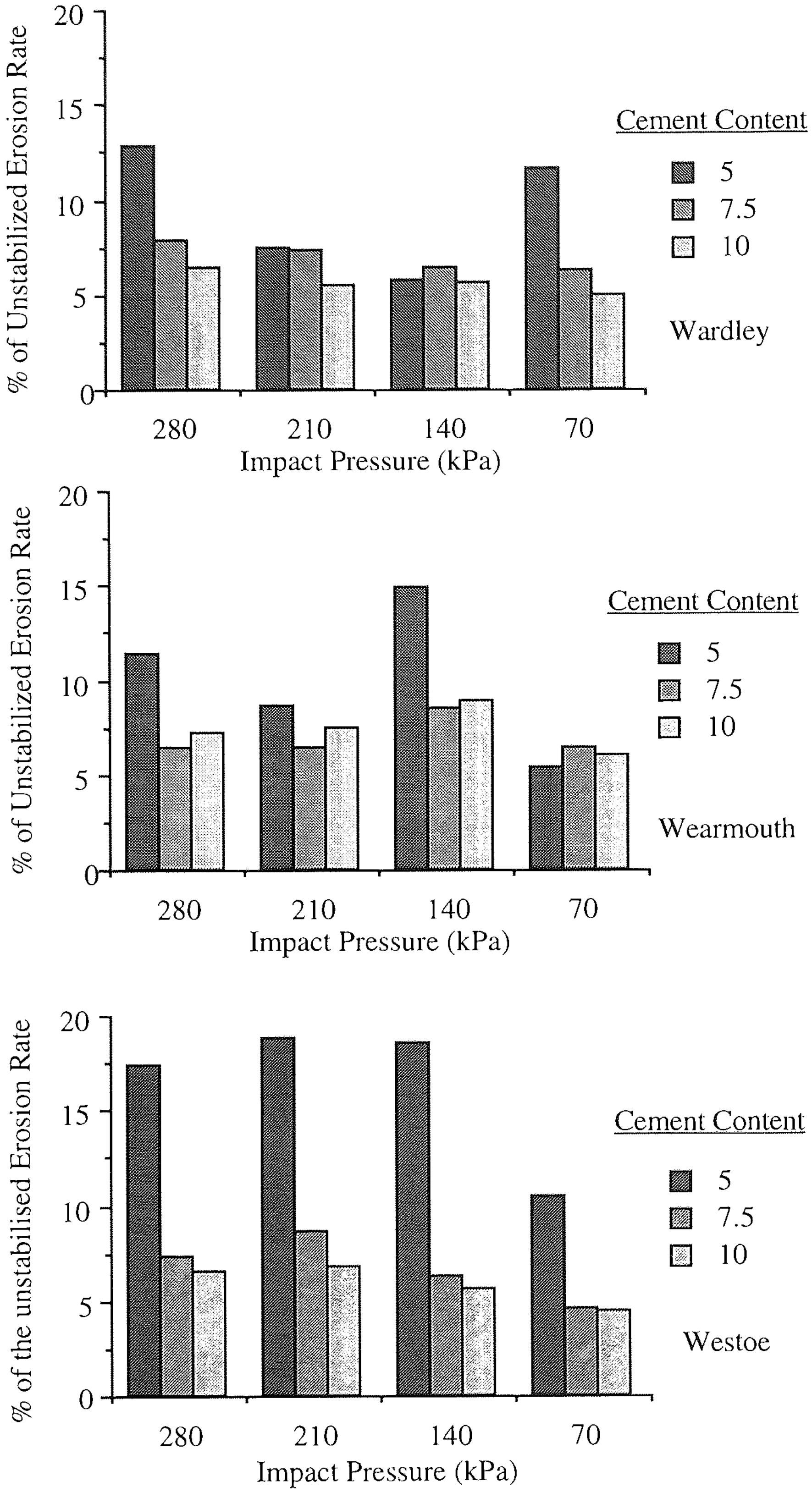


Figure 10.6 The erosion rate of stabilized minestone plotted as a percentage of the unbound rate to illustrate the improvement in erosion resistance.

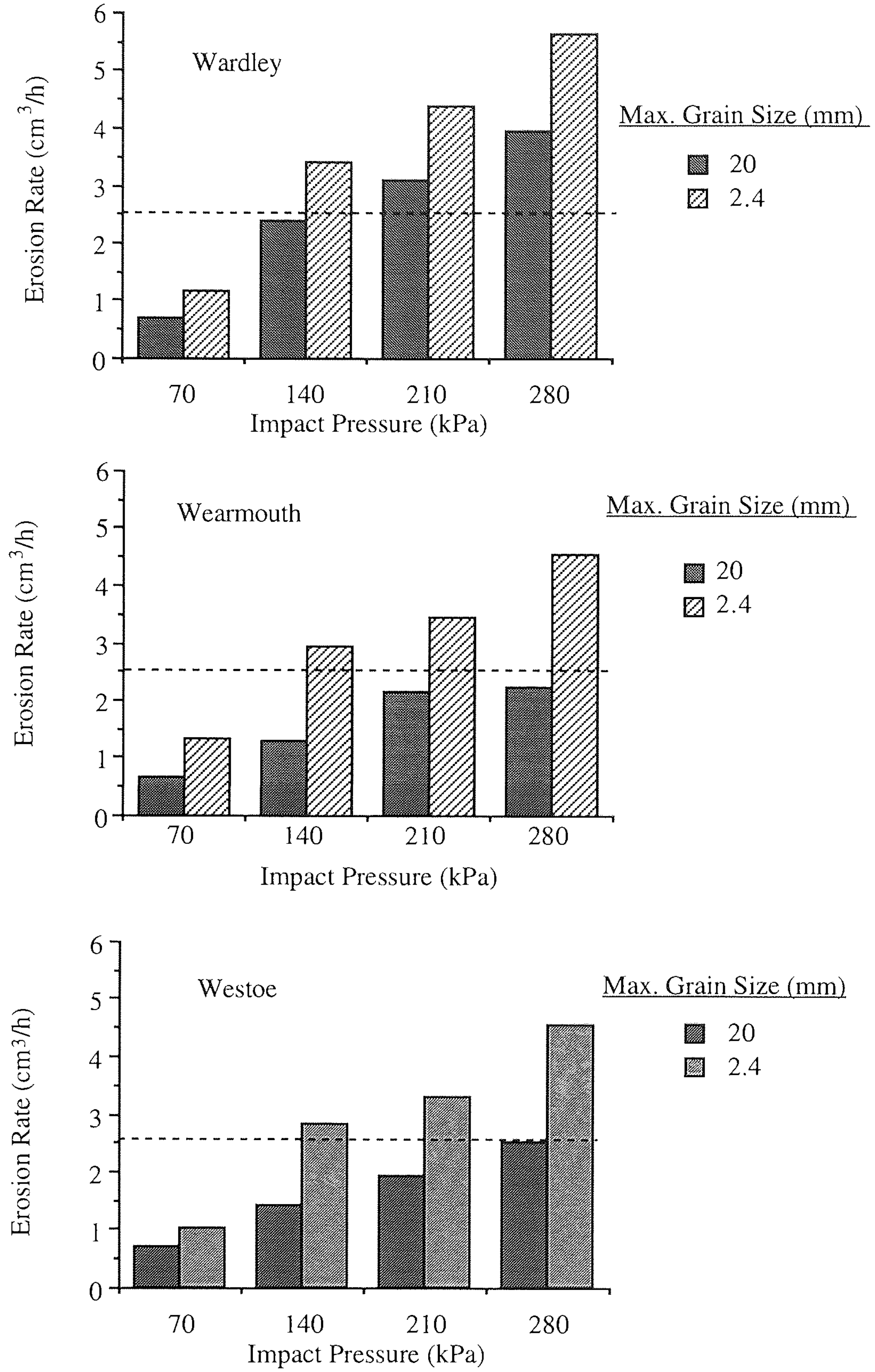


Figure 10.7 The influence of maximum grain size upon the erosion rate for stabilized minestone.

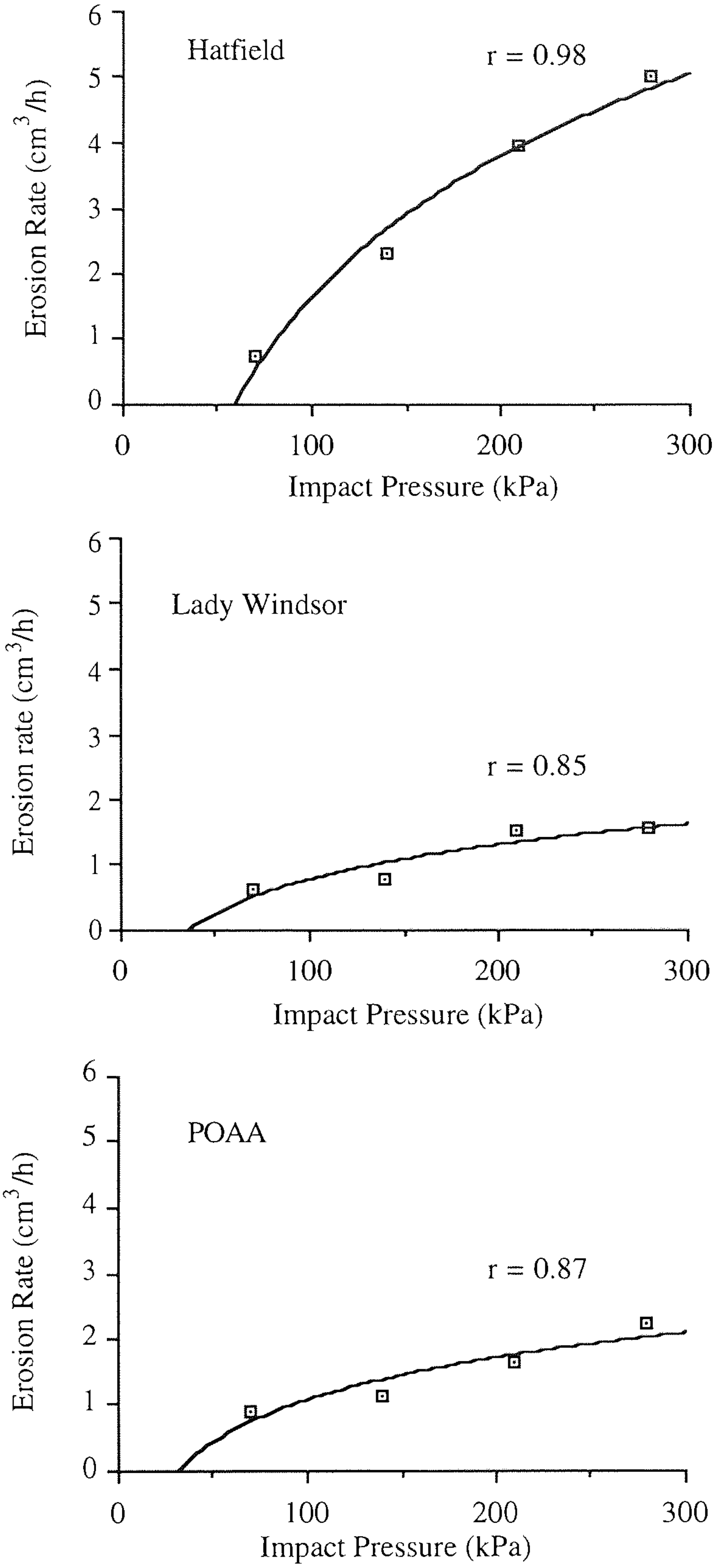
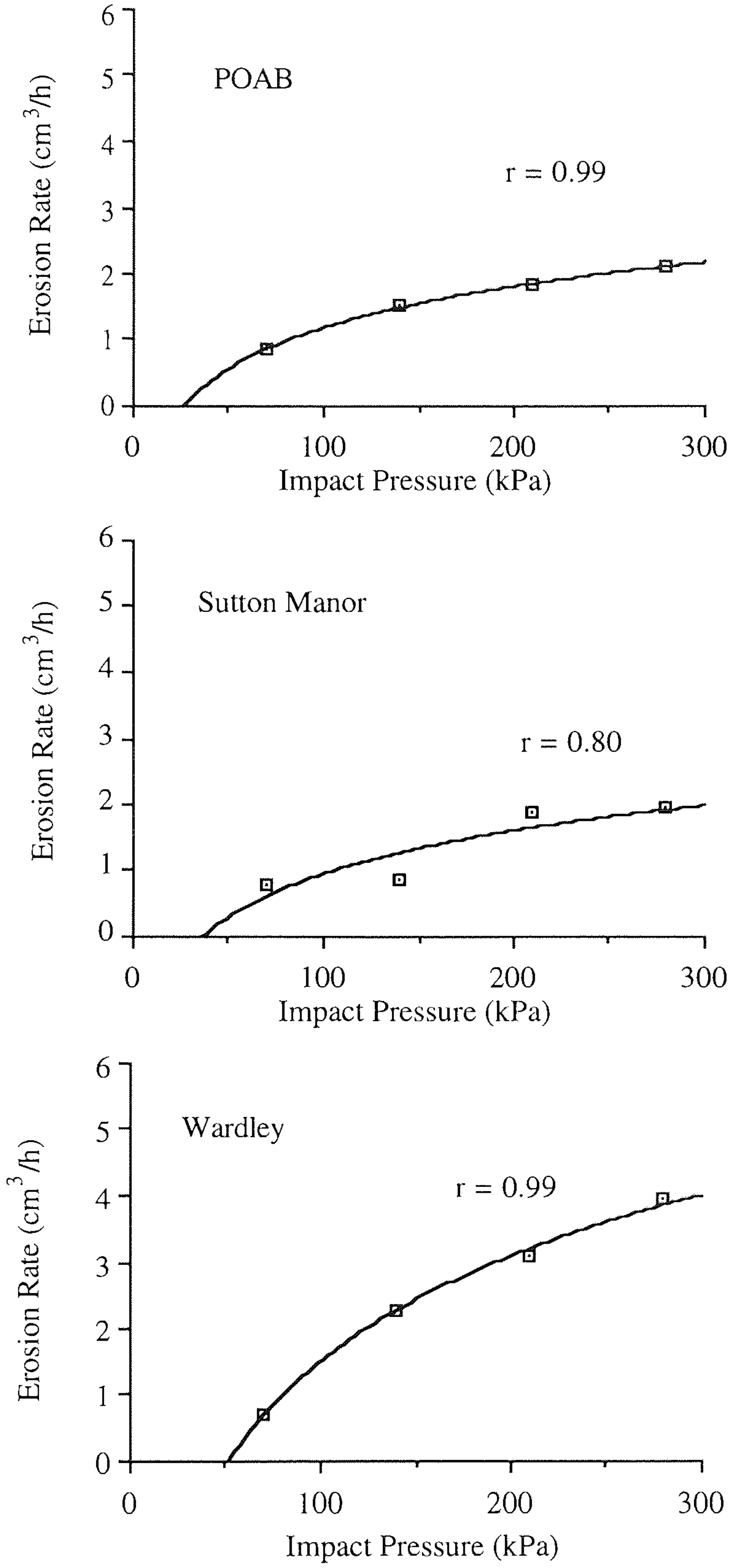


Figure 10.8 The relationship between impact pressures and erosion rates for minestone plus 10 per cent OPC.



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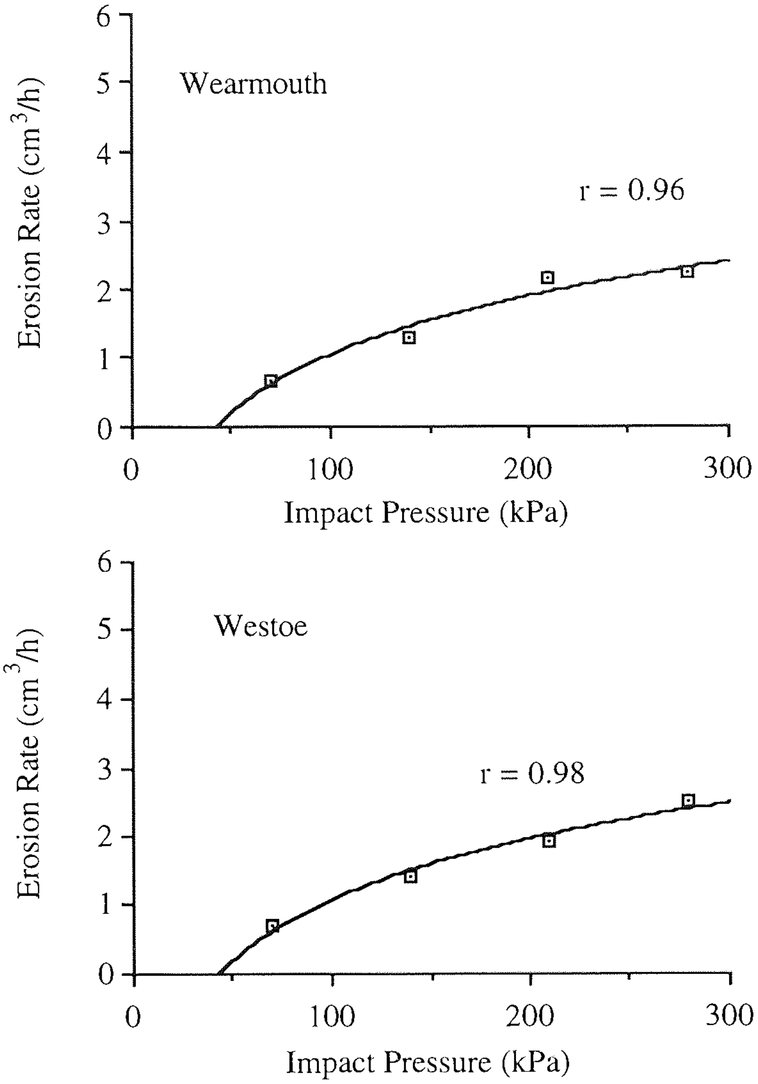

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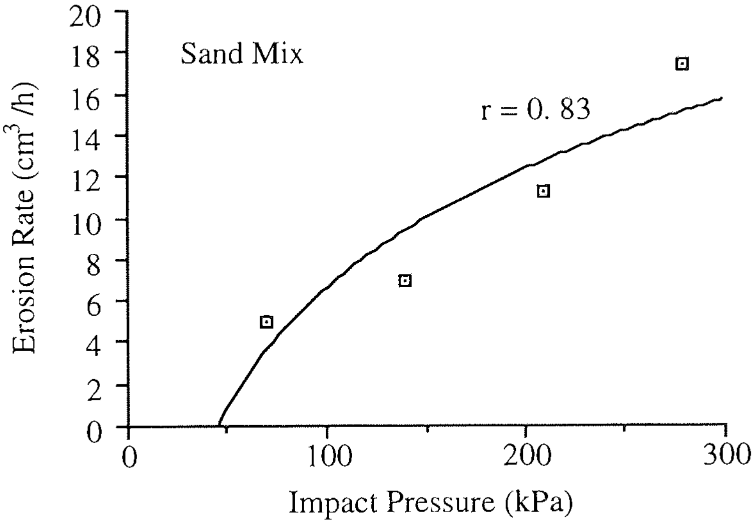


Figure 10.9 The relationship between impact pressures and erosion rates of the sand mix stabilized with 10 per cent OPC.

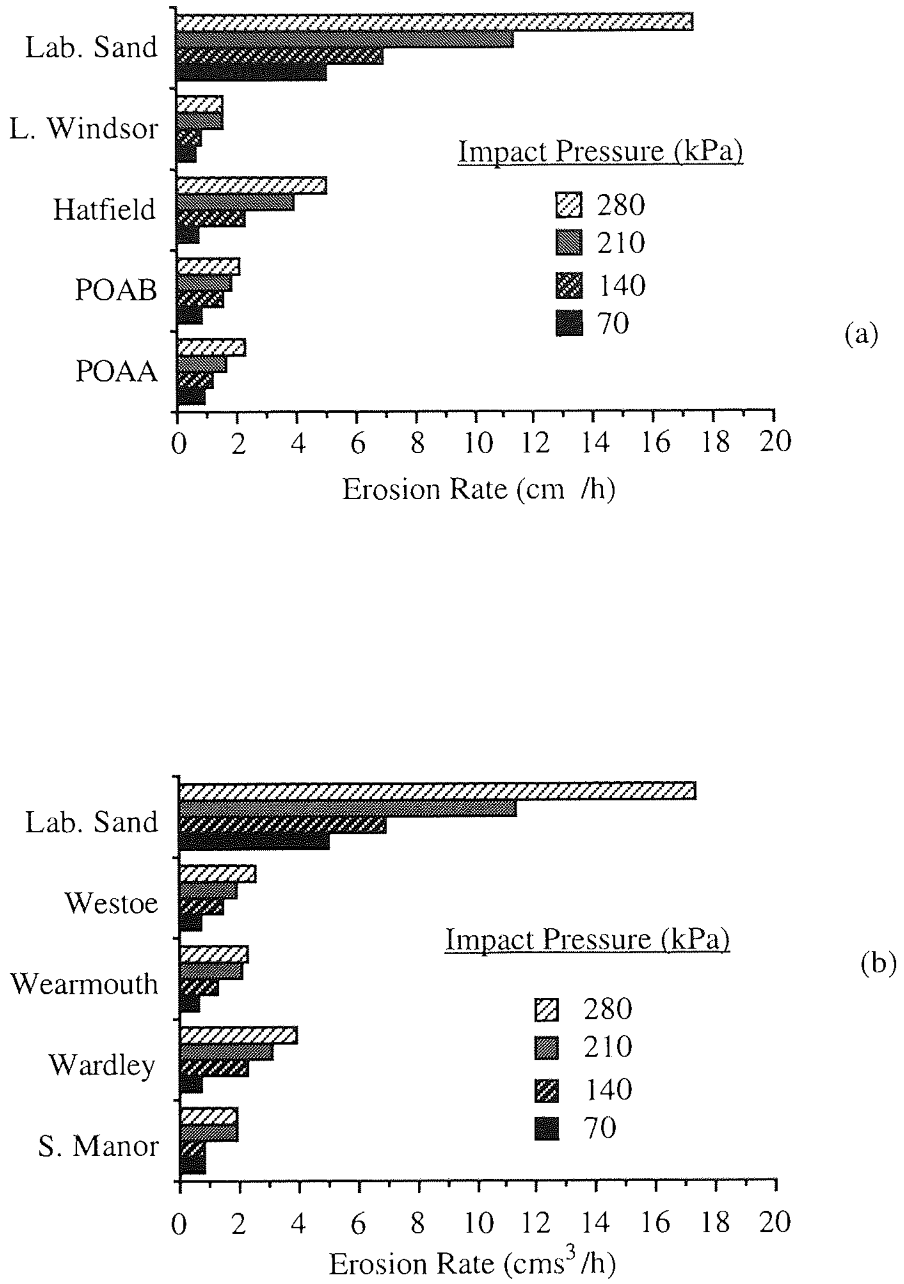


Figure 10.10 A comparison of the erosion rate for test materials plus 10 per cent OPC at various impact pressures.

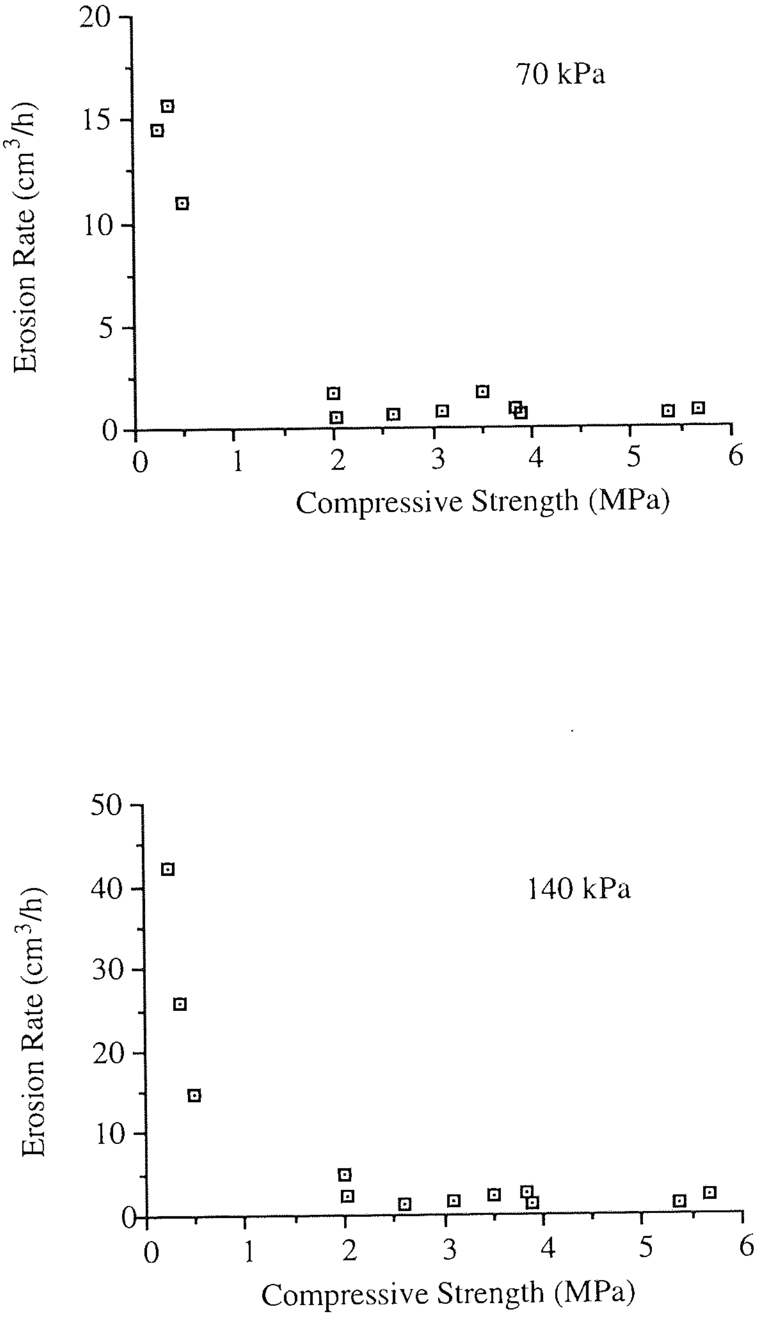
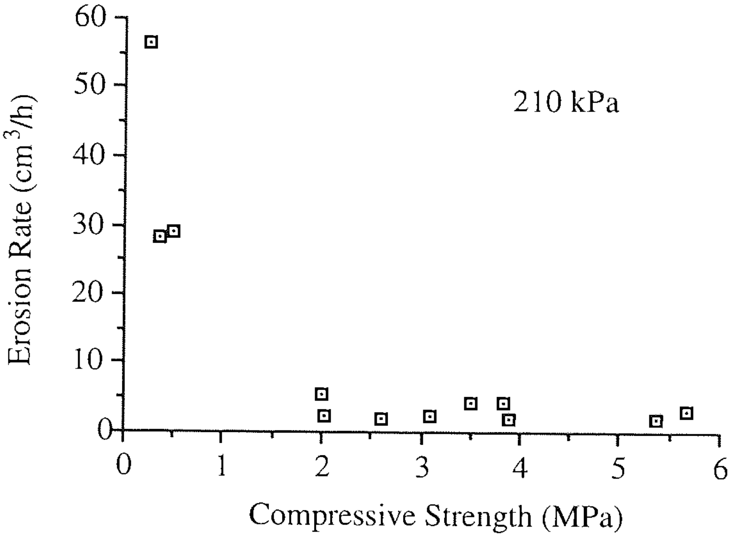


Figure 10.11 The exponential relationship between the erosion rate and compressive strength of mine [illegible]



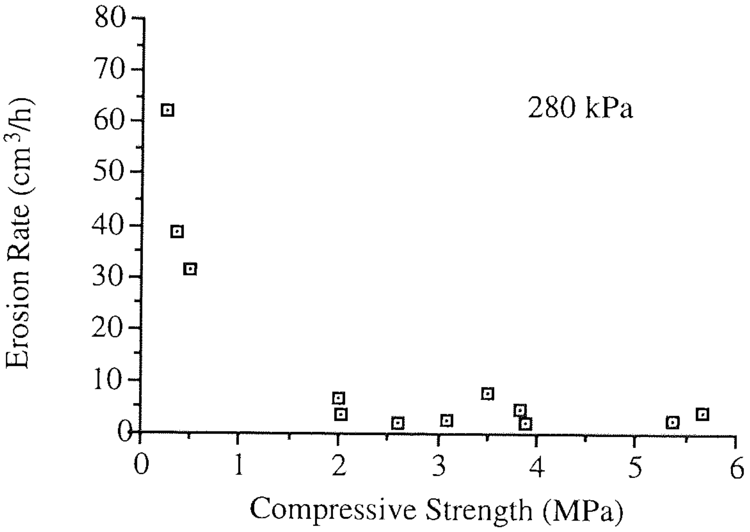
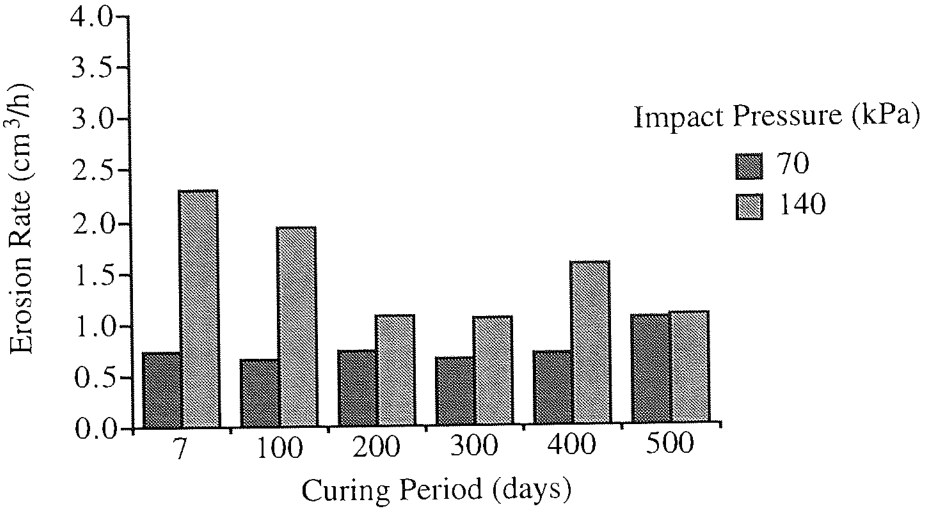


Figure 10.11 (continued)



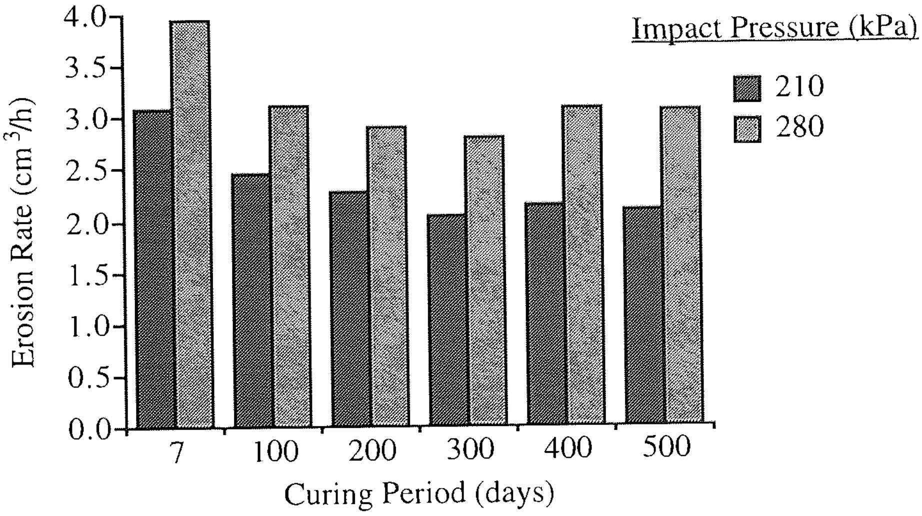
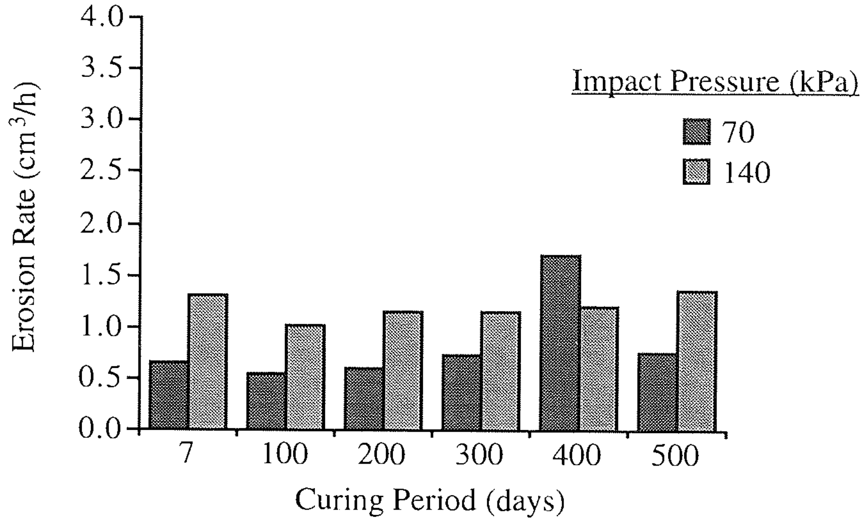


Figure 10.12 The effects of long term curing (normal) on the erosion rate for stabilized Wardley minestone



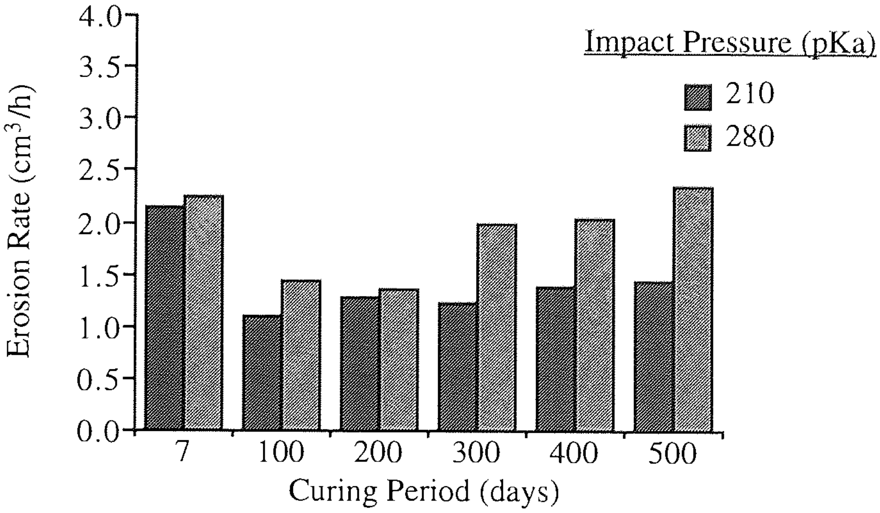
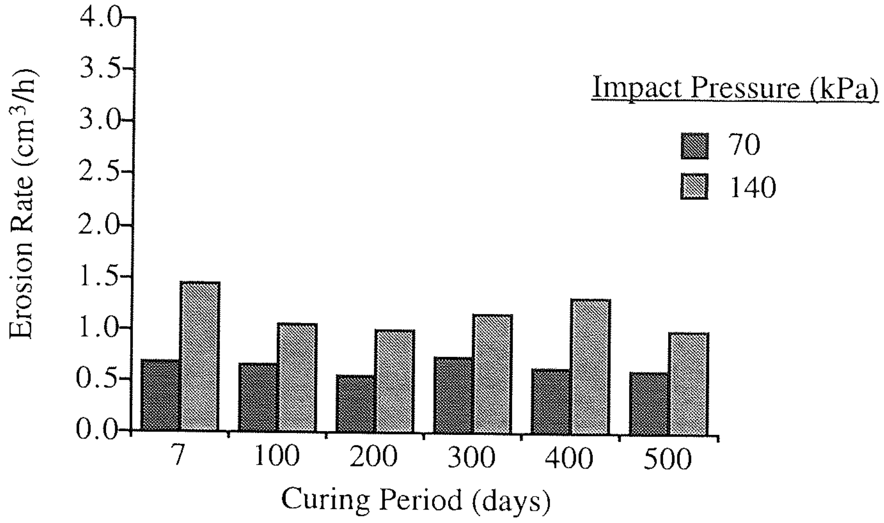


Figure 10.13 The effects of long term curing (normal) on the erosion rate for stabilized Wearmouth minestone



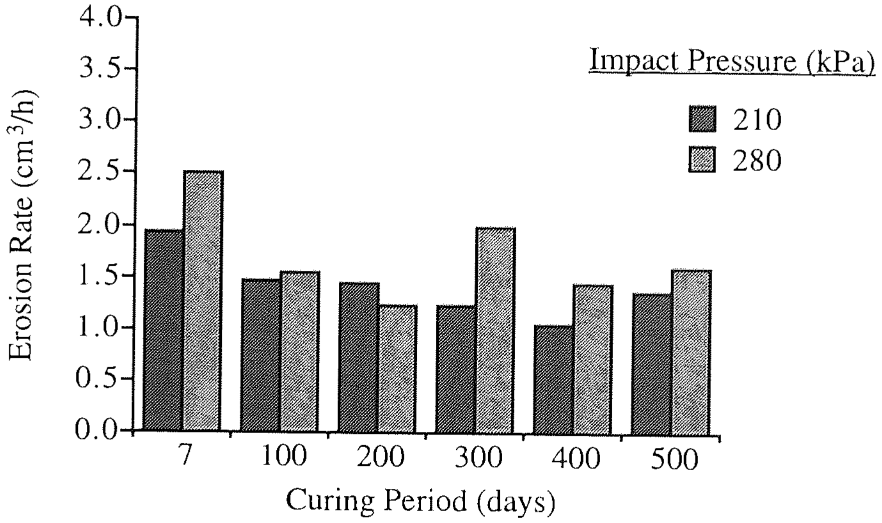
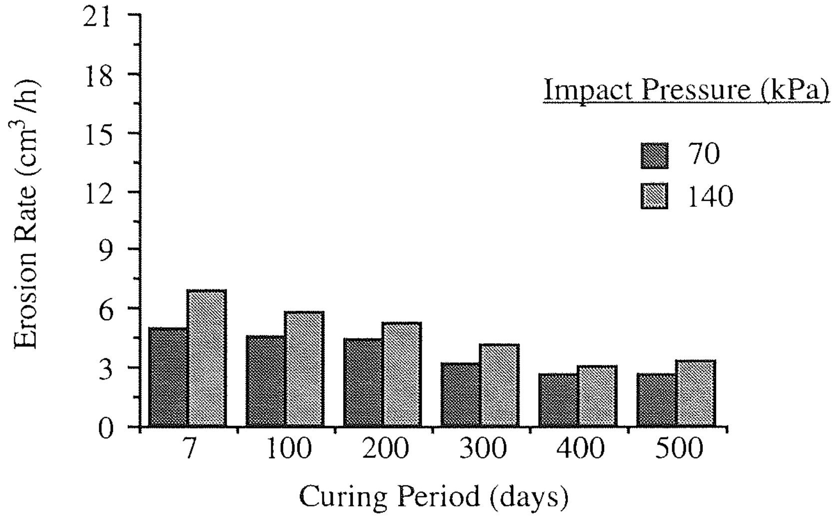


Figure 10.14 The effects of long term curing (normal) on the erosion rate for stabilized Westoe minestone.



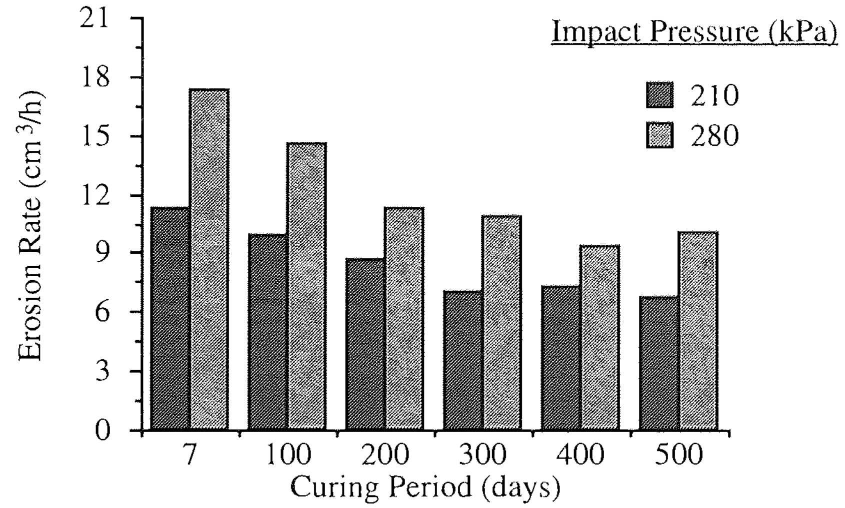


Figure 10.15 The effects of long term curing (normal) on the stabilized sand mix plus 10 per cent OPC.

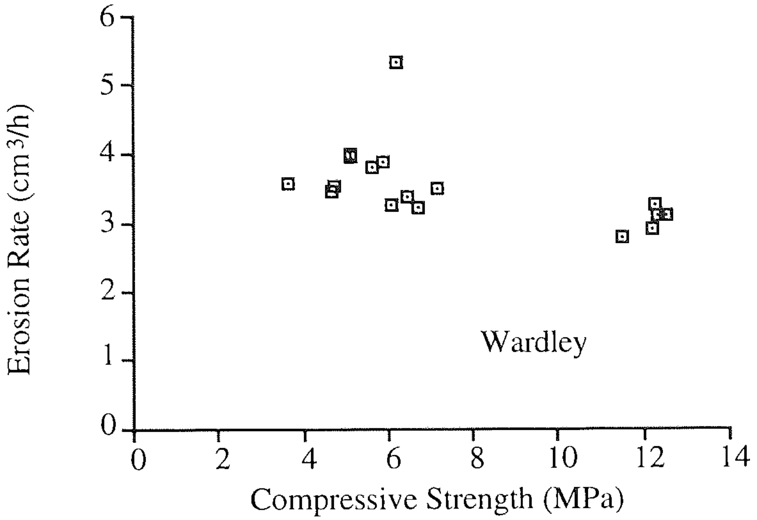
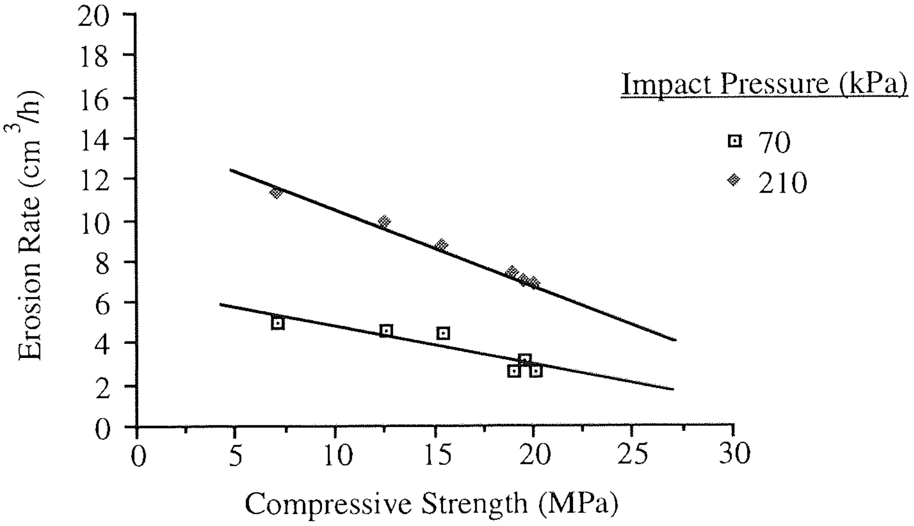


Figure 10.16 The relationship between compressive strength and erosion rate for minestone eroded at an impact pressure of 280kPa.



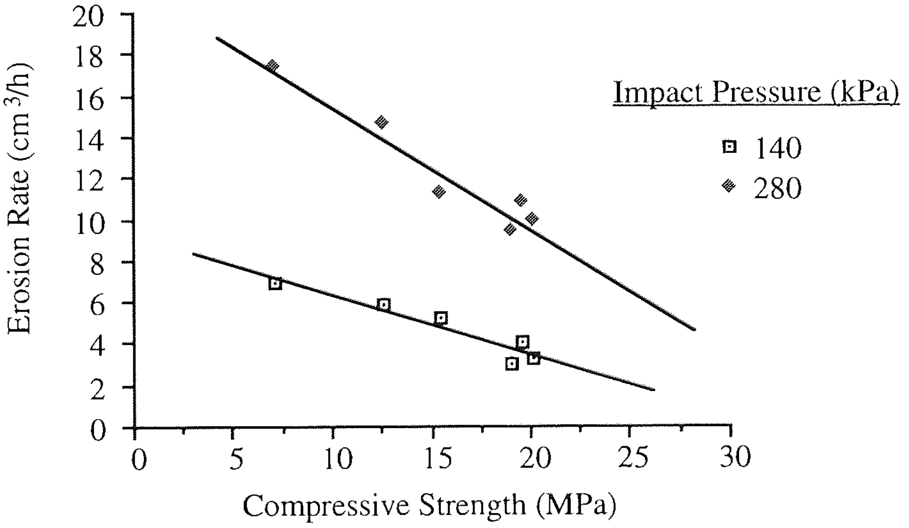


Figure 10.17 The relationship between the compressive strength and erosion rate for the stabilized sand mix plus 10 per cent OPC.

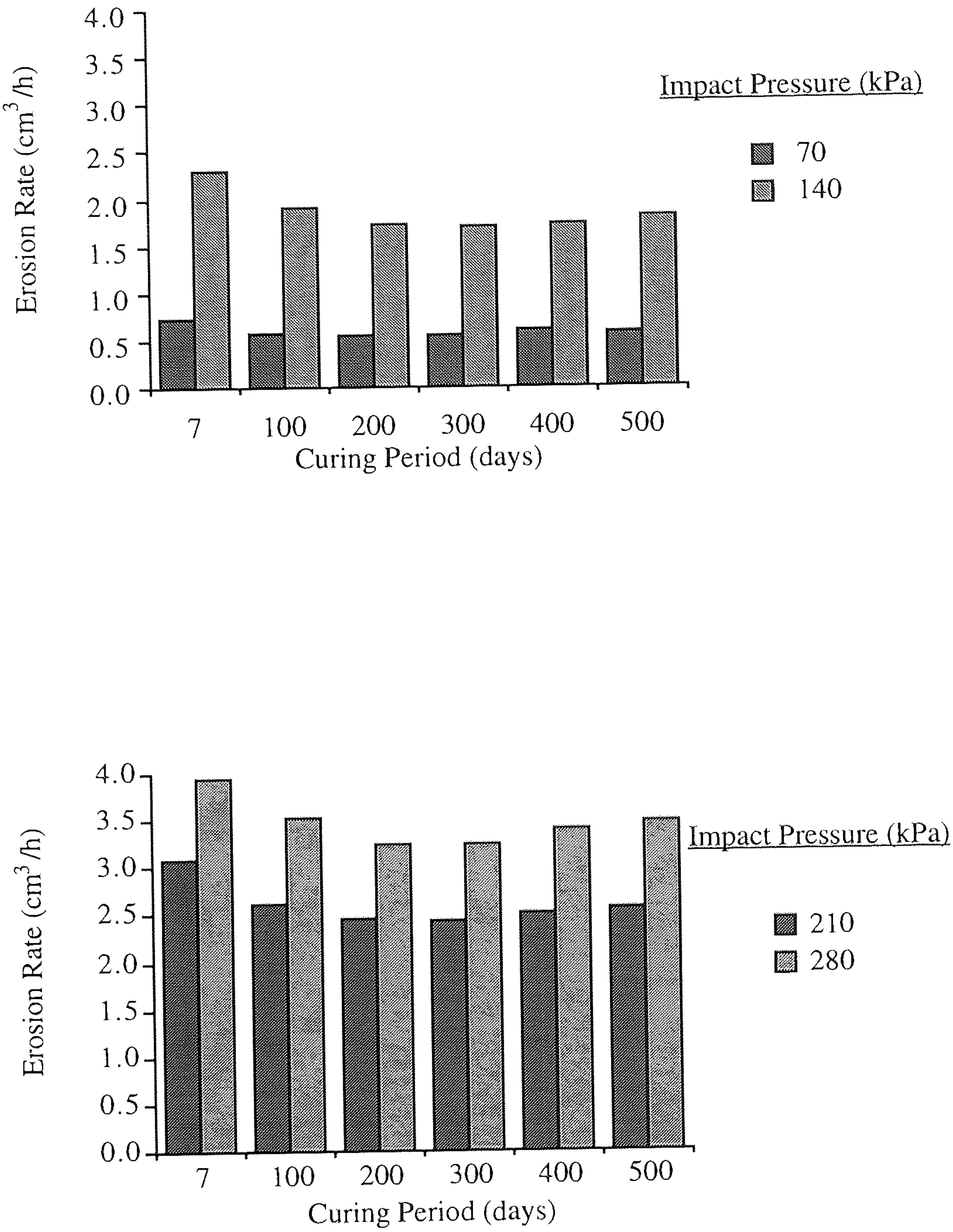


Figure 10.18 The effects of long term immersion in distilled water on the erosion rate for Wardley minestone plus 10 per cent OPC.

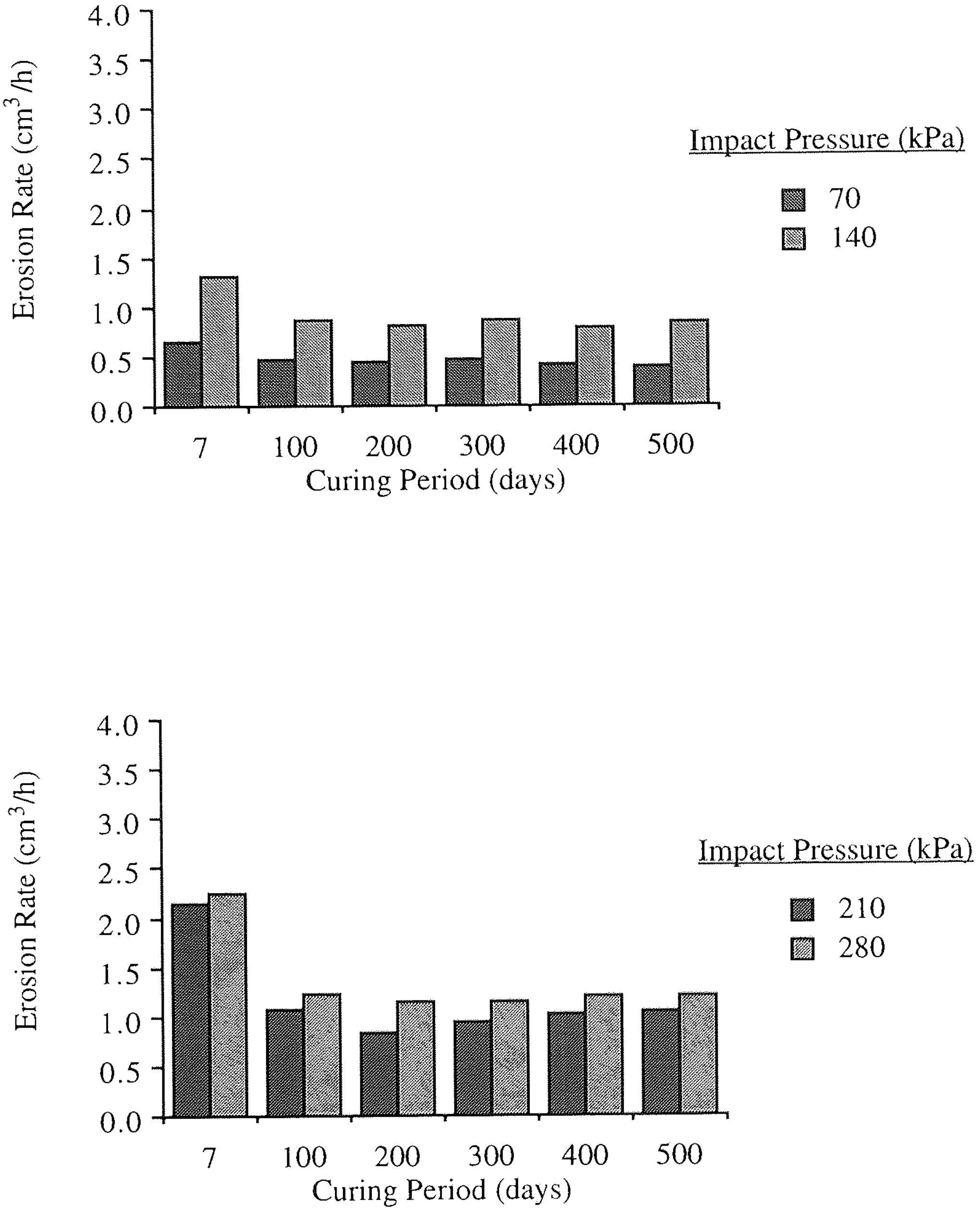
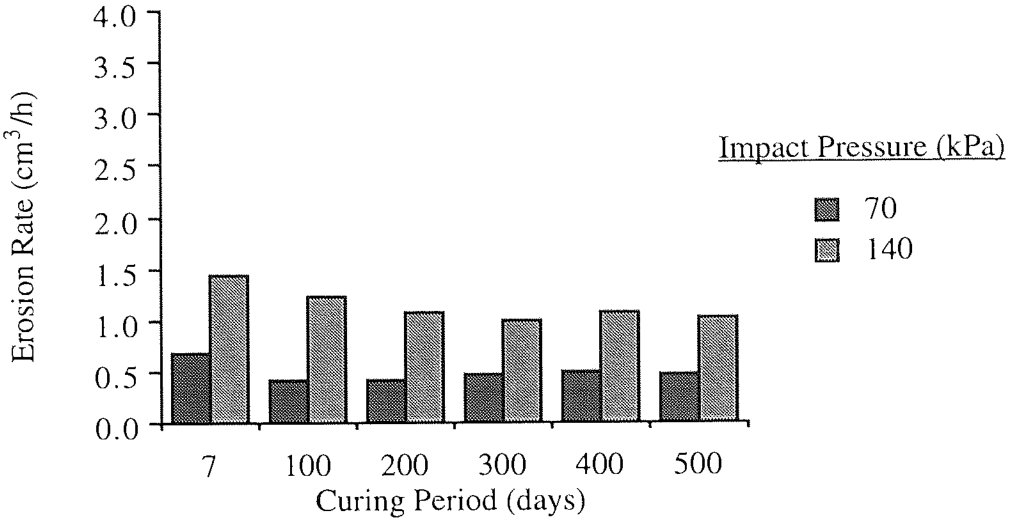


Figure 10.19 The effects of long term immersion in distilled water on the erosion rate for Wearmouth minestone plus 10 per cent OPC.



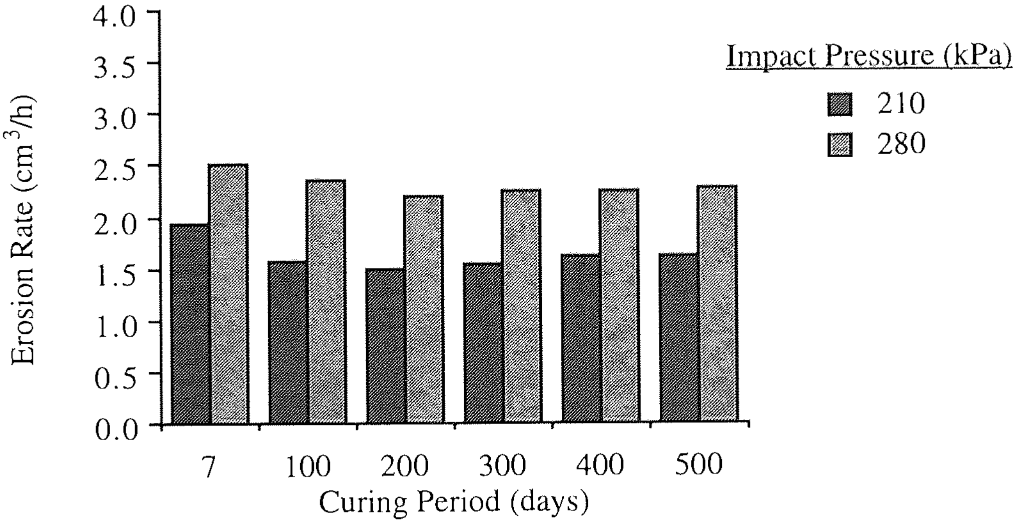


Figure 10.20 The effects of long term immersion in distilled water on the erosion rate for Westoe minestone 10 per cent OPC.

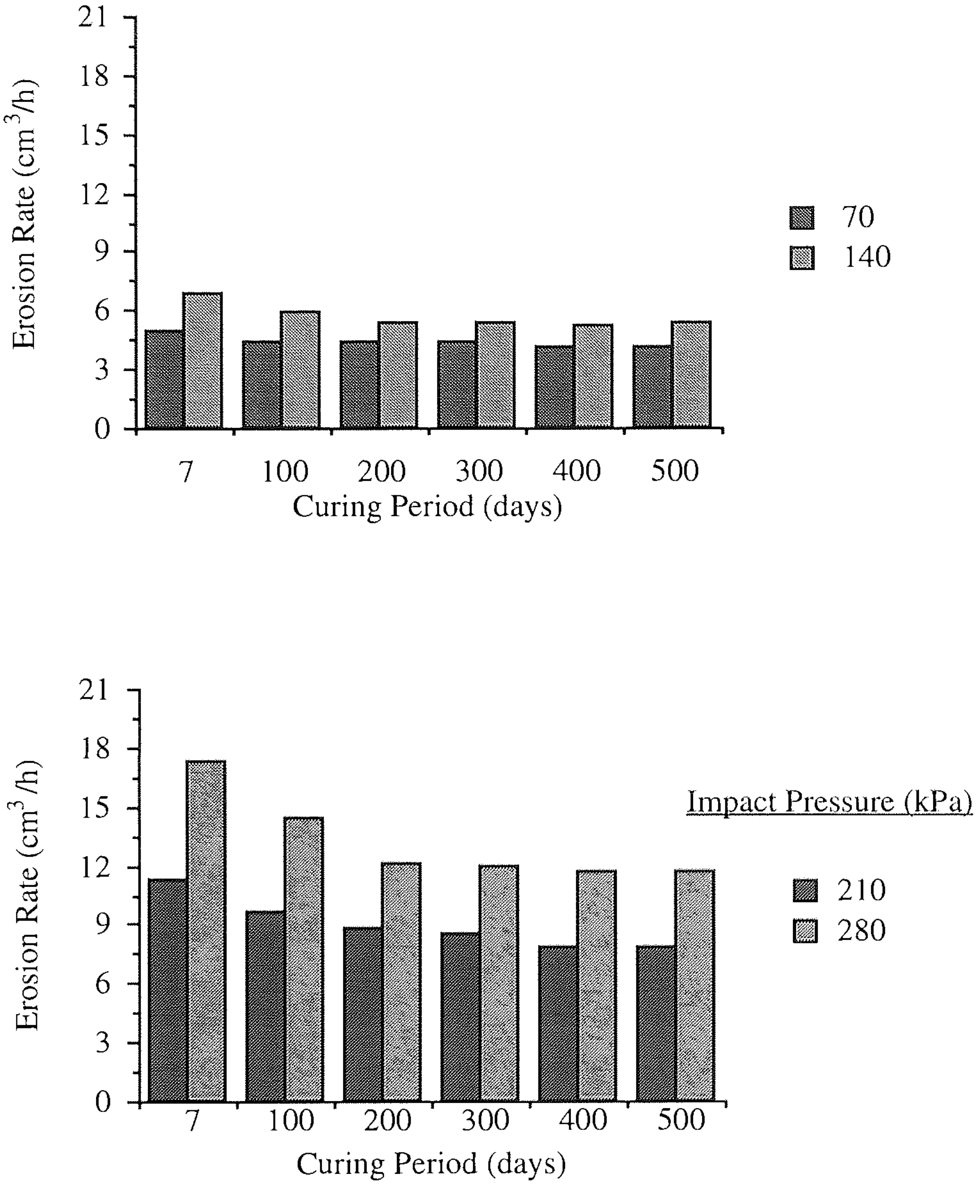


Figure 10.21 The effects of long term immersion in distilled water on the erosion rate for the sand mix plus 10 per cent OPC.

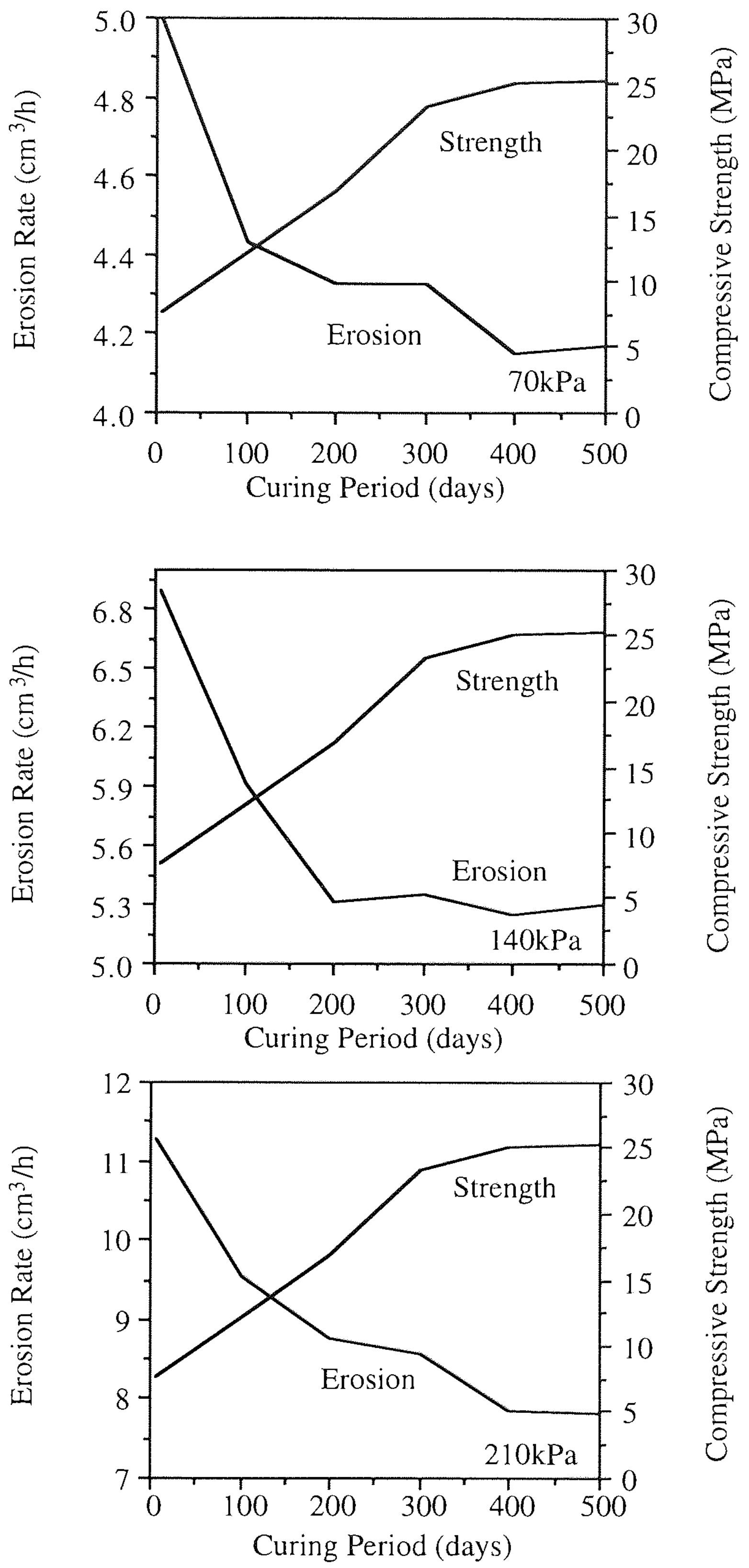


Figure 10.22 The relationship between the erosion rate and compressive strength of the sand mix following immersion in distilled water.

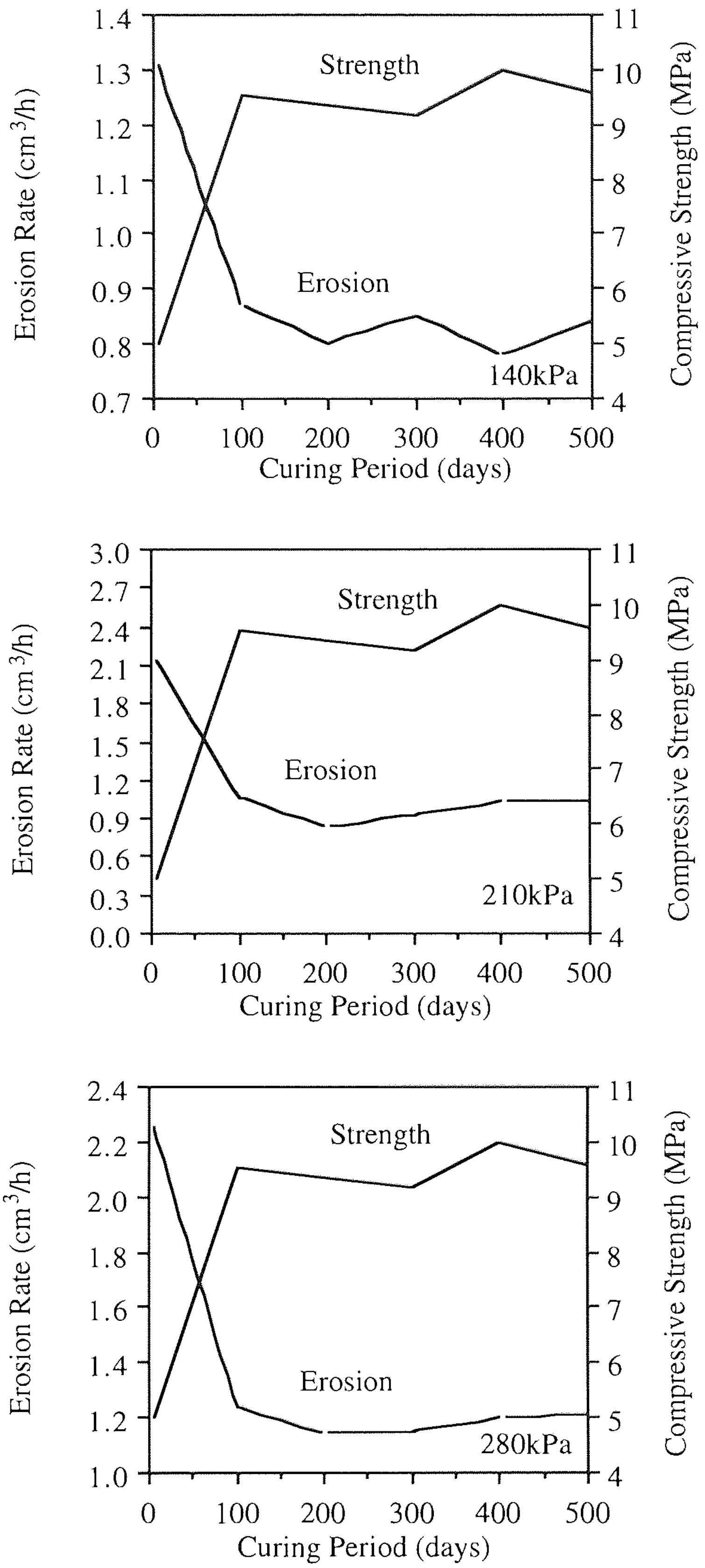


Figure 10.23 The relationship between the erosion rate and compressive strength for Wearmouth minestone following immersion in distilled water.

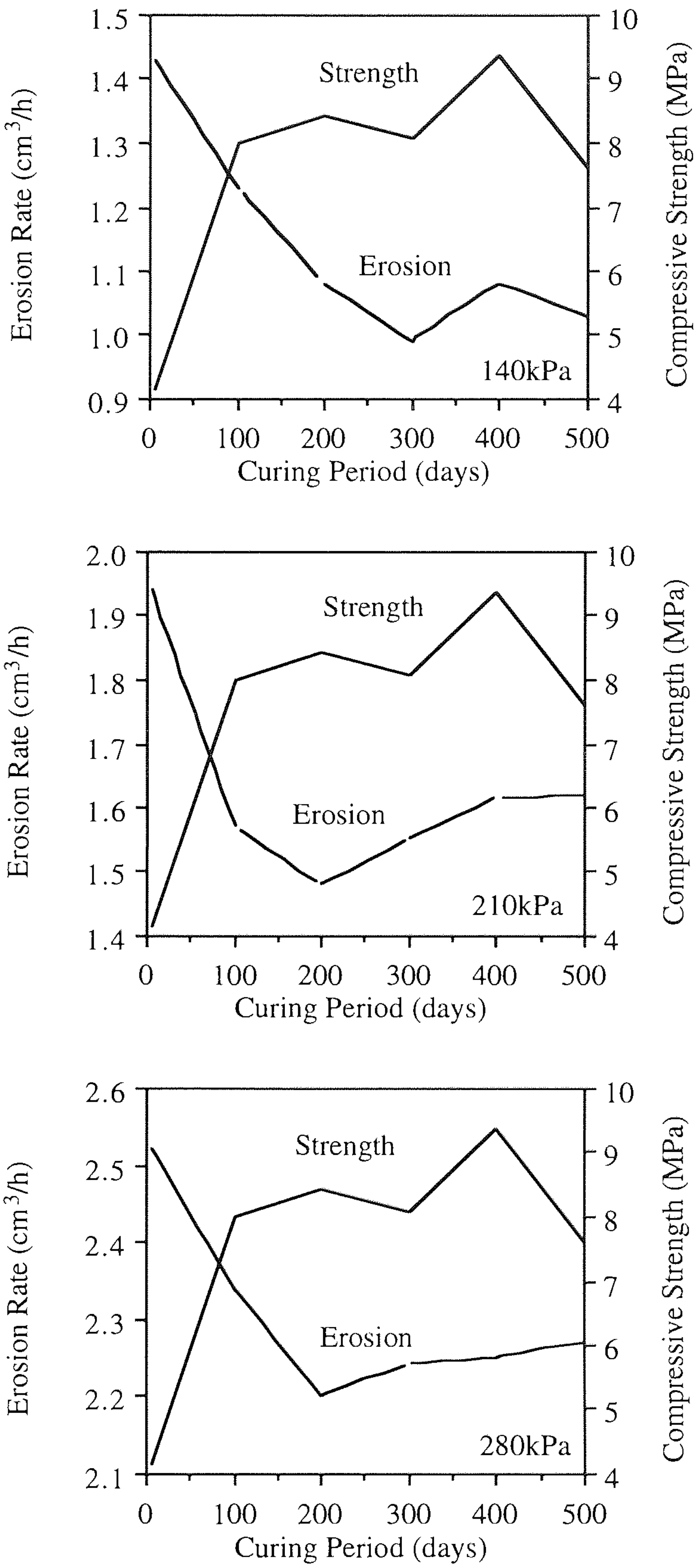


Figure 10.24 The relationship between the erosion rate and compressive strength for Westoe minestone following immersion in distilled water.

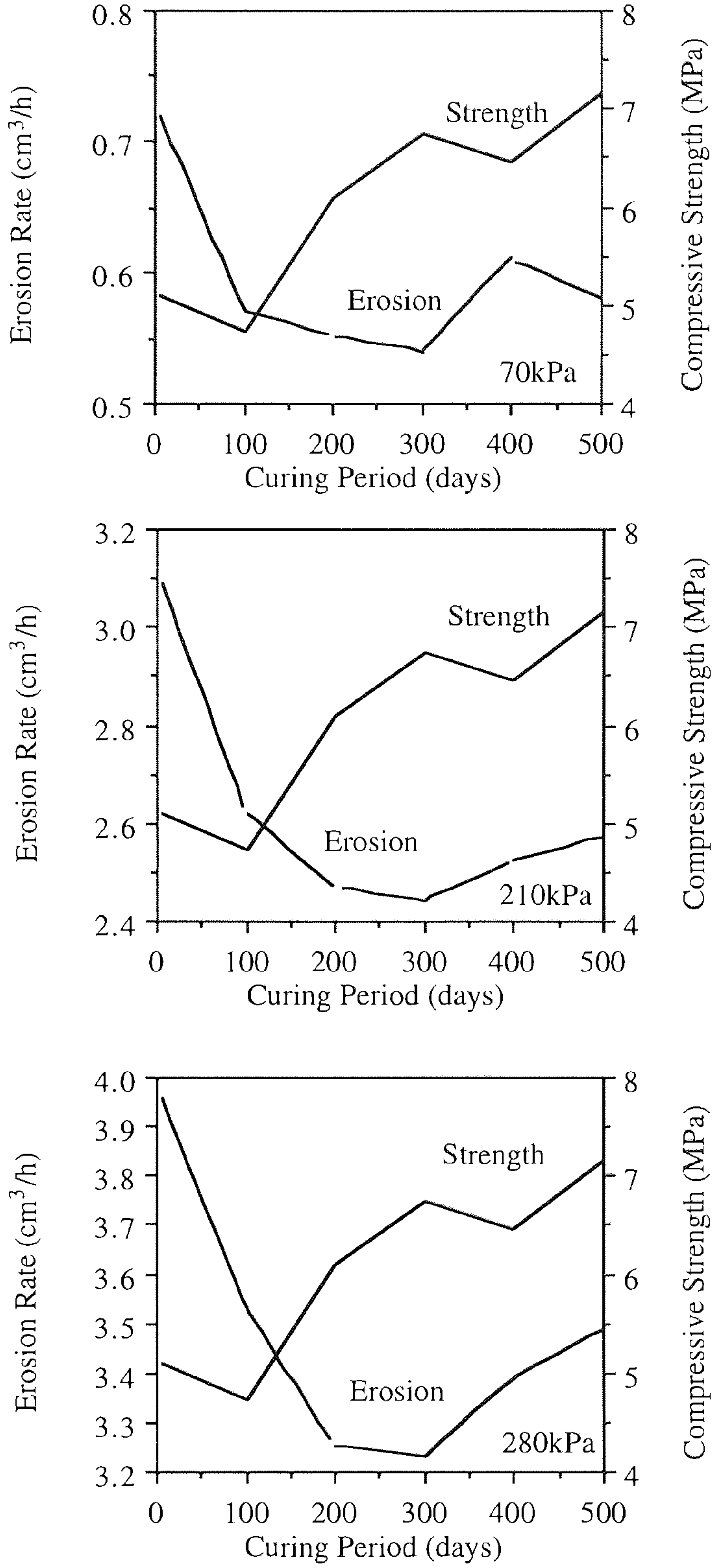
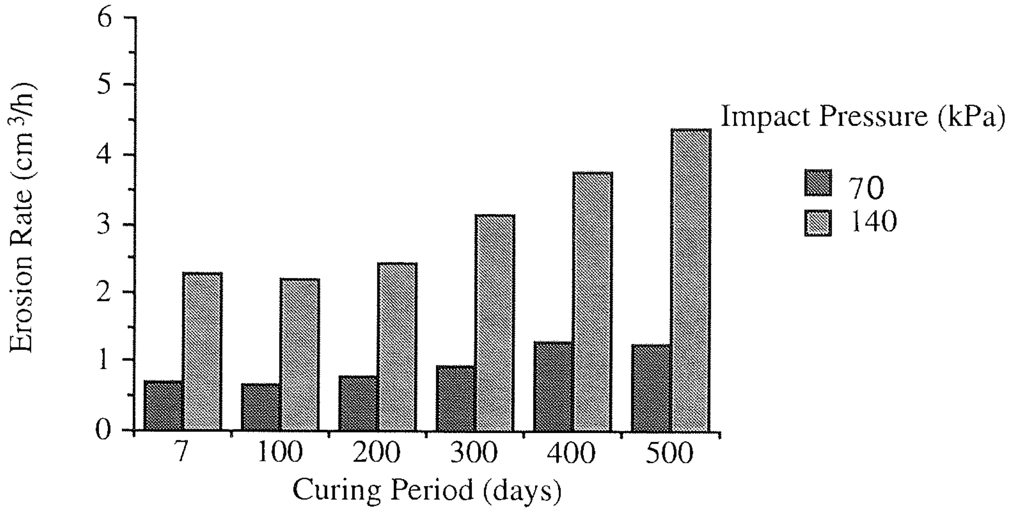


Figure 10.25 The relationship between the erosion rate and compressive strength for Wardley minestone following curing in distilled water.



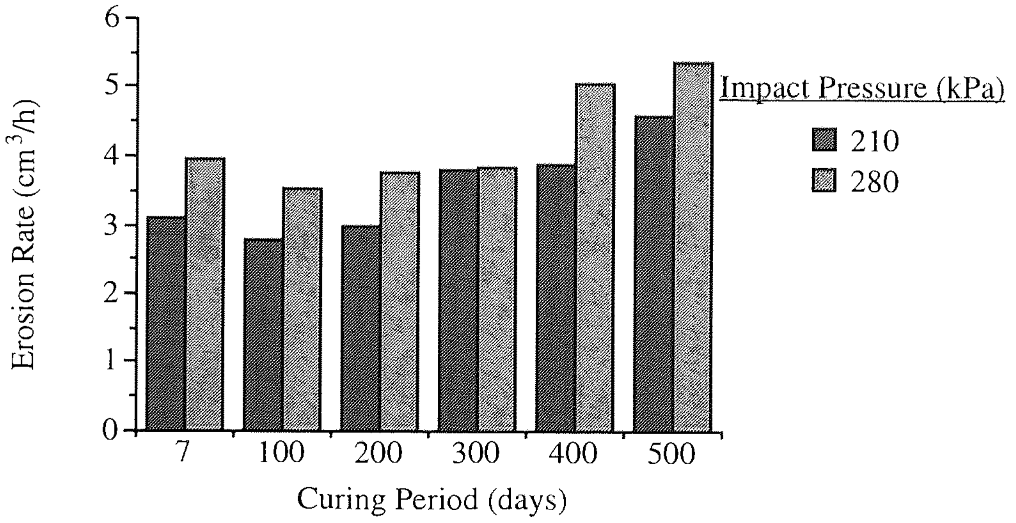
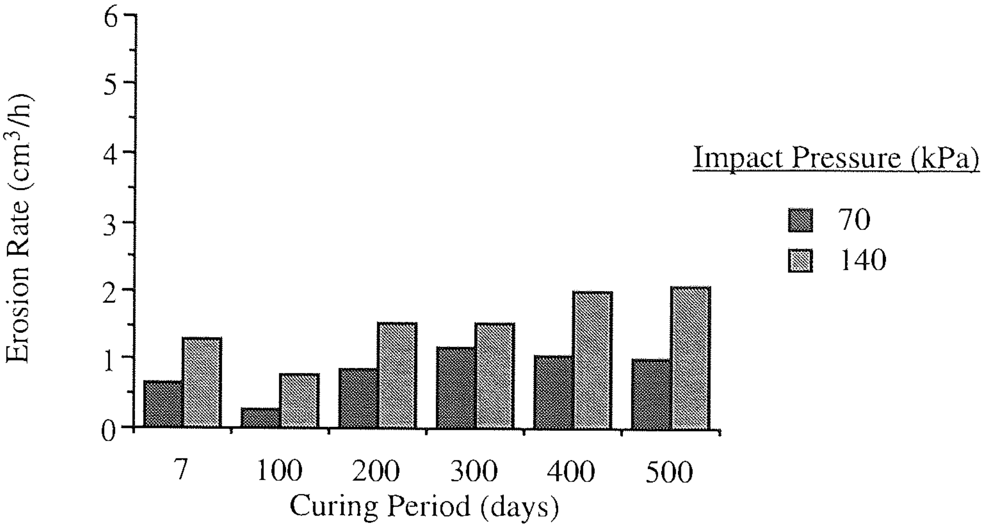


Figure 10.26 The effect of long term immersion in sea water on the erosion rate for stabilized Wardley minestone.



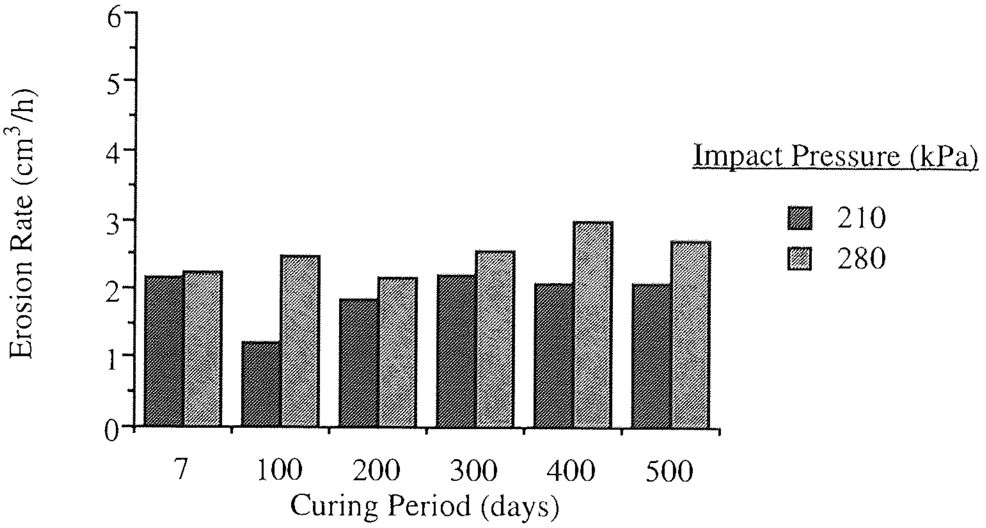
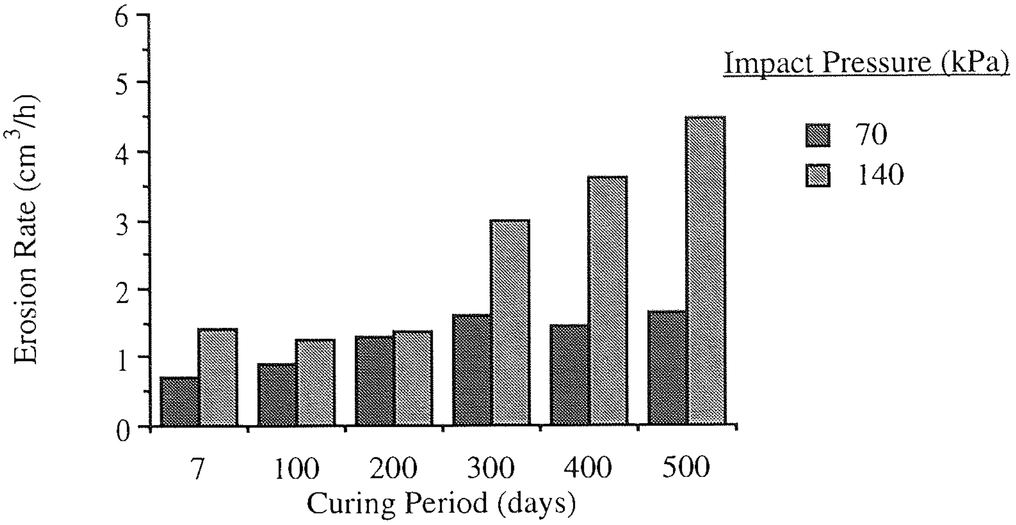


Figure 10.27 The effect of long term immersion in sea water on the erosion rate for stabilized Wearmouth minestone.



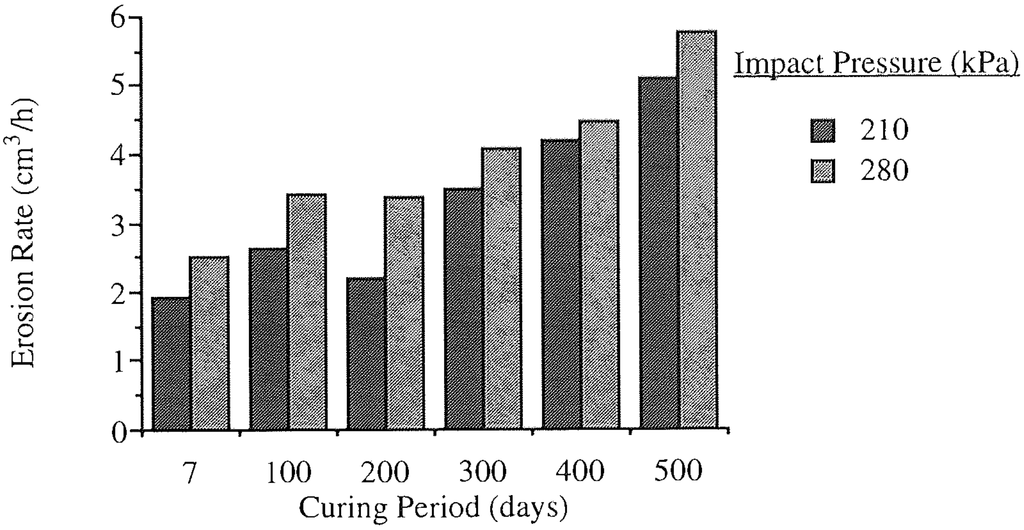


Figure 10.28 The effect of long term immersion in sea water on the erosion rate for stabilized Westoe minestone.

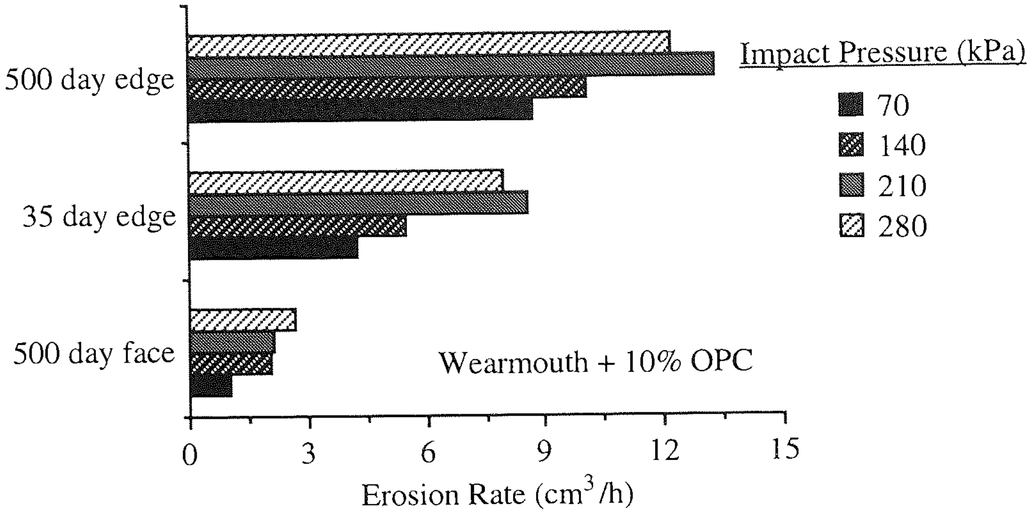


Figure 10.29 A comparison between the erosion rates at the edges and corners with those taken at the face of the sample.

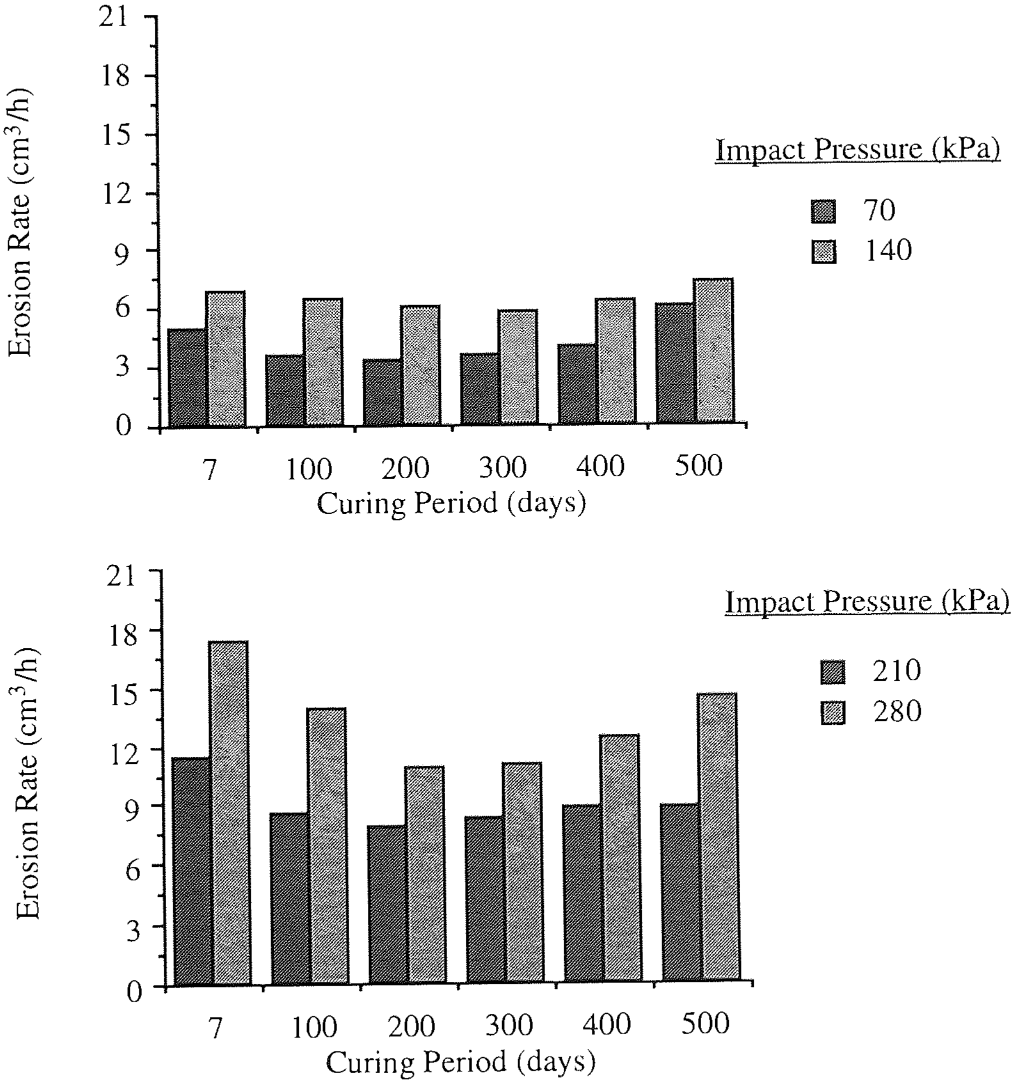


Figure 10.30 The effects of long term immersion in sea water on the erosion rate fot sand the mix plus 10 per cent OPC.

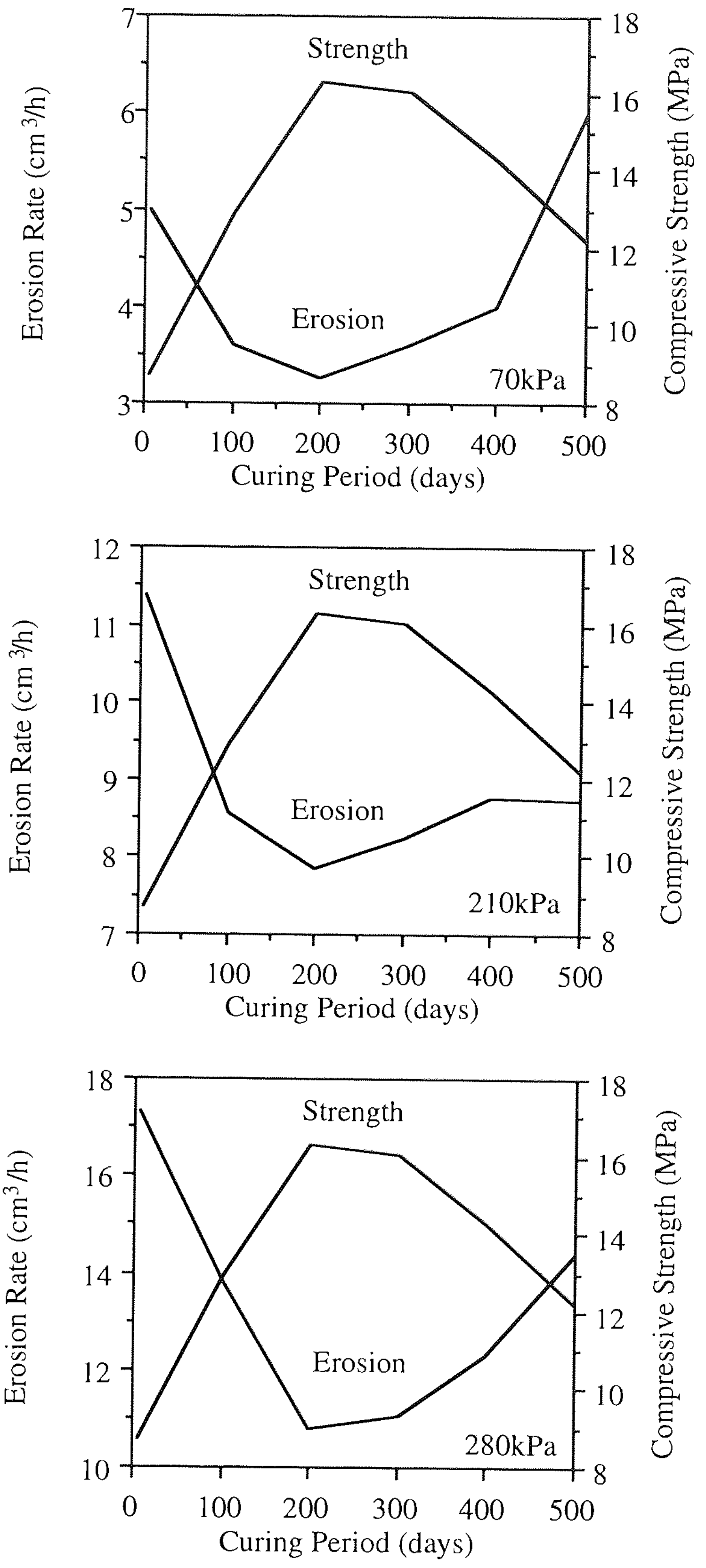


Figure 10.31 The relationship between the erosion rate and compressive strength for the sand mix following immersion in sea water.

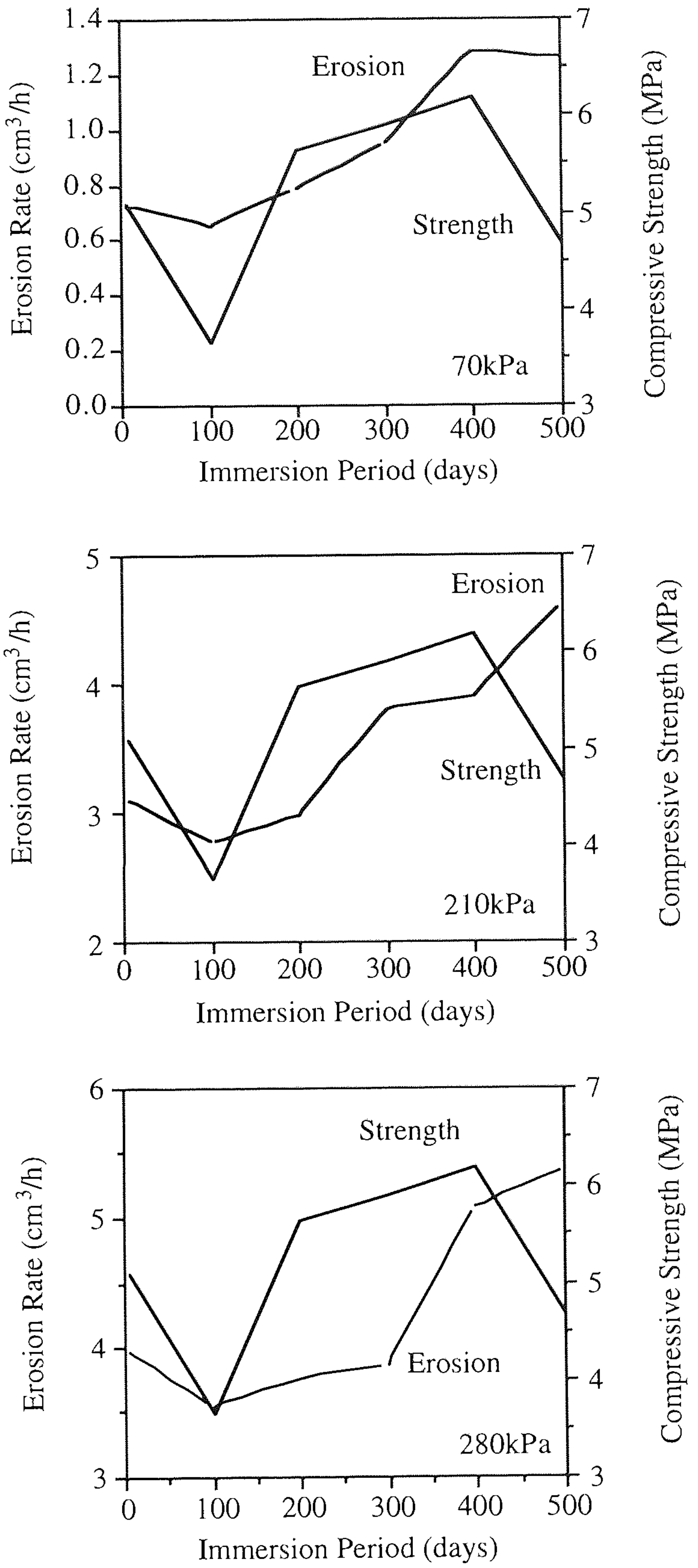


Figure 10.32 The relationship between the erosion rate and compressive strength for Wardley minestone following immersion in sea water.

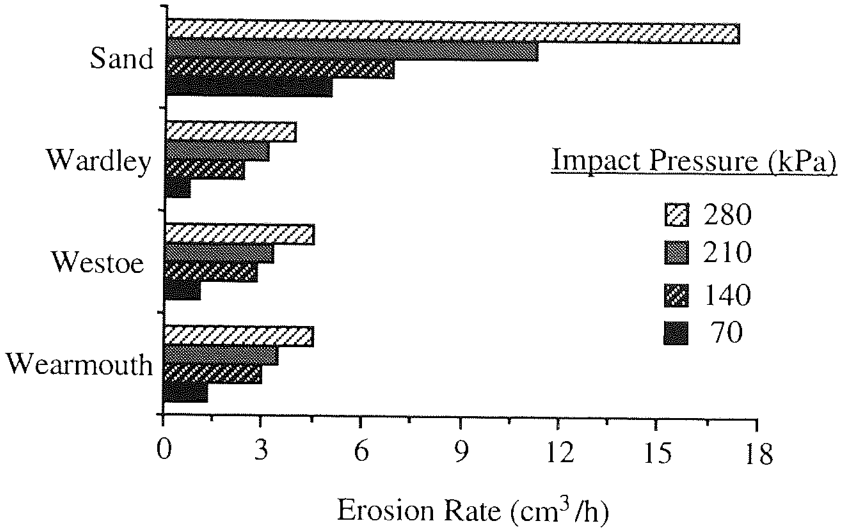


Figure 10.33 Comparison between the erosion rates for stabilized minestone with a maximum particle size of 2.4mm and stabilized sand samples.

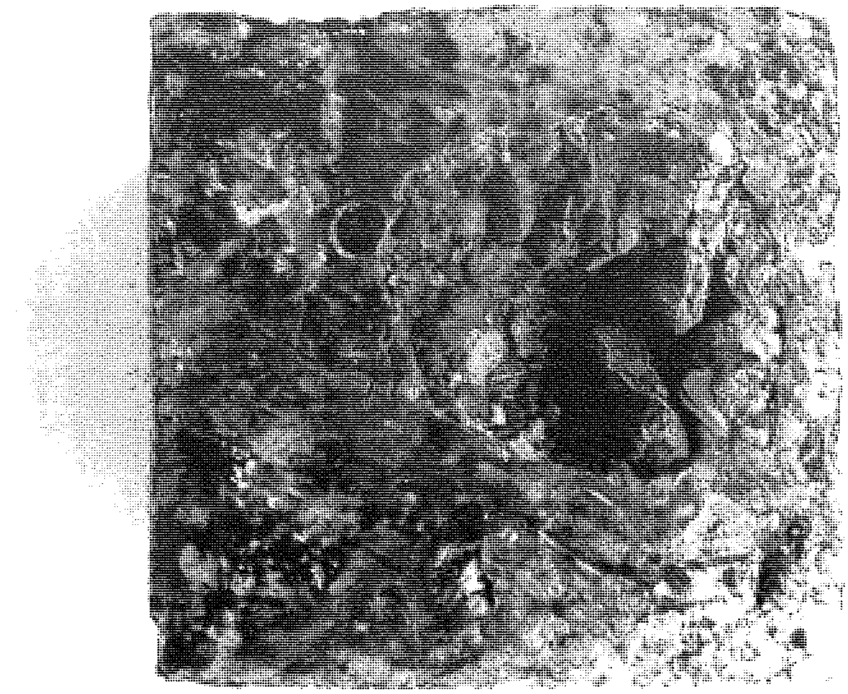


Plate 10.1 Typical damage inflicted on unstabilized minestone by the erosion/abrasion test (70kPa).

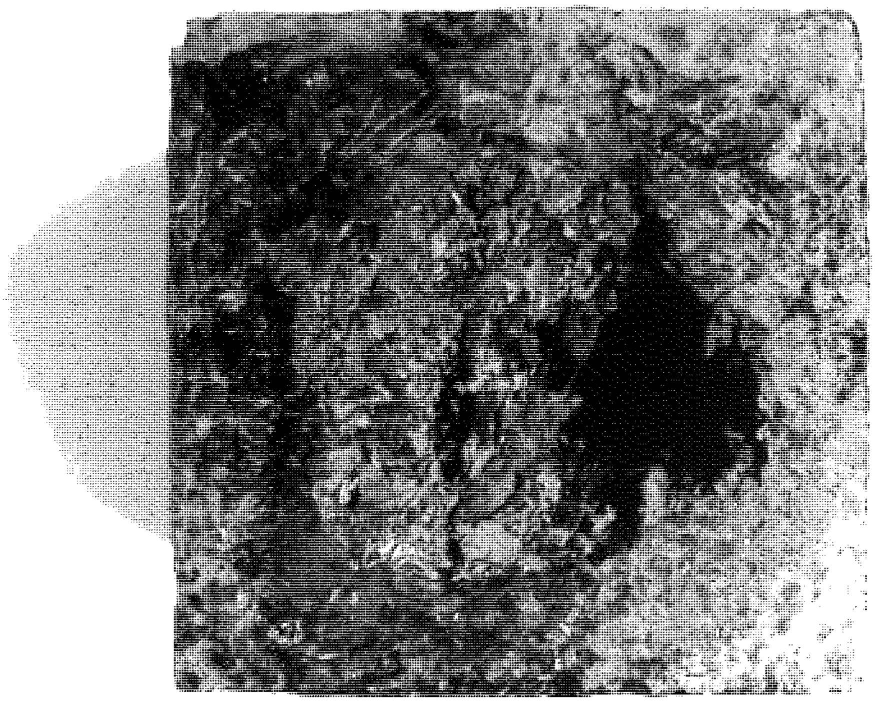


Plate 10.2 Typical damage inflicted on unstabilized minestone by the erosion/abrasion (140kPa).

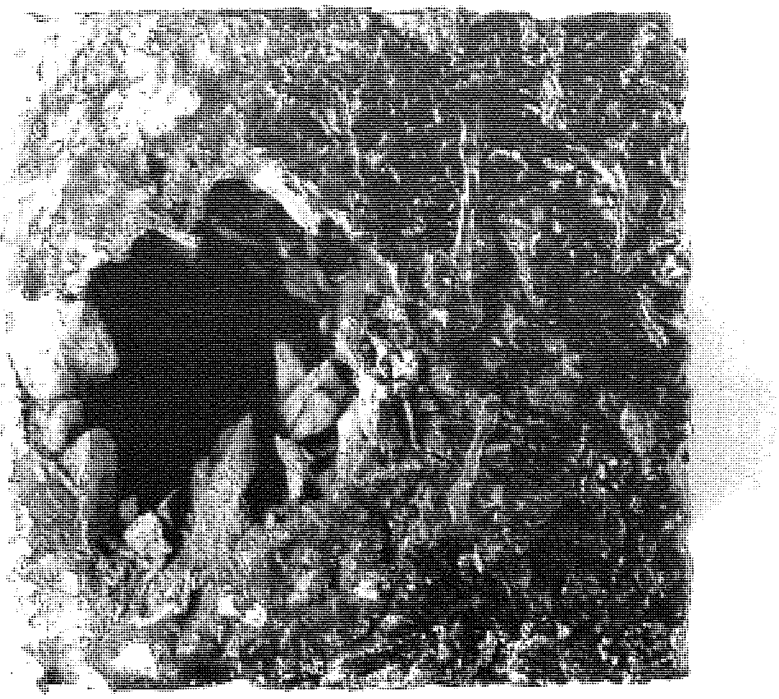


Plate 10.3 Typical damage inflicted on unstabilized minestone by the erosion/abrasion test (280kPa).

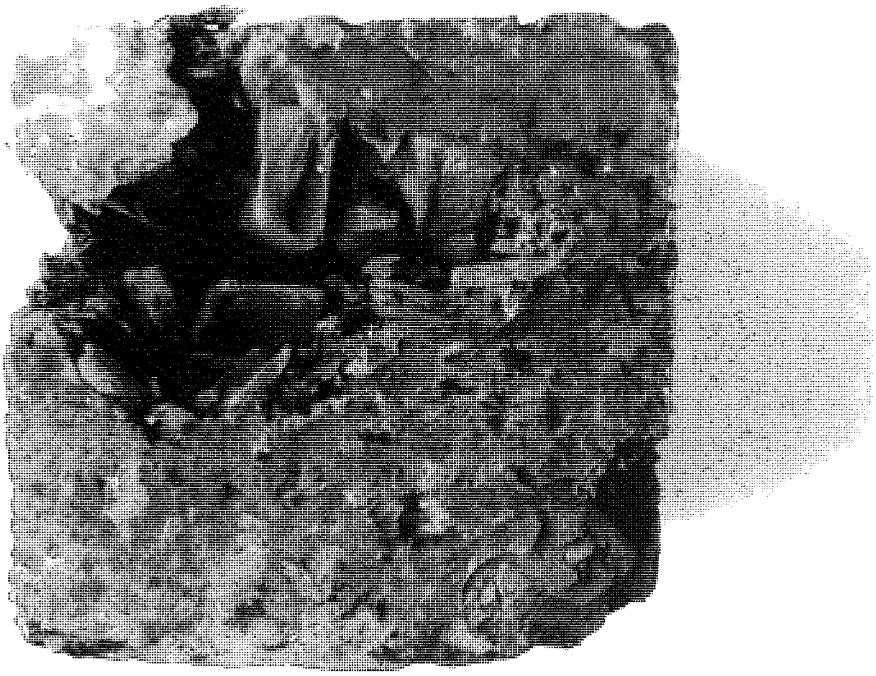


Plate 10.4 Irregular cavity as a result of differential erosion along the lines of least resistance, mainly by the winnowing away of vunerable matrix.

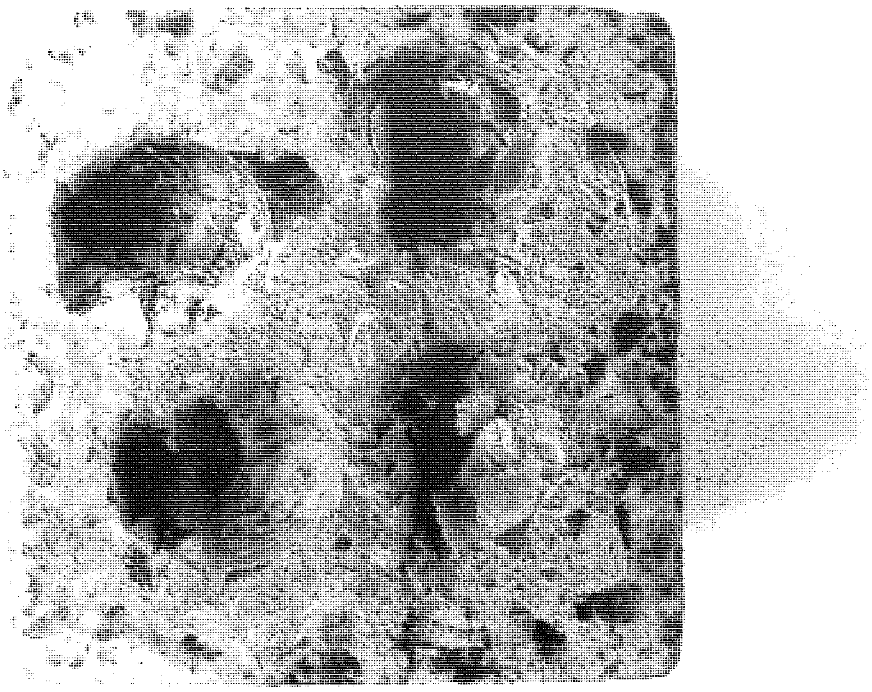


Plate 10.5 Typical erosion inflicted on CSM by the erosion/abrasion test (70kPa and 140kPa).

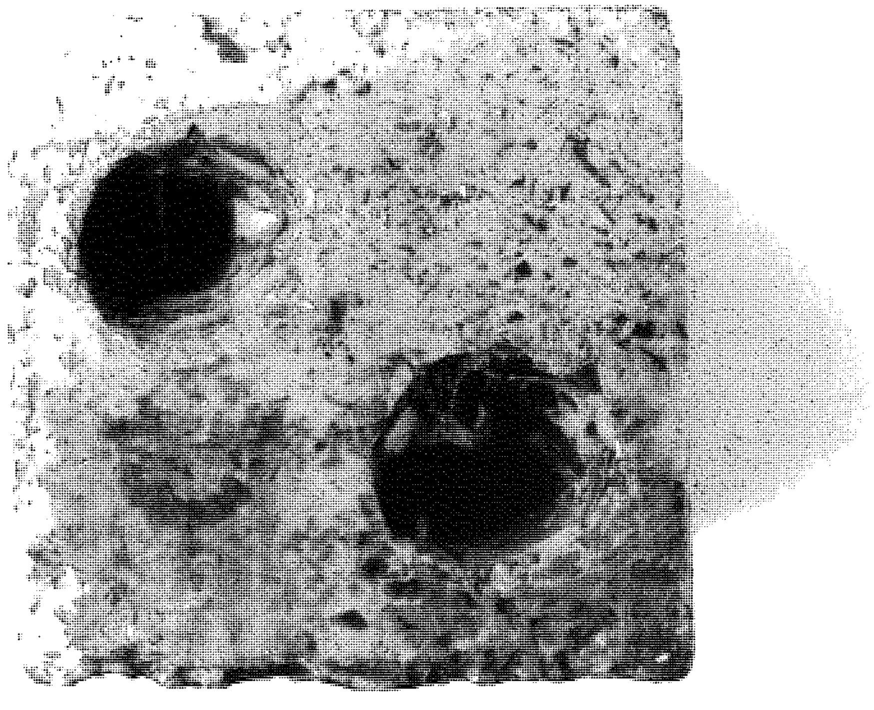


Plate 10.6 Typical erosion inflicted on CSM by the erosion/abrasion test (280kPa).

11. ANALYSIS AND DISCUSSION OF THE ROTATING CYLINDER TEST RESULTS

11.1 Introduction

The rotating cylinder test in its various forms has been a popular tool for investigating the erodability of both unstabilized and stabilized soils considered for the facing and lining of hydraulic embankments and channels. Its forte is its ability to determine the critical shear stress of the material and hence the point at which serious detrimental erosion would begin. Although the apparatus has been used to test unstabilized soils for 30 years and stabilized soils for at least 20 years, until now it has not been used to investigate the erodability of minestone. Previous studies on soils (Moore and Masch, 1962; Arulanandan *et al.* 1975; Chapuis, 1986) suggest that the test is a useful and powerful piece of research apparatus for the study of the hydraulic erosion caused by a unidirectional flow regime. The object of this research was to determine the potential of the rotating cylinder test as a tool for investigating the erodability of minestone in a hydraulic environment and also to implement the potential of the test. The emphasis was placed on investigating minestone in the unstabilized state, as it is in this condition that it is most likely to be used.

11.2 Test Feasibility Study Results

A feasibility programme was devised to determine whether the rotating cylinder apparatus would be a suitable for testing minestone. This programme involved testing both unstabilized and stabilized samples, the stabilizing agent being ordinary portland cement. Five sets of samples were tested using Wearmouth and Hatfield minestone :-

1. Unstabilized minestone.
2. 1 per cent cement and minestone.
3. 3.5 per cent cement and minestone.
4. 5 per cent cement and minestone.
5. 10 per cent cement and minestone.

11.2.1 Unstabilized Samples

Although the unstabilized samples were durable when handled and could be mounted within the apparatus without too much difficulty, severe (and insurmountable) problems arose prior to this stage. On being extruded from the moulds, the samples were seen to be perfectly smooth and in peak condition for shear stress testing. However, following the immersion of the samples in the eroding medium, severe disruption occurred at the surface. As soon as the samples come into contact with the fluid, particles began to slough rapidly from the surface and, in some cases, this action was quite explosive. The cause of this disintegration was primarily the result of escaping air bubbles breaking violently through the surface and overcoming the weak cohesive forces that bound the particles together. Presumably this reaction would continue until the samples were completely saturated with eroding fluid. Such action would result in a significant percentage of the sample being lost before the erosion test had commenced. As the samples needed to be placed into the eroding fluid to equilibriate prior to testing, severe disruption would often occur before the sample was mounted in the apparatus. Once this damage had occurred, handling of the samples became more difficult. A further consequence of this slaking and sloughing was for severe roughening to occur around the entire circumference of the sample. Such roughening impaired the suitability of the sample for the rotating cylinder test. Similar problems were encountered by Fan (1988) when using the rotating apparatus as a means to predict the erosion of cut and fill slopes. Indeed he recommended that the device would be unsuitable for non-cohesive, partially saturated or loose soil samples because of the slaking and instability associated with such samples. Although, in this research, a cohesive element was present in all the minestone samples, they were predominantly composed of non-cohesive silt and sand size particles together with a gravel component, and the reaction when immersed in water, suggests that the cohesive element was insufficient to maintain the integrity of the sample sufficiently long for the test to be executed.

11.2.2 Stabilized Samples

The stabilizing agent for the stabilized samples was ordinary portland cement and four sets of samples were made containing 10 per cent, 5 per cent, 3.5 per cent and 1 per cent cement by dry weight of the minestone. As expected these samples were durable and sufficiently firm to facilitate mounting in the apparatus following 24 hours of curing. All the samples were able to be placed in the eroding fluid without surface disruption from escaping air bubbles.

Following 7 days of normal curing the samples containing 10 and 5 per cent cement were found to be too erosion resistant when tested for, even at the highest rotational speed, the shear force exerted upon the surface of the sample was insufficient to produce any marked erosion. The samples containing 3.5 per cent remained intact at low and intermediate rotational speeds though, at high speeds, erosion was observed but at a low rate with the measured critical shear stress occurring near to the maximum rotational speed of the rotating cylinders. In the case of the Wearmouth samples stabilized with 3.5 per cent cement, clouding of the erosional fluid was not observed suggesting that the critical shear stress had not been reached during the test. The samples containing 1 per cent cement eroded at low rotational speeds with sufficient quantities of eroded material being produced to allow accurate measurement and subsequent calculation of the critical shear stress. However, considerable care was needed when handling the 1 per cent samples during the soaking and mounting operations as serious damage could be inflicted as a result of their low strength and durability. Even low levels of damage were liable to render the samples unsuitable for subsequent testing.

It was concluded from the feasibility study that unstabilized minestone could not be tested in the rotating cylinder apparatus with confidence and that the results would not be accurate and meaningful. The unstable nature of the samples, partly due to the lack of cohesiveness coupled with the effects of slaking, would not allow accurate measurement of either the erosion rates or critical shear stresses. For minestone to be tested within the rotating cylinder device a stabilizing agent was required to thoroughly bind the particles together.

It was found that ordinary portland cement could only be used as a stabilizing agent for the rotating cylinder test, in quantities of less than 3.5 per cent (with a 7 day normal curing period). If greater quantities of cement were used the critical shear stress of the sample at which pronounced erosion began, was not reached using the range of rotational speeds generated by the particular apparatus. With a cement content of 1 per cent the behaviour of the samples was satisfactory, with slaking being prevented and significant erosion being observed during the subsequent testing. It must be stated, however, that research carried out by Van Wijk (1985) showed that the critical shear stress was relatively low for crushed stone aggregate even with the addition of 7 per cent cement.

It was therefore decided that the main test programme would be carried out on minestone samples from Hatfield, Lady Windsor, Point of Ayr, Sutton Manor, Wardley, Wearmouth, and Westoe collieries which had been processed to pass through a 6.5mm sieve and stabilized initially with 1 per cent cement.

11.3 Rotating Cylinder Test Results and Discussion (Minestone Plus 1 per cent OPC)

Plate 11.1 shows the typical effects of shear erosion upon minestone stabilized with 1 per cent cement following a test within the rotating cylinder apparatus. It was noticeable, with the samples, that the surface was eroded evenly around the entire circumference, with very little evidence of any localised areas of severe erosion. This suggests that these samples possessed a weak fabric that was uniformly susceptible to erosion around the entire circumference, compared to the uneven pattern of erosion shown by a similar specimen with 10 per cent cement (*see Plate 11.2*). Another general erosion characteristic of these samples was the suddenness at which failure occurred. The failure of the sample was characterised by a dramatic decrease in the shear resistance as a result of disintegration and collapse. Care was taken to prevent this situation occurring as it did not significantly add to the information on the erodability but seriously delayed further testing as complete dismantlement of the apparatus was necessary to remove the debris from the collapsed sample.

Table 11.1 presents the critical shear stress for each of these stabilized minestones together with other salient properties. For each minestone *Figure 11.1* illustrates the relationship between the applied shear stress and the amount of surface erosion rate. The figures clearly show two phases in these relationships. The first stage of erosion is characterised by a slow increase in the erosion rate with rising shear stress while the second stage is represented by a dramatic increase in the erosion rate for only small increments of shear stress. The point at which the dramatic change in erosion rate occurs is known as the critical shear stress and is represented by the intersection of two straight lines plotted through the data points. The pattern of erosion was noted in the results of previous studies by Moore and Masch (1962), Espey (1963), Arulanandan *et al.* (1973; 1975) Chapuis (1986), Van Wijk (1985) and Lo and Lovell (1983) and Fan (1988).

The critical shear stresses for minestone stabilized with 1 per cent cement range from 1.5Pa for the POAA samples to 7.3Pa for the Sutton Manor samples. A relationship is seen to exist between the critical shear stress and the compressive strength of the samples. As the compressive strength of the samples increases the resistance to shear erosion is also seen to increase (*see Figure 11.2*). Further examination reveals a relationship between the quantity of material present that passed a 63µm mesh and compressive strength, illustrated in *Figure 11.3*. However, the strongest relationship was found to exist between the critical shear stress of the samples and the percentage of the sample that was sub 63µm in size. The relationship between these two characteristics has a correlation factor of 0.93 and is illustrated in *Figure 11.4*.

Samples from POAA, Wardley and Sutton Manor (representing the least resistant and most resistant of these samples) were cured in distilled water for a week prior to being tested in the rotating cylinder device. The POAA sample suffered complete disintegration during immersion and was not suitable for testing in the shear apparatus. Although the Wardley and Sutton Manor samples showed signs of cracking and fabric disruption, their integrity was sufficient to allow testing to take place. *Figures 11.5* and *11.6* compare the relationships between the critical shear stress and the erosion rate for samples cured in air for seven days (normal) and for those cured for a further 7 days in distilled water. It can be seen that the erosion resistance of both the Wardley and the Sutton Manor samples suffered a significant drop when immersed in distilled water for a week. The Wardley samples had a critical shear stress of 3.8Pa when cured normally for 7 days. Following a further 7 days of curing in distilled water the threshold for serious disruption had been lowered to 1Pa. The critical shear stress for the Sutton Manor samples showed a similar decrease, dropping from 7.5Pa when cured normally to 5Pa when cured in distilled water.

It is unlikely that minestone would ever be stabilized with just 1 per cent cement considering its susceptibility to crack and disintegrate following immersion. However, the previous results give a certain amount of guidance in determining the critical shear stress of unstabilized minestone, since unstabilized minestone is unable to be tested in the rotating cylinder device. The critical shear stress for unstabilized minestone must fall below the critical shear stresses shown by the samples stabilized with 1 per cent cement. Thus if, in a field situation, shear stresses are measured in excess of those experienced by the 1 per cent samples, it can be assumed that minestone in the unstabilized condition would be unsuitable for use.

The appropriateness of the rotating cylinder device for testing minestone is further thrown into doubt when consideration is taken of the size of the particles present in raw minestone and the maximum particle size able to be used in the present rotating cylinder test. Between 40 and 80 per cent of the particles in raw minestone will be larger than the maximum particle size capable of being tested in the rotating cylinder test. Although the rotating cylinder test would be able to determine the behaviour of the finer fraction of a minestone, the effect of the larger particles on the erosion resistance of the finer fraction would remain unknown. Consequently the rotating cylinder test is unable to account for the behaviour of a large proportion of a minestone structure in field conditions. The test is not totally inappropriate for a structure built with compacted minestone which would possess an outer skin of finer material, a natural phenomenon associated with the rolling and compaction of a soil material. The erodability of this layer could be estimated using the rotating cylinder test.

As a result of the difficulties encountered in fabricating a suitable piece of apparatus during the study period, the extent of this phase of the research was limited, especially in relation to long term testing. However, a small number of tests were carried out on Wearmouth minestone plus 10 per cent cement cured for 28 days in air, distilled water and sea water. *Figures 11.7* and *11.8* show the erosion rates for samples cured in the different environments for 7 and 28 days respectively, while *Figures 11.9* to *11.11* compare the erosion rates at 7 and 28 days for the individual curing environments. It can be seen from these figures that the critical shear stress was not reached following 7 days of curing in any of the three curing environments. This was also so for the samples cured in air and distilled water for 28 days. *Figures 11.9* and *11.11* show that the erosion rate was lowered slightly following 28 days curing in air and distilled water. However, the samples cured in sea water for 28 days showed a marked degree of deterioration with the critical shear stress being sufficiently low to be registered (*see Figure 11.10*). Following 28 days of curing the critical shear stress for the samples had fallen from 90Pa to 53Pa.

For the samples stabilized with 1 per cent cement the zone of erosion was an even, continuous band around the entire circumference of the sample giving a uniformly roughened surface, however for the 10 per cent samples immersed in sea water for 28 days the erosion was distributed in individual, discontinuous patches probably reflecting lines of weakness such as cracking which were in evidence following immersion in sea water. *Plate 11.2* illustrates the form erosion takes for the samples stabilized with 10 per cent cement.

11.4 Further Considerations Concerning the Rotating Cylinder Device.

Until recently, it was thought that the critical shear stress obtained from the rotating cylinder device was a close representation of the critical shear stresses experienced in the field. However, a recent study has challenged this proposition suggesting that the critical shear stresses obtained using the device are well below the critical shear stresses experienced in natural environments and hence exhibits increased erositivity (Rohan and Lefebvre, 1991). The findings in their study are based upon two hydraulic aspects, the streamline curvature and fluctuations in the radial component of velocity.

11.4.1 Streamline Curvature

The hydrodynamic analysis of flow within the device is based upon the consideration that water flowing within the annular space is closely analogous to flow in a curved rectangular pipe, which can be represented by straight flow between two plates with respect to streamline and shear stress profiles (Rohan and Lefebvre, 1991). The lower plate, representing the sample, is fixed whilst the upper plate has a steady constant velocity equal to the outer cylinder of the rotational device.

Flow is shown to equate closely for rectilinear conditions and rotating cylinders when laminar flow is experienced. Shear stresses and coefficient of friction have been shown by calculations to be equal for these two sets of hydraulic conditions. It was these calculations, based on laminar flow, that Moore and Masch (1962) used to equate the hydraulic conditions present in the rotational device with those experienced in the field.

Rohan and Lefebvre (1991) believe that the high rotational velocities involved in the test can only result in turbulent flow. The centrifugal forces generated in the annular flow (curved rectangular flow) do not exist for straight-forward rectilinear flow. The dividing point between laminar flow and the stable vortices known as Taylor Rotational Vortices, can be given by the critical Reynolds number derived from the following relationship :

$$\mathrm{Re}_{cr} = \frac{2\pi N \rho}{\eta} \alpha^{1.5} \quad r_i^2 > 41.3$$

(*Equation 11.1*)

where

Re_{cr} = Critical Reynolds number.
ρ = fluid density (kg/m^3).
N = rad/s/2π.
η = dynamic viscosity (Ns/m^2).
r_e = external radius (m).
r_i = internal radius (m).
α = r_e/r_i .

According to *Equation 11.1*, the critical Reynolds number will be very low, hence the vortices will develop at very low velocities.

In the rotating cylinder test, flow towards the exterior of the cell will be subjected to a far greater centrifugal force than flow towards the interior of the cell. The gradient in centrifugal force causes a depression to form externally allowing the fluid to move away from the internal surface (i.e. the sample) and helping to set up helicoidal flow. Helicoidal flow is far more efficient at dissipating energy than rectilinear flow and has a different surface interaction with the sample.In a series of equations and calculations, Rohan and Lefebvre (1991) demonstrate that the mean shear stress experienced in the rotating cylinder would always be higher than the mean shear stress set up by a rectilinear flow for the same flow velocity. A summary on how they derived such a proposition follows.

In order to simplify matters and to avoid working with the complex hydrodynamics of helicoidal flow, the relationship between rotational curved flow and rectilinear flow was derived using the well established equations that govern flow within a rectilinear pipe, known as White's Formula (*Equation 11.2*) and Prandtl's expression (*Equation 11.3*)

$$\frac{\left(\frac{f}{2}\right)_{rot}}{\left(\frac{f}{2}\right)_{rect}} = 1 + 0.075 \left[\mathrm{Re}\left(\frac{D}{D_S}\right)^2\right]^{0.25}$$

(*Equation 11.2*)

For $1.5 \times 10^4 < \mathrm{Re} < 10^5$

Where

f = Roughness coefficient.
D = Pipe diameter.
D_s = Average diameter for the spires of the coil.
Re = Reynolds number.

and Prandtl's expression :

$$\frac{\left(\frac{f}{2}\right)_{rot}}{\left(\frac{f}{2}\right)_{rect}} = 0.288\,(D_N)^{0.36} \qquad (\textit{Equation 11.3})$$

Where D_N = Dean's number

and applicable when

$$160 < D_N < 4000$$

To make *Equation 11.2* applicable to the analysis, the relationship between roughness coefficient (f) and the shear stress (τ) within a turbulent flow must be applied :

$$\frac{f}{2} = \frac{\tau}{\rho V_d^2} \qquad (\textit{Equation 11.4})$$

Where V_d = mean velocity in the duct
ρ = density of the fluid

When *Equation 11.4* is introduced and by using the same flow velocity value, *Equation 11.2* becomes :

$$\frac{\tau_{rot}}{\tau_{rect}} = 1 + 0.075\left[Re\left(\frac{D}{D_s}\right)^2\right]^{0.25} \qquad (\textit{Equation 11.5})$$

whilst *Equation 11.3* becomes :

$$\frac{\tau_{rot}}{\tau_{rect}} = 0.288\,(D_N)^{0.36} \qquad (\textit{Equation 11.6})$$

To circumvent the difficulty of calculating D_s in *Equation 11.5*, two extreme cases, the most favourable and least favourable, were used to establish the range of values for τ_{rot}

For *Equation 11.5*

$$Re = 10^4 \text{ and } \frac{D}{D_s} = 0.003 \text{ then } \tau_{rot} = 1.04\ \tau_{rect}$$

D = Pipe diameter.
D_s = Average diameter for the spirals of the coil.

and

$$Re = 10^5 \text{ and } \frac{D}{D_s} = 0.15 \text{ then } \tau_{rot} = 1.51\ \tau_{rect}$$

Hence

$$1.04\ \tau_{rect} < \tau_{rot} < 1.51\ \tau_{rect}$$

Further evidence that the mean shear stress for rotating cylinders will be greater than the shear stress experienced in rectilinear flows is provided by *Equation 11.6* which applies for a maximum Reynolds number of 8000 (often exceeded by the rotational device)

$$DN = 160 \text{ with } \tau_{rot} = 1.79\,\tau_{rect}$$

$$DN = 4000 \text{ with } \tau_{rot} = 5.70\,\tau_{rect}$$

Hence

$$1.79\ \tau_{rect} < \tau_{rot} < 5.70\ \tau_{rect}$$

Rohan and Lefebvre concluded that the mean value of τ_{rot} would always be greater than τ_{rect} for the same flow velocity. Natural streams have a rectilinear type flow with only macro-radial flow around large bends. In comparison the flow radius within the rotating cylinder is micro, suggesting that a comparison of shear stress between the two is prone to error.

11.4.2 Fluctuations in the Radial Component of Velocity

The stability of the boundary layer in the presence of three dimensional disturbances, can be given using the Taylor number in the form (Schlichting, 1979) :

$$Ta = \frac{V_e\, d}{\nu} \left[\frac{d}{r_i} > 41.3 \right]$$ (*Equation 11.7*)

d = the width of the gap
r_i = inner radius
V_e = peripheral velocity of the outer cylinder
ν = the kinematic viscosity

Three regimes of flow can be recognised using the Taylor number

$Ta < 41.3$: Laminar Couette flow.
$41.3 < Ta < 400$: Laminar flow with Taylor vortices.
$Ta > 400$: Turbulent flow.

The Taylor numbers that exist in the high velocity regime of the rotational device suggest that full turbulent flow will always be attained.

Although no consensus has been reached concerning the influence of fluctuations in the radial component of velocity (V_r'), they have been shown to exceed the tangential fluctuations (V_t') by a factor of four in the internal zone of an annular space (Townsend, 1980). This situation is different to the rectilinear flow associated with natural channels. The relationship between shear stress and turbulent fluctuations is given by Rohan and Pilon (1984) as :

$$\tau = \rho\, \phi\, (Re)\, \overline{V}^2$$ (*Equation 11.8*)

where
$\phi\,(Re)$ = A form factor dependant upon the Reynolds number (Re).
$\overline{V}$ = turbulent fluctuation of velocity.
τ = Shear stress.

Using *Equation, 11.8* Rohan and Lefebvre (1991) calculated that, if the turbulent fluctuations in the radial component were 20 per cent greater than the tangential fluctuations, the shear stress would be 50 per cent higher than the shear stresses measured using the rotating cylinder test. The difference may further increase once erosion is initiated and surface roughness becomes more marked. Using the extreme differences between the radial and tangential components of velocity, that Townsend (1980) reported, the actual shear stress experienced by the sample may be a magnitude higher than the shear stress derived from the torque measurement established with the rotational device.

Rohan and Lefebvre (1991) were also concerned with the eroded material suspended within the eroding fluid. Both the rate of erosion and the critical shear stress will be influenced by the suspended load, leading to an underestimation of the critical shear stress and compounding even further the inadequacies of the rotating cylinder test .

11.4.3 The Bearing of the Rohan and Lefebvre Analysis on the Present Research

The rotating cylinder apparatus used in the present research was selected, fabricated and extensively employed prior to the publication of the analysis detailed by Rohan and Lefebvre (1991). The test was selected for the present research on the basis that it had been considered to mirror natural hydraulic environments (Moore and Masch 1963). The Rohan and Lefebvre analysis has cast serious doubts upon the capabilities of the test to provide accurate, quantitative results. If their suggestions are accepted, the critical shear stress results obtained for the minestone during the present research programme must be viewed as being poor indicators of natural field conditions and be regarded with reservation. Thus the expectations of the apparatus for providing valuable information relating test results to the erositivity of minestone in field structures have not been realised. Although the results might be considered as poor indicators of the absolute values of the critical shear stress, they may still provide useful information to be used on a comparative basis to assess the relative performance of the various samples.

The rotating cylinder test was heavily relied upon during the present research to tie in laboratory work with real situations. If the recent analysis proves substantive it would mean that failure has occurred in forging this link, thus seriously undermining this section of the research programme. An underestimation of the shear stress value of up to an order of magnitude may be expected once account is taken of the streamline curvature and fluctuations in the radial component of velocity. With such suggested errors existing between the measured and the true critical shear stress, the quantitative values of the rotating cylinder test must be considered flawed. However, it does have a valuable use as a comparative test, helping to distinguish the erositivity levels of different materials and enabling them to be ranked in order of resistance. Although the results of the present research have failed to elucidate the precise mechanisms of minestone erosion, the employment of the results to determine the reactions of different minestones to similar erosive stresses, can be an invaluable comparative tool.

Minestone	Critical Shear Stress (Pa)	Compressive Strength (MPa)	% of Sample Finer than 63μm
Hatfield	4.5	0.45	11
Lady Windsor	4.0	0.43	14
POAA	1.5	0.14	25
POAB	7.2	0.83	5
Sutton Manor	7.3	1.11	4
Wardley	3.8	0.43	15
Wearmouth	6.3	0.92	6
Westoe	4.5	0.52	10

Table 11.1 Critical shear stress results for minestone stabilized with 1 per cent OPC.

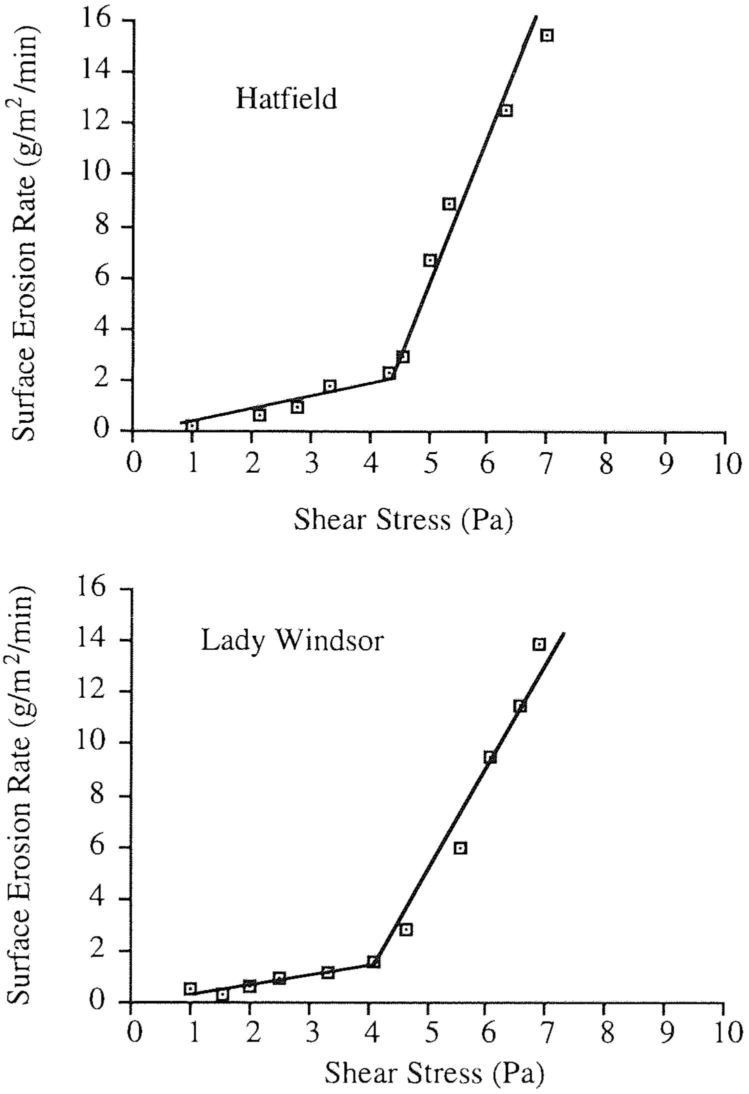
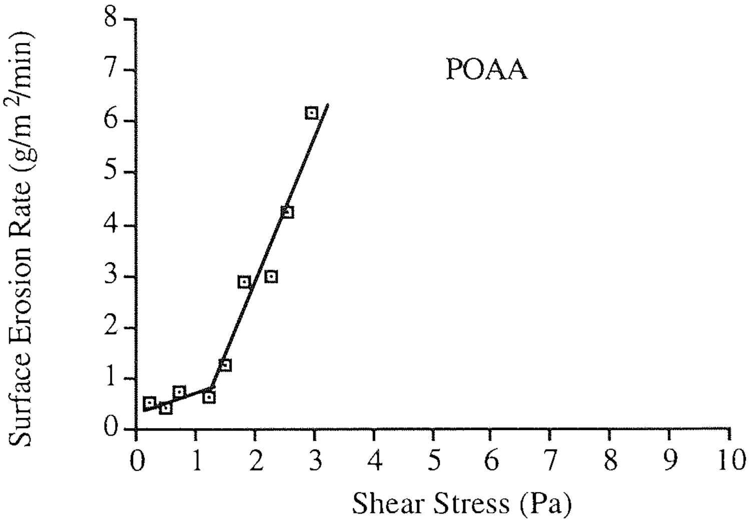
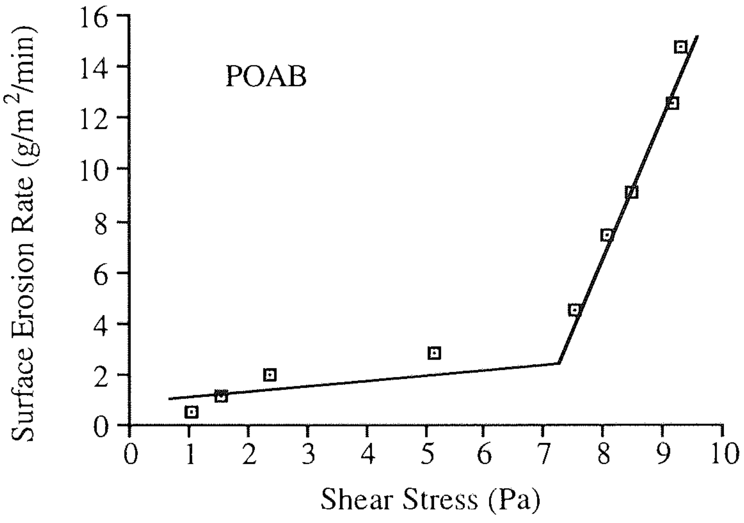


Figure 11.1 Erosion rate versus shear stress for minestone plus 1 per cent OPC.





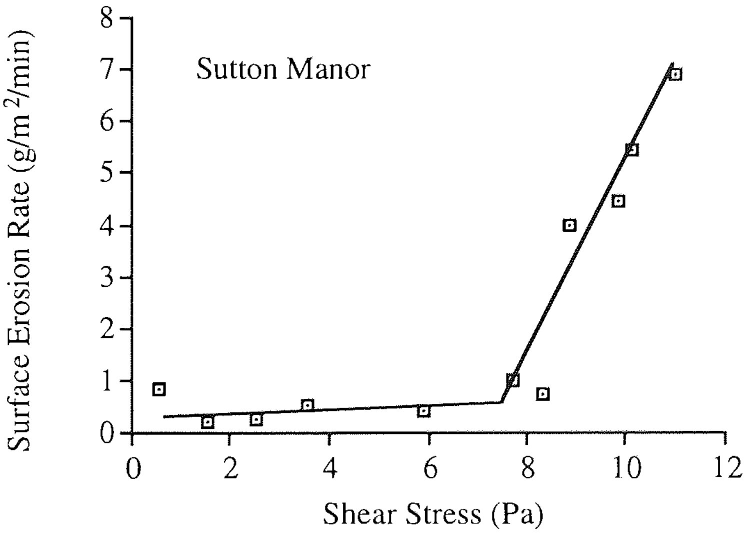
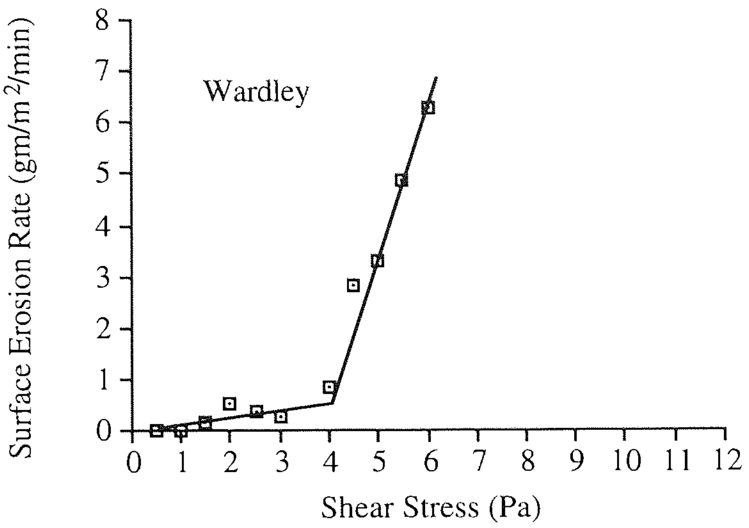
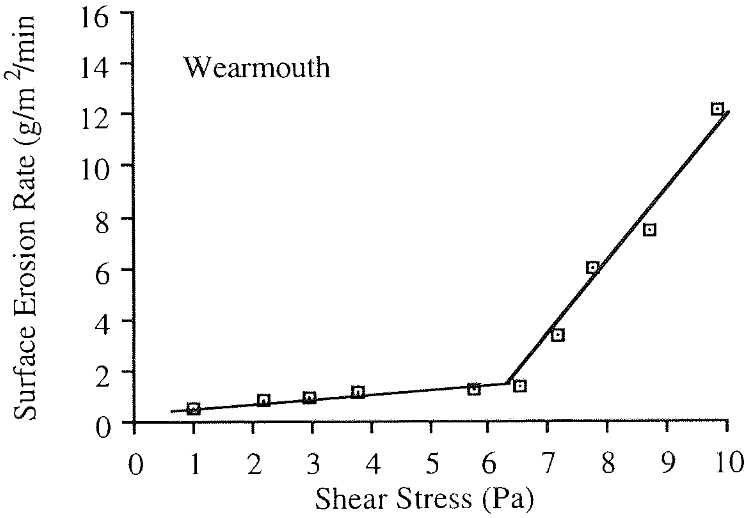


Figure 11.1 (continued)





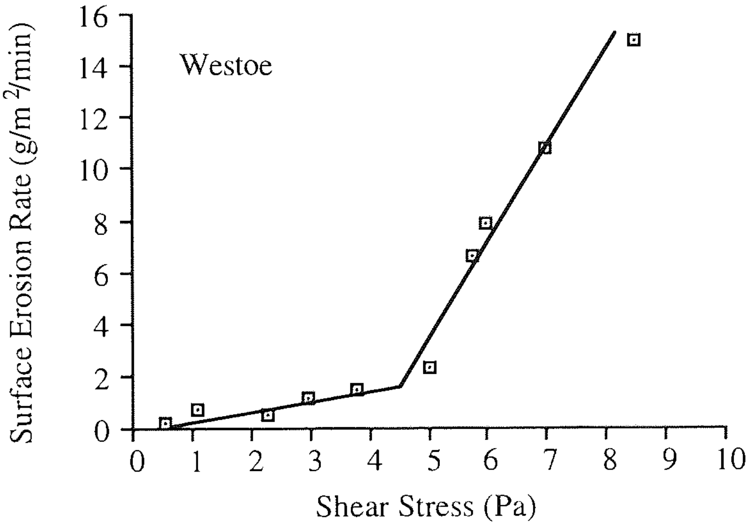


Figure 11.1 (continued)

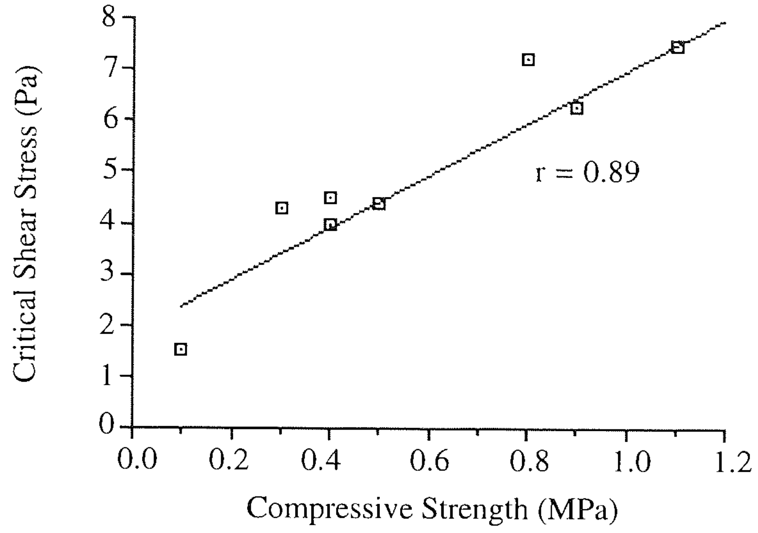


Figure 11.2 The relationship between compressive strength and the critical shear stress of minestone stabilized with 1 per cent OPC.

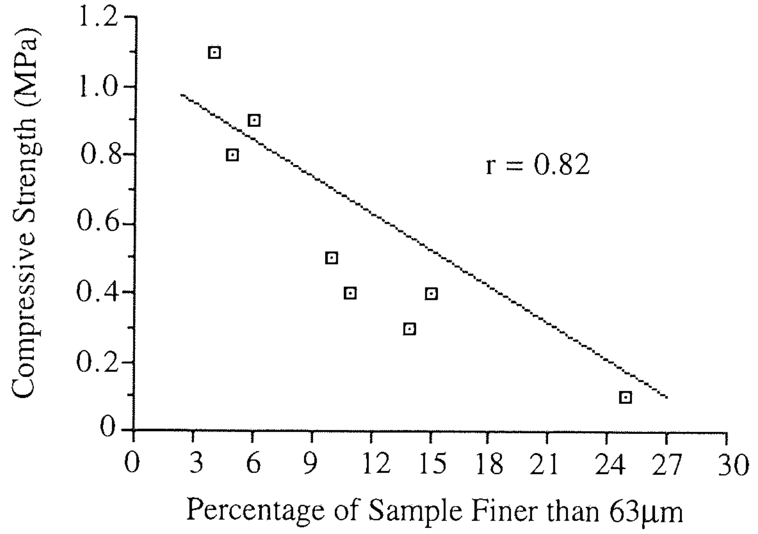


Figure 11.3 The relationship between compressive strength and the percentage of the sample passing a 63 μm sieve.

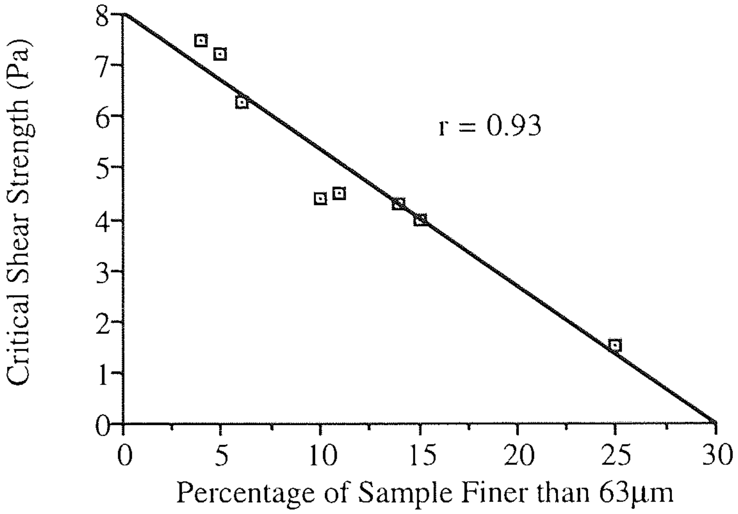


Figure 11.4 The relationship between critical shear stress and the percentage of sample passing the 63 μm sieve.

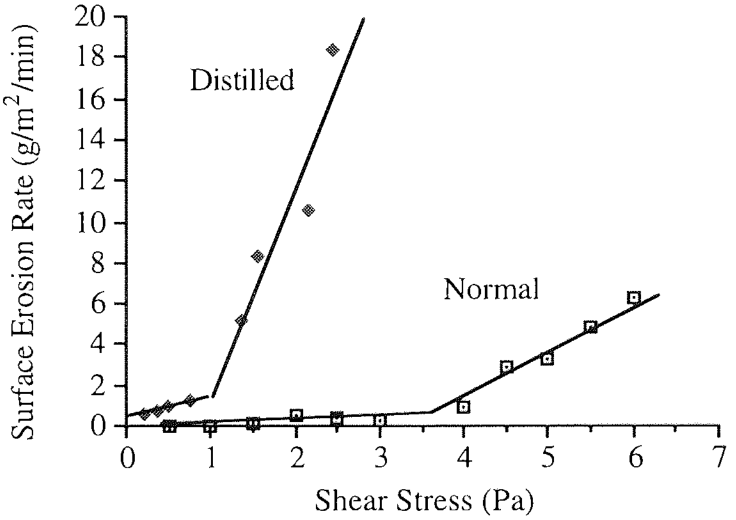


Figure 11.5 The effects of curing in distilled water for 7 days on the erosion rate for Wardley minestone stabilized with 1 per cent OPC.

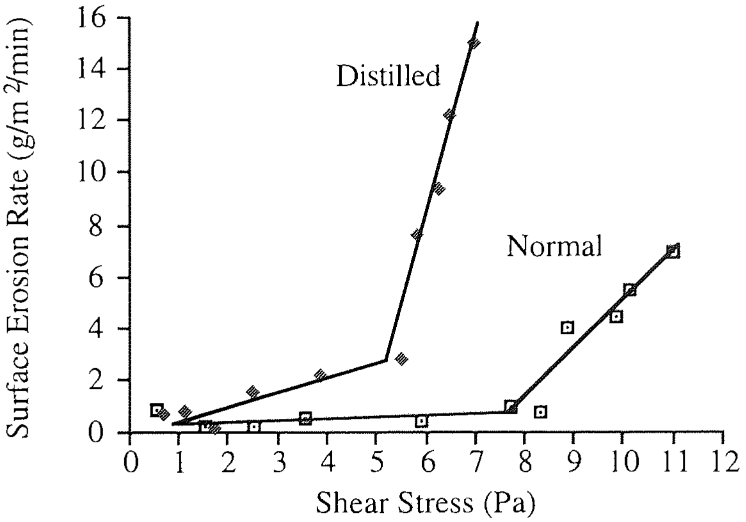


Figure 11.6 The effects of curing in distilled water for 7 days on the erosion rate for Sutton Manor minestone stabilized with 1 per cent OPC.

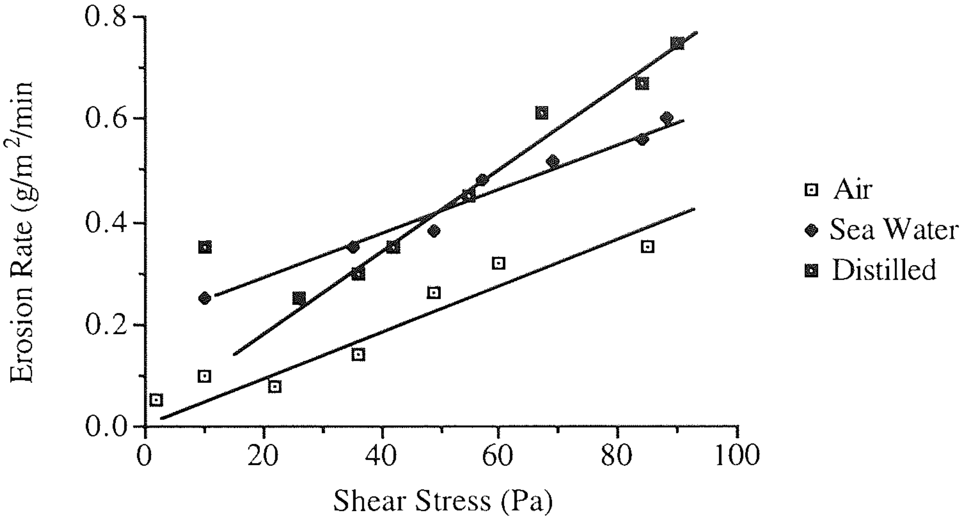


Figure 11.7 The effects of differing curing mediums on the erosion rate of Wearmouth minestone stabilized with 10 per cent OPC following 7 days.

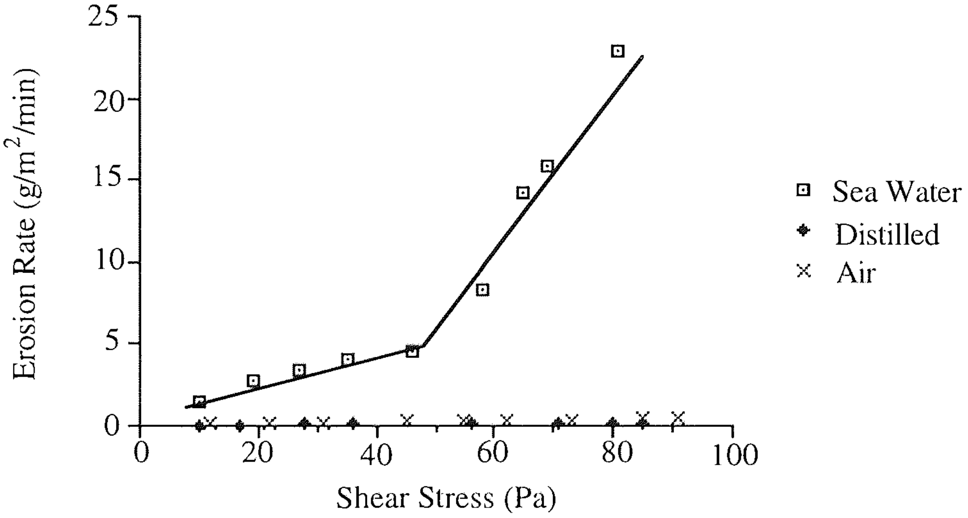


Figure 11.8 The effects of differing curing mediums on the erosion rate of Wearmouth minestone stabilized with 10 per cent OPC following 28 days.

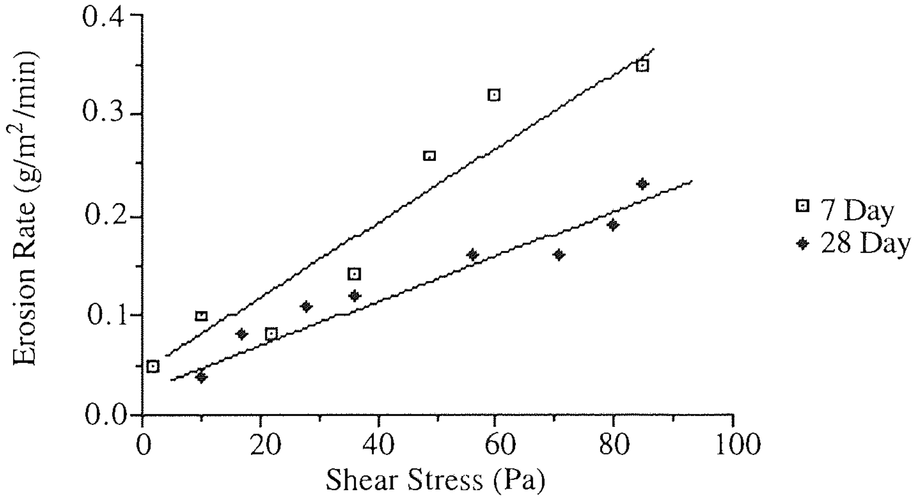


Figure 11.9 The effect of curing in air on the erosion rate for Wearmouth minestone plus 10 per cent OPC.

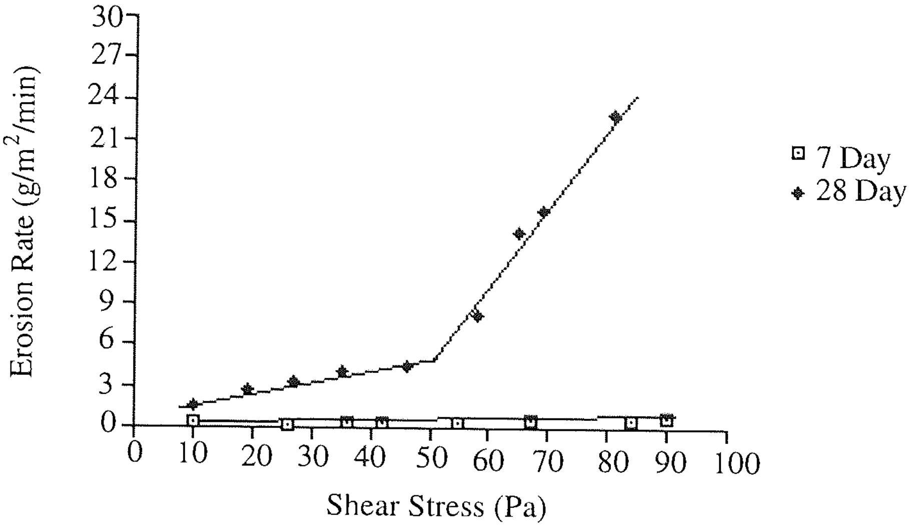


Figure 11.10 The effect of immersion in sea water on the erosion rate for Wearmouth minestone plus 10 per cent OPC.

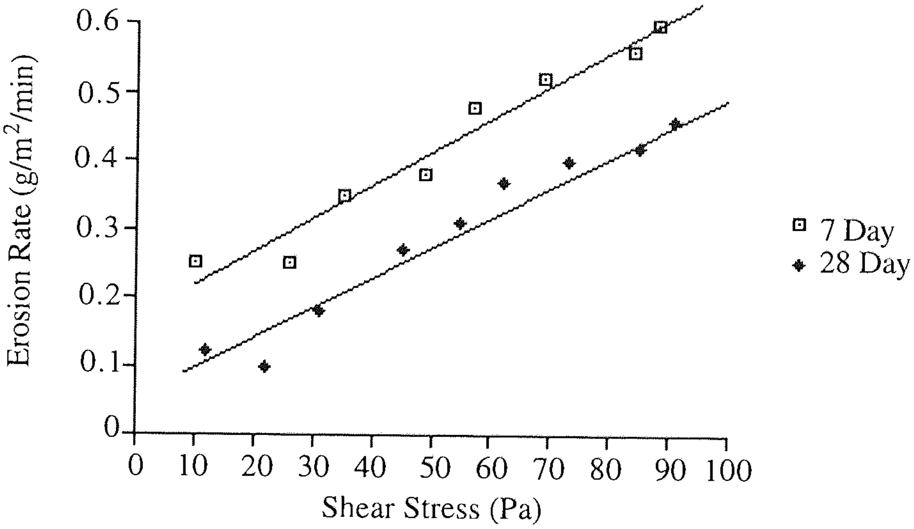


Figure 11.11 The effect of immersion in distilled water on the erosion rate for Wearmouth minestone plus 10 per cent OPC.

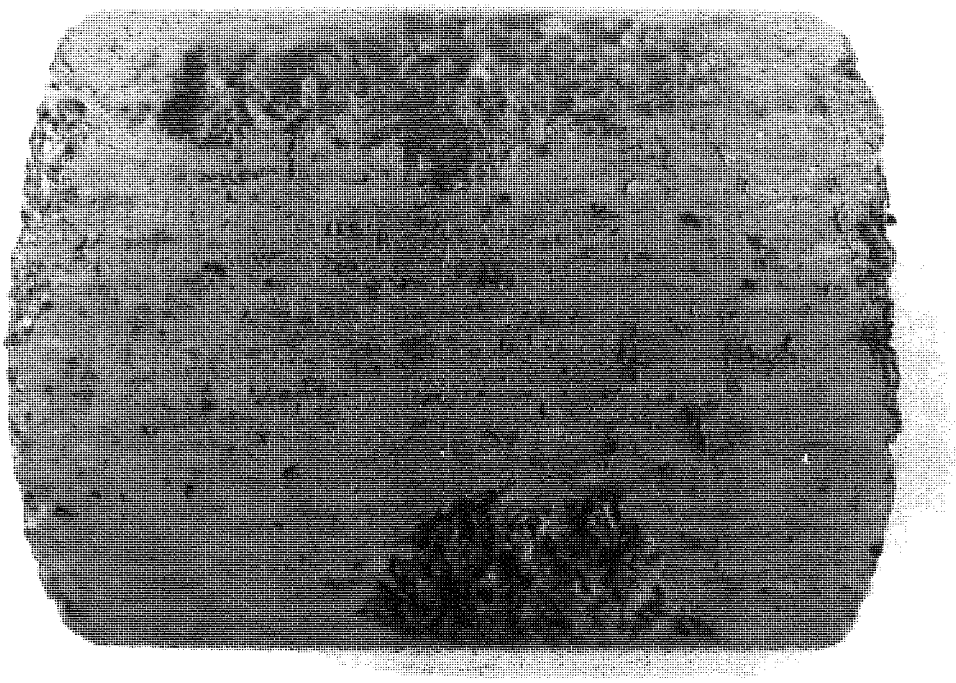


Plate 11.1 Typical damage to stabilized minestone (1 per cent OPC) as a result of shear erosion. Note the even nature of the erosion pattern.

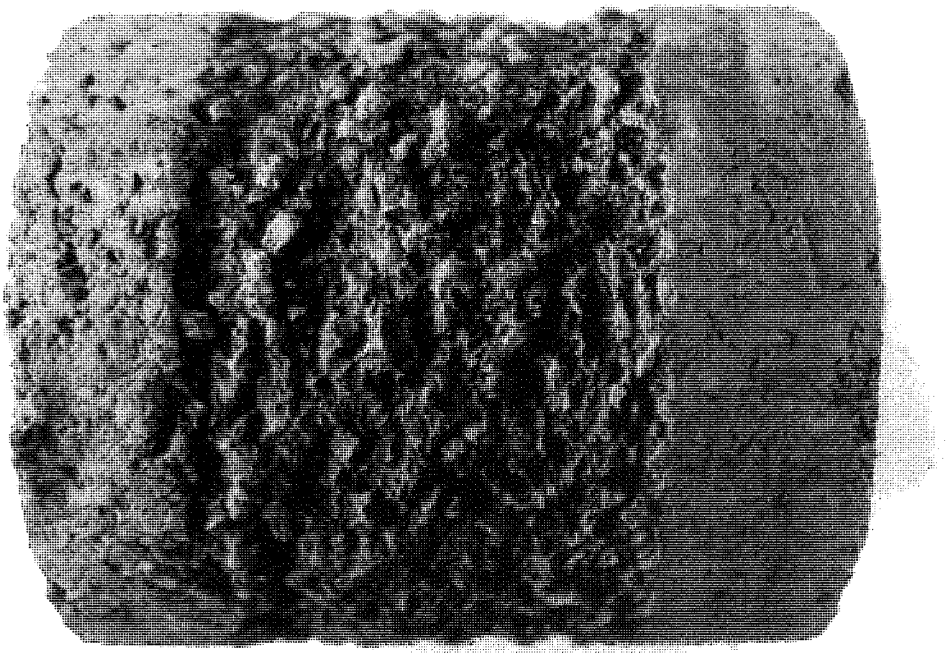


Plate 11.2 Typical damage to stabilized minestone (10 per cent OPC) as a result of shear erosion. Note the uneveness of the erosion pattern.

12.0 CONCLUSIONS

12.1 Introduction

The erosion resistance of unstabilized minestone and cement stabilized minestone (CSM) was investigated in the laboratory. Further tests were carried out to investigate the response of these materials to separate immersion in sea water and distilled water. The programme of tests examined the erosion resistance using the erosion/abrasion test, the resistance to shear erosion using the rotating cylinder test, slake durability, compressive strength and immersion resistance. Although the erosion tests were found to have limitations with regard to testing minestone, the results suggest that both unstabilized minestone and CSM have potential for use as construction materials in hydraulic structures.

12.2 General Conclusions Relating to Erosion Resistance

If either unstabilized minestone or CSM is to be used as a replacement for the more traditional, bulk, hydraulic materials it must be shown to have comparable properties and/or advantages over the traditional materials. Although some limitations became apparent when the erosion test was used for minestone specimens, both the unstabilized and cement stabilized minestone produced a better performance than the artificial dredged sand. The following sections detail the conclusions reached during the erosion study.

12.2.1 Erosion/Abrasion Test

12.2.1.1 Unstabilized Minestone

1. Erosion of the unstabilized material is predominantly a function of the impact pressure of the water jet, with the abrasive element being of only secondary importance.

2. In the unstabilized state, both minestone and sand mixes exceed the limit for negligible erosion for the range of impact pressures used in the test. Thus, if unstabilized minestone is to be used in a hydraulic structure, it would need to be protected from direct wave action or flow action using a stabilized layer, rip-rap, graded layers and/or geo-textiles. As the range of impact pressures used in this investigation represent maximum wave heights and not average wave conditions, it is envisaged that unstabilized minestone could be used in more sheltered situations as a sacrificial material.

3. Unstabilized minestone was typically 3 to 6 times more resistant to erosion than the laboratory sand mix. This greater resistance has been attributed to the presence of larger aggregate particles and to the cohesive nature of the matrix.

4. The performance of the individual minestones depends on the grading characteristics of each material. Well graded samples, with a balance between matrix and aggregate, develop the greatest resistance to erosion. Samples with a predominance of coarse particles and, consequently insufficient matrix exhibit intermediate resistance, while samples with an excess of fine material exhibit the greatest susceptibility to erosion.

5. Field observations at the Point of Ayr site suggest that the presence of clays in the matrix was an important factor in erosion resistance. The cohesive nature of the clays appears to help protect the aggregate from wave buffeting and subsequent self-degradation.

12.2.1.2 Stabilized Minestone

1. Abrasion is the predominant erosion agent in the degradation of the cement stabilized test materials. Without the presence of abrasive particles the force of the water is not significant in the degradation of the samples. However, a significant correlation exists between the impact pressures of the water jet and the resultant erosion rates when the jet contains abrasive materials.

2. By stabilizing minestone with ordinary portland cement a dramatic improvement in the erosion resistance of the material is achieved. The relatively low cement content of 5 per cent improves the resistance to erosion of the eight minestones by between 5 and 22 times compared to that of unstabilized material.

3. A cement content of 5 per cent is sufficient to ensure that only a negligible erosion rate occurs at the lower impact pressures of 70 kPa and 140 kPa. Cement contents of 7.5 per cent and 10 per cent are required to achieve a negligible erosion rate at the higher impact pressures of 210 kPa and 280 kPa.

4. It is indicative from the erosion rate trend that significant improvement in the erosion resistance would not be achieved by the addition of further quantities of cement above 10 per cent. It is the resistance of the minestone aggregate itself that controls the erosion rate at cement contents of 10 per cent or above and not the cement stabilized matrix.

5. At a given cement content, stabilized minestone manufactured from the coarser samples is more resistant to erosion than those produced from finer material.

6. Stabilized minestone is more resistant to erosion than the laboratory sand mix, when equivalent quantities of cement are used. The cement stabilized sand mix fails to achieve a negligible rate of erosion at all the test impact pressures, whereas the majority of the CSM samples, are able to achieve the suggested resistance to erosion even at the highest impact pressures.

7. The difference in performance between the various CSM samples and the cement stabilized sand is attributed to the characteristics of the particles, in particular both their size and the nature of their fabric, their relationship to each other within the soil skeleton and the means by which they transmit shock waves following an impact by an abrading particle. The maximum grain size in the samples is a major influence in determining the erosion resistance of the test materials.

8. With the CSM, much lower compressive strengths are required to sustain a negligible erosion rate than are necessary for cement stabilized sands and soils. The relationship between compressive strength and erosion rate is less clearly pronounced for the CSM than for both the cement stabilized sands tested in this research and soil-cements tested in a previous Canadian study.

9. Erosion resistance is increased following prolonged immersion in distilled water. However, following immersion in sea water, the resistance to erosion is eventually lowered in the long term, though the degradation caused by the erosion/abrasion test was considered to be less than was expected when consideration was taken of the severely disrupted state of the fabric at the end of immersion. Although prolonged immersion in sea water is detrimental to the erosion resistance of the CSM samples, the samples immersed for 500 days in sea water still show more resistance than cement stabilized sand immersed for a similar period of time.

12.2.2 Rotating Cylinder Test

The rotating cylinder test was chosen for its reported ability to accurately determine the shear resistance of a material but, unfortunately, this study was beset with problems. The apparatus was redesigned twice during the research period to incorporate recent modifications that came to light during the later stages of the literature review. Eventually a piece of apparatus was fabricated that was considered capable of giving accurate and reproducible results although further problems became apparent during the feasibility tests. The test was basically chosen to aid research into unstabilized minestone, but the unstable nature of raw minestone, with its propensity to slake and its relatively non-cohesive nature, made accurate testing impossible. The results of the feasibility study also indicate [illegible] only samples with a low percentage of cement are capable of being tested within the [illegible] as the minestone samples undergo a period of detrimental conditioning prior to [illegible]

The appropriateness of the rotating cylinder device for testing minestone was further thrown into doubt when consideration was taken of the size of the particles in raw minestone and the maximum particle size able to be used in the rotating cylinder apparatus. Between 40 and 80 per cent of the particles in the raw minestone are larger than the maximum particle size capable of being tested in the device. Consequently the test is unable to account for the behaviour of a large proportion of a minestone structure in field conditions. Towards the end of the laboratory study, a report into the hydrodynamics of the rotating cylinder test was published (Rohan and Lefebvre 1991) suggesting that the results from the rotating cylinder test would not represent the true critical shear stresses for materials such as minestone and that additionally, the test does not truly simulate field conditions.

Despite the problems encountered with the rotating cylinder test and the concern over the veracity of the critical shear stress results some general conclusions concerning the shear erosion of CSM can be made :-

1. The resistance to shear erosion is increased by the addition of cement.

2. At low cement contents, there is evidence that a relationship exists between the critical shear stress and the compressive strength of a CSM sample.

3. A further relationship is shown to exist between the sub 63μm percentage of the sample and the critical shear stress. The higher the percentage of fines within a sample, the lower the critical shear stress and consequently the higher the susceptibility to erosion.

4. The susceptibility of CSM to erosion by shear forces is greatly increased by only a short period (28 days) of immersion in sea water.

12.3 The Stability of Immersed Minestone

The exposure, to weathering and erosional agents, of an element in a hydraulic structure depends on its relative position with respect to the various zones of weathering. These zones range from complete submergence, a relatively constant environment, through a dynamic zone that suffers from cyclic change, to a subaerial zone that is influenced intermittently by the various agents of weathering. The response of minestone to the prevailing conditions in those various weathering zones, especially with regard to the slake durability, was investigated and the conclusions are outlined in the following section.

12.3.1 General Conclusions From the Slake Durability Study.

1. All the minestones exhibit some degree of slaking when immersed in sea water or distilled water.

2. The slaking characteristics of the test minestones is not controlled by the mineralogical or lithological nature of the material but is governed by the nature of the fabric, especially the pore size distribution.

3. A relationship is shown to exist between the total pore surface area per gram of sample and the standard slake durability index. Hatfield has a strong propensity to slake which is attributed to the presence of pores with a diameter of less than 0.006μm. Such pores are absent or negligible in the other test samples, all of which had a greater degree of resistance to slaking than the Hatfield minestone.

4. Minestone is shown to have a greater resistance to slaking when immersed in sea water than when immersed in distilled water. The improvement in resistance ranges between 0.5 and 8 per cent using Laan's rate of fineness test and 2 and 25 per cent using the modified slake durability test.

5. Slaking is inhibited when the test samples are saturated or close to saturation prior to immersion. This suggests that 'air breakage' is the predominant slaking mechanism involved in the minestone degradation.

6. Minestone that is permanently submerged will suffer no physical degradation by way of slaking, once the initial period of slaking has ended or if the material is saturated or near saturated prior to immersion. This is important when considering that minestone will more often than not be in the saturated state when collected from a spoil heap or colliery or at least in equilibrium with the interstisial fluid.

7. The tidal simulation tests indicate that minestone present in a structure within the tidal zone would not drain or dry sufficiently during the low tide period to allow repeated slaking to occur. Limited slaking could occur on the surface where action by wind and sun would accelerate drying, though the material derived from this degradation may help to seal the surface thus further enhancing protection for the material below.

8. The tests indicate that minestone will be highly susceptible to breakdown in the subaerial zone. Once again, however, it is suggested that a protective skin of finer degraded material would form to protect the inner zones of the structure from extremes of temperature, air movement, and help to retain moisture within the structure. It is thought that the use of a sealing layer, or the presence of vegetation, would help to limit the effects of slaking within this zone.

9. Generally the rate of slaking increases with decreasing grain size and increasing particle surface area, although towards the finer end of the particle size range, the rate of increase is reduced. The production of clay size particles through slaking is negligible.

12.3.2 Conclusions Concerning the Compressive Strength and Dimensional Stability of CSM

1. With all the cement stabilized samples a cement content of 10 per cent is required to achieve the normally specified 7 day compressive strength of 4.5MPa, although 7.5 per cent is sufficient with some of the minestones. A cement content of 10 per cent also enables all the samples to pass the US Bureau of Reclamation strength requirements for soil-cement to be used in hydraulic structures.

2. The early strength development of normally cured CSM is slightly inhibited when sea water, rather than potable water, is used for moulding.

3. The short-term immersion durability of CSM is slightly greater when immersed in sea water compared to immersion in distilled water. This improved performance is attributed to the lower slaking susceptibility of minestone in sea water.

4. The short term strength development and immersion durability of CSM in distilled water is governed by the finer fraction of the minestone and by the properties of the material. Minestones with a higher percentage of fines and a greater propensity to slake produces CSM with a lower strength and durability. Consequently, the removal of the finer fraction of the minestone by wave action improves the strength and immersion durability of CSM.

5. None of the samples exhibits excessive linear expansion following immersion, less than 1 per cent in distilled water and less than 2 per cent in sea water. However, the samples immersed in sea water for prolonged periods show a high degree of surface disruption.

6. The processes associated with sulphate attack begin almost immediately following the immersion of CSM in sea water. The sulphates derived from sea water cause surface disruption within 10 days of immersion All the CSM samples immersed in sea water exhibit sulphate attack regardless of their intrinsic sulphate content.

7. All the samples immersed in distilled water for 500 days exhibit higher compressive strengths than at 7 days following normal curing. This is also valid for the majority of samples immersed in sea water. Despite the apparent severity of fabric disruption following prolonged periods immersed in sea water, the relatively high compressive strength values suggest that a high percentage of the deterioration was of a superficial nature affecting mainly the edges and free surfaces.

8. The strength loss of the samples immersed sea water may have been exaggerated by the test conditions. In practice, sea water would be expected to circulate through and around the CSM and, as the presence of chloride in sea water makes the gypsum and ettringite produced by sulphate attack more soluble, they would expect to be carried away by the circulating water. Hence the build up of these voluminous and detrimental minerals would be reduced. In the laboratory the water is virtually static so these minerals were not removed but continued to build up, and help to weaken the fabric of the CSM.

12.4 Limitations of Study and Recommendations for Future Work

The present investigation into the hydraulic performance of unstabilized minestone and CSM was confined to a laboratory study carried out on relatively small test specimens. While such laboratory studies provide valuable information concerning the basic qualities of a material and are relatively inexpensive and straight-forward to undertake, there is ultimately no substitute for large scale field experiments, especially when dealing with a variable bulk material such as minestone. However, small scale studies are invaluable for both identifying the factors that control the performance of a material and selecting possible sources for large scale projects, but, the true response of a bulk material in the real environmental can only be gauged when scale is taken into consideration and this necessitates full scale field experimentation.

A major factor in the present study is the relatively narrow range in the quality and character of the selected minestones, although this arose primarily because the study was directed at minestones that were being considered, or had been used for hydraulic structures. With the exception of Hatfield and the POAA samples, all the other samples are very similar. This makes it difficult to reach conclusions on the relationship between various characteristics and properties of the minestones in relation to their performance, and so further work should include minestones with a wider range of properties so that the validity of the relative relationships can be extended.

The long term study into the stability of CSM in distilled water and sea water was terminated after 500 days. Studies of longer duration would be required to determine whether trends recognised in the present study will continue and to assess the effects of such trends on the long term performance of CSM.

The poor performance of the rotating cylinder test prevented the shear erosion resistance of minestone from being determined adequately. This may be righted if a test programme involving the Erositivity Test (Rohan and Lefebvre 1986) is implemented.

A full scale porosimetry investigation into minestone may help to reveal important information concerning the breakdown of minestone by the action of slaking.

The erosion resistance of minestone alone is inadequate to maintain a negligible rate of erosion. A study into the influence of geo-textiles on the erosion resistance of unstabilized minestone is thus proposed.

It can be concluded from the present study that both minestone and cement stabilized minestone compare very favourably with the more traditional bulk materials that have been used for river, coastal and sea defences when tested for erosion resistance. If judicial material selection is practised, together with stringent construction control on site, the author believes that both minestone and its stabilized equivalent would be viable alternatives to the more traditional bulk hydraulic materials such as dredged sand. As the interest in tidal barrage schemes grows and the need for improved coastal management becomes more apparent, the results and conclusions of the present study will gain greater relevance especially as the economic and environmental costs of more traditional materials increase.

REFERENCES

ADASKA, W.S. 1991. *"Soil Cement- A Material with Many Applications"* Concrete International, January, p. 49-52.

AASHO. 1970. "*Standard Specifications for Highway Materials and Methods of Sampling and Testing.*", Report N145, American Association of State Highway Officials, Washington D.C.

AKKY, M.R. 1974. "*The Erodibility of Cement-Stabilized Soils.*", PhD Thesis, University of California, USA.

AKKY, M.R. and SHEN, C.K. 1973. "*Erodibility of a Cement-Stabilized Sandy Soil.*", Special Report 135, Highway Research Board, Washington D.C, USA, p. 20-29.

ALLEN, R.T.L. and TERRETT, F.L.1968. "*Durability of Concrete in Coast Protection Works.*", Proceedings of the11th Conference. on Coastal Engineering, Sept, American Society of Civil Engineers, 6843.101, p. 1200-1210.

AMERICAN CONCRETE INSTITUTE 1970. "*Expansive Cement Concretes-Present State of Knowledge.*", ACI 223, ACI Materials Journal, Proceedings, 67, 8, p. 583-610.

AMERICAN CONCRETE INSTITUTE 1987. "*Erosion of Concrete in Hydraulic Structures.*", ACI Materials Journal, Committee 210 Report, Title No. 210R-87.1987.

AMERICAN CONCRETE INSTITUTE COMMITTEE 1990. "*State of the Art Report on Soil-Cement.*", ACI Materials Journal, 87, p. 395-417.

ANDREWS, D.C. 1966. "*Lime-Soil Stabilization.*", PhD Thesis, Leeds University, U.K.

ARIATHURAI, R. and ARULANANDAN, K.1978. "*Erosion Rates of Cohesive Soils.*", Technical Notes, Proceedings of the American Society of Civil. Engineers, 104 (No HY2), p. 279-283.

ARMAN, A. and SAIFAN, F. 1967. "*The Effect of Delayed Compaction on Stabilized Soil-Cement.*", Highway Resarch Record 198, p. 30-38.

ARMAN, A. and DANTEN, T.N. 1969. "*Effects of Admixtures on Layered Systems Constructed with Soil-Cement.*", Bulletin No. 263, Highway Research Board, Washington D.C., USA.

ARULANANDAN, K, LOGANATHAN, P. and KRONE, R.B. 1973. "*Application of Chemical and Electrical Parameters to Prediction of Erodibility.*", Special Report, 135, Highway Research Board, Washington DC, USA. p. 42-51.

ARULANANDAN, K., LOGANATHAN, P. and KRONE, R.B. 1975. "*Pore and Eroding Fluid Influences on Surface Erosion of Soils.*", ASCE Journal of the Geotechnical Engineering Division, 101(GT1), p. 51-66.

ASHMEAD, D. 1955. "*The Influence of Bacteria in the Formation of Acid Mine Waters.*". Colliery Guardian, June, p. 694-698.

BADGER, C.W, CUMMINGS, A.D. and WHITMORE, R.L.1956. "*The Disintegration of Shales in Water.*", Journal of the Institute of Fuel, October, p. 417-423.

BASCOM, W. 1980. "*Waves and Beaches.*" Anchor Press/Doubleday, Garden City, New York.

BEHRMAN, S. and THURLOW, G.G. 1980. "*The Use of Colliery Shale for Construction.*", in "An Environmental Appraisal of Tidal Power Stations", Edited by T.L. Shaw, Pitman, London. p. 197-200.

BELLY, R. T. and BROCK, T. D. 1974. "*Ecology of Iron-Oxidizing Bacteria in Pyritic Materials Associated with Coal.*" Journal of Bacteriology, 117, p. 726-732.

BERGLUND, M. 1986. Fine Homebuilding, Aug-Sept 1986, p. 35-59.

BERKOVITCH, I., MANACKERMAN, M. and POTTER, N. M. 1959. "*The Breakdown Problem in Coal Washing : Part 1.*", Journal of the Institute of Fuel, 32, p. 579-589.

BESSEY, G.E. AND LEA, F.M. 1953. "*The Distribution of Sulphates in Clay Soils and Ground Water.*" Proceedings of the Institute of Civil Engineers, 2 (1), p. 159-81.

BRIDLINGTON FREE PRESS 1988. "*Coal Mine Waste Project Cancelled.*", 24th November.

BRITISH COAL 1989. "*Geotechnical Properties of United Kingdom Minestone.*", British Coal, Hobart House, London.

BRITISH COAL 1990. "*Minestone Services Data Pack.*", Minestone Services, British Coal, Hobart House, London.

BRITISH STANDARDS INSTITUTION 1975a. "*Methods of Test for Stabilized Soils.*", BS1924., British Standards Institution, London.

BRITISH STANDARD INSTITUTION 1975b. "*Methods of Test for Soils for Civil Engineering Purposes.*", BS1377, British Standards Institution, London.

BROCK, T.D. 1975. "*Effects of Water Potential on Growth and Iron Oxidation by Thiobacillus ferrooxidans.*", Applied Microbiology, 29(4), p. 495-501.

BROS, B.E. 1990. "*Seepage Phenomena and Control of Phreatic Surface within two Lagoon Embankments Constructed of Coarse Coal Waste.*", Proc. 3rd Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, British Coal, Sept, Glasgow, U.K., p. 275-282.

BROWN, E.T. 1981. "*Rock Characterization, Testing and Monitoring.*". ISRM Suggested Methods. Pergamon, London.
CALDWELL,J.A., SMITH, A. and WAGNER, J. 1984. "*Heave of Coal Shale Fill.*", Canadian Geotechical Journal, 21, p. 379-383.

CANIBANO, J.G. and LEININGER, D. 1987. "*The Characteristics and Use of Coal Wastes.*", Proc. 2nd Int. Symp.on the Reclam, Treat and Util. of Coal Mining Wastes, Sept, Nottingham, U.K., p. 111-122.

CARR, C.E. and WITHERS, N.J 1987. "*The Wetting Expansion of Cement-Stabilized Minestone- An Investigation of the Causes and Ways of Reducing the Problem.*", Proc. 2nd Int. Conf.on the Reclam., Treat. and Util. of Coal Mining Waste, Sept, Nottingham, U.K., p. 545-559.

CARUCCIO, F.T, GEIDELL, G. and SEWELL, J.M. 1976. "*The Character of Drainage as a Function of the Occurrence of Framboidal Pyrite and Ground Water Quality in Eastern Kentucky*", Proc. 6th Symp Coal Mine Drainage Research, Louisville, Kentucky, p. 1-16.

CASIAS, T.J. and HOWARD, A.K. 1984."*Performance of Soil-Cement Dam Facings: 20-Year Report.*", Report No. REC-ERC-84-25, U.S. Bureau of Reclamation, Denver.

CASIAS, T.J. 1991. "*Bureau of Reclamation Soil-Cement Slope Protection.*", Concrete International, January, p. 59-64.

CASWELL, R. H. and TRAK, B.,1985. "*Some Geotechnical Characteristics of Fragmental Queenstone Shale.*", Canadian. Geotechnical. Journal., 22, p. 403-408.

CHAPUIS, R.P. 1986. "*Quantitative Measurement of the Scour Resistance on Natural Solid Clays.*", Canadian Geotechnical Journal, 23, p. 132-141.

CHAPUIS, R.P. and GATIEN, T. 1986. "*An Improved Rotating Cylinder Technique for Quantitative Measurements of the Scour Resistance of Clays.*", Canadian Geotechnical Journal, 23(1), p. 83-87.

COLBACK, P.S.B.and WIID, B.L.1965. "*The Influence of Moisture Content on Compressive Strength of Rocks.*" Proc.Rock Mech.Symp.Toronto 1965,p. 65-83.

COLLINS, R.J. 1976. "*A Method for Measuring the Mineralogical Variation of Spoil from British Collieries.*", Clay Minerals 11, p. 31-49.

COOKE, R.U. 1979. "*Laboratory Simulation of Salt Weathering Processes in Arid Environments.*",Earth Surface Processes, 4, p. 347-59.

COUPER, A.S. and MONTGOMERY, H. 1990. "*Minestone Fill for a Maritime Village Development at Port Edgar, South Queensferry, Scotland.*", Proc 3rd Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes. Sept, Glasgow, U.K., p. 377-390.

CROFT, J.B. 1967. "*The Influence of Soil Mineralogical Composition on Cement Stabilization.*", Geotechnique, 17, p. 119-135.

CURTIS, C.D. 1976. "*Chemistry of Rock Weathering.Fundamental reactions and Controls in Geomorphology and Climate.*", Wiley,London.

DAVIDSON, D.T. 1961. "*Soil Stabilization with Portland Cement.*", Bulletin No.292, Highways Research Board, Washington DC, USA. p. 1-27.

DAVIS, F.J., GRAY, W.E. and JONES, C.W. 1973. "*The Use of Soil-Cement for Slope Protection.*", Trans. 11th Int. Congress on Large Dams, Vol.III, p. 237-255

DEGROOT, G. 1971. "*Soil-Cement Seepage Test Section, Lubbock Regulating Reservoir Canadian River Project, Texas.*", Report No. REC-ERC-71-13, U.S. Bureau of Reclamation, Denver.

DEPARTMENT OF ENERGY 1981. "*Tidal Power from the Severn Estuary; Volume II.*", Energy Paper No. 46, H.M.S.O., London.

DEPARTMENT OF ENERGY 1989. "*The Severn Barrage Project: General Report.*", Energy Paper No. 57, H.M.S.O., London.

DEPARTMENT of the ENVIRONMENT 1970. "*A Guide to the Structural Design of Pavements for New Roads.*", Road Note 29, HMSO, London.

DEPARTMENT OF TRANSPORT 1976. "*Specifications for Road and Bridge Works.*", HMSO, London.

DEPARTMENT OF TRANSPORT 1986. "*Specifications for Road and Bridge Works.*", HMSO, London.

DIAMOND, S. and KINTER, E.B. 1966. "*Mechanisms of Soil-Lime Stabilization.*" Public Roads, 33, No. 12, p. 260-273.

DIXON, K., SKIPSEY, E. and WATTS, J.T. 1964. "*The Distribution and Composition of Inorganic Matter in British Coals-Initial Study of Seams from the East Midlands Division of the N.C.B.*", J. Inst. Fuel, Nov, p. 485-493.

DULANEY, R.L. 1960. "*The Structural Strength of Coal Mine Floors.*" MSc Thesis,VPI 74 pp.

DUNBAVAN, M. 1990. "*Utilizing Coarse Refuse for Tailings Dam Construction.*", Proc 3rd Int. Symp. on the Reclam.,Treat and Util. of Coal Mining Wastes. Sept, Glasgow, U.K. p. 267-274.

DUNN, J.R. and HUDEC, P.P. 1965. "*The Influence of Clays on Water and Ice in Rock Pores.*", Contribution (unpublished), Dept. of Geology, Renssalaer Polytechnic Institute, Canada, p. 1-149.

DUNN, J.R. and HUDEC, P.P. 1972. "*Frost and Sorbtion Effects in Argillaceous Rocks.*", Highway Research Records, No. 393, p. 65-78.

DUPAS, J.M. and PACKER, A. 1979. "*Static and Dynamic Properties of Sand-Cement.*", Proceedings. ASCE, 105, GT3, p. 419-436.

EMERSON, W.W. 1977. "*Physical Properties and Structure.*" in "Soil Factors in Crop Production in a Semi-Arid Environment", Edited Russell, J.S. and Greacen, E.L, Queensland University Press, Brisbane, p. 78-104.

ESPEY, W.H. 1963. "*A New Test to Measure the Scour of Cohesive Sediments.*", Technical Report No. HYD 01-6301., Hydraulic Engineering Laboratory, Dept. of Civil Engineering, University of Texas

FAN, J.C. 1988. "*Prediction of Erosion on Cut and Fill Slopes.*", Final Report, Joint Highway Research Project, FHWA/IN/JHRP-88/4, Purdue University, Indiana.

FELT, E.J. 1955. "*Factors Influencing Physical Properties of Soil-Cement Mixtures.*", Bulletin No.108 Highway Research Board, Washington DC, USA, p. 138-162.

FOOKES, P.G. 1991. "*Geomaterials.*", Quart. J. Engn. Geol., 24, p. 3-15.

FOOKES, P.G. and POOLE, A.B. 1981. "*Some Preliminary Considerations on the Selection and Durability of Rock and Concrete Materials for Breakwaters and Coastal Protection Works.*", Quart. J. Eng. Geol, 14, p. 97-128.

FOOKES, P.G, GOURLEY, C.S. and OHIKERE, C. 1988 ."*Rock Weathering in Engineering Time.*", Quart. J. Eng. Geol, 21, p. 33-57.

FRANKLIN, J.A. and CHANDRA, R. 1972. "*The Slake Durability Test.*", Int. J. Rock Mech. Min. Sci., 9, p. 325-341.

FRENCH, W.J. 1991. "*Concrete Petrography: A Review.*", Quart. J. Eng. Geol, 24, p. 17-48.

GARRELS, R.M. AND THOMSON, M.E. 1960. "*Oxidation of Pyrite by Iron Sulphate Solutions.*", Am. J .Sci.,258A, p. 57-67.

GILLOT, J.E, PENNER, E. and EDEN, W.J., 1974. "*Microstructure of Billings Shale and Biochemical Alteration Products, Ottowa , Canada.*", Can. Geotech. J., 11, p. 482-489.

GLOVER, H.G. 1984. "*Environmental Effects of Coal Mining Waste Utilization.*", Proc. Int. Symp. on Reclamation, Treatment and Utilization of Coal Mining Wastes. Sept. Durham, U.K., p. 17.1.

GOUDIE, A.S.,COOK, R.U. and EVANS, I.S. 1970. "*Experimental Investigation and Rock Weathering by Salts.*", AREA, 4, p. 42-48.

GRAF, W.H. 1970. "*Hydraulics of Sediment Transport*", McGraw-Hill, New York.

GRAINGER, P 1984. "*The Classification of Mudrocks for Engineering Purposes.*", Quart. J. Eng. Geol., 17, p. 381-387.

GRICE, R.H.1968."*The Effects of Temperature-Humidity on the Disintegration of Non-Expandable Shales.*", Bull. Assoc. Eng. Geol, 5, p. 69-77.

GRIFFIN, G.M. 1971. "*Interpretation of X- Ray Diffraction Data.*", in Procedures in Sedimentary Petrology, Edited by Carver R.E., Wiley, New York, p. 541-569.

GRIM, R.E. 1968. "*Clay Mineralogy.*", Mc Graw-Hill, New York.

GRIMER, F.J. and KRAWCZYK, J. 1963. "*Relations between Strength and Age for Soil-Cement with Particular Reference to the Prediction of Later Strengths from Earlier Strengths.*", Mag. Conc. Res. 15 (43), p. 21-30.

GRIMER, F.J. and ROSS, N.F. 1957. "*The Effect of Pulverization on the Quality of Clay Cement.*", Proc. 4th Int. Conf. on Soil Mech. and Found.Eng. p. 109-113.

HALE, B.C. 1979. "*The Development and Application of a Standard Compaction-Degradation Test for Shales.*" Joint Highways Research Project, BJHRP-79-21, Purdue University, Indiana

HAMILTON, W.A.H. 1984 . "*A Colliery Shale Sea Wall at Deal.*", Proc. 1st Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Durham, U.K., p. 45.1.

HANSEN, K.D. 1991. "*Soil-Cement is Big in Texas Water Projects.*", Concrete International, January, Vol.13 No.1. p.49-52.

HAUSS, K.D. and HEIBAUM, M.H. 1990. "*Minestone in German Waterway Engineering.*", Proc. 3rd Int. Symp. on the Reclam, Treat. and Util. of Coal Mining Wastes, Sept, Glasgow, U.K., p. 244-256.

HAUSS, K.D. and MAYER, B.K. 1990. "T*ests Concerning Reclamation, Treatment and Utilization of Coal Mining Wastes in Waterway Engineering.*", Proc 3rd Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Glasgow, U.K., p. 243-248.

HAWKINS, A.B. and PINCHES, G.M. 1987. "*Sulphate Analysis on Black Mudstones.*", Geotechnique, 37, p. 191-196.

HEMPHILL, R.W. and BRAMLEY, M.E. 1989. "*Protection of River and Canal Banks.*", CIRIA, Butterworths, London.

HENDERSON, F.M. 1966. "*Open Channel Flow.*", MacMillan, New York.

HERZOG, A. 1963. "*Evidence of a Skeletal Matrix Structure in Clays Stabilized With Portland Cement.*", 5th Australian and New Zealand Conference on Soil Mechanics and Foundation Engineering. p. 55-61.

HERZOG, A. and MITCHELL, J. K. 1963. "*Reactions Accompanying Stabilization of Clay with Cement.*" Highway Research Record, 36, p. 146-171.

HIGHWAY RESEARCH BOARD 1961. "*Soil Stabilization with Portland Cement.*", Bulletin 867:292, Highway Research Board, Washington DC, USA.

HILLEL, D. 1971. " *Soil and Water:Physical Principles and Processes.*" Academic Press, New York.

HOCKLEY, D.E. and VAN DER SLOOT, H.A. 1991. "*Long-Term Processes in a Stabilized Coal-Waste Block Exposed th Seawater.*", Environ. Sci. Technol., 25, p. 1408-1414.

HOPE, G. 1987. "*Minestone for Hydraulic Applications.*", Interim Report No.1, Civil Engineering Department, Aston University, Birmingham, U.K.

HOPKINS, T.C. AND DEEN, R.C. 1984. "*Identification of Shales.*", Geotechnical Testing Journal, GTJODJ, 7(1), p. 10-18.

HORTON, A.E., MANACKERMAN, M. and RAYBOULD, W.E. 1964. " *The Shale Breakdown Problem in Coal Washing : Part 2.* "Journal of the Instiute of Fuel, 37, p. 52-58.

HUDEC, P.P. 1973."*Weathering of Rocks in Arctic and Subarctic Environments.*", Proc. Symp. on the Geology of the Canadian Arctic, Geol Assoc. Canada, p. 313-365.

HUDEC, P.P. 1977. "*Deterioration and Dimensional Changes of Carbonate Rocks as a Function of Adsorbed Water Content.*", Proc. 2nd Int Symp. on Water Rock Interaction, Strasbourg, 4, p. 38-42.

HUDEC, P.P. 1983."*Statistical Analysis of Shale Durability Factors.*", Transport Research Record, 873, p. 28-35.

HUDEC, P.P. 1989. "*Durability of Rock as a Function of Grain Size, Pore Size and Rate of Capillary Adsorption of Water.*", Journal of Materials in Civil Engineering, 1(1). ASCE. p. 3-9.

INSTITUTION OF CIVIL ENGINEERS 1989. "*Coastal Management.*", Institution of Civil Engineers, Thomas Telford, London.

IVARSON, K.C., ROSS, G., and MILES, N 1978. "*Alterations of micas and feldspars during microbial formation of basic ferric sulphates in the laboratory.*" Soil Science Society of America Journal, 42, p. 518-524.

JOHNSON, V.E. 1963. "*Mechanics of Cavitation.*", Journal of the Hydraulics Division, Proceedings ASCE, 89, HY3, p. 251-275.

KALOUSEK, G.L., PORTER, L.C. and BENTON, E.J. 1972. "*Concrete for Long Time Service in Sulphate Environments.*", Cement and Concrete Research, 2(1), p. 79-89.

KELLER, W.D. 1957. "*The Principles of Chemical Weathering.*", Lucas Bros, Columbia, Mississippi, U.S.A.

KENNARD, M.F., KNILL, J.L. and VAUGHAN, P.R. 1967. "*The Geotechnical Properties and Behaviour of Carboniferous Shale at the Balderhead Dam.*", Quart. J Eng. Geol., 1, p. 3-24.

KENNEDY, J. 1979. "*The Economic Viability of Using Cement Stabilized Materials for Pavements", Cem. Conc. Res., Tech. Note DN/1048*

KETTLE, R.J. 1973. "*Freezing Behaviour of Colliery Shale.*" Ph.D.Thesis, University of Surrey, Guildford, U.K.

KETTLE, R.J. 1978. "*The use of Colliery Shale in Highway Construction.*", Int. Conf. Materials of Construction for Developing Countries. Thailand.

KETTLE, R. J. 1990. "*The Influence of Particle Size on the Performance of Cement Bound Minestone.*" Proc. 3rd Int.Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Glasgow, U.K., p. 33-40.

KETTLE, R.J. and WILLIAMS, R.I.T. 1969. "*Preliminary Study of Cement Stabilized Unburnt Colliery Shale.*", Proc. Int. Conf. Use of By-Products and Waste in Civil Engineering, Vol ii, Ecole Nationale Des Ponts et Chaussees, Paris, p. 475-81.

KETTLE, R.J. and WILLIAMS R.I.T 1977. "*Frost Action in Cement-Stabilized Colliery Shale.*",Transport Research Record 641, p. 41-47.

KING, S.W.H. 1968. "*Discussions on the Properties of Concrete Under Complex States of Stress.*" Proc. Int. Conf. on the Structure of Concrete, Cement and Concrete Association, London, p. 293.

KLUTH, D.J. 1984. "*The Use of Minestone in the Selby-Wistow-Cawood Barrier Bank.*", Proc. 1st Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Durham, U.K., p. 46.1.

KNUTZ, H. and PLOGMANN, H.1979. "*L'utilization des Schistes Houillers du Point de vue Materiers Premieres, Journees D'information.*" Nouveaux Procedes de Valorisation du Charbon", Luxembourg, Sept,vol.2.

KUZNETZOV, S.L., IVANOV, M.V. and LYALIKOVA, C.H. 1963. "*Introduction to Geological Microbiology.*", McGraw Hill, New York.

LAAN, G.,VAN WESTEN, J.M. and BATTERINK, L. 1984. "*Minestone in Hydraulic Engineering. Application, Deterioration and Quality Control.*", Proc. Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Durham, U.K., p. 5.1.

LANE, E.W. 1955. "*Design of Stable Channels.*", Trans. American Soc. Civil Engineers, 1120, p. 1234-1279.

LASAN, T.C. and CODY, P.D. 1967. "*Identification of Aggregates Exhibiting Frost Susceptibility.*", NCHRP, final report project 4-3, Dept. Civil Engineering, Penn. State University, Pennsylvania.

LAUTRIDOU, J.P. and OZOUF, J.C. 1982. "*Experimental Frost Shattering.*", Progress in Physical Geography, 6, p. 215-232.

LAWRENCE, J.A. 1972. "*Some Properties of South Wales Colliery Discards.*", Colliery Guardian, June.

LAZAROFF, N. 1963. "*Sulphate Requirement for Iron Oxidation by Thiobacillus ferrrooxidans.*", J. Bacteriology, 106, p. 78-82.

LEA, F.M. 1970. "*The Chemistry of Cement and Concrete.*", 3rd Edition, Arnold, London.

LEATHEN, W.W., KINSEL, N.A., and BRALEY, S.A. 1956. "*Ferrobacillus ferrooxidans:A Chemosynthetic Autotrophic Bacterium.*", J. Bacteriology, 99, p. 700-703.

LEEDER, M.R. 1983. " *Sedimentology.Process and Product.*", George Allen and Unwin, London.

LEININGER, D, LEONHARD. W.ERDMANN, W. and SCHEIDER,T. 1987. "*Research on Suitability of Coal Refuse in Civil Engineering in FDR.*" Proc. 2nd Int. Symp.on the Reclam, Treat and Util. of Coal Mining Wastes.", Nottingham, U.K., p. 55-68.

LIGHTSEY, G.R. and CALLIHAN, C.D. 1970. "*Changes in the Characteristics of Soil-Cement Stabilized by the Addition of Excess Compaction Moisture.*", Record No. 315, Highway Research Board, Washington DC, USA, p. 36-45.

LILLEY, A.A. 1971. "*Cement Stabilized Materials for Road and Airfield Pavements.*", Technical Report No. 42. 458 , Cement and Concrete Research Association, Slough U.K..

LILLEY, A.A. 1979. "*Cement Stabilized Materials-Past, Economics and Future.*", Civil Engineering, December, p. 25-29.

LITTLE, J.A., ADAM, E.A. and LUNT, M.G. 1983. "*Traditional and Improved Methods of Low-Cost Housing in the Sudan.*", Int. J. for Development Technology,1, p. 109-120.

LITTON, L.L. 1982. "*Soil-Cement for use in Stream Channel Grade Stabilization Structures.*", MSc. Thesis, Iowa State University, Ames, Iowa.

LITTON, L.L. and LOHNES, R.A 1982. "*Soil-Cement for Use in Stream Channel Grade Stabilization Structures.*", Transport Research Record No. 839, p. 33-37.

LITTON, L.L. and LOHNES, R.A 1984. "*Attrition Rates of Soil-Cement Subjected to Water Jets.*", Transport Research Record No. 941, p. 18-22.

LITVAN, G.G. 1972. "*Phase Transitions of Adsorbates iv, Mechanism of Frost Action in Hardened Cement Paste.*", J. American Ceramic Society, 55, p. 38-42.

LITVAN G.G. 1975. "*Phase Transition of Adsorbants vi.Effects of Deicing Agents on the Freezing of Cement Paste.*", J. American Ceramic Society, 58, p. 26-30

LIVESEY, L. 1988. "*Lime Stabilization : Current Applications.*", Civil Engineering, Oct 1988, p. 52.

LO, Y.K.T. and LOVELL, C.W. 1983. *"Prediction of Soil Properties from Simple Indices"* Transportation Research Record 873, Highway Research Board, p. 43-49.

LOCKWOOD P.M. 1990. *Personal Communication*, Southampton University, Southampton, UK.

LOUGHNAN, F.C. 1969. "*Chemical Weathering of the Silicate Minerals.*" Elsevier Publishing Co. New York.

LOWELL, S. and SHIELDS, J.E. 1981. "*Contact Angle in Mercury Porosimetry.*", International Laboratory, 84, March, 1984.

LOWSON, R.T. 1982. "*Aqueous Oxidation of Pyrite by Molecular Oxygen.*", Chemical Reviews, 82(5), p. 461-497.

MAFF, 1992 "*Flood Defence.*", Environmental Matters, Ministry of Agriculture, Fisheries and Food, no.1, June 1992.

MASATOMO, I. 1989. "*Slaking of Coal Measure Rock: Effects of Water on Mechanical Properties of Coal Measure Rocks.*", J. Min. Nat. Processes Ins. Japan, 105, no.3 and no.4. p. 279-283, p. 432-438.

MASCH, F.D., ESPEY, W.H. and MOORE, W.L. 1963. "*Measurements of the Shear Resistance of Cohesive Sediments.*", Publication No. 970, Agricultural Research Service Washington D.C. p.15-155.

MATHER, M. B. 1979. "*Concrete Need not Deteriorate.*", Concrete International, 1(9), p. 32-37.

MATHER, M. B. 1982. "*Concrete in Sea Water.*", Concrete International, 4(3).

Mc NULTY, A.T. 1985. "*Durability of Cement Bound Minestone.*", PhD Thesis, Aston University, Birmingham, United Kingdom.

MICROMERITICS, 1988. "*Poresizer 9310 Instruction Manual.*" Micromeritics P/N : 931/42801/01.

MITCHELL, J.K. and EL JACK, S.A. 1965. "*The Fabric of Soil-Cement and its Formation.*", Proc.14th National Conference on Clays and Clay Mineralogy. p. 297-304.

MOH, Z. 1965. "*Reactions of Soil Minerals with Cement and Chemicals.*", Record No. 86, Highways Research Board, p. 39-61.

MOORE, W.L. and MASCH, F.D. 1962. "*Experiments on the Scour Resistence of Cohesive Sediments.*", J. Geophysical Research, 57, No. 4, p. 1437-1449.

MORIWAKI, Y. 1975. "*Causes of Slaking in Argillaceous Materials.*", PhD Thesis University of California, Berkley, 290 pp.

MORIWAKI, Y.and MITCHELL, J.K. 1977. "*The Role of Dispersion in the Role Slaking of Intact Clay.*", ASTM Special Technical Publication, No.623, p. 287-302.

MORTON, J.M., KETTLE, R.J. and THOMAS, M.D.A. 1984 ."*Some Experimental Observations on the Potential Expansion of Cement Bound Minestone.*" Proc. 2nd Int. Symp. on the Reclam.,Treat and Util. of Coal Mining Wastes. Sept, Durham, U K. , p. 56.1.

MOSS, A. J., GREEN, P., HUKA, J. 1981. "*Static Breakage of Granite Detritus by Ice and Water in Comparison with Breakage by Flowing Water.*", Sedimentology, 28, p. 261-272.

MUELLER, P.E. 1991. "*Mixing Procedures for Soil Cement.*", Concrete International, Jan. p. 53-55.

MUGRIDGE, S.J. and YOUNG, H.R. 1983. "*Disintegration of Shale by Cyclic Wetting and Drying and Frost Action.*", Can. J. Earth Sci., 20, p. 568-576.

MURAYAMA, S. 1966. "*Swelling of Mudstone due to Sucking of Water.*" Proc.1st Int. Congr. Rock Mech., Lisbon 1, p. 495-498.

NANKANO, R. 1967. "*On Weathering and Change of Properties of Tertiary Mudstone Related to Landslide.*", Soil and Foundations,7, p. 1-14.

NATIONAL COAL BOARD 1964. "*The Effect of Changes in Moisture Content of Shale on its Breakdown Characteristics.*", National Coal Board. Scientific Dept, Report SBWWP/P/64.

NATIONAL COAL BOARD 1971. "*Application of British Standard 1377:1967 to the Testing of Colliery Spoil.*", National Coal Board Technical Memorandum, Joint Working Party, Coal House, Harrow, Middlesex.

NATIONAL COAL BOARD 1983. "*Cement Bound Minestone: User's Guide for Pavement Construction.*", Minestone Executive, National Coal Board, Hobart House, London.

NEVILLE, A.M. 1981. "*Properties of Concrete.*", 3rd Edition, Pitman, London.

NEW CIVIL ENGINEER, 1991. "*News Analysis.*", New Civil Engineer, 18th Nov. 1991.

NEW SCIENTIST, 1993a. " *When the Tides Come in* " New Scientist 2nd Jan, 1993.

NEW SCIENTIST, 1993b. " *Can Load of Old Rubbish Save Essex from the Sea* .", New Scientist, 9th Jan1993

NICHOLS, T.C. 1980. "*Rebound, its Nature and Effects on Engineering Works.*", Quart. J. Eng. Geol., 13, p. 133-152.

NICOL A.W. 1975. (Editor). "*Physicochemical Methods of Mineral Analysis.*" , Plenum Press, New York.

NOBLE, D.F. 1967. "*Reactions and Strength Development in Portland Cement-Clay Mixtures.*", Record No.198,Highway Research Board,Washington DC, USA, p. 39-56.

NUSSBAUM, P.J. and COLLEY, B.E. 1971. "*Dam Construction and Facing with Soil-Cement.*", Portland Cement Association, Report No.RD010.01w.

OBIKA, B.,FREER-HEWISH, R.J. and FOOKES, P.G. 1989. "*Soluble Salt Damage to Thin Bituminous Road and Runway Surfaces.*", Quart. J. Eng. Geol., 22, p. 59-73.

OKAMOTO, R., SUGAHARA, H. and HIRANO, I. 1981. "*Slaking and Swelling Properties of Mudstone.*", Proc. Int. Symp. on Weak Rock, Tokyo, Sept. 1981, p. 283-295.

OLIVIER, H.J. 1979. "*Some Aspects of the Influence of Mineralogy and Moisture Redistribution on the Weathering Behaviour of Mudrocks.*" Proc. 4th Int. Congress Rock Mech., Montreaux, Switzerland, 3-1-18.

OLLIER, C.D. 1984 . "*Weathering.*", Longmann, London.

OSWELL, J.M. and JOSHI, R.C. 1986. "*Development of an Erosion Test for Soil-Cement.*", Geotechnical Testing Journal, GTJODJ, 9 (1), p. 19-23.

OWENS, P.L. 1989. "*Water and its Role in Concrete.*", Concrete International, 11(11) 1989.

PENNER, E., GILLOT, J.E. and EDEN, W.J. 1970. "*Investigation of Heave in Billings Shale by Mineralogical and Biochemical Methods.*", Can. Geotech. J., 7, p. 333-338.

PERRY, E.F. and ANDREWS, D.E.1983. "*Slaking Modes of Geologic Materials.*", Transport Research Record, 873, p. 22-28.

PETTIJOHN, F.J., POTTER, P.E. and SIEVER, R. 1972. "*Sand and Sandstone.*", Springer, New York.

PIHLAJAVAARA, R.E.1974. "*A Review of Some of the Main Results of a Research on the Aging Phenomena of Concrete: Effects of Moisture Conditions on Strength Shrinkage and Creep of Mature Concrete.*" Cement and Concrete Research, 4, (5), p. 761-771.

POOLE, A.B. 1991. "*Rock Quality in Coastal Engineering.*", Quart. J. Eng. Geol. 24, p. 83-90.

PORTLAND CEMENT ASSOCIATION 1971. "*Soil-Cement Laboratory Handbook.*", Engineering Bulletin No. EBO52S, P.C.A. Skokie, 62 pp.

PORTLAND CEMENT ASSOCIATION 1975a. "*Suggested Specifications for Soil-Cement Linings for Lakes, Reservoirs and Lagoons.*", Information Sheet No. IS186W, P.C.A., Skokie, 4 pp.

PORTLAND CEMENT ASSOCIATION 1975b. "*Thickness Design of Soil-Cement Pavements for Heavy Industrial Vehicles.*", Information Sheet No. IS187S, P.C.A., Skokie, 12 pp.

PORTLAND CEMENT ASSOCIATION 1976. "*Suggested Specification for Soil-Cement Slope Protection for Embankments (Central-Plant-Mixing Method).*", Information Sheet No. IS052, P.C.A., Skokie, 4 pp.

PORTLAND CEMENT ASSOCIATION 1984. "*Soil-Cement Slope Protection for Embankments: Planning and Design.*", Information Sheet No. IS173W, P.C.A., Skokie, 10 pp.

PORTLAND CEMENT ASSOCIATION 1986. "*Soil-Cement for Facing Slopes and Lining Channels, Reservoirs and Lagoons.*" Information Sheet No. IS126W, P.C.A., Skokie, 8 pp.

PORTLAND CEMENT ASSOCIATION 1988. "*Soil-Cement Slope Protection: Construction.*", Information Sheet No. IS167W, P.C.A., Skokie, 12 pp.

PRICE, N.J. 1960. "*The Compressive Strength of Coal Measure Rocks.*" Colliery Engineering, 37, p. 283-292.

PUGH, C.E.,HOSSNER, L.R. and DIXON J.B. 1984. "*Oxidation Rate of Iron Sulfides as Affected by Surface Area, Morphology, Oxygen Concentration and Autotrophic Bacteria.*", Soil Science, 137 (5), p. 309-314.

PYE, K. and MILLER, J.A. 1990. "*Chemical and Biochemical Weathering of Pyritic Mudrocks in a Shale Embankment.*", Quart. J. Eng. Geol., 23, p. 365-381.

QUIGLEY, R.M. and VOGAN, R.W. 1970. "*Black Shale Heaving at Ottowa, Canada.*", Can. Geotech. J., 7, p. 106-112.

QUIGLEY, R.M., ZAJIC, J.E., Mc KYES, E. and YONG R.N, 1973. "*Oxidation and Heave of Black Shales, Detailed Observations and Interpretations.*" Can. J. Earth Science, 10, p. 1005-1015.

RAINBOW, A.K.M. 1982. "*Colliery Spoil - its Production, Properties and use in a Cement Stabilized Form.*" Seminar on Waste Materials in Concrete, Cement and Concrete Research (TDH8610).

RAINBOW, A.K.M. 1983. "*An Investigation of Some Factors Influencing the Suitability of Minestone as the Fill in Reinforced Earth Structures.*", N.C.B., Hobart House, London.

RAINBOW R.K.M. 1984. "*Research into Reinforced Minestone in the United Kingdom.*" Proc. 1st Int. Conf. on the Reclam.,Treat. and Util. of Coal Mining Wastes, Sept, Durham , U.K.

RAINBOW, A.K.M. 1986. "*The Use of a Problematic Rock Soil Material.*" Civil Engineering, July,1986.

RAINBOW, A.K.M. and SLEEMAN, W. 1984. "*The Effect of Immersion in Water on the Strength of Cement-Bound Minestone.*", Proc. 1st Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Durham, U.K. , p. 55.1.

RAINBOW, A.K.M. and NUTTING M. 1986. "*Geotechnical Properties of British Minestone Considered for Landfill Projects.*",Symp. on Environ. Geotechnol., Allentown, Penn. USA.

RAINBOW, A.K.M. and SKARZYNSKA, K.M.1987. "*Minestone Impoundment Dams.*", Proc. 2nd Int. Conf. on the Reclam.,Treat. and Util. of Coal Mining Wastes, Sept, Nottingham, U.K., p. 219-236.

RANSOM, W.H. 1963. "*Soil Stabilization - A Review of Principles and Practice.*", Building Research Station, DSIR, Tropical Building Studies 5.

RAYBOULD, W.E.1966. "*Washery Water Clarification.*" J. Coal Preparation, 2, p. 9-13.

READ, H.H. 1976. "*Rutley's Elements of Mineralogy.*" Thomas Murby, London.

RICHARDSON, D.N. and LONG, J.D. 1987. "*The Sieved Slake Durability Test.*" Bull. Assoc. Engng. Geologists, 26(2), p. 247-258.

ROHAN, K. and LEFEBVRE, G. 1991. "*Hydrodynamic Aspects in the Rotating Cylinder Erosivity Test.*", Geotech. Testing Journal. GTJODJ,14(2), p. 166-170.

ROHAN, K., LEFEBVRE, G., DOUVILLE, S. and MILETTE, J.P. 1986. "*A New Technique to Evaluate Erosivity of Cohesive Material.*", Geotechnical Testing Journal, GTJODJ, 9, No. 2, p. 87-92.

ROHAN, K. and PILON, G. 1984. "*Influence de la Turbulence sur la Force Tractrice dans les Canaux Rectangulaires Lisses.*", Can. J. Civ. Eng., 11, p. 655-662.

ROWLAND, J. 1988. "*Greenwatch.*", Sunday Times, 4th December, 1988.

RUSSELL, D.J. 1982. "*Controls on Shale Durability : the Response of Two Ordovician Shales in the Slake Durability test.*", Can. Geotech. J.,19(1), p. 1-13.

SCHLICHTING, H. 1979. "*Boundary Layer Theory.*", McGraw Hill, New York.

SEGUIN, S. 1966. "*Instability of Iron Carbonate in Air.*", Am. J. Sci., 264, p. 562-568.

SENKAVI, A.L.,DIXON, J.B. and HOSSNER, L.R. 1986. "*Alteration Products of Siderite in East Texas Lignite Overburden.*", Soil Science 142(1), p. 36-42.

SERADA, P.J. and RAMACHANDRAN ,V.S. 1975. "*Predictability Gaps Between Science and Technology of Cements-2, Physical and Mechanical Behaviour of Hydrated Cements.*",J. Amer. Ceramic Society, 58(5-6), p. 249-53.

SEVERN TIDAL POWER GROUP 1986. "*Tidal Power from the Severn. Engineering and Economic Studies: Cardiff Weston Scheme.*", Thomas Telford, London.

SHAW, T.L. 1980. "*An Environmental Appraisal of Tidal Power Stations: With Particular Reference to the Severn Barrage.*", Pitman, London.

SHEN, C.K. and AKKY, M.R. 1974. "*Erodibility and Durability of Cement-Stabilized Loam Soils.*", Highway Research Record, No.501, p. 54-58.

SHERWOOD, P.T. 1958. "*The Effect of Sulphates on Cement Stabilized Clay.*", Bulletin No. 198, Highway Research Board,Washington DC, USA, p. 45-54.

SHERWOOD, P.T. 1961. "*Soil Stabilization by the use of Chemical Admixtures. A Review of the Present Position.*", Rds. and Rd. Constr. April, p. 102-110.

SHERWOOD, P.T. 1962. "*The Effect of Sulphates on Cement and Lime Stabilized Soils.*", Rds and Rd Construction, February, p. 34-40.

SHERWOOD, P.T. 1968. "*The Properties of Cement Stabilized Soils.*", Report LR205, Road Research Laboratory, Ministry of Transport.

SHERWOOD, P.T. and RYLEY, M.D. 1970. "*The effect of sulphates in colliery shale on its use for roadmaking.*",RRL Report LR324, Road Research Laboratory, Ministry of Transport.

SHULTZ, L.G.1964. "*Quantitative Intepretation of Mineralogical Composition from X-Ray and Chemical Data for the Pierre Shale.*", U.S. Geological Survey,Prof. paper, p. c1-c31.

SILVERMAN, M.P. 1967. "*Mechanism of Bacterial Pyrite Oxidation.*", J. Bacteriology, 94, p.1046-1051.

SKARZYNSKA, K.M. and POREBSKA, M. 1990. "*Influence of Sodium Chloride on the Freezing of Minestone.*", Proc. 3rd Int. Symp. on the Reclam., Treat and Util. of Coal Mining Waste, Sept., Glasgow, U.K., p. 431-436.

SKARZYNSKA, K.M.,RAINBOW, A.K.M. and KOZIELSKA-SROKA, E.1990. "*Swelling of Minestone in Relation to its Petrographic Composition.*", Proc. 3rd Int. Symp. on the Reclam, Treat and Util of Coal Mining Wastes, British Coal. Sept., Glasgow,U.K., p. 437-444.

SLEEMAN, W. 1984 . "*Practical Application of Cement Bound Minestone within the British Coal Mining Industry.*", Proc. 1st Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Durham, U.K., p. 55.1.

SLEEMAN, W.1987. "*The Economics of Minestone Utilization.*", Proc. 2nd Int. Conf. on the Reclam.,Treat. and Util. of Coal Mining Wastes, Sept, Nottingham, U.K. , p. 1-20.

SLEEMAN, W. 1990. "*Environmental Effects of the Utilization of Coal Mining Wastes.*", Proc. 3rd Int. Symp. of Reclam., Treat. and Util of Coal Mining Wastes, Sept., Glasgow, U.K., p. 65-76.

SPEARS, D.A.,TAYLOR, R.K., TILL, R. 1970. "*A Mineralogical Investigation of a Spoil Heap at Yorkshire Main colliery.*", Quart. J. Eng. Geol., 3, p. 239-52.

SPEARS, D.A. and TAYLOR, R.K. 1972. "*Influence of Weathering on the Composition and Engineering Properties of Insitu Coal Measure Rocks.*" Int. J. Rock Mech. Min. Sci., 9 , p. 129-56.

STEINOUR, H.H. 1960. "*Concrete Mix Water-How Impure Can It Be?*"., Journal, Portland Cement Association Research and Development Laboratories, 2(3), Sept, p. 32-50.

STOCKER, P.T. 1963. "*A Research Project on the Mechanisms of Cement -Stabilized Black Soils.*", Australia Road Research Board, March, p. 13-28.

SURENDRA, M.,LOVELL, C.W. and WOOD, L.E. 1981. "*Laboratory Studies of the Stabilization of Nondurable Shales.*",Transport Research Record, 790, p. 33-41.

TANDANANT, S. 1987. "*Time Dependant Behaviour of Coal Measure Rocks: Adsorption Rate and Strength Degradation.*",U.S. Bureau of Mines, Rep. Invest. 9131, 1987, pp. 18.

TANFIELD, D.A. 1971. "Construction Uses for Colliery Shale.", Contract J, January, p. 141-3.

TAYLOR, R.K. 1984. "*Composition and Engineering Properties of British Colliery Discard*" Mining Department, N.C.B., Hobart House, London.

TAYLOR, R.K. 1988. "*Coal Measure Mudrocks: Composition, Classification and Weathering Processes.*", Quart. J. Eng. Geol., 21, p. 85-99.

TAYLOR, R.K. and SMITH, T.J. 1986. "The *Engineering Geology of Clay Minerals : Swelling, Shrinking and Mudrock Breakdown*", Clay Minerals, 21, p. 235-260.

TAYLOR, R.K. and SPEARS, D.A.1970. "*Breakdown of British Coal Measure Rocks.*", Int. J. Rock Mech. Min. Sci.,7, p. 481-501.

TEMPLE, K.L. and COLMER, A.R. 1951. "*The Autotrophic Oxidation of Iron by a new Bacterium: Thiobacillus ferrooxidans.*", J. Bacteriol.,62, p. 606-611.

TEMPLE, K.L. and DELCHAMPS, E.W. 1953. "*Autotrophic Bacteria and the Formation of Acid in Bituminous Coal Mines.*", App. Microbiology, 1(5), p. 255-258.

TEMPLE, K.L. and KOEHLER, J. 1958. "*Drainage from Bituminous Coal Mines.*", West Virginia Univ. Agric. Exp Stn. Res Bull. 25.

TERZAGHI, K. and PECK, R.B. 1967. "*Soil in Engineering Practice.*" 2nd Edition, Wiley, New York.

THOMAS M.D.A, 1984. "*Study of Stabilized Minestone.*" Interim Report A, Aston University, Birmingham. (Unpublished)

THOMAS, M.D.A. 1987. "*The Performance of Cement Stabilized Minestone.*", PhD Thesis, Aston University, Birmingham, United Kingdom.

THOMAS,M.D.A, KETTLE, R.J. AND MORTON, J.A 1990. "*The role of the mixing moisture content in determining the strength and durability of CSM.*", Proc. 3rd Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Glasgow, U.K., p. 1-8.

TIMES, 1993. " *Minister Curbs the Gravel Grabbers.*", The Times, 4th January 1993

TOWNSEND, A.A. 1980. "*The Structure of Turbulent Shear Flow.*", University Press, Cambridge, Great Britain.

TREHAILE, A.S. and MERCAN, D.W. 1984. "*Frost Weathering and the Saturation of Coastal Rock.*", Earth Surface Processes and Landforms, 9, p. 321-331.

TURNBULL, D. 1984. "T*he Role of the Minestone Executive in British Mining and Civil Engineering.*", Proc. Int. Symp. Reclam, Treat and Util. of Coal Mining Wastes, Sept, Durham , U.K., p. 1.1.

USBR 1986. "*Design Standards No. 13-Embankment Dams.*", Chapter 17, Soil-Cement Slope Protection, (DRAFT), U.S.Bureau of Reclamation, Denver.

VARLEY, P.M. 1990. "*Susceptibility of Coal Measure Mudstone to Slurry During Tunnelling.*", Quart. J. Eng. Geol., 23, p. 147-160.

VAN EEKHOUT, E.M.1976. "*The Mechanisms of Strength Reduction due to Moisture in Coal Mine Shales.*" Int. J. Rock Mech. Min. Sci.& Geochem. Abstr.,13, p. 61-67.

VAN EEKHOUT, E.M. and PERG ,S.S.1975. "*The Effect of Humidity Compliance of Coalmine Shales.*" Int. J. Rock Mech. Min. Sci. Geochem. Abstr.12, 3D, 340.

VAN WIJK, A.J. 1985. "*Rigid Pavement Pumping: Sub-base Erosion: Economic Modelling.*", Informational Report, Joint Highway Research Project, JHRP-85-10,Purdue University Indiana.

VEAR, A. and CURTIS, C.D. 1981. "*A Quantitative Evaluation of Pyrite Weathering.*", Earth Surface Processes and Landforms, 6, p.191-198.

VENTER, J.P.1981. "T*he Behaviour of Some South African Mudrocks due to Temperature and Humidity Changes with Particular Reference to Moisture Content and Volume Change.*" Proc. Int. Symp. on Weak Rock,Tokyo, p. 205-212.

VERBECK, G. 1958. "*Carbonation of Hydrated Portland Cement.*", Cement and Concrete, STP-205, ASTM, p.17-37.

WALKER, S.and BLOEM, D.L. 1960. "*The Effects of Curing and Moisture Distribution on Measured Strength in Concrete.*" Proc. Highways Research Board, 36, p.334-46.

WASHA, G.W. and WENT, K.F. 1975. "*Fifty Year Properties of Concrete.*", J. Amer. Conc. Inst., 72(1), p. 20-28.

WASHBURN, E.W.1921. "*Note on a Method of Determining the Distribution of Pore Sizes in a Porous Material.*", Proc. Nat. Acad. Sci., U.S., 7, p.115-116.

WEI, C. and De FAZIO, F. 1982. "*Simulation of Free Jet Trajectories on the Design of Aeration Devices on Hydraulic Structures.*", Proc. 4th Int. Conf. on Finite Elements in Water Resources, Hanover, Ashurst-Southampton, U.K.

WEST, G. 1959. "*A Laboratory Investigation into the Effect of Lapse Time after Mixing on the Compaction and Strength of Soil-Cement.*", Geotechnique, 9, No.1, p. 22-28.

WILLIAMS, P.J. 1982. "*Surface of the Earth. Introduction to Geotechnical Science.*", Longman, London.

WILLIAMS, R.I.T. 1986. "*Cement-Treated Pavements.*", Elsevier, London.

WILSON, M. J. 1987. "*A Handbook of Determinative Methods in Clay Mineralogy.*", Blackie, New York.

WITHERS, N.J. 1992. Personal Communication.

YOUN, K.M. and KI, K.I.1990. "*A Study on Utilization of Coal Mining Wastes in Korea.*", Proc. 3rd Int. Symp. on the Reclam., Treat. and Util. of Coal Mining Wastes, Sept, Glasgow, U.K., p. 369-376.

APPENDIX A1 CATION EXCHANGE CAPACITY DETERMINATION

Minestone Sample

A representative sample was air dried at room temperature (25^0C) and graded into the required sub-samples.

Sample Saturation

A sub sample of 2g was accurately weighed out and placed into a 250ml conical flask, to which 100ml of 0.2 molar barium nitrate solution was added. The resultant suspension was continuously agitated for 2 hours.

Sample Cleaning

The suspension was filtered through a double millipore filter (0.45µm) then flushed with distilled water until no more sulphuric acid precipitates out barium sulphate

Cation Leaching and Stripping

The filter was transferred together with all the minestone sample to a 250ml beaker and washed with exactly 100ml of 0.1molar nitric acid. The filter and suspension was then agitated for 15 minutes to ensure complete leaching.The procedure is repeated using a clean filter paper as a control.

The resultant solution was then analysed using atomic absorption spectrophotometry for barium.

APPENDIX A2 CLAY ANALYSIS

A detailed account of the theory and methodology related the X-ray diffraction analysis of clays can be found in several previous works such as Nicol (1975) and Wilson (1987), however a basic outline of the method used during the present research would be appropriate.

Sample Preparation

A 5g sample was placed in a 1 litre measuring cylinder and vigorously shaken to fully disperse the material. The colloidal suspension was left to stand for a set period time (calculated using Stokes Law) allowing the fraction greater than 2µm to settle. A clay suspension sample was then taken using a pipette.

Sample Mounting

Using a vacuum pump the suspension was drawn rapidly through an unglazed ceramic tile resulting in the deposition of an even non-segregated clay film on the upper surface of the tile. A ceramic tile was used to aid the later procedures of heat treatment.

Sample Analysis

As with Thomas (1987), the semi-quantitative method described by Griffin (1971) was chosen for clay analysis. This is a simplified method suitable for routine analysis, but it does rely upon three basic assumptions :

1. The sample is considered to be composed of 100 per cent clay minerals.
2. The refractive property of the clay minerals is constant.
3. The relationship between the ratio of 3.58Å kaolinite peak to the 3.54Å peak is a 1.1 linear relationship.

To determine the specific clay mineral content of each sample, three treatments are performed on the specimens.

1 The sample is untreated prior to analysis i.e. air dried.
2. Ethylene glycol treatment. The samples are placed in a sealed desiccator. with a reservoir of ethylene glycol for 24 hours.
3. Heating to 180°C for 1 hour.

A diffractogram was obtained for the untreated sample and immediately after the other two treatments.

Mineral Analysis Calculations

By measuring the peaks displayed upon the three diffractograms the relative proportions of the four, salient clay groups may be assessed in the following manner :

1. On the heat treated diffractogram, measure the heights of the 7 and 10Å peaks. Calculate the total percent kaolinite and/or chlorite as follows :

$$\%(K{+}C) \quad = \quad \frac{\dfrac{h_{7A(180^{\circ})}}{2.5}}{h_{7A(180^{\circ})} + h_{10A(180^{\circ})}} \quad x \quad 100\%$$

2. On the heat treated pattern measure the heights of the 3.54 and 3.59Å peaks. These heights are used to apportion the 7Å peak into percent kaolinite and percent chlorite as follows :

$$\%K \quad = \quad \frac{h_{3.59\text{Å}}\,(180^{\circ})}{h_{3.59\text{Å}}\,(180^{\circ}) \;+\; h_{3.54\text{Å}}\,(180^{\circ})} \quad x \quad \%(K + C)$$

$$\%C \quad = \quad \%(K + C\,) \;-\; \%K$$

3. Calculate the total percent illite and/or mixed layer as follows, using peak heights from the heat treated pattern :

$$\%(I + M) \quad = \frac{h_{10\text{Å}}\,(180^{\circ})}{\dfrac{h_{7\text{Å}}\,(180^{\circ})}{2.5} \;+\; h_{10\text{Å}}\,(180^{\circ})} \quad x \quad 100$$

4. Measure the 7Å and 10Å peak heights on the ethylene glycol pattern. Use these measurements and the previous ones on the heat treated pattern to calculate percentages of illite and mixed layer clays as follows:

$$\%I = \frac{h_{10\text{Å}\,(EG)}\left[\frac{h_{7\text{Å}(180^{\circ})}}{h_{7A\,(EG)}}\right]}{h_{10\text{Å}\,(180^{\circ})}} \times \%(I + M)$$

$$\%M = \%\,(I + M) - \%I$$

Where %K, %C, %I, %M = Percentage of kaolinite, chlorite, illite, and montmorillonite respectively.

^{h}Å (180^{o}), ^{h}Å (EG) = The height of the diffraction peaks at x Å following heating at 180^{o}c and treatment with ethylene glycol respectively.

Montmorillonite is not found as a separate phase in British Coal Measure rocks. (Collins 1976).The % M fraction of the sample will in all probability represent an illite-montmorillonite mixed-layer clay.

APPENDIX A3 MINESTONE LITHOLOGY

Minestone	Sample Lithology (% weight of sample)				
	Mudstone	Sandstone	Siltstone	Pyrites/ Ironstone	Coal
Hatfield	100	---	---	---	---
L. Windsor	38	---	---	62	---
POAA	45	---	15	12	28
POAB	50	50	---	---	---
Sutton Manor	100	---	---	---	---
Wardley	---	---	---	---	---
Wearmouth	---	---	100	---	---
Westoe	---	---	---	---	---

Lithological composition of minestone particles greater than 50mm in size.

Minestone	Sample Lithology (% weight of sample)				
	Mudstone	Sandstone	Siltstone	Pyrites/ Ironstone	Coal
Hatfield	66	11	15	3	3
L. Windsor	54	18	---	20	8
POAA	67	6	1	22	4
POAB	27	9	49	10	5
Sutton Manor	69	6	17	---	8
Wardley	65	16	12	---	7
Wearmouth	47	16	34	1	2
Westoe	54	13	29	1	3

Lithological composition of minestone particles between 20mm and 50mm.

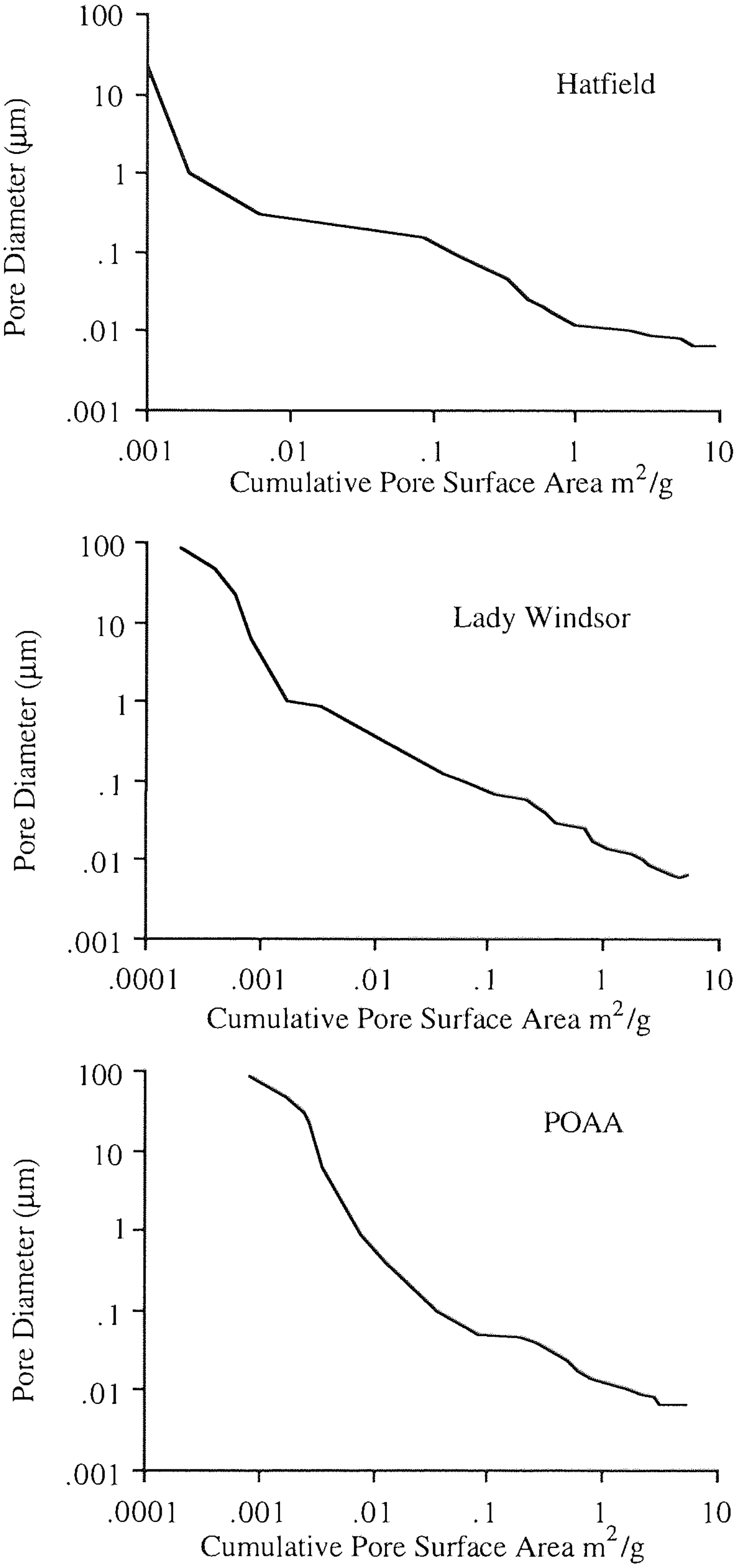
Minestone	Sample Lithology (% weight of sample)				
	Mudstone	Sandstone	Siltstone	Pyrites/ Ironstone	Coal
Hatfield	68	8	17	6	1
L. Windsor	80	3	2	5	10
POAA	62	12	13	4	9
POAB	49	1	39	4	7
Sutton Manor	74	4	7	2	13
Wardley	66	10	16	---	8
Wearmouth	46	5	35	4	9
Westoe	28	19	46	1	6

Lithological composition of minestone particles between 20mm and 10mm.

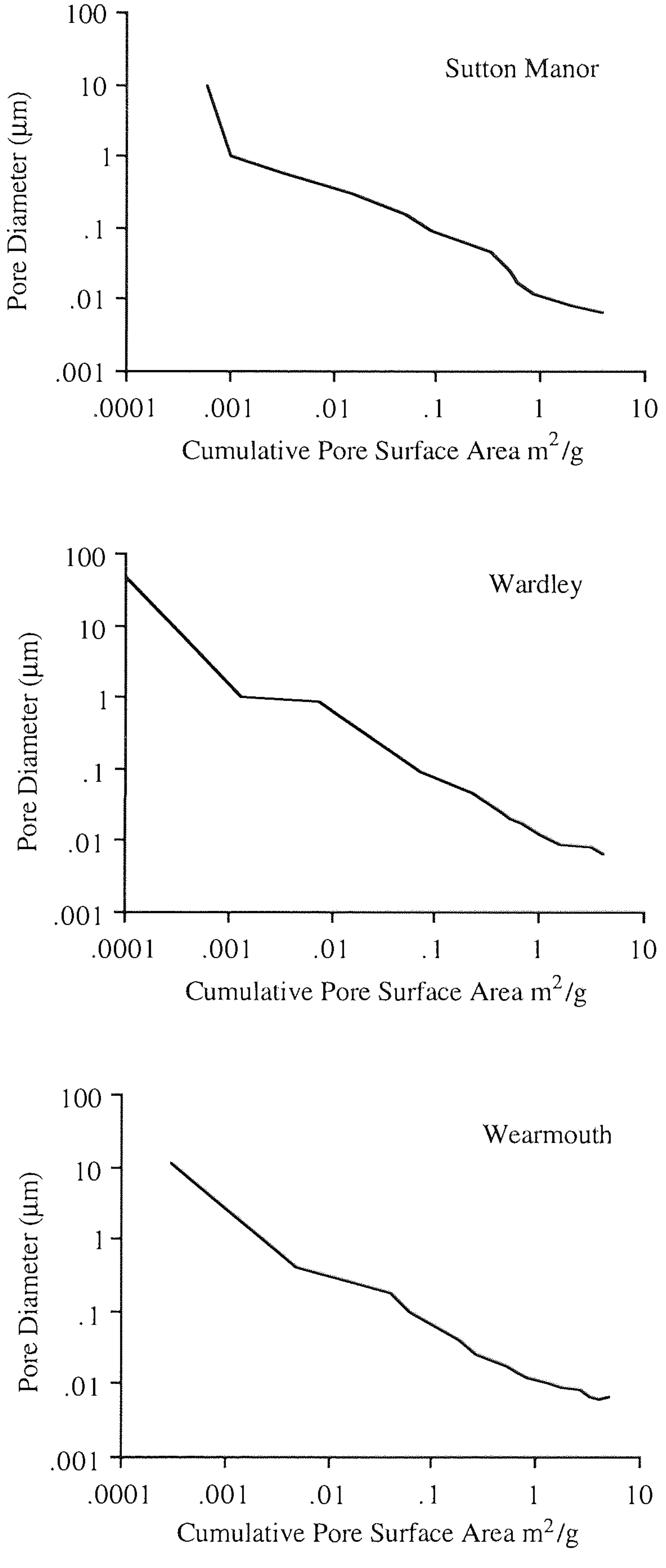
Minestone	Sample Lithology (% weight of sample)				
	Mudstone	Sandstone	Siltstone	Pyrites/ Ironstone	Coal
Hatfield	61	21	17	---	1
L. Windsor	89	---	4	---	7
POAA	73	11	10	3	7
POAB	49	1	39	4	3
Sutton Manor	74	4	7	2	13
Wardley	64	7	18	---	11
Wearmouth	69	8	6	3	13
Westoe	53	3	37	1	6

Lithological comparison of minestone particles between 10mm and 5mm.

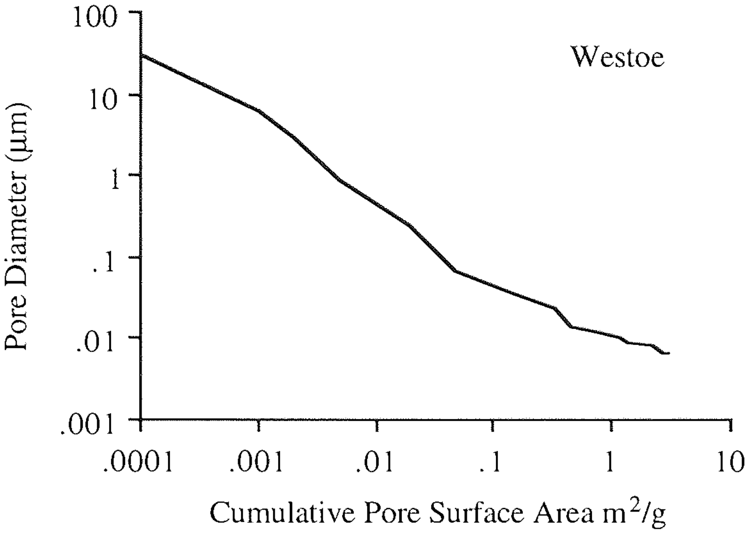
APPENDIX A4 Relationship Between Pore Diameter and Total Pore Surface Area



The relationship between pore diameter and cumulative pore surface area



The relationship between pore diameter and the cumulative pore surface area.



The relationship between pore diameter and cumulative pore surface area.

APPENDIX A5 LONG TERM COMPRESSIVE STRENGTH RESULTS

Minestone	Curing Period (Days) in Air Compressive Strength (MPa)									
	7	**14**	**21**	**28**	**50**	**100**	**200**	**300**	**400**	**500**
Hatfield	4.81	5.21	5.33	6.07	8.01	9.39	10.15	10.22	9.72	10.06
L. Windsor	4.45	5.76	6.14	6.35	7.56	7.88	8.20	9.04	8.30	9.78
POAA	4.04	4.50	5.53	6.79	7.46	7.44	7.98	10.06	9.56	11.05
POAB	6.71	7.52	8.95	9.57	10.57	10.93	10.39	11.30	12.42	12.46
S.Manor	5.93	9.00	12.58	13.47	13.63	13.52	14.29	14.94	14.88	15.44
Wardley	5.09	5.34	6.35	9.11	10.06	10.46	10.49	9.91	11.52	10.51
Wearmouth	4.99	5.81	7.73	9.96	10.58	12.64	12.63	12.97	12.45	12.71
Westoe	4.17	5.52	6.33	8.07	10.03	12.33	12.24	11.51	12.53	12.25

Long term strength development of CSM cured in air.

Minestone	Immersion Period (Days) in Distilled Water Compressive Strength (MPa)									
	7	**14**	**21**	**28**	**50**	**100**	**200**	**300**	**400**	**500**
Hatfield	4.81	3.69	3.50	3.19	3.36	3.94	6.13	6.05	7.53	10.00
L. Windsor	4.45	2.15	3.15	3.63	2.46	4.64	6.32	8.02	8.11	7.69
POAA	4.04	3.10	2.59	3.46	2.24	3.62	6.61	6.71	6.43	5.35
POAB	6.71	6.15	6.24	5.99	6.41	6.43	9.12	8.36	9.52	10.45
S.Manor	5.93	7.74	7.93	8.08	7.27	8.17	9.70	10.13	11.66	11.05
Wardley	5.09	4.50	4.01	3.51	3.15	4.74	6.09	6.74	6.46	7.16
Wearmouth	4.99	5.50	5.56	6.34	5.49	7.53	9.36	9.15	10.00	9.57
Westoe	4.17	4.30	5.01	4.79	6.93	8.01	8.43	8.09	9.37	7.60

Long term strength development of CSM cured in distilled water.

Minestone	**Immersion Period (Days) in Sea Water Compressive Strength (MPa)**									
	7	**14**	**21**	**28**	**50**	**100**	**200**	**300**	**400**	**500**
Hatfield	4.81	3.69	4.13	3.92	3.53	3.19	2.93	5.11	6.00	5.96
L. Windsor	4.45	2.36	2.65	2.44	2.02	3.74	4.90	6.45	6.60	5.65
POAA	4.04	3.03	3.54	2.64	2.83	3.28	3.42	3.63	4.61	3.17
POAB	6.71	6.19	5.91	5.64	5.83	5.21	7.16	6.58	5.46	6.36
S.Manor	5.93	8.28	8.30	9.01	8.69	10.81	10.80	9.63	7.46	7.77
Wardley	5.09	4.80	4.61	4.49	4.27	3.65	5.65	5.89	6.18	4.68
Wearmouth	4.99	5.90	6.32	5.51	7.22	5.46	5.63	5.53	5.25	5.01
Westoe	4.17	4.60	5.07	5.22	6.70	5.63	5.68	5.06	4.98	5.10

Long term strength development of CSM cured in sea water.

APPENDIX A6 DIMENSIONAL CHANGES FOLLOWING IMMERSION

Minestone	Immersion Period in Distilled Water (days) Volume Change (%)							
	1	**7**	**35**	**100**	**200**	**300**	**400**	**500**
Hatfield	0.33	0.66	0.73	1.20	1.61	1.76	1.80	1.89
L. Windsor	0.60	1.17	1.56	1.80	2.01	2.15	2.24	2.34
POAA	0.53	1.57	1.87	2.07	2.28	2.43	2.49	2.58
POAB	0.30	0.49	0.60	1.11	1.72	1.86	1.95	1.97
S. Manor	0.51	0.77	1.14	1.40	1.48	1.64	1.75	1.78
Wardley	0.66	1.14	1.61	1.89	1.92	1.97	2.03	2.05
Wearmouth	0.48	0.68	1.08	1.20	1.35	1.48	1.57	1.70
Westoe	0.56	0.98	1.24	1.49	1.66	1.85	1.92	1.98

Percentage volume change for CSM following immersion in distilled water.

Minestone	Immersion Period in Sea Water (Days) Volume Change (%)							
	1	**7**	**35**	**100**	**200**	**300**	**400**	**500**
Hatfield	0.60	1.08	1.81	2.11	2.55	3.02	3.41	3.64
L Windsor	0.54	1.09	2.19	2.68	3.26	3.57	3.61	3.66
POAA	0.50	1.51	2.41	3.38	3.77	4.24	4.47	4.57
POAB	0.12	0.21	0.82	2.27	2.41	2.51	2.66	2.81
S. Manor	0.49	0.72	1.49	1.82	2.01	2.22	2.39	2.54
Wardley	0.71	0.81	1.58	2.13	2.72	3.23	3.41	3.51
Wearmouth	0.42	0.60	1.12	1.39	1.83	2.11	2.41	2.55

Percentage volume changes for CSM following exposure in sea water.

Minestone	**Immersion Period in Distilled Water (days)** **Linear Change (μm)**							
	1	**7**	**35**	**100**	**200**	**300**	**400**	**500**
Hatfield	126	238	256	370	520	572	580	610
L. Windsor	192	398	506	604	652	692	734	770
POAA	174	516	620	686	774	818	842	862
POAB	98	140	180	438	504	554	608	622
S. Manor	170	282	366	508	550	568	606	628
Wardley	180	342	512	606	632	648	670	682
Wearmouth	174	226	380	410	468	494	542	584
Westoe	180	304	442	508	560	624	648	682

Linear dimension changes (μm) for CSM exposed to distilled water immersion.

Minestone	**Immersion Period in Sea Water (days)** **Linear Change (μm)**							
	1	**7**	**35**	**100**	**200**	**300**	**400**	**500**
Hatfield	184	354	558	678	808	962	1100	1188
L. Windsor	164	350	742	888	1032	1174	1190	1208
POAA	162	474	790	1076	1246	1420	1522	1572
POAB	50	92	286	648	864	904	952	1028
S.Manor	154	274	484	598	662	732	788	844
Wardley	194	236	504	726	922	1072	1140	1174
Wearmouth	146	194	292	456	610	708	808	868

Linear dimension changes (μm) for CSM exposed to sea water immersion.

APPENDIX A7 EROSION/ABRASION RESULTS ON SAMPLES FROM THE COAL RESEARCH ESTABLISHMENT

Minestone	Bulk density	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Francis	1.91	16.72	8.75	5.47	19.00	14.50
		7.80	4.08		10.00	
		12.54	6.57		14.50	
Hatfield	2.16	31.46	14.43	14.81	17.10	20.05
		37.46	18.24		18.00	
		27.34	19.31		26.60	
L.Windsor	2.02	18.32	9.07	7.16	23.00	15.20
		14.86	7.36		12.20	
		10.20	5.05		10.40	
P of Ayr	1.95	5.14	2.64	5.90	8.00	15.80
		12.72	5.52		16.00	
		18.58	9.53		22.40	
S.Manor	2.14	57.76	26.99	21.08	30.30	24.10
		25.96	12.13		17.50	
		51.60	24.11		24.50	
Tilmanstone	2.07	22.80	11.01	16.19	20.90	23.30
		37.76	18.24		22.10	
		39.98	19.31		26.80	

Results for the erosion/abrasion test upon CRE unstabilized samples at impact pressures of 140 kPa (20 psi).

Minestone	Bulk density	Eroded mass (g/h)	Erosion rate (cm^3/h)	Average erosion rate (cm^3/h)	Cavity depth (mm)	Average cavity depth (mm)
Francis	1.89	38.42	20.32	14.94	33.50	28.30
		23.86	12.62		25.40	
		22.46	11.88		26.10	
Hatfield	2.17	41.12	18.95	19.97	31.10	36.20
		43.86	20.21		37.80	
		45.02	20.75		39.90	
L.Windsor	1.98	57.38	28.98	23.00	40.90	33.10
		15.88	8.02		17.30	
		63.36	32.00		41.10	
P.of Ayr	1.96	32.70	16.68	14.60	30.50	25.70
		14.82	7.56		10.40	
		38.34	19.56		34.60	
S.Manor	2.14	55.12	26.76	35.13	31.80	42.30
		96.92	47.05		40.00	
		65.08	31.59		55.00	
Tilmanstone	2.22	53.08	23.89	26.64	40.40	40.30
		60.74	27.36		36.30	
		63.64	28.67		43.40	

Results for the erosion/abrasion test upon CRE unstabilized samples with impact pressures of 280 kPa (40 psi).