

GEOTECHNICAL PROPERTIES OF CARBONATE SOILS WITH
REFERENCE TO AN IMPROVED ENGINEERING CLASSIFICATION

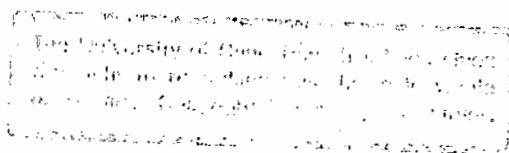
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B. STERIANOS
September 1988

SYNOPSIS

The engineering behaviour of carbonates differ substantially from quartz based soils. A review of the literature was undertaken in an attempt to identify relevant parameters which could provide the basis for an improved engineering classification. Carbonate content, cementation, crushability, particle size distribution and Atterberg limits were found to be relevant to engineering behaviour of carbonates, and should be included.

The maximum obtainable void ratio (e_{\max}), which is related to the particle shape distribution, has been proposed as an additional index for carbonate sands. Results from direct shear tests showed that this parameter correlates well with crushability and compressibility. It is recommended that this parameter (e_{\max}), as determined in the test devised by Kolbuszewski (1948), should be adopted as an index property for classifying carbonate sands.

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LIST OF SYMBOLS

ϕ	angle of internal shearing resistance (degrees)
ϕ_m	maximum angle of internal shearing resistance (degrees)
ϕ_u	angle of shearing resistance (direct shear) at maximum apparatus displacement (degrees)
ϕ_s	secant angle of internal shearing resistance (degrees)
ϕ_μ	angle of sliding friction (degrees)
δ	angle of skin friction (degrees)
σ_1	maximum principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
σ_n	applied normal stress in direct shear (kN/m^2)
σ_c	confining pressure (triaxial test)
$\bar{\sigma}_{vm}$	apparent preconsolidation pressure (oedometer)
σ'_{vo}	effective overburden pressure
ρ	density (kg/m^3)
γ	specific unit weight (kN/m^3)
ν	Poisson's ratio
ϵ_f	axial strain at failure (triaxial test)
τ	shear stress
μ	coefficient of sliding friction
A_f	pore pressure parameter at failure (triaxial)
C_k	crushing coefficient as measure of sample crushability
C_c	compression index
D_{60}	particle diameter at which 60 percent of soil by weight is finer
D_{10}	particle diameter at which 10 percent of soil by weight is finer
D_r	relative density (%)
E	Young's modulus

G_s	particle relative mass density (SG)
K_o	coefficient of earth pressure at rest
LL	liquid limit
PI	plasticity index
Q_s	pile shaft resistance
c	cohesion
\bar{c}	cohesion intercept w.r.t. effective stress
e	void ratio
e_u	unconsolidated void ratio (direct shear)
e_o	consolidated void ratio (direct shear, initial condition)
e_{max}	void ratio of soil in loosest condition
e_{min}	void ratio of soil in densest condition
f	value of limiting pile skin friction (p.7.8)
h_u	unconsolidated sample height (direct shear)
h_o	consolidated sample height (direct shear, initial condition)
Δh	consolidation settlement (direct shear)
Δh_s	maximum change in sample height during shear (direct shear) as measure of sample "compressibility"
m_s	mass of solids
n	porosity
s	standard deviation
\bar{x}	mean of measured values

1. INTRODUCTION

Soils composed of quartz grains have been shown to behave substantially differently from soils in which the particles are composed of calcium carbonate. Because the planned production platform near Mossel Bay will be founded on carbonate soils, an understanding of the behaviour of carbonates is of particular relevance at the present time. Knowledge of the anomalous behaviour of carbonate soils has been gained mainly as a result of construction experience of piled foundations, and to a lesser extent, shallow foundations, so that the discussion that follows is from this perspective.

The development of an engineering classification system for carbonates is important because :

- Material properties of carbonate soils can show a large variability within short distances.
- Carbonate soils are particularly prone to sampling disturbance.
- There is limited data for correlating results of conventional site investigation techniques (e.g. SPT, "Dutch cone", etc.) with standard methods of design.
- Cost and time constraints associated with offshore site investigations often restrict the number of in situ tests that can be performed.
- Due to the susceptibility of carbonate materials to sampling disturbance, laboratory samples are often unrepresentative of the in situ condition.
- The unique material properties of some carbonates can give rise to misleading results (e.g. consolidation) or to large variability in results (e.g. Atterberg Limits).

- The relation between the behaviour of the material in laboratory strength and index tests, and the likely field behaviour (e.g. low pile capacities) is still not clearly understood.

Chapters 1 to 8 contain a review of the literature on carbonate soils with particular reference to laboratory study of their properties, in situ testing and associated design considerations. Emphasis is placed on the following aspects :

- Identification of soil properties which cause anomalous engineering behaviour.
- The possibility of improving the existing classification systems through measurement of these properties (laboratory and in situ).
- Sources of error and variability in testing techniques as a result of the unique material characteristics of carbonates.
- The applicability of parameters such as relative density, that might provide a basis for improved classification.

A new classification parameter related to particle shape has been proposed. Direct shear tests have been undertaken (Chapters 9 to 11) with the objective of evaluating the relevance of this parameter to the engineering behaviour of carbonate soils.

2. BACKGROUND TO CARBONATE SOILS AND PROCESSES

Carbonate or calcareous soils can be generally regarded as those sediments which contain an appreciable amount (> 30%) of CaCO_3 . The various types of sediment can be further differentiated on the basis of their origin and mode of formation. Thus, a rough distinction can be made between carbonate deposits formed in the marine environment and those formed in arid or semi-arid regions :

- marine environment : formed by the settlement of calcium carbonate rich skeletons of marine organisms; direct inorganic precipitation occurs on a limited scale
- arid regions : formed mainly by the inorganic precipitation of CaCO_3 . The carbonates can derive either from weathering of the parent material, influx of carbonate rich eolian dust or from carbonates dissolved in rain water.

The anomalous engineering properties displayed by these sediments can be ascribed to factors related to their mode of formation and both physical and chemical compositions.

(a) Mode of deposition

Carbonate sediments are generally formed within the basin of deposition, so that carbonate contents can range from virtually 0 to 100 percent with the non-carbonate component usually consisting of quartz particles and clay minerals.

(b) Physical composition

The carbonate component contains organic and chemically precipitated components in varying proportions. Grains of biological origin, the major component of marine sediments, can display a wide variety in size, shape and geometry.

(c) Diagenesis

Carbonate sediments are susceptible to post-depositional diagenesis (Section 2.3) at ambient pressures and temperatures. Sediments can be cemented to various degrees to form deposits ranging from loose calcareous sands to firmly cemented limestone rock.

(d) Chemistry and mineralogy

The sediments contain CaCO_3 in the form of aragonite, high Mg-calcite and low Mg-calcite.

2.1 LOCATION AND DISTRIBUTION

It is estimated (Lee, 1982) that approximately 48 percent of the world's seabed is covered by calcareous ooze (containing more than 30% CaCO_3). Since calcareous materials are predominantly produced by living organisms, primary deposition can occur only in locations where water conditions favour calcium producing marine organisms. These conditions are determined mainly by salinity and temperature.

Present-day deposition occurs predominantly in waters that are warmer than 18°C throughout the year. As shown in Fig. 2.1, this zone generally lies between 30°N and 30°S latitude. This zone can however not be considered to be a precise determinant for the occurrence of carbonate soils, because :

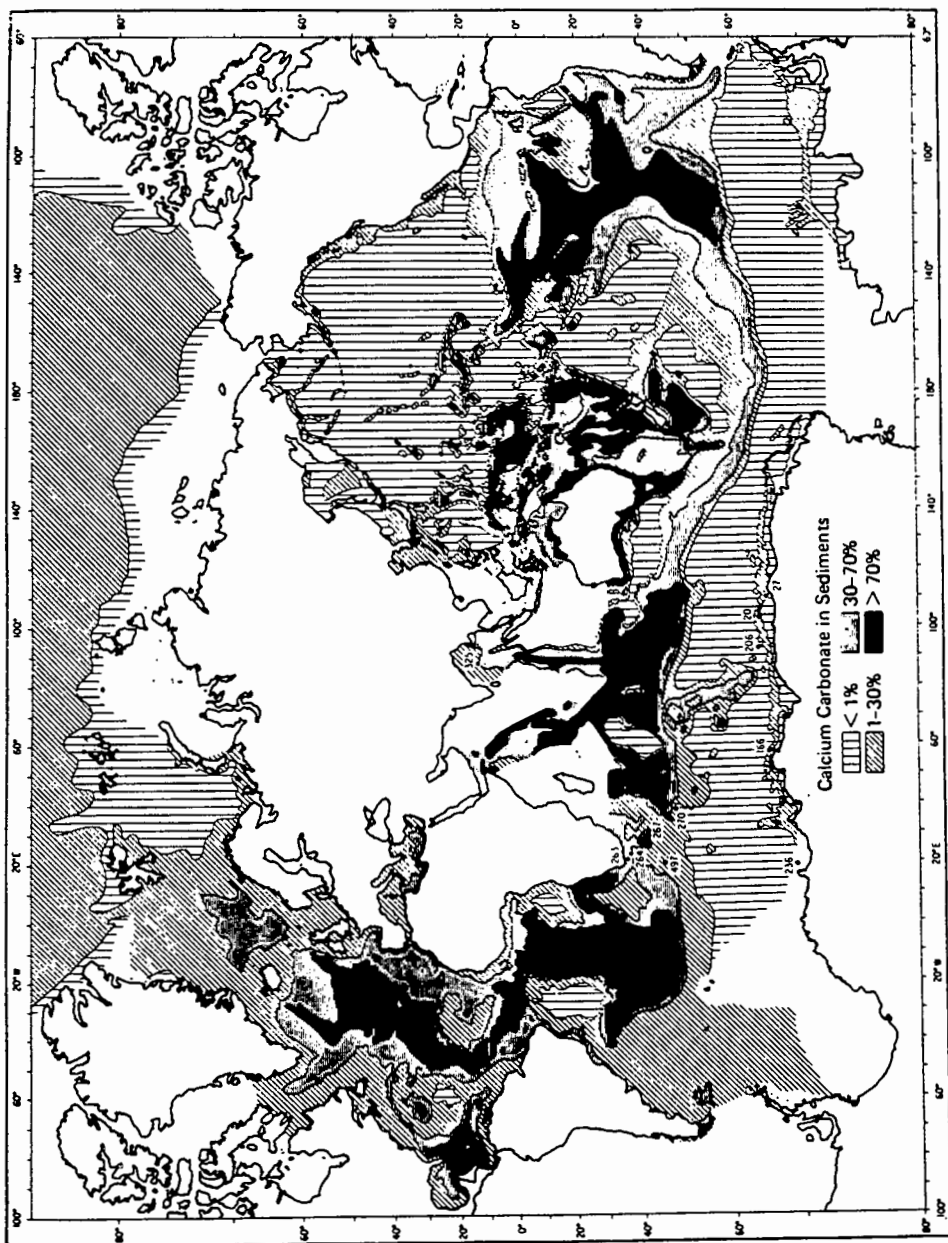


Figure 2.1 : Calcium carbonate distribution in surface sediments of the world ocean; percent of dry sediment (after Kennett, 1982)

- fluctuations in climatic and environmental conditions have occurred;
- warm water currents extend beyond the zone mentioned;
- precipitation of carbonate material is also controlled by factors such as water depth, pressure and solubility of carbonate materials.

Fig. 2.2 shows the distribution of surficial deposits offshore the Republic of South Africa, and the location of the proposed oil and gas FA platform, approximately 90km offshore Mossel Bay, in a water depth of 106m. Surficial deposits at the site are unusual in their high calcium carbonate content (7 to 90% by dry weight). It is believed that the warm Agulhas current creates the favourable environmental conditions for carbonate deposition.

2.2 ORIGIN AND MODE OF DEPOSITION

The biogenic fraction in carbonate sediments is composed primarily of the skeletons of marine organisms living in the upper waters of the ocean. These organisms utilize dissolved calcium carbonate to construct portions of their bodies, usually in the form of low-magnesium calcite or aragonite. Their remains settle to the seafloor, and if the rate of sedimentation is greater than the rate of dissolution, a bed of calcareous sediment forms.

The rate of sedimentation of calcite is greater than the rate of dissolution down to a level termed the calcite compensation depth (CCD). This generally occurs at water depths of between 3.5 and 5.0km in the world's oceans. At depths greater than the CCD calcite does not precipitate. Aragonite is more soluble than calcite, so that the aragonite compensation depth is less.

Marine carbonate sediments differ from terrigenous sediments in that the constituent materials are formed mostly at or near the site of deposition.

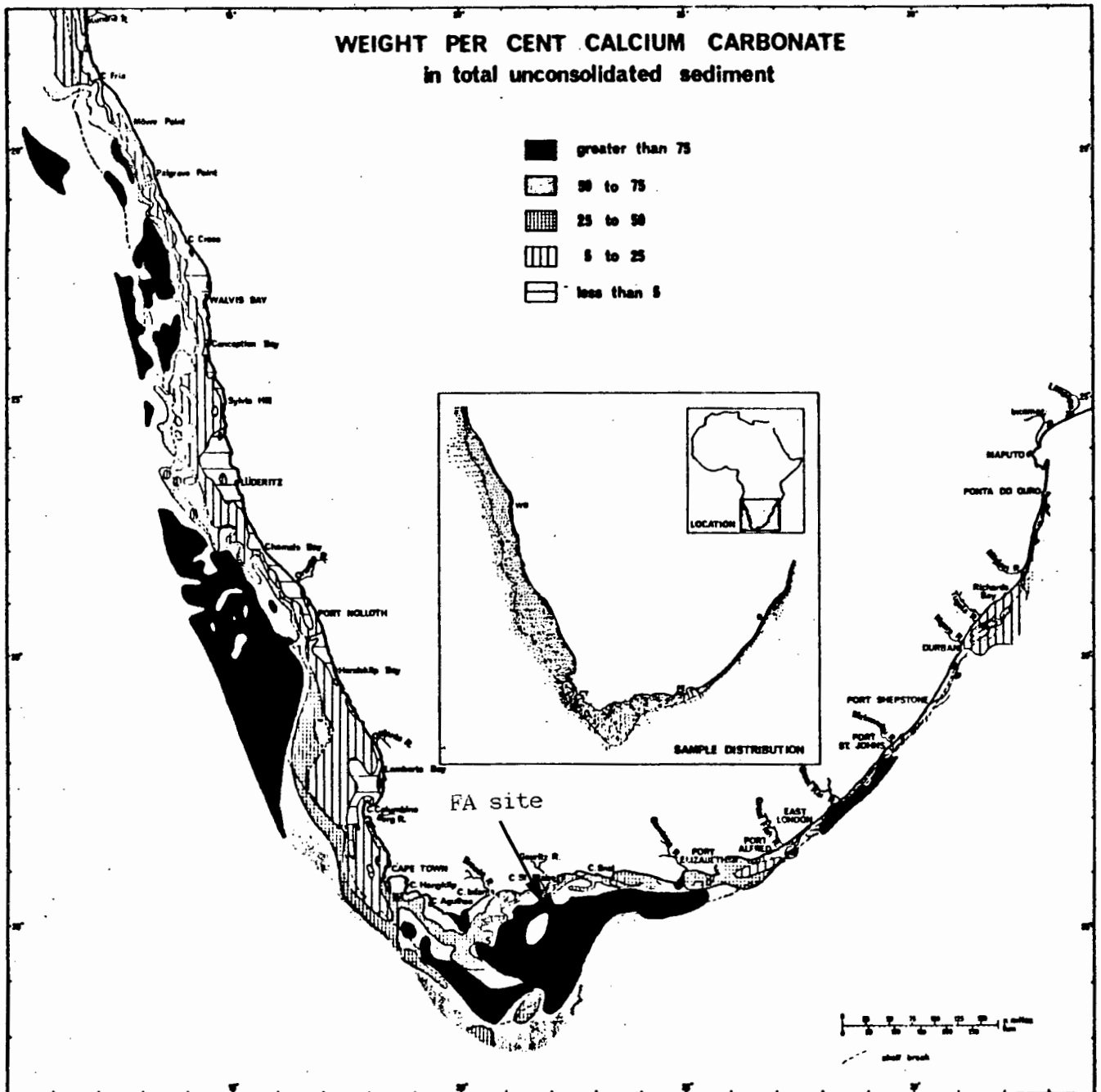


Figure 2.2: Distribution of surficial carbonate deposits offshore the Republic of South Africa.

The grain size and shape of the carbonate particles that reach the bottom are controlled by biological productivity and dissolution factors and not by distance from shore. Proximity to shore however determines the distribution and nature of sand and other terrigenous sediments that enter the marine environment. Calcium carbonate rich sediments consisting of two sets of particles are therefore found; one whose grain size and shape depend on biological productivity (carbonate component) and the other whose nature depends on distance from shore or large rivers (terrigenous component). Such bimodal material can exhibit significantly different engineering behaviour, depending on which of the basic constituents dominate.

2.3 DIAGENESIS

Diagenesis can be defined as "the changes which occur in the character and composition of sediments, beginning from the moment of deposition and lasting until the resulting materials (rocks) are either moved into the realm of metamorphism or become exposed to the effects of atmospheric weathering" (Larsen and Chillingar, 1979). The diagenetic processes of geotechnical significance are outlined below.

(a) Consolidation

As particles settle, the weight of accumulating sediment reduces the pore space and the contained water. When clay particles are subjected to consolidation loading they tend to cohere.

Carbonate sediments generally do not consolidate to as low a void ratio as non-carbonate sediments during deposition. Void ratios are often larger than would be expected for a given overburden pressure. This appears to be due to early cementation.

Evidence for the mechanism of the poor consolidation of carbonates is suggested by the deformation characteristics of some types of fossils in sediments. Graptolite fossils preserved in shale are commonly compressed and flattened, whereas in limestones they are generally not deformed (Dunbar and Rodgers, 1957).

(b) Cementation and dissolution

Cementation is the process by which crystals grow into pore spaces from particle surfaces. The pore spaces may be primary (result of depositional process) or secondary (produced by dissolution or fracturing).

Cementation of carbonate sediments can vary from a light bonding (which is easily broken) of the individual particles, to an almost total solidification. Fig. 2.3 illustrates a range of carbonate sediments cemented to various degrees. Discontinuous layers and lenses of cement are commonly encountered (Angemeer and McNiellan, 1982).

The cementing crystals are formed by precipitation of calcium carbonate from solution. The mineralogy of the carbonate cements depend on the composition of the fluid from which the crystal grows. In seawater it generally consists of aragonite and high-magnesium calcite, while in freshwater, only calcite crystals form.

The precipitation of CaCO_3 is controlled by the following reaction (Leeder, 1982) :

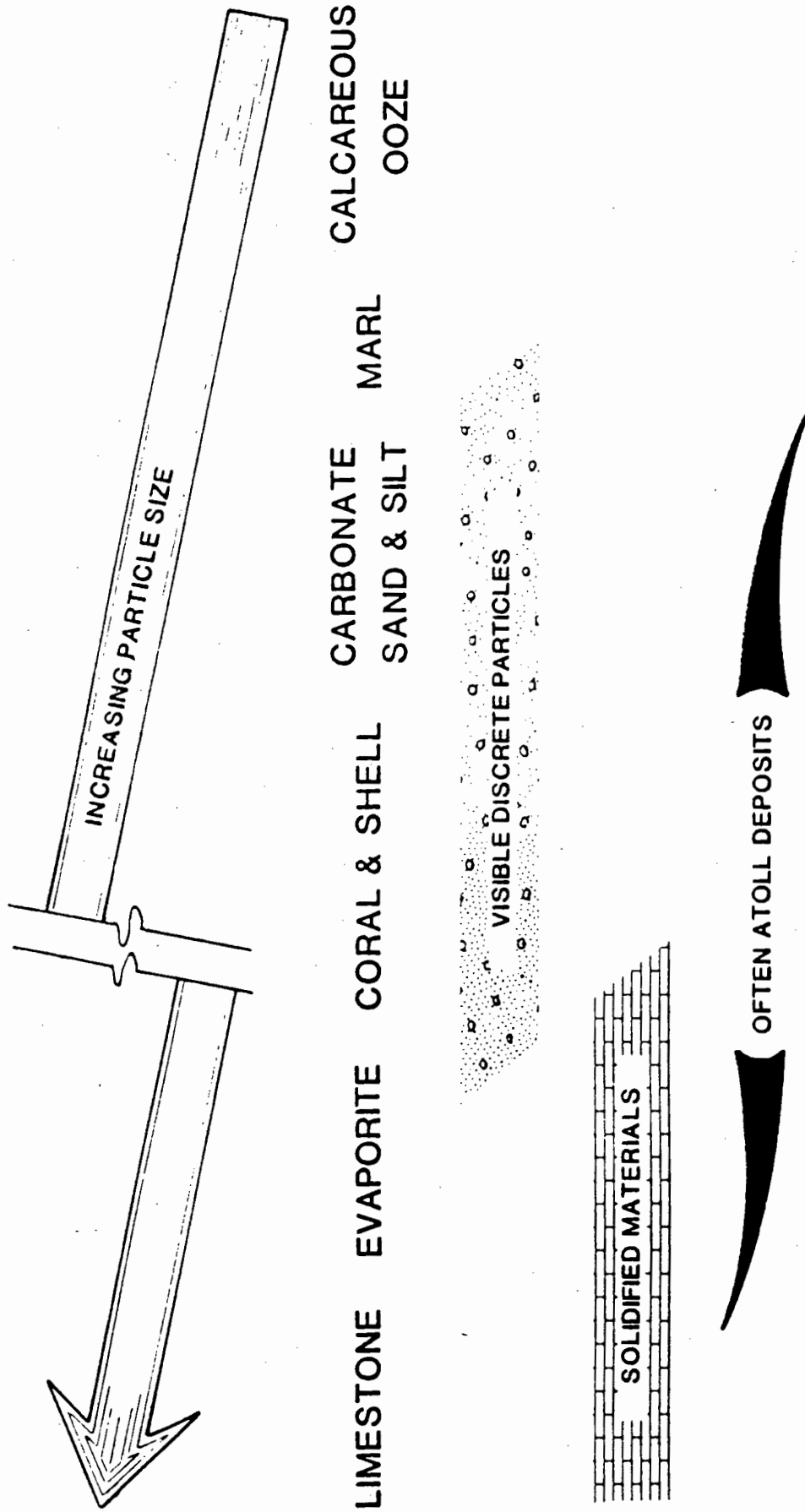
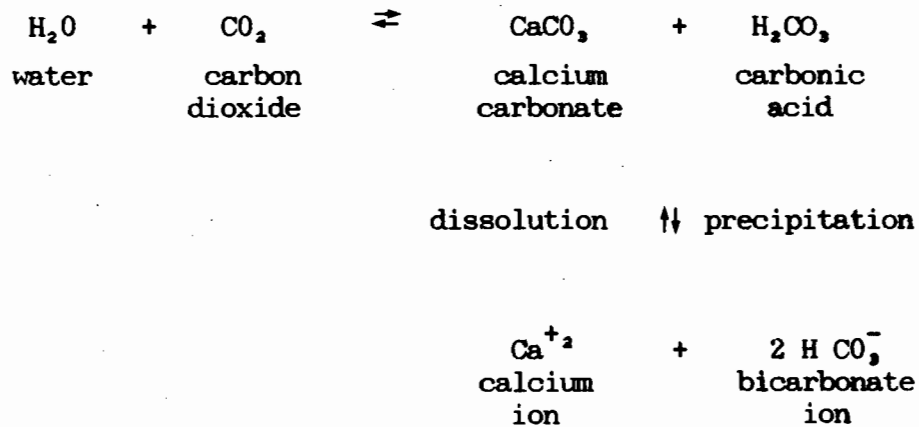


Figure 2.3 : Categories of carbonate materials (after Augemeer and McNiellan, 1982)



The precipitation of CaCO_3 is therefore favoured by processes which decrease the amount of CO_2 available to the solution, such as increase in temperature and photosynthesis. The reverse process of dissolution is favoured by factors increasing the CO_2 content, such as decay of organic matter. Dissolution of CaCO_3 increases the void ratio of the sediment and weakens it by destruction of cementation bonds.

(c) Recrystallization

After burial, less stable (thermodynamically) mineral particles tend to change to a more stable form. Shells originally made from more soluble aragonite may be changed to calcite or may simply be slowly dissolved. Dissolved material may then be reprecipitated on fragments of calcite to build out their crystal form, or deposited between pores, solidifying the sediment. Minute needles of aragonite are especially susceptible to such alteration.

2.4 PARTICLE TYPES IN CARBONATE SEDIMENTS

Most marine carbonate rocks and soft sediments consist of calcium carbonate particles graded from sand to fine clays. Since the nature of the particles have a significant influence on the engineering properties, they are briefly

discussed below. Particle properties and environments in which they occur are listed in Table 2.1 for some typical marine carbonate materials.

(i) Carbonate sand and silt

(a) Skeletal particles

Carbonate sand particles can be derived from the calcareous hard parts of invertebrates and calcareous algae. The type of grain and its mineralogy depend on the organisms present in the environment and the age of the sediment.








The ease of identification of the hard parts of the sediment depends upon the degree of physical and chemical breakdown. The smaller the particle the less precisely one can place it in a zoological classification. Mechanical breakdown of particles can result from several processes :

- Transport by waves and currents may break skeletons into smaller grains.
- Invertebrates such as crabs may crush shelled organisms.
- Organisms that consume sediment for organic content can cause internal disintegration.
- Boring sponges and algae may penetrate larger particles, weakening them for further disintegration by other burrowing organisms.

These mechanisms can result in grain size differences of up to a thousandfold in carbonate sediments.

Where sediment is lithified or has become rock, thin-section analysis techniques must be used for identification. In this situation many particles may be difficult to identify because under 'thin section' they

Table 2.1 : Marine carbonate materials (after Chaney et al., 1982)

Name	Shape	Smallest Sub-particle Shape	Depth Location m	ENVIRONMENT		Porosity	Overall Material Behaviour ^a	Chemical Constituent
				High-Energy Area	Low-Energy Area			
Foraminiferan shells (tests)			3500	benthic forams	planktonic forams globigerina	porous (porcellaneous)	cohesionless (sandy-silt)	calcite (Mg-calcite in small quantities)
Pteropod shells (tests)		no discrete subparticles	3000	tropical and subtropical waters	porous (porcellaneous)	cohesionless (sand)	aragonite
Coccolithic plants (nannofossils)			8500 to 5000	mainly latitudinal variation of species type	no inherent porosity (hyaline)	cohesionless (silt)	calcite shells of the algal family - skeletons may be protected by organic or MgCO ₃ coatings
Corals		no discrete subparticles	0 to 35	branching, massive and encrusting corals	foliose and encrusting corals	porous (porcellaneous)	cohesionless (sand-gravel)	Mg-calcite aragonite
Precipitate		no discrete subparticles	35 to 200	oolites	pelletoids	no inherent porosity (hyaline)	cohesionless (sand)	Mg-calcite
Benthic materials	highly variable		35 to 200	branching coraline algae, forams, planktonic feeders	mollusks, benthic forams, echinoid fragments	Mg-calcite

^a Qualitative description of material response (that is, sand, silty-sand, sand-gravel, silt) based on assumed particle size.

show many different shapes. In addition, diagenetic dissolution of minerals may degrade the original shell structure.

Some of the commonest carbonate secreting organisms contributing to the sediments are described in references such as Blatt *et al.* (1980) and Kennett (1982) :

Foramifera : These are mainly benthonic or planktonic forms, in which there is a considerable morphological variation. They are sub-divided into three groups based on the character of the skeleton or 'test', which may be of porcellaneous calcite, clear calcite or arenaceous material with a cement which is partly calcite.

Foram skeletons are hollow particles with spherical to globular shapes. The shell walls are often porous and sieve-like (Fig. 2.4). Typical diameters are in the order of 100 to 300 μm , with wall thicknesses of 4 to 10 μm . These particles are the primary constituent of sediments termed "calcareous ooze".

Calcareous planktonic algae : These single-celled algae secrete tiny calcitic discs, known as coccoliths, which are oval to circular in shape. The coccolith plates are typically 2 - 20 μm in diameter. Because the particles are so small, they are also called nanofossils.

Mollusca : These shells may be either external univalve, external bivalve or internal. The shells may comprise complex alterations of aragonite and calcite layers arranged in various geometrical ways with several characteristic microstructural types.

Echinodermata : Tests are composed of calcareous plates or 'spicules'. The skeleton differs from that found in all other invertebrates in that it is secreted by the middle rather than the outer body layer so that the test is enveloped in soft body tissue. Each plate is a single crystal of calcite.

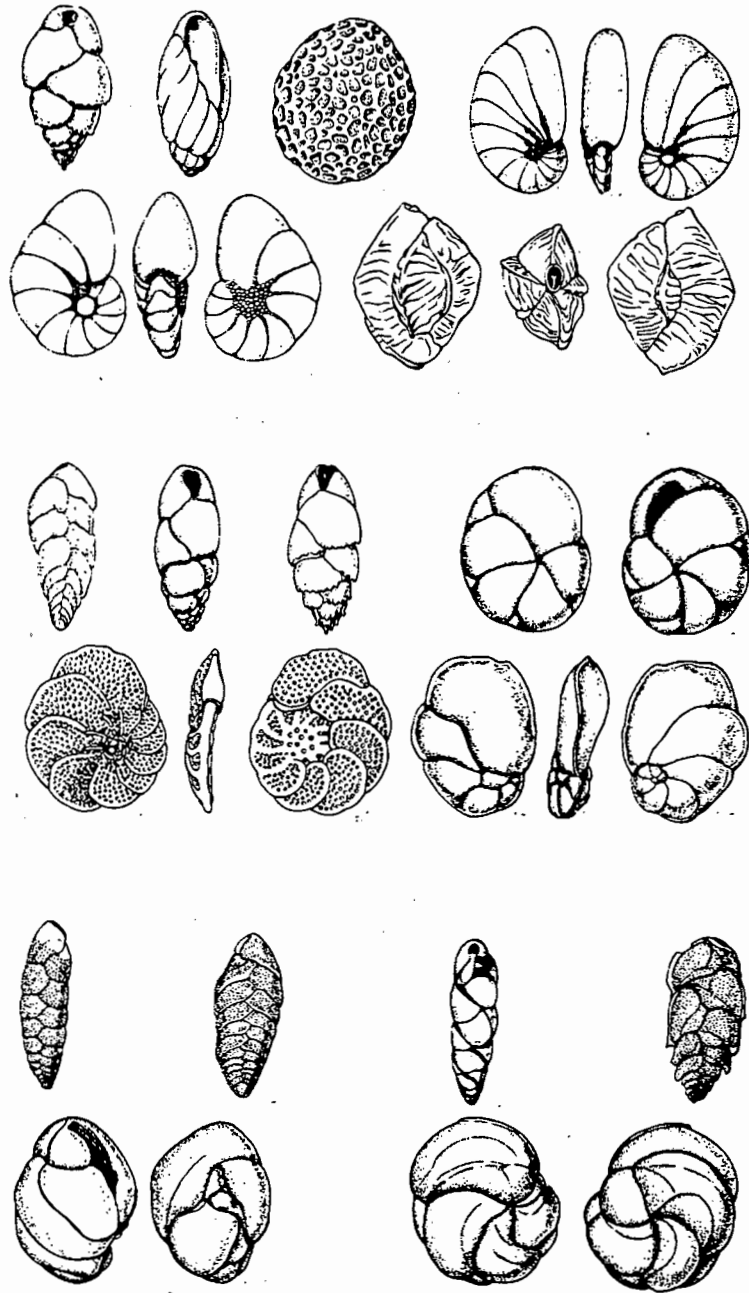


Figure 2.4 : Variable shapes of foraminifera particles
(after Kennett, 1982)

Pelletoid refers to similar appearing particles formed by the micritization (reduced to micro-crystalline form) of other particles such as skeletal grains or ooliths. In advanced stages of micritization, the original particle may be converted into a uniform homogeneous mass of microcrystalline carbonate and be indistinguishable from a faecal pellet.

(d) Intraclasts

These particles are fragments of partially lithified carbonate sediment that was eroded from the sea bottom or adjacent tidal flats. The prefix "intra" indicates that they formed as particles within the general area of deposition of the host sediment. They originate from sources such as :

- Fine carbonate sediments which develop mud cracks and curls when they are exposed to air by a drop in sea level.
- Carbonate sands on beaches within the intertidal zone which are cemented into beach rock by fibrous aragonite. Eroded fragments of such material produces intraclasts.

(ii) Carbonate muds

Micrite or microcrystalline carbonate is a common constituent of carbonate rocks. Individual crystals in ancient rocks are usually less than $5\mu\text{m}$ in diameter and usually have been converted to calcite. In modern carbonate sediments, most carbonate mud is composed of individual crystals averaging approximately $3\mu\text{m}$ in length and less than $0.5\mu\text{m}$ in width. These needlelike crystals are predominantly aragonite.

The origin of clay size carbonate mud is a controversial topic, as the abundance of aragonite crystals appears to be disproportionate to

origination from one source. Three general sources are thought possible (Leeder, 1982) :

- **Mechanical or biological abrasion of larger particles to yield fine material**
Large carbonate particles can be mechanically or biologically abraded to carbonate mud size fractions. (With the possible exception of the fibrous aragonite layers of some shells, mechanical or biological abrasion will not produce needle shaped aragonite crystals.)
- **Direct inorganic precipitation of aragonite from seawater**
There appears to be no evidence from normal or near-normal seawater for inorganic precipitation of individual aragonite needles on a scale compatible with the abundance of this type of particle as occurs in carbonate sediments.
- **Production of aragonite needles within the tissues of calcareous algae**
Organic tissue of some living calcareous algae contains abundant needles of aragonite that are similar to those found in bottom sediment. When the plant dies and the organic tissue decays, the needles are freed and added to the bottom sediment.

2.5 MINERALOGY AND CHEMISTRY

2.5.1 Mineralogy

The amount of carbonate material found in marine sediment can range from 0 to nearly 100 percent by dry weight. Most carbonates contain one major cation (Ca^{++}), two minor cations (Mg^{++} , Sr^{++}) and a number of trace cations (Fe^{++} , Mn^{++} , Ni^{++} , K^{++}). The major anion is CO_3^{-2} , while significant amounts of SO_4^{--} , PO_4^{--} , OH^- and Cl^- may also be present (Chaney et al., 1982).

Minerals formed by the major cation, minor cation (Mg^{++}) and major anion are:

Aragonite	$CaCO_3$
Calcite	$(Ca_{1-x}, Mg_x) CO_3$
Dolomite	$Ca Mg (CO_3)_2$
Magnesite	$MgCO_3$

Specific gravities and other properties of these crystals are listed in Appendix A.1.

As indicated in the previous section, the predominant minerals in modern marine carbonate sediments are aragonite and calcite containing a small percentage of magnesium. Both organic and inorganic calcite can have magnesium substituting for calcium, but the amount of substitution possible is much greater for organic calcites. Little substitution takes place in either organic or inorganic aragonite. A distinction can therefore be made between low-magnesium calcite (containing less than 4 percent $MgCO_3$), high-magnesium calcite, and aragonite. Calcite (high and low Mg) is thermodynamically more stable than aragonite, while high-magnesium calcite has a higher stability than aragonite, which in turn has a higher solubility than low-magnesium calcite (Brownlow, 1979). In most diagenetic environments, aragonite will therefore be eliminated with time.

Fig. 2.5 illustrates the distribution of mineral species according to phylum for the common carbonate contributing species.

Lowenstam (1964) paid particular attention to the temperature-dependence of the aragonite/calcite ratio. He found that temperature is one of the factors influencing skeletal mineralogy, but that there are numerous

exceptions. He divided organisms secreting calcareous shells into four types :

- (i) orders composed of forms that secrete skeletons entirely of aragonite, but in which the number of species is markedly greater in the warmer than in the colder water;
- (ii) classes or subclasses in which all species of a given order secrete either all calcite or all aragonite, but in which the orders with aragonite depositing species are confined to the warmer waters;
- (iii) a transitory type of (ii) which secretes trace amounts of calcite in the colder climates;
- (iv) genera with species in which skeletons are composed of mixtures of both aragonite and calcite, with the aragonite content increasing with higher temperatures and the calcite with lower temperatures, the temperature effect depending on the species.

In addition to temperature, the mineralogy of skeletons is influenced by salinity and environmental factors. The relationship between these variables is complex. Unfortunately, it appears that there is no simple relationship between mineralogy and organism type which can be linked to the likely engineering behaviour and properties of a sediment. The mineralogy of carbonate sediments could nevertheless have geotechnical significance because :

- it is affected by post-depositional diagenesis, in particular its susceptibility to dissolution as a result of sampling and changes in pore water conditions;

- it could serve as a site-specific indicator of variability in sample composition (relevant to SPT sampling where disturbed samples only are available), and aid in the dating of strata;
- the ratio of magnesium to calcium at a specific site could be related to geotechnical properties (see Section 2.5.3).

2.5.2 Identification of Minerals in a Sediment

The nature of the carbonate minerals in a sediment can be identified by x-ray analysis, differential thermal analysis, microprobe examination, staining, acetate peels and insoluble residues. Staining techniques are among the fastest, simplest and cheapest methods for getting reliable chemical data on carbonate phases. Since it does not require sophisticated and expensive equipment, it can be performed in any laboratory and is suitable for geotechnical purposes.

Staining as a diagnostic technique has developed mainly from the pioneering work of Friedman (1959), Dickson (1966) and others. For details of the exact applications, these references should be consulted. Fig. 2.6 outlines a staining scheme which can be used for the identification of aragonite, calcite, dolomite, (Mg) calcite and (Fe) calcite.

2.5.3 Relative Abundance of Calcium and Magnesium in Carbonate Sediments

After calcium, magnesium is the element most commonly found in combination with the carbonate ion. To assess the relative abundance of magnesium in carbonate sediments, it is useful to examine the chemistry of :

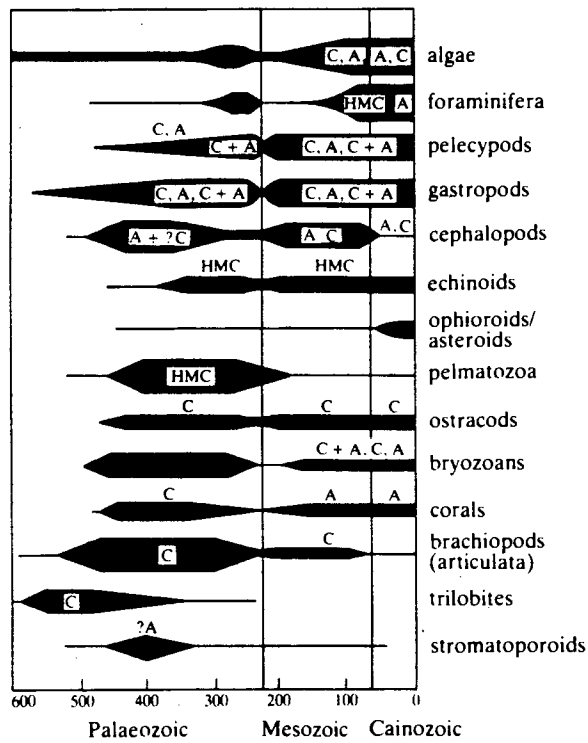


Figure 2.5 : Distribution and relative importance of calcareous animal and plant phyla through geological time. Width of bars represent the relative importance as sediment contributors. C - calcite; HMC - high Mg-calcite; A - aragonite (after Leeder, 1982)

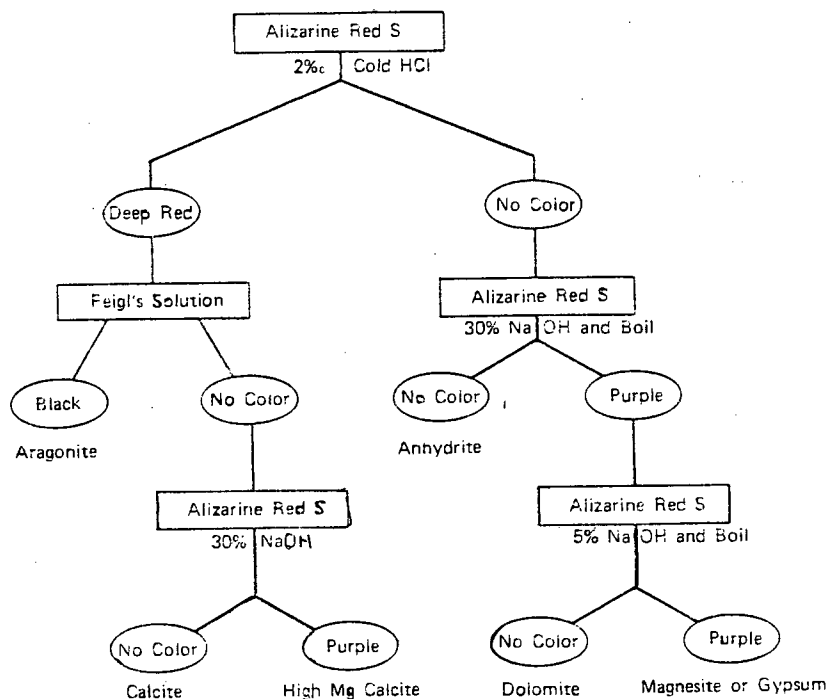


Figure 2.6: Recommended staining procedure (after Friedman, 1959).

- carbonate rocks which represent lithified ancient carbonate sediment; (because these sediments are subject to post-depositional diagenesis, the original chemical composition may have been altered);
- recent carbonate deposits.

Table 2.2 contains an analysis of the chemical composition of three typical limestones which illustrate the large variability in particle type and texture. These materials were formed from carbonate mud, cemented fossil fragments and oolites respectively. Despite these variations, the chemical composition of the three types are remarkably similar. Expressed as oxide, CaO and CO₂ are the only major components, while the Mg content is less than 3% in each case.

Chaney *et al.*, (1982) state that the maximum amount of MgCO₃ and FeCO₃ found in nature is 3% by dry weight. The validity of this assumption must be evaluated in the light of the following possible exceptions :

- The occurrence of magnesium in the mineral dolomite and the rock composed of this mineral.
- Some calcite-secreting organisms contain quantities of magnesium in their shells far in excess of the theoretical maximum of 4% for inorganic calcite. Table 2.3 lists the chemical and mineralogical composition of modern marine invertebrates, from which it can be seen that some organisms contain up to 29% MgCO₃. X-ray analysis has shown that the phase present is calcite, and not dolomite or magnesite (Chave, 1952). It is therefore theoretically possible to find modern sediments with MgCO₃ content in excess of 3% if these are predominantly composed of high-Mg calcite.

Table 2.2 : Chemical composition of typical limestones (after Brownlow, 1979)

	Oolitic	Lithographic	Fossiliferous
SiO ₂	0.75	1.15	0.29
TiO ₂	0.03	-	-
Al ₂ O ₃	0.25	0.45	0.26
Fe ₂ O ₃	0.64	-	0.11
FeO	-	0.26	-
MnO	-	-	0.01
MgO	2.14	0.56	0.70
CaO	52.30	53.80	55.53
Na ₂ O	0.01	0.07	0.07
K ₂ O	0.04	0.07	0.02
P ₂ O ₅	-	-	0.05
H ₂ O ⁺	0.50	0.69	-
H ₂ O ⁻	-	0.23	-
CO ₂	43.54	42.69	43.42
SO ₃	-	-	0.15
	100.20	99.90	100.61

Table 2.3 : Chemical and mineralogical composition of modern marine invertebrates (after Brownlow, 1979)

Type of organism	Most common mineralogy	CaCO ₃ (range, %)	MgCO ₃ (range, %)	SrCO ₃ (mean, %)
Foraminifera	Calcite	77-90	1-16	0.363
Calcareous sponges	Calcite	71-85	5-14	0.116
Madreporarian corals	Aragonite	98-99	0.1-0.8	1.33
Aleyonarian corals	Calcite	73-99	0.3-16	0.385
Echinoids	Calcite	78-92	4-16	0.334
Crinoids	Calcite	83-92	7-16	0.244
Asteroids	Calcite	84-91	9-16	0.225
Ophiuroids	Calcite	83-91	9-17	0.257
Bryozoa	Calcite, aragonite	63-97	0.2-11	0.383
Calcareous brachiopods	Calcite	89-99	0.5-9	0.190
Phosphatic brachiopods	Chitinophosphatic	?-8	2-7	-
Annelid worms	Calcite, aragonite	83-94	6-17	0.707
Pelecypods	Calcite, aragonite	98.6-99.8	0-3	0.258
Gastropods	Calcite, aragonite	96.6-99.9	0-2	0.234
Cephalopods	Aragonite	93.8-99.5	Trace-0.3	0.492
Crustaceans	Calcite, calcium phosphate	29-83	1-16	0.573
Calcareous algae	Calcite, aragonite	65-88	7-29	0.322

Although high magnesium contents are associated with specific types of organisms, for reasons given below, magnesium content cannot serve as an indicator of the type of organism present in the sediment :

- High-Mg calcite is the least stable carbonate mineral and will tend to change to low-Mg calcite. The magnesium content will therefore also depend on factors such as the diagenetic environment, the rate of deposition and the age of the sediment.
- The magnesium content of shells of a given invertebrate group is not constant, but depends on water temperature (typically higher in warmer temperatures) and salinity.

Parker (1972) determined the calcium and magnesium content of high carbonate content (> 85%) sediment cores from the Bahama platform. For these sediments, he found that the shear strength and void ratio were closely correlated to the carbonate content and variations in mineralogy, expressed as the magnesium/calcium ratio. It is therefore possible that Mg/Ca ratio could potentially be linked to engineering properties of sediments at specific sites.

3. CLASSIFICATION FOR CARBONATE SOILS

3.1 NEED FOR ENGINEERING CLASSIFICATIONS

From an engineering viewpoint, it is desirable that classification of soils should be based on index properties which discriminate on engineering behaviour. Index properties of non-carbonate soils are usually based upon field identification and relatively inexpensive laboratory tests. The index properties used in classification of such soils can be divided into two categories :

- soil grain properties : these deal with the individual grain's size, geometry, mineralogy and crystal structure;
- soil aggregate properties : dealing with the assemblage of particles and the microstructure.

Classification systems developed for non-carbonate soils, for example the Unified Classification System (UCS), are not applicable to carbonate soils since they do not take account of their unique properties. Similarly, terminology and classification systems developed by geologists are of limited use to the geotechnical engineer since they appear to give little indication of the engineering behaviour of the soils.

3.2 PROPOSED SYSTEMS

Fookes and Higgenbotham (1975) first proposed an engineering classification system for carbonate soils. This was later extended by Clark and Walker (1977) and further modified by Beringen *et al.* (1982). These classification

systems are shown in Figures 3.1 to 3.3. The main criteria for their classification are :

(a) Carbonate content

The system shown in Fig. 3.3 applies to soils containing 90 to 100 percent calcium carbonate. For lesser carbonate contents, Beringen *et al.* proposed that the prefix should be altered as follows :

<u>carbonate content %</u>	<u>prefix</u>
90 to 100	carbonate
50 to 90	siliceous carbonate
10 to 50	calcareous silica
0 to 10	silica

(b) Degree of cementation or induration

Induration is the process of hardening of sediments through cementation, pressure, heat or other causes. In general, the term cementation refers to sands or silts and induration applies for clays where interparticle cementation for carbonate soils cannot be distinguished from natural cohesion. The degree of cementation is quantified on the basis of unconfined compressive strength (Clark and Walker system) and cone resistance (Beringen system).

(c) Particle size

The words "clay", "silt", "sand" and "gravel" are indicative of particle size, as for non-carbonate soils.

(d) Geological origin of carbonate component

This parameter is subdivided into the most frequently occurring categories :

clastic - grains of inorganic origin, transported and deposited as grains

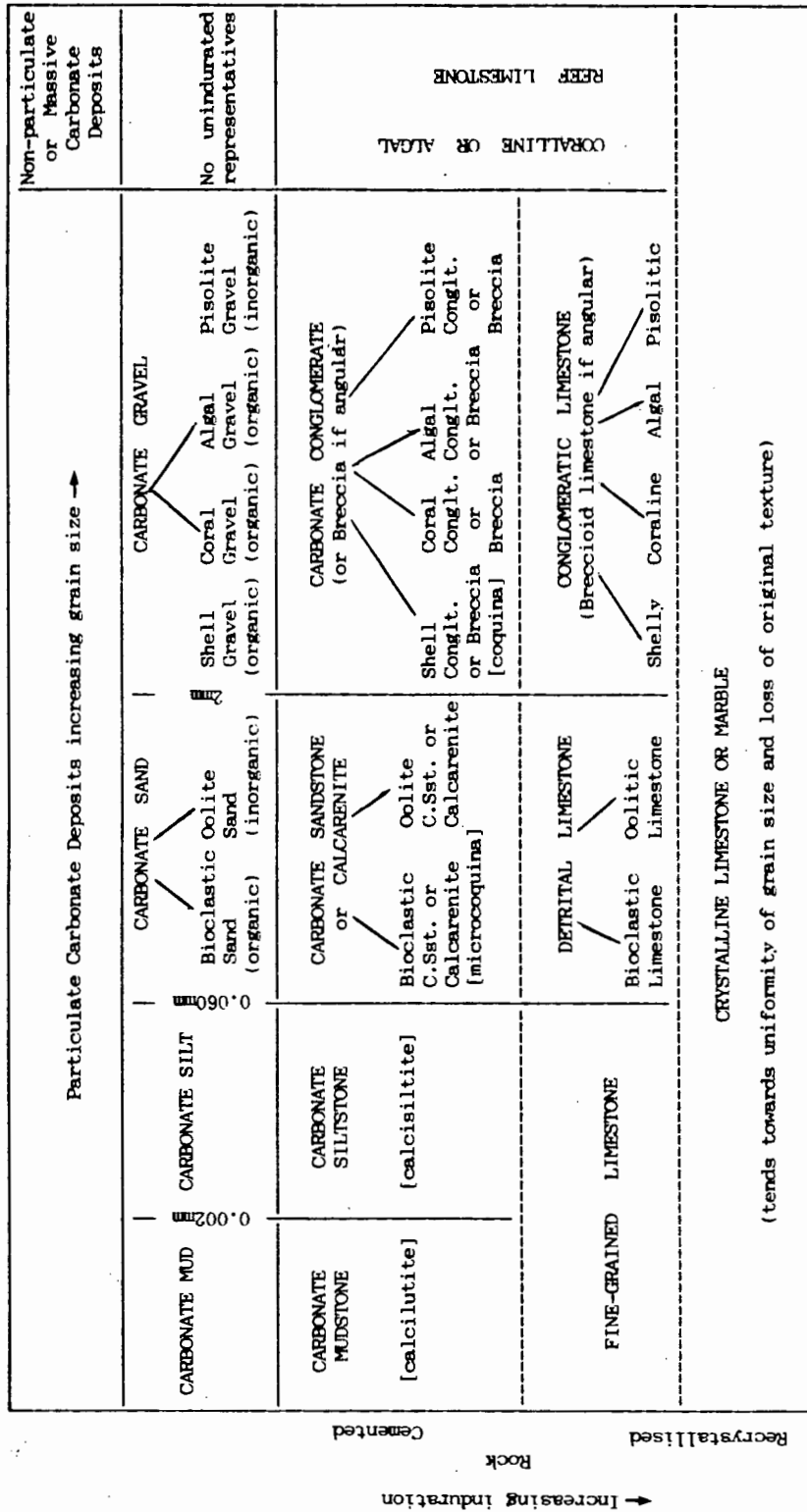


Figure 3.1 : Proposed classification of pure carbonate sediments for engineering purposes (Fookes and Higgenbottom, 1975)

CARBONATE CLASSIFICATION

Additional Descriptive Terms Based on Origin of Constituent Particles		Increasing Grain Size of Particle Deposits			
		0.002mm	0.074mm	4.76mm	76mm
Degree of Induration	Non-Indurated (Soils)	Carbonate Clay Calcareous Clay (3) Clay	Carbonate Silt (1) Calcareous Silt (1) Silt	Carbonate Sand (1) Calcareous Sand (1) Silica Sand	Mixed Carbonate and Non-carbonate Gravel (2) Gravel
	Moderately Indurated (Rock)	Limestone Argillaceous Limestone Calcareous Claystone Claystone	Limestone Fine-grained Limestone Calcareous Siltstone Siltstone	Limestone Arenaceous Limestone Calcareous Sandstone Sandstone	Limestone Conglomerate or Breccia Calcareous Conglomerate or Breccia Conglomerate or Breccia

Not Discernible Bioclastic Oolitic (Organic) Shell (Organic) (Inorganic) Coral (Organic) (Organic) Algal (Organic) (Organic) Pisolithic (Inorganic) (Inorganic)

Terms Related to Carbonate Classification

- Algal** - Composed of the remains of calcareous-secreting algae.
- Arenaceous** - Having a notable proportion of detrital quartz sand or silt.
- Argillaceous** - Having a notable proportion of clay.
- Authigenic** - Formed in place by chemical or biochemical action.
- Bioclastic** - Consisting of fragmental remains of organisms.
- Coral** - Calcareous skeleton of a coral or group of corals.
- Detrital** - Derived of pre-existing rock fragments.
- Oolitic** - Made up of ooliths (0.25 to 2mm, spherical particles, usually carbonate).
- Pisolithic** - Made up to pisoliths (2 to 10mm round particles, usually carbonate).
- Shell** - The generally hard rigid covering of an animal, commonly calcareous.
- Siliceous** - Containing abundant quartz or silica, generally cryptocrystalline.

Degree of Cementation

- (a) Well cemented - cannot be manually broken but grains can be dislodged.
- (b) Weakly (or lightly) cemented - can be manually broken down without difficulty; individual grains can be dislodged.
- (c) Moderately cemented - intermediate.

Notes :

- (1) Non carbonate constituents are likely to be siliceous apart from local concentration of minerals such as feldspar and mixed heavy minerals.
- (2) In description, the rough proportions of carbonate and non carbonate constituents should be quoted and details of both the particle minerals and matrix minerals should be included.
- (3) Calcareous is suggested as a general term to indicate the presence of unidentified carbonate. When mineral identification is possible, calcareous referring to calcite or aragonite, etc.

Figure 3.2 : Classification of carbonate sediments (Clarke and Walker, 1977)

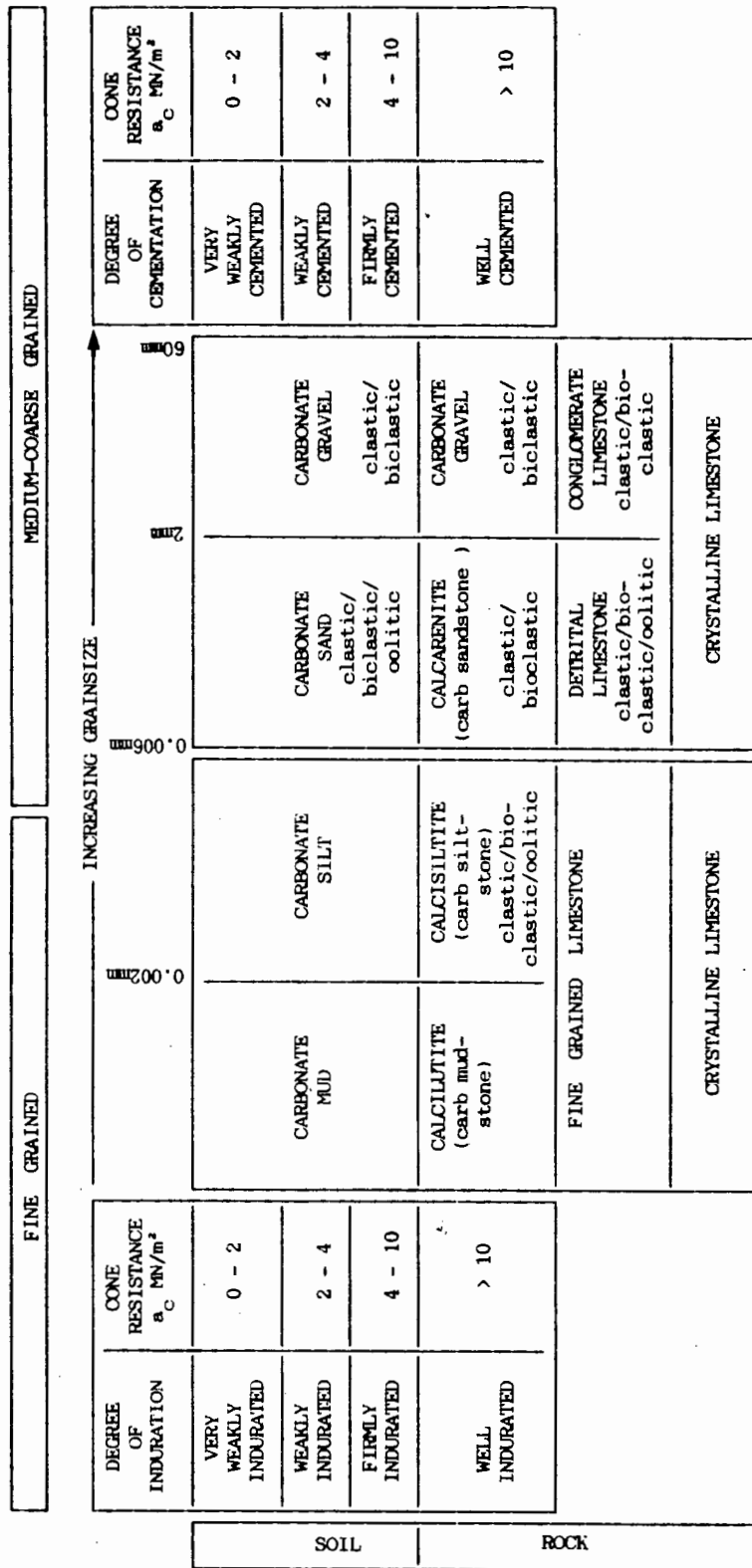


Figure 3.3 : Classification of carbonate sediments (90 - 100% carbonate) (Beringen, 1982)

- bioclastic - grains of organic origin, transported and deposited as grains, fossils, and fossil fragments
- oolitic - spherical grains coated with precipitated calcium carbonate
- pellitic - elliptical grains coated with precipitated calcium carbonate
- reefoidal - grains consisting of old reef fragments.

Other parameters of minor importance which are also described in the three classification systems are :

- bedding and lamination
- colour
- composition of non-carbonate material.

3.3 LIMITATIONS OF THE EXISTING SYSTEMS

The schemes proposed by Clark and Walker, Fookes and Higgenbotham and Beringen *et al.* provide for the entire spectrum of carbonate soils and rocks, but have been severely criticised by Datta *et al.* (1982) because in covering the "width", they seem to lack the "depth" necessary for serving as an indicator of engineering behaviour. In this respect, the following points have been raised :

- the utility of carbonate content as an index property is limited, since it is not only the amount of carbonate material present that controls the engineering behaviour, the nature of the carbonate material often plays a significant, if not dominant, role;
- the schemes fail to differentiate soils on the basis of crushing susceptibility. Two sands can both be classified as "bioclastic carbonate sands" and yet have significantly different engineering

behaviours on account of their different susceptibilities to particle crushing;

- terms such as "clayey carbonate silt/mud" and "calcareous silt/clay" fail to indicate that the former soil is uncemented and behaves like a normally consolidated soil whereas the latter is cemented and exhibits apparent overconsolidation;
- in these classification systems the proposers have neither specifically stated the relevance of nor attempted to incorporate the usual classification parameters like gradation and plasticity (e.g. Atterberg Limits as in UCS).

The development of an effective engineering classification system is complicated by the following factors :

- there is as yet no method for readily identifying and quantitatively expressing the degree or uniformity of cementation;
- there appears to be no simple method for quantitatively assessing the susceptibility to crushing of carbonate particles;
- visual or microscopic studies provide only a qualitative indication of crushability;
- at this stage, a clear understanding of how different non-carbonate components of soils are influenced by the presence of carbonate material is lacking.

Datta *et al.* (1982) believe that in view of these restrictions, it is at present only possible to formulate a system of description for carbonate soils. They have suggested that a system of description should include the following :

- cementation
- grain size distribution and plasticity
- nature of carbonate component
- nature of non-carbonate component.

Table 3.1 gives detailed observations that can purposefully be recorded for each of the above aspects, together with some relevant procedures and information.

3.4 IDENTIFICATION AND MEASUREMENT OF RELEVANT INDEX PROPERTIES

In the following section, the effect of soil "aggregate" properties, particularly in respect of its influence on shear strength behaviour, consolidation and settlement and piling behaviour will be examined. For each of these "aggregate" properties an attempt will be made to identify those soil grain properties which have an influence, and which should thus be incorporated in an engineering classification/description scheme for carbonate soils. Methods for measuring and quantifying these properties will be investigated.

Table 3.1 : Proposed system of description (Datta *et al.*, 1982)

Description of	Remarks
<p>1. Cementation</p> <p>(a) No cementation</p> <p>(b) Weak cementation</p> <p>(c) Strong cementation</p> <p style="padding-left: 20px;">(i) uniform</p> <p style="padding-left: 20px;">(ii) partial</p>	<p>the soil has a soft rock-like appearance. Unconfined compressive strength should be indicated</p> <p>the soil contains cemented aggregates - this should be noted</p>
<p>2. Grain Size Distribution (GSD) and Plasticity</p> <p>(a) Grain size distribution</p> <p>(b) Plasticity</p>	<p>for strongly cemented soils, GSD is not very relevant; for uniform cementation, size of constituent particles should be indicated; for partial cementation, GSD of soil after removing aggregates should be indicated and size and proportion of aggregates noted separately</p> <p>for fine-grained soils in which intraparticle voids cause error in GSD and Atterberg Limits, field classification procedures may be used for providing the relevant information in a qualitative sense</p>
<p>3. Nature of Carbonate Component</p> <p>(a) Carbonate content</p> <p>(b) Particle size of carbonate material</p> <p>(c) Particle characteristics and origin</p> <p>(d) Mineralogy</p> <p>(e) Geologic name</p>	<p>soils having more than 30% carbonate content should be termed carbonate soils</p> <p>the carbonate content in the sand and in the silt-clay fractions should be determined separately and indicated. Microscopic studies mentioned below will also give information about particle size</p> <p>microscopic studies - optical microscope for sands and scanning electron microscope for fine-grained soils - should be conducted. Presence of thin-walled material and intraparticle voids should be highlighted</p> <p>X-ray diffraction analysis should be performed</p> <p>if possible to identify, the geologic name may be indicated</p>
<p>4. Nature of Non-carbonate Component</p> <p>(a) Particle size</p> <p>(b) Particle characteristics</p> <p>(c) Mineralogy</p>	<p>information on non-carbonate material is determined by dissolving the carbonate material in HCl, separating the remaining soil, and conducting the following tests on it</p> <p>grain size distribution analysis as for carbonate soils</p> <p>microscopic studies</p> <p>X-ray diffraction analysis</p>

4. RELEVANT INDEX PROPERTIES AND THEIR MEASUREMENT

4.1 CARBONATE CONTENT

4.1.1 The Effect of Carbonate Content on the Engineering Properties

The engineering properties of a soil mixture consisting of a carbonate and non-carbonate component could be influenced by the following variables :

- (a) For a given carbonate fraction and composition, the nature and composition of the non-carbonate component (e.g. clay or sand, sensitivity of clay fraction, etc.).
- (b) For a given non-carbonate fraction and composition, the nature and composition of the carbonate component (e.g. particle size, skeletal or non-skeletal particles, etc.).
- (c) For a given carbonate and non-carbonate composition, the relative proportions of each in the soil.
- (d) Whether or not the carbonate fraction has a cementing effect either on the carbonate component, the non-carbonate component or both.

The environment, regional location and sedimentary and geological history of the soil to a large extent determines its nature, composition and state of cementation. Thus, a marine clay could be expected to be affected differently by carbonate content than a terrestrial clay.

It has been reported that carbonate content has a definite beneficial effect on shear strength in the case of calcareous clays (Agarwal *et al.*, 1977), while the deleterious effect of crushable carbonate particles in some sands are discussed in Section 4.3. McKown and Ladd (1982) reported that

carbonate content (beyond a minute threshold level) has little effect on cementation of a terrestrial shale, while Kelly *et al.* (1974) found indications of an increase in cementation with carbonate content for a marine clay.

These examples illustrate the difficulty of generalizing about the effect of carbonate content without reference to the nature and composition of the material. Notwithstanding these difficulties, it is recognised that for a given soil, carbonate content could have a decisive influence on the strength and stress-strain behaviour. For design purposes, it is useful to delineate the carbonate contents, i.e. where it ceases to have a beneficial/deleterious effect on the engineering strength properties.

Little systematic research on the effect of carbonate content (for example through artificially constituted laboratory samples) have been conducted, especially for calcareous clays. Much of the available information is based on site-specific data and cannot be assumed to be universally applicable.

(i) Shear strength

Good correlation between bulk density and carbonate contents has been reported for some types of carbonate sediments (Curry and Lohman, 1986; Kominz *et al.*, 1977). Lee (1982) investigated the relationship between bulk density, carbonate content and shear strength of several deep-sea calcareous sediments. The shear strength of the sediments depended mainly on whether the non-carbonate matrix or a carbonate framework dominated, and on the physical nature of the non-carbonate matrix. A model based on relations between bulk density and carbonate content was developed to delineate three types of behaviour : matrix-dominant, transition and carbonate framework dominant. The model was quantitatively evaluated using vane shear, triaxial

shear strength, density and carbonate content data from four studies of deep-sea calcareous sediments. The following important trends emerged :

- Division between the three behaviour types were determined primarily by the character of the non-carbonate matrix.
- In the matrix dominant zone, vane shear strength was independent of carbonate content.
- In the transition and carbonate framework dominant zone, vane shear strength increased with increasing carbonate content. The cause of the increase (granular nature of carbonate particles or cementation) could not be ascertained.

Demars *et al.* (1976) performed consolidated undrained triaxial tests on natural calcareous clays with carbonate contents ranging from 10 to 90 percent. A summary of the results is given in Table 4.1. The criterion for failure was chosen as the strain at which maximum porewater pressure parameter-A occurred, typically between 3 and 4 percent strain. With an increase in carbonate content, an increase in the effective friction angles was observed, with a corresponding decrease in porewater pressure parameter-A at failure. The trend was from a cohesive behaviour to a granular behaviour with increasing carbonate content. Demars *et al.* suggest that the data shows the importance of particle shape and packing on strength behaviour.

Agarwal *et al.* (1977) investigated the effect of carbonate content on the direct shear behaviour of calcareous sands, clays and silts obtained at three sites offshore of India. It was found that the carbonate content had a significant influence on the stress-strain behaviour of the sands, particularly at higher normal stress and carbonate contents in excess of 45

percent. For sands with high carbonate contents, the behaviour and pattern of stress-strain curve appeared to signify the effect of cementation and the possible breakage of aggregated composition. The effect of quartz sand was prominent only for carbonate contents below 30 percent. Above 45 percent, carbonate content had a detrimental effect on the strength of the soil. An increase in the carbonate content of clay samples appeared to have a beneficial effect on the strength properties. This was confirmed by Demars *et al.* (1976).

(ii) Compressibility

Bryant *et al.* (1974) performed more than 120 consolidation tests on samples with varying carbonate contents, obtained from the Gulf of Mexico. The results indicated that the compression index (C_c) generally decreased with increasing carbonate content. The variation of C_c with carbonate content for normally consolidated and stiff overconsolidated samples are shown in Fig. 4.1. Bryant *et al.* (1974) concluded that the results of oedometer tests on carbonate soils were similar to those for non-carbonate silty clays, with the exception that the carbonate sediments did not compress to such low final void ratios. Such decreasing compressibility with increasing carbonate content appears to be applicable only to some fine grained carbonate-clay mixtures, since it is known that carbonate sands are generally more compressible than silica sands (e.g. McCarel and Beard, 1984).

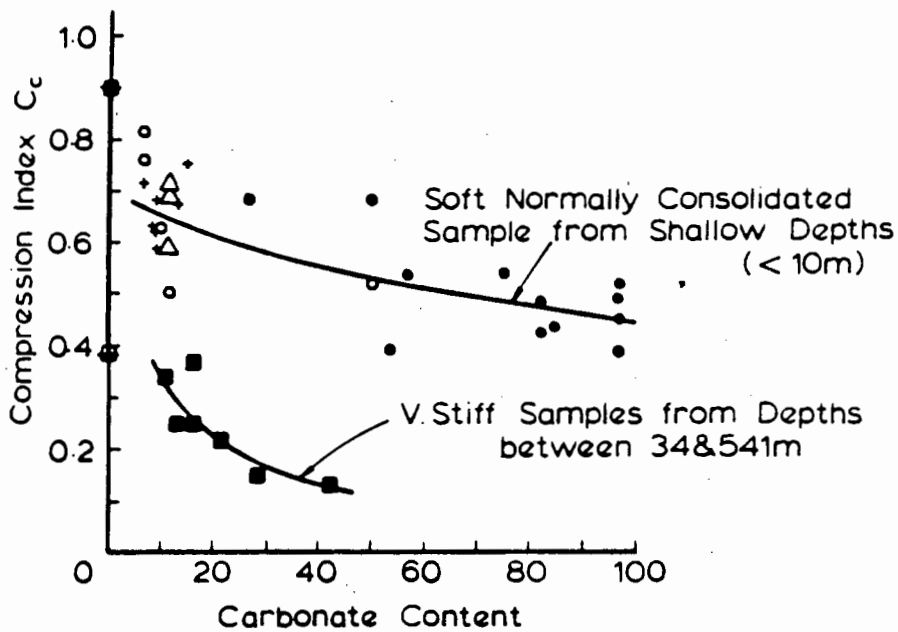
(iii) Piling and skin friction

McCarel and Beard (1984) investigated the effect of carbonate content on the pile driving resistance of model piles. For dense soil samples, the carbonate content appeared to control the amount of resistance (decrease with increasing carbonate content) that the soil mobilized during pile

Table 4.1 : Consolidated Undrained Triaxial Test Results
(after Demars *et al.* (1976))

CaCO ₃ (%)	\bar{c} (PSI)	$\bar{\theta}$ (degrees)	A _f
0 - 25	0	27.7	0.7
25 - 40	0	29.4	0.55
40 - 60	0.1	31.0	0.40
> 60	0.1	31.3	0.25

Failure strain, ϵ_f , taken at maximum pore-pressure parameter-A, A_f - typically
 $\epsilon_f = 3$ to 4%



Note: Compression Index C_c taken between $\sigma' = 100$ & 1000kPa.

Figure 4.1 : Variation of C_c with carbonate content
(after Bryant *et al.*, 1974)

driving. For loose soil samples, the amount of carbonate sand did not affect the amount of resistance mobilized by the soil.

Apart from the above study, there appears to be no published data on the effect of carbonate content on piling behaviour. On the basis of direct shear tests, Agarwal *et al.* (1977) recommended that sands with less than 45% carbonates be treated as conventional sands. Other authors, for example Datta *et al.* (1982) suggested that the nature of the carbonate particles play a more important role, but there seems to be widespread agreement that sands with very high carbonate contents (> 80%) clearly act as calcareous and have the potential for abnormally low pile capacities (Murff, 1987).

4.1.2 Determination of Calcium Carbonate Content

(i) Appropriate technique for geotechnical engineering

A number of laboratory procedures for the measurement of calcium carbonate content have been developed. These can be divided into two classes : techniques that determine the calcium ion (Ca^{++}) concentration and techniques that determine the carbonate ion concentration (CO_3^{--}).

Chaney *et al.* (1982) evaluated the suitability of methods based on these techniques for geotechnical engineering purposes. They concluded that the pressure calcimeter method or "Karbonat-Bombe" is the most suitable. Selection of this method was based on the following factors :

- Accuracy obtainable : The stated error varies typically from 2% to 5%, with errors approximating 1% for high carbonate contents. Suggestions and precautions for achieving improved accuracies with this method are discussed by Dunn (1979).

- Operator skill required : Minimal skill is required for this method.
- Relative equipment cost : Initial equipment cost is low compared to that of other methods.
- Analytical speed : The high analytical speed minimizes unit costs. Up to 50 specimens can be analysed daily.

These favourable characteristics makes the method ideal for shipboard determinations of carbonate content and has led to its adoption for the Deep Sea Drilling Project. A comparison of the "Karbonat-Bombe" with other methods, based on the above criteria, is presented in Table 4.2.

(ii) Limitations of the "Karbonat-Bombe"

The basic assumption of the method is that all carbonate ions (CO_3^{--}) are combined as calcium carbonate. If the relative abundance of magnesium in carbonate sediments is less than 3 percent by dry weight (Section 2.5.3), the error should remain within the stated range.

The method cannot be used to identify the nature of the carbonate minerals in the sediment, and staining schemes should be used for this purpose. Where detailed understanding of the trace element chemistry of the sediment is needed, x-ray fluorescence, microprobe, atomic absorption, or cathodoluminescence techniques may be used.

The accuracy of the "Karbonat-Bombe" technique decreases with decreasing carbonate content. Carbonate contents of as little as 2% can sometimes affect the engineering behaviour of some clays (McKown and Ladd, 1982). In such cases alternative techniques with high accuracy at low carbonate contents, such as atomic absorption spectrophotometry must be used.

Table 4.2 : Methods of determining calcium or calcium carbonate in soils (after Chaney et al., 1982)

Method	Relative Accuracy ^b	Analytical Speed ^c specimens/day	Initial Equipment Cost	Skill of Operator	Reagents Used	Comments
Atomic absorption spectrophotometry	accurate	20	high	skill and expertise required for setup and calibration		1. accurate at low concentrations 2. separate determination of Ca ⁺⁺ and Mg ⁺⁺ can be made from the same solution
EDTA titration	accurate	6 to 10	low	some chemical expertise required	cyanide ammonia	separate determinations of Ca ⁺⁺ and Mg ⁺⁺ can be made from the same solution by an additional titration
Calcium-specific ion electrode	good	10	low	some chemical expertise required	Na ₂ H ₃ O ₂	1. determinates Ca ⁺⁺ 2. 0.5 N sodium acetate (NaC ₂ H ₃ O ₂) having a pH of 8.2 must be used to limit free CaCO ₃
Vacuum-distillation and titration method	good	8	moderate	some chemical expertise required	Ba(OH) ₂ HCL SnCl ₂	foaming is frequently excessive, especially with soils high in carbonate
Gravimetric method	good	10	low	some chemical expertise required	H ₂ SO ₄ F ₂ SO ₄	accuracy of this method is dependent on the accuracy of weighings and the ability of the absorbent to retain all the CO ₂
Acid-soluble weight loss methods	good	50	minimal	minimal skill	HCL	accuracy and precision decrease markedly for specimen weights less than 2.0g
Volume calcimeter method	good	10	low	some chemical expertise required	HCL FeCl ₂	accuracy dependent on (1) vigorous shaking of reaction flask, (2) uniform temperature of environment, and (3) standardized input of HCL
Pressure calcimeter methods	accurate	10	low	some chemical expertise required	HCL	1. Hg normally employed in apparatus pressure system
Gasometric Methods Karbonat Bombe	good ^d	50	low	minimal skill	HCL	2. Range of calibration from 3 to 110%

Table 4.2 contd.

Method	Relative Accuracy ^b	Analytical Speed ^c specimens/day	Initial Equipment Cost	Skill of Operator	Reagents Used	Comments
Gravimetric method for loss of CO ₂	good	4	low	minimal skill	HCL	accuracy of this method is dependent on the accuracy of weighings and upon the degree to which CO ₂ retained in solution is compensated for by water vapour losses
Acid-neutralization method	good	20	minimal	some chemical expertise required	HCL NaOH	estimate of carbonate will usually be somewhat high due to other constituents reacting to some degree with the acid

^b Classes of accuracy based on the following : 1 accurate, generally less than $\pm 1\%$; 2 good, generally less than $\pm 5\%$; and 3 rough, generally over $\pm 5\%$.

^c Estimates based on 8-h day with one apparatus and one technician.

^d Based on results from 10 prepared specimens consisting of various amounts of laboratory grade CaCO₃ mixed with an inert sand.

4.2 CEMENTATION

4.2.1 The Effect of Cementation on the Engineering Properties

Carbonate sediments can be cemented to various degrees, forming lightly cemented deposits at the one end of the spectrum and highly indurated limestone rock at the other. The degree of cementation can vary over the entire stratigraphic interval of a particular formation, and on a smaller scale, variability can be high even within the dimensions of the average laboratory sized specimen. Variability of cementation results in materials with a wide range of properties, so that it is difficult to generalise about its effect on engineering behaviour. The difficulty in obtaining undisturbed samples, especially in the marine environment, further hampers research. The effect of cementation can be either detrimental (preventing development of adequate lateral pressure) or beneficial (additional strength), depending on the nature of the cemented material, degree and uniformity of cementation and the type of structure founded on the cemented material.

(i) Apparent overconsolidation

The most significant effect of cementation on carbonate clay in terms of one-dimensional consolidation tests is an apparent overconsolidation behaviour when normal reasons for overconsolidation are absent. That overconsolidation behaviour can be attributed to physico-chemical bonding between particles, rather than preloading, has been shown by the leaching experiments of Moore *et al.* (1977). Removal of organic compounds (non-carbonate) caused clays which had shown overconsolidation characteristics prior to treatment to behave as normally consolidated sediments.

McKown and Ladd (1982) investigated the effect of calcite cementation on the compressibility of Pierre Shale. Relatively small amounts of calcium carbonate, above a threshold value of two percent by weight, appeared to cement the structure of the shale, causing measured $\bar{\sigma}_{vm}$ values (apparent maximum past pressure) several times larger than the maximum past overburden stress inferred from geological history. For calcium carbonate contents in excess of this threshold value (even up to fifty percent or more) little further increase in $\bar{\sigma}_{vm}$ was observed.

Kelly *et al.* (1974) performed oedometer tests on cemented marine sediments and found that the overconsolidation ratio increased with carbonate content, possibly as a result of increased cementation. They believe that there is some evidence to suggest that the cemented structure of the soil is a transient phenomena which is destroyed with burial, after which the sediments behave as normally consolidated soils. Thus, cementation of near-surface ocean sediments has particular significance for the design of shallow foundations. If contact pressures are small and the cemented structure is not destroyed, settlements will be limited. If the cementation strength is exceeded the large deformations associated with soft, normally consolidated sediments can be expected.

Frydman (1982) discusses a similar phenomenon with reference to the thick calcareous deposits overlying the Mediterranean coastal plain of Israel. Considerable breakdown of cementing bonds occur under conditions of high stress, with the result that the material behaves as an essentially cohesionless material, sometimes having an extremely high compressibility. Fig. 4.2 shows curves of compressibility vs. confining pressure for cemented samples as determined from triaxial consolidation tests. For a given confining pressure, the five cemented samples had higher compressibilities

than the uncemented non-carbonate reference sand. With increasing confining pressures, the compressibility of the cemented samples decreased, until at a certain confining pressure ($\pm 220 \text{ kN/m}^2$ and $500\text{--}600 \text{ kN/m}^2$), a sudden increase was again noted. This sudden increase in compressibility is believed to be due to breakdown of cementation bonds. Frydman has also shown that such increased compressibility as a result of breakdown of the cemented skeleton can lead to increased susceptibility to liquefaction during cyclic loading.

In addition to settlement, the apparent overconsolidation behaviour of cemented deposits have important practical implications regarding an estimation of the in situ K_0 , which in turn affects the design of underground structures and selection of safety factors for slope stability.

(ii) Shear strength

The strength behaviour of a soil is determined by the three components of shearing resistance; cohesion, dilatancy and friction. The cohesion component includes any natural or artificial cementation or interparticle bonds. The friction and dilatancy components are hard to separate, but are both a direct function of the force acting normal to the shear surface.

Saxena and Lastrico (1978) investigated the strength behaviour of a lightly calcite cemented non-calcareous sand. At low strain levels ($< 1\%$), the cohesion portion of the shearing resistance was predominant. Beyond a certain strain level ($\pm 1\%$), a gradual breakdown of cementation started, and the frictional resistance became predominant. At very high strains, there was a complete breakdown in structural cementation and the shearing resistance was entirely frictional in nature. In the majority of samples tested, the value of the pore pressure parameter A reached a peak within one

percent strain and then continued to decrease with increasing strain. This behaviour is typical of dilating or dense material.

Frydman (1982) performed drained triaxial tests on undisturbed calcareous soils cemented to various degrees. Despite large variations in the internal structure of the weakly cemented samples, a common strength line with strength parameters of $c' = 13 \text{ kN/m}^2$ and $\phi' = 37 \text{ deg}$, could be used to represent all points of this group. Moderately cemented specimens approximated to the same line. Two well-cemented specimens were tested, yielding strength parameters of $c' = 190 \text{ kN/m}^2$ and $\phi' = 37 \text{ deg}$. It therefore appears as if the cementation primarily affects the cohesion value and has little effect on the angle of internal friction of this soil. Failure stress values were taken from the peaks of the stress strain curves obtained in the test, occurring at axial strains of between two and five percent. The peak cohesion however, was developed at much lower values of axial strain, between one and half a percent (see Fig. 4.3).

Kelly *et al.* (1974) performed consolidated undrained triaxial tests on lightly cemented marine carbonate sediments. At small strains ($\pm 0.5\%$), virtually no pore water pressure was developed. As the strain increased, cementation broke down, with the pore water parameter-A reaching a maximum value of 1.2 at axial strains of approximately five percent. A similar change in pore water pressure response was noted in consolidated drained tests.

The available data appears to indicate that cementation contributes to the strength only at relatively low levels of strain. Destruction of cementation occurs when the apparent overconsolidation pressure is exceeded or when certain levels of axial strain are exceeded.

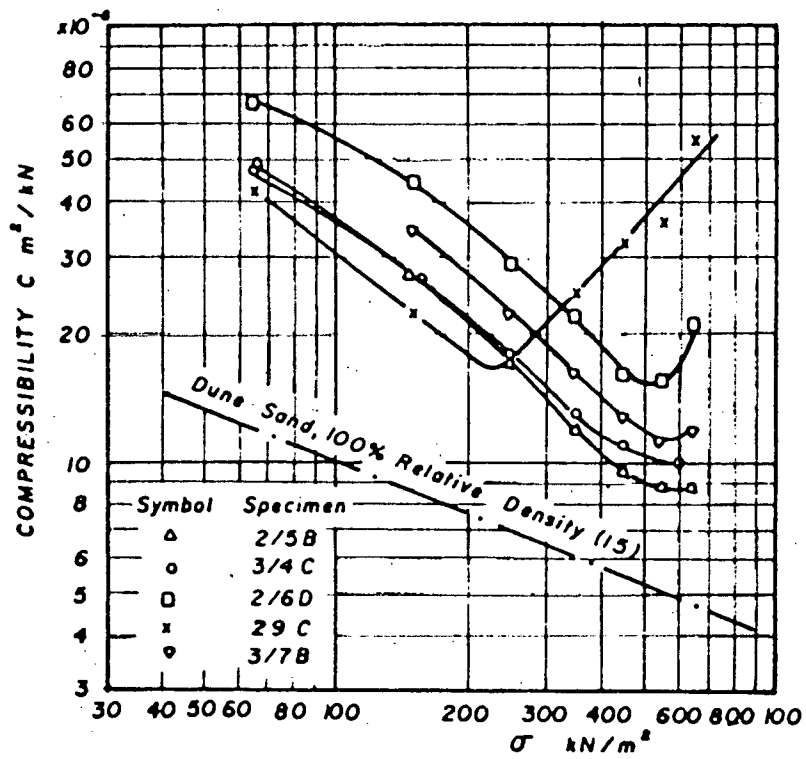


Figure 4.2 : Compressibility C versus confining pressure, where C is defined as $d\epsilon_v/d\sigma_3$ for increments of confining pressure up to 700 kN/m^2 (after Frydman, 1982)

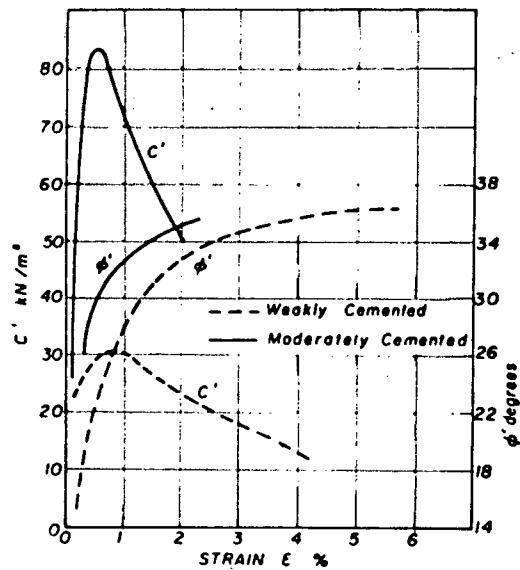


Figure 4.3 : Development of cohesion and friction angle with strain (after Frydman, 1982)

This aspect has important design implications, the availability of additional strength due to cementation being highly strain dependent.

(iii) Piling and skin friction

Driven piles offer significantly lower support capacity in calcareous soils than predicted by conventional soil mechanics theory. Skin friction capacities can be as low as one tenth of the value expected for similar piles in quartz based sands (King *et al.*, 1980). This is thought to be a result of a combination of factors due to particle crushing and the presence of weak cementation preventing development of significant lateral pressure. There is evidence to suggest that both the nature and degree of cementation have an important effect on the magnitude of skin friction development (McClelland, 1980). In the Arabian Gulf, where cementation is strong and uniform, difficulty has been experienced in driving piles to a predetermined depth. In the Bass Strait of Australia, where cementation is weak and variable, little driving resistance has been experienced to depths of up to several hundred feet. Variable cementation results in variable driving resistance. In the case of prestressed concrete piles, this can lead to pile damage.

Ertec Western Inc. (1983) conducted laboratory studies on the effect of cementation on pile driving and skin friction. The following trends were noted :

- the amount of crushing during driving increased with increasing cementation;
- at each prescribed cement content, pile driving resistance was at the same order of magnitude for higher density silica and calcareous soils;

- for lower density carbonate sands, increase in cement content did not significantly increase driving resistance;
- in silica sand, there was a general tendency for skin friction resistance to increase with increasing cement content;
- for calcareous sands, there was no definite pattern; it is believed that the effect of cementation on pile pullout resistance was either partially or overly compensated by effects of increasing crushability during driving.

Where cementation is strong and uniform, it could have a positive effect on the end bearing capacity of the pile. Conventional design theory probably overestimates the contribution of skin friction for driven piles in calcareous sands, while it underestimates the available end bearing capacity (Datta *et al.*, 1980).

4.2.2 Quantification of Cementation

At present there is no laboratory procedure or test to quantitatively express the degree or uniformity of cementation. For uniformly, well cemented samples, the unconfined compressive strength has been used in some classification systems, but for weakly or non-uniformly cemented samples no laboratory tests have been suggested. The qualitative system of description proposed by Datta *et al.* (1982) appears to be the limit of refinement achievable at this stage, i.e. :

- no cementation;
- weakly cemented;
- strongly cemented;
- uniform : determine the unconfined compressive strength and indicate size of constituent particles;
- partial : note the size and proportion of the aggregates.

It has been noted by several authors that the presence of weak cementation in particular appears to contribute to low pile capacities in carbonate soils. It would seem that a refinement in the identification and description of especially weakly cemented samples would have practical significance. An investigation into artificially cemented samples in the laboratory, simulating the natural cements, could aid in this regard.

4.2.3 Artificial Cementation

The difficulty of obtaining undisturbed cemented samples and the non-uniformity of cementation in naturally cemented deposits necessitates the use of artificially cemented samples for controlled laboratory investigations into the effect of cementation on various engineering properties. A process for inducing artificial cementation should ideally satisfy the following conditions :

- (a) the artificial cement should simulate the natural cement as far as possible;
- (b) the method should be practical in that sufficient cementation is produced within a reasonable time span;
- (c) the method should be practical in that prohibitively complex and expensive equipment and procedures are not required.

A survey of the literature appears to indicate that the processes used in the engineering research field generally satisfies conditions (b) and (c) above at the expense of condition (a). This approach is dictated by the volume of cemented sample required for such research and practical difficulties associated with satisfying conditions (a) and (b) simultaneously.

Ertec Western Inc. (1983) produced cemented samples of shell fragment by adding Portland cement Type I (between 0.5 and 2 percent by weight) to the dry sand, mixing thoroughly and adding water. Poulos *et al.* (1982) attempted to produce cementation of Bass Strait sands using four different procedures :

- Samples of uncemented carbonate sands were left standing in an oedometer under a stress of 248 kPa, for a period of 10 weeks, in the presence of a supersaturated solution of calcium carbonate.
- Two of the above samples were subjected to an increase in stress to 310 kPa, and left standing for a further 6 to 8 weeks.
- Some samples were left standing in a lime rich solution under the same stress conditions as above.
- Some samples were mixed with 6 percent Portland cement.

Only the samples to which cement was added showed any visible signs of cementation. These experiments indicate that cementation does not appear to be redeveloped in a static calcium carbonate solution in a laboratory environment. However, experiments by McKown and Ladd (1982) indicate that even circulatory pore water conditions is in itself not sufficient to ensure precipitation of large quantities of calcium carbonate. Specimens of natural clays were leached for about four months with a saturated calcium carbonate solution, using hydraulic gradients up to 9600. A total of about 35 void volumes of fluid was passed through the specimens. Chemical, x-ray diffraction analysis and index properties examination before and after the tests indicated that no significant changes had occurred. From these experiments, the following important observations can be made :

- (a) Significant precipitation and hence cementation is unlikely to be induced by merely "treating" a sample with a supersaturated solution of calcium carbonate.
- (b) Precipitation of calcium carbonate resulting from changes in pore water conditions due to sampling is unlikely to affect the engineering properties of the sediment significantly.
- (c) Artificially cemented samples used in studies of their engineering properties are not satisfactory in terms of simulating natural cementation.

Artificial cementation has been studied more systematically in the field of sedimentology, mainly as a means of deducing the physical environment and chemical conditions prevailing at the time of cement formation. It is believed that methods developed by researchers in this field can provide useful guidelines for preparation of artificially cemented samples to be used in engineering/geotechnical investigations.

Various attempts to simulate natural cementation processes under laboratory conditions are mentioned by Badiozamani *et al.* (1977) :

- Experiments conducted at low temperatures have been hindered by lack of precipitation of significant amounts of cements in reasonable periods of time (e.g. Rezak, 1971).
- To overcome this difficulty, Thorstenson *et al.* (1972) designed an experimental setup based on CO₂-degassing of calcium carbonate waters initially saturated at one atmosphere CO₂ , to produce significant cementation of carbonate skeletal sands in a reasonably short time span.

Based on the above method, Badiozamani *et al.* (1977) successfully produced carbonate cementation under various conditions of temperature, solution composition and physical environment. Cements analogous to their natural counterparts reported in the literature resulted. Three different experimental designs were used to simulate 1) vadose cementation, 2) phreatic cementation by CO₂ evasion, and 3) cementation due to evaporation. Duration of experiments ranged from 6 days for high temperature freshwater experiments to more than 2 months for seawater experiments. The procedures and experimental setup required do not appear to be unduly complicated and expensive, and the time span to produce sufficient cementation is reasonable.

4.3 CRUSHABILITY

4.3.1 The Engineering Significance of Particle Crushing

The susceptibility of the individual grains of some carbonate soils to crush under relatively low stresses has a number of important effects on their engineering behaviour. Thus unsafe designs for structures founded in these soils can result if parameters and theories developed for conventional soils are applied.

(i) Compressibility and Settlement

Calcareous soils are significantly more compressible than silica soils, their compressibility resulting from grain crushing and the collapse of grain structure, so that volume changes are usually permanent (McCarel and Beard, 1984). Similarly, shallow foundations on crushable material undergo more settlement (Poulos *et al.* 1984), and embedded anchors and heavily loaded footings founded on coarse grained calcareous oozes can induce

stresses sufficient to cause large displacements or even failure of the anchors and footings (Valent, 1974).

(ii) Shear strength and bearing capacity

It has been established by various researchers that the friction angle of crushable soils in general (Fedaa, 1971; Lee, 1967) and calcareous soils in particular (Datta *et al.*, 1982; Poulos, 1982) decreases with an increase in confining pressure (triaxial test) or normal stress (plane strain). This reduction has been linked to grain crushing. It would appear that increased grain crushing induces decreasing shearing resistance until a limiting value of shearing resistance is reached. It might therefore be expected that crushing of sand grains under the tip of a pile causes a reduction in ϕ and hence a reduction in end-bearing resistance (Datta *et al.*, 1980). In general therefore, the highly compressible nature of some calcareous sands results in substantially lower bearing capacities than for quartz sands (Poulos and Chua, 1985). Because shearing can cause grain crushing, large volumetric reductions can be associated with shearing stresses (McCarel and Beard, 1984).

(iii) Piling and skin friction

The results of pile load tests conducted in the Bass Strait (Angemeer *et al.*, 1973) revealed that side friction of steel piles in calcareous soils can be very low and that the unit end bearing measured was substantially less than would be expected for piles bearing on quartz sands. Similarly, driven piles have encountered low driving resistance and have shown a low static load capacity when compared to corresponding piles in quartz sand (Poulos, 1982).

It has been suggested that as a pile is driven into a carbonate stratum, crushing of the particles, and hence a volume reduction, occurs due to crushed fines moving into the voids. As a consequence of the crushing, the lateral stress between the pile and soil is poorly developed. This results in low skin friction.

Investigations by King *et al.* (1980) and Poulos (1984) revealed that further significant degradation of pile skin friction can be expected under cyclic loading conditions, possibly as a result of an increase in the amount of crushing.

4.3.2 Factors Influencing Crushability

In examining the factors influencing crushability, information gained from research into crushing of other types of material can provide insight into relevant factors affecting calcareous materials. It should nevertheless be recognised that some calcareous soils have unique properties which are not simulated by the crushing behaviour of other material types.

On the basis of material type, research into crushability of granular media can be divided into three categories :

- the crushing of rockfill grains;
- crushing of alluvial sands and gravels at high stresses;
- crushing of sands with weak grains.

For each of these cases a different set of factors will influence the extent of crushing. Nevertheless, factors causing crushing can be related, quantitatively at least, for each of these material types. Thus, weak

grained sands have been used to model the behaviour of rockfill material (Fedda, 1971).

The crushing susceptibility of a soil will depend on both the inherent material properties, such as the hardness of the individual soil particles, and externally imposed factors, such as type and intensity of loading.

(i) Inherent material properties affecting crushing

(a) Particle hardness

Carbonate soil particles are composed of the minerals calcite and aragonite, which are relatively soft compared to quartz-based sands (3 to 4 Mohs scale of hardness, 7 for quartz-based sands).

(b) Particle shape and textures

Particles of carbonate soils are composed of a wide variety of shapes and textures, especially where these are of biological origin. Datta *et al.* (1982) performed triaxial experiments on carbonate soils of various shapes and textures and found that crushability increases with an increase in :

- the amount of thin walled shell fragments
- angularity of grains
- coarseness of grains
- amount of particles having large intraparticle voids.

Thus a calcareous soil composed of rounded non-skeletal particles (ooliths) may display crushing comparable with quartz sand. They therefore proposed that more emphasis be placed on the nature of the soil particles, rather than the calcium carbonate content alone, in assessing the crushing potential.

(c) Cementation

Both the amount of cementing material and its hardness have an influence on the amount of crushing. Due to practical difficulties associated with simulation of cemented conditions in artificially composed laboratory samples (Section 4.2.2) and particle size analysis of both natural and artificially cemented samples, data on the effect of cementation on crushing is limited and highly qualitative.

Ertec Western Inc. (1983), in a model laboratory study of piling in artificially cemented carbonate soils, found that crushability increased with an increase in "cement" content, for both higher density calcareous sand and quartz sand. The degree of cementation had little effect on crushing in low density sands of both types.

(d) Particle size

The potential for breakage of a soil particle increases with its size. This results from the fact that the normal contact forces in a soil element increase with particle size, and the fact that the probability of a defect in a given particle increases with its size. Large particles will break under moderate stresses, while high stresses are required to break silt size particles.

(e) Particle size distribution

The potential for particle breakage increases with the uniformity of gradation. This seems to have a particularly important effect in the case of some calcareous soils composed of hollow or plate-like particles.

Valent (1974) performed consolidation tests on material containing coarse-sized hollow foraminefera shells, and found no significant crushing under consolidation stresses of up to 1530 kPa. Removal of the finer material resulted in significant crushing at stresses as low as 50 to 200 kPa. It is believed that the fine-size fraction acts to distribute the load on the coarse-size shell surfaces.

(f) Void ratio

Carbonate sediments often have high void ratios, typically between 1 and 2, with void ratios as large as 3.7 occurring in some natural sediments. In addition to containing large spaces between particles as a result of the complexity of particle shapes and early cementation, hollow and porous particles containing large intraparticle voids are often present. Such hollow particles will be more susceptible to crushing.

The complex shapes of the particles will allow greater interlocking when these particles are in a dense state of packing. It could be hypothesized that crushing of the soft particles will be greater as the amount of interlocking increases, so that more crushing would be expected for a larger relative density. This hypothesis appears to be supported by experimental results obtained by Ertec Western Inc. (1983), who found that the amount of grain crushing increases as the void ratio decreases for piles driven into uncemented calcareous sand.

(ii) External factors

(a) Effective stress path

Particle fragmentation is known to have a major effect on the stress-strain behaviour of rockfill material (Marsal, 1967), alluvial sands tested under

high pressures (Vesic and Clough, 1967) and sand with weak grains (Datta et al., 1982; Poulos, 1982).

Different types of loading will produce different stress-strain behaviour during isotropic, triaxial and confined compression upon identical soil specimens which initially had the same void ratio and carried the same vertical stress (Lambe and Whitman, 1979). Figure 4.4 illustrates the relative effect that different modes of confinement have on the stress-strain behaviour of a typical terrigenous soil. Given the interdependence of particle fragmentation and the stress path followed during loading of a soil (e.g. Hardin, 1986), it is to be expected that a given soil will show different extents of crushing under different types of loading.

Feda (1971) found that for non-calcareous weak-grained soils, isotropic consolidation of samples, even at comparatively high cell pressures, results only in slight grain crushing. For the calcareous soils and quartz sand evaluated by Datta et al., (1982), crushing during shear was significantly more than crushing during isotropic compression.

The stresses induced by dynamic impact of driven piles result in different degrees of crushing compared to that induced by application of consolidation or shear stresses. Consolidation stresses applied through isotropic (triaxial) and confined (K_0) conditions do not follow the same stress path, similarly shear stresses applied through plane strain and triaxial conditions (Lambe and Whitman, 1979).

(b) Stress level and strain energy

The amount of crushing experienced by a given soil increases with the level of effective stress applied to the soil under all stress path conditions.

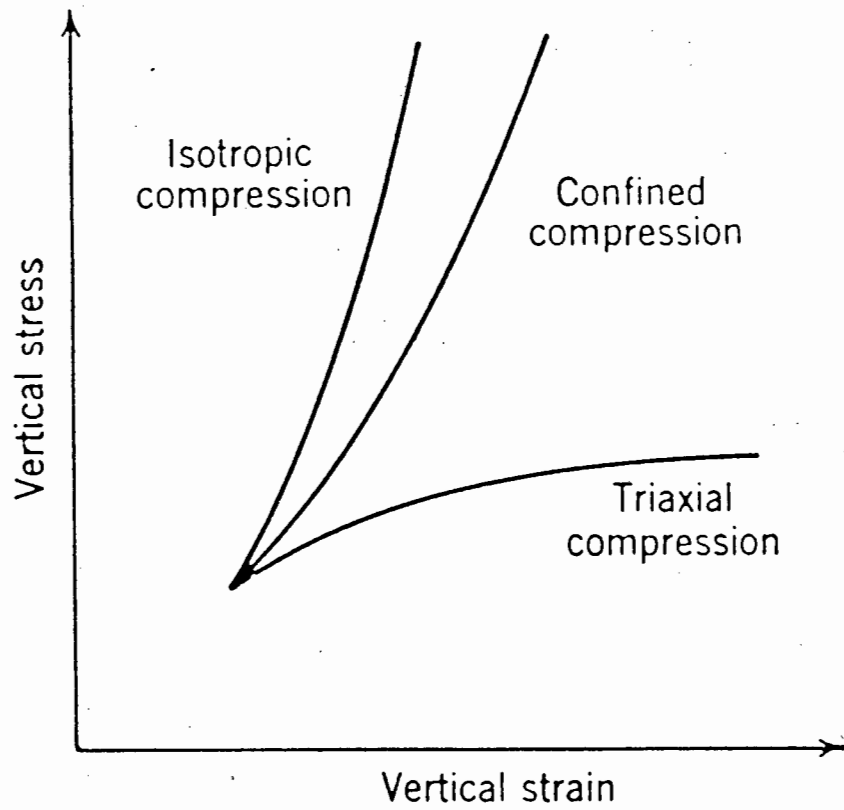


Figure 4.4: Comparison of stress-strain curves for three types of compression. (after Lambe and Whitman, 1979)

For many crushable soils, a threshold effective stress at which significant crushing commences can be identified. This threshold value however appears to be stress path dependent, i.e. significant particle crushing for a given soil will not commence at the same effective stress under K_0 compression (e.g. oedometer) loading and triaxial compression. As particle crushing increases, the particle size distribution of the material changes, the total contact area increases, the contact stresses between particles decreases and crushing slows down and eventually reaches a limit for a given level of effective stress.

Under triaxial and plane strain conditions, for a particular confining stress or applied normal pressure, Feda (1971) found that the amount of crushing is highly dependent on the amount of strain energy supplied to the sample in the course of its loading. Intensive grain crushing occurred only after the amount of strain corresponding to the peak strength of the sample had been applied.

(c) Laboratory/scale effects

The different stress paths in the case of direct shear and triaxial compression may not be sufficient to result in a significant difference in crushing. However, the size of the shear zone affected by crushing relative to the total sample size will have an important influence on the evaluation of crushability by particle size analysis. A similar effect could result in using the same stress path (e.g. direct shear), with different size samples.

(d) Presence or absence of water

Because of the ease of forming and testing dry samples as compared to moist and saturated samples, laboratory strength tests are often performed on air

dry soil (Lee *et al.*, 1967). This approach is appropriate only for soils falling into the category of clean sands, i.e. soils which will flow freely when dry and whose strength is defined by an angle of internal friction only. The practice is justified on the assumption that moisture content does not significantly affect the effective strength characteristics of soils, an assumption supported by considerable evidence for many soils.

There does not appear to be any published literature discussing the effect of moisture on the strength or crushability of calcareous soils. For some granular materials however, moisture can have a significant effect on the strength properties. Bishop and Eldin (1953) performed triaxial tests on a fine clean sand and found a decrease in the angle of friction of up to 5° for sand tested in the saturated condition. Sowers *et al.* (1965) studied the compressibility of rockfill material and found that moisture has a significant effect on the amount of crushing. They concluded that this is possibly due to water entering the microfissures in highly stressed contact points, causing a local increase in stress and additional failure. The moisture sensitive strength behaviour of Ottawa river sand was ascribed to the same phenomena by Lee *et al.* (1967);

"It would appear moisture sensitivity is likely to be greatest in granular soils whose particles either contain cracks or are susceptible to the formation of fine cracks during loading."

Considering the structure and composition of calcareous soil particles, it is clear that many carbonate soils could potentially display such moisture sensitive strength behaviour. The complex shapes and porous surfaces could act as natural stress concentrators. For particles with complex shapes,

small contact areas can give rise to high contact stresses with formation of minute cracks at contact points. Furthermore, the solubility of carbonate minerals in water could further promote weakening mechanisms.

4.3.3 Measures of Particle Breakage

The amount of particle breakage during loading of a soil sample is defined by the particle size distribution curves measured before and after loading. For correlation of crushing with material behaviour under a given loading condition, it is most convenient to express the amount of breakage as a single number.

In current practice, the degree of crushability of calcareous sands is usually quantified by a crushability index C_k , defined by Datta *et al.* (1979) as :

$$C_k = \frac{\text{percentage of particles of the sand after being subjected to stress finer than } D_{1.0} \text{ of the original sand}}{\text{percentage of particles of the original sand finer than } D_{1.0} \text{ of the original sand}}$$

where :

$D_{1.0}$ = particle diameter at which 10 percent of the soil by weight is finer.

The denominator of C_k is therefore by definition equal to 10.

The measure C_k as defined by Datta *et al.* (1979), together with other measures of particle breakage which have been proposed are shown in Fig.

4.5.

Leslie (1963) used the percent passing the sieve on which the original material was 100% retained, and later (1975) used the increase in percent passing the sieve on which 90% of the original sample was retained, which is the same as the Datta definition.

Marsal (1965) proposed a breakage measure based on the increase in percent passing a single sieve size. The increase in percent passing is calculated for the sieve diameter suffering the greatest increase.

Lee and Fahroomand (1967) proposed a breakage measure which is related to the particle size scale instead of the percent finer scale. The parameter was defined as the ratio of D_{15i}/D_{15a} , where :

D_{15i} = diameter for which 15% of original sample is finer

D_{15a} = diameter for which 15% of original sample is finer after crushing.

This ratio is determined by the horizontal distance between particle size distribution curves at 15% finer (Fig. 4.5).

In addition, the area between the grading curves for a sand before and after it has been subjected to shear have been used to quantitatively express the degree of crushing. The advantage of this definition is that it integrates the breakage occurring in the various size fractions of the material.

Beringen *et al.* (1982) investigated a large variety of crushing coefficients based on changes in particle size distribution. They found that the coefficient defined by Datta *et al.* (1979) produced the largest numerical increase for the majority of test data, and it would thus appear to be the most sensitive indicator of crushing. The sensitivity of crushing

GRAIN SIZE CURVES

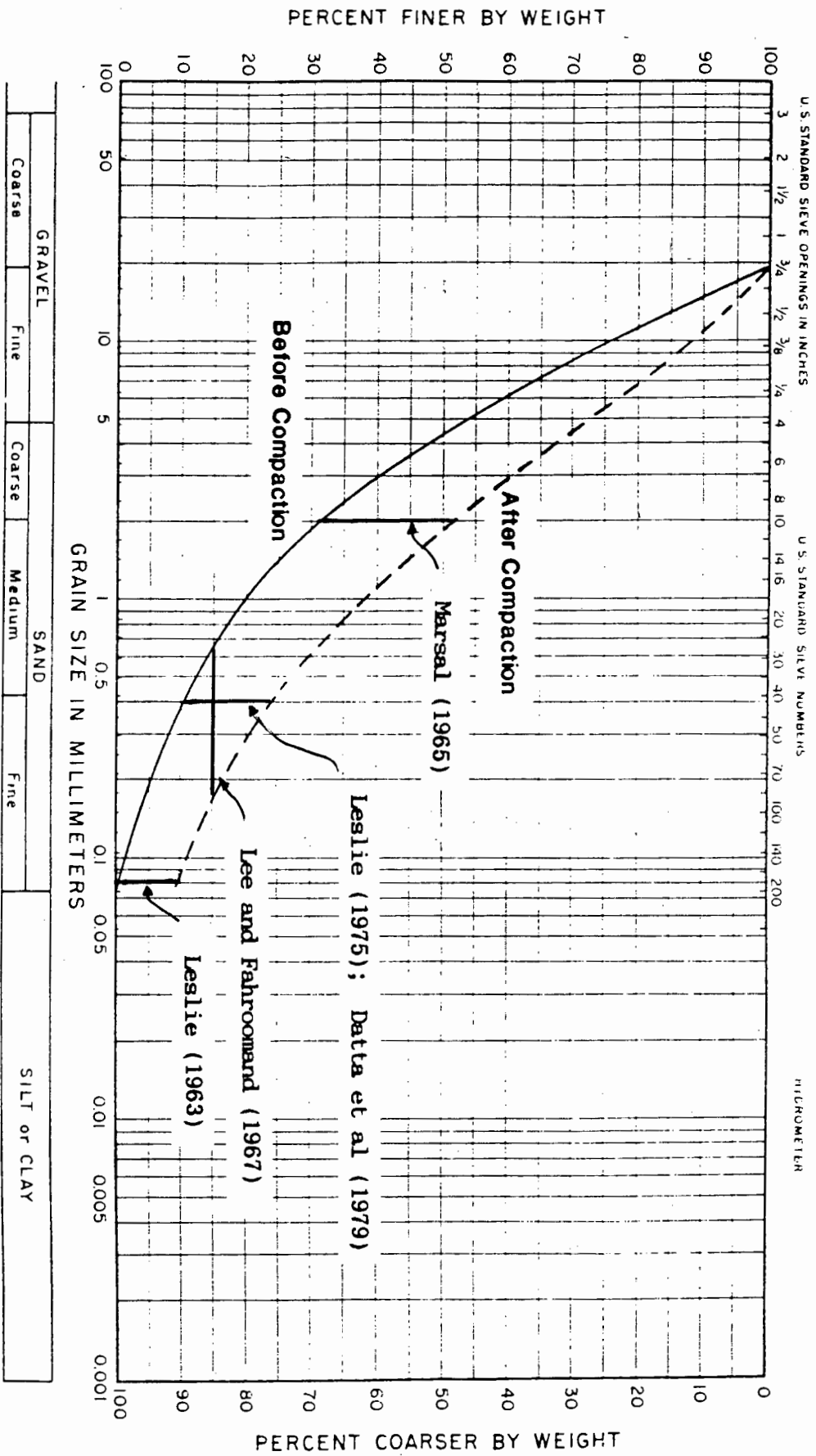


Figure 4.5 : Crushing coefficients as defined by various researchers

coefficient however appears to be a function of the particle size distribution before and after the test, and the Datta coefficient has been found to be an inappropriate measure by some investigators.

Datta *et al.* (1979) obtained values of C_k up to three and higher for coarse sands containing little fines. For calcareous sands with a high fines content, a coefficient based either on D_{60} or D_{80} appears to give more sensitive results. Ertec Western Inc. (1983) used a "fines content", defined as the percent of soil by weight finer than the 200 mesh sieve, since D_{10} for the soil tested was finer than the 200 mesh sieve, so that its determination would have required hydrometer analysis.

The quantitative measure of particle breakage is clearly dependent on the definition of the crushing coefficient, so that if this parameter is to be used as an "index" for the soil, attention should be given to appropriate and standardized expressions for the amount of crushing.

4.3.4 Correlation of Crushing with Engineering Behaviour

The tendency of the individual particles of carbonate soils to crush under relatively low stresses can significantly influence their engineering behaviour (Section 4.3.1). Although particle crushing has an important effect on engineering properties, its usefulness as an index property remains limited due to the difficulty of quantifying this effect in a consistent and rational manner.

Angemeer and McNeilann (1982) suggested that crushing could be quantified by an index test based on the change in particle size distribution after the sample has been reworked by some standard mechanical energy input. This is

a useful suggestion, which recognises the necessarily arbitrary nature of any such index test, leaving to consideration only the type of energy input to which the sample should be subjected.

The effect of crushing on carbonate soils is of interest primarily in cases where the soil is subjected to compressive loading, cyclic and shear stresses and impact loads created by pile driving. The extent of crushing that can be expected under each of these loading conditions vary considerably. It would therefore be more useful to relate the amount of particle crushing to the effect that it has on the engineering property of interest.

It has been established by a number of researchers (Poulos *et al.*, 1982; Datta *et al.*, 1982) that one of the most significant manifestations of crushing in terms of strength behaviour is the decrease of the angle of internal friction of the material with increasing confining pressure, i.e. a curved Mohr failure envelope results. Poulos *et al.* (1982) found that variation of ϕ' with σ_c for Bass Strait sands approximated well to a relationship of the type :

$$\phi' = a - b \log (\sigma_c)$$

where

$$\sigma_c = \text{confining pressure in kPa}$$

$$a, b = \text{constants, depending on the soil type.}$$

Datta *et al.* (1979, 1982) performed triaxial tests on calcareous soils (coarse grained) and found a relationship between his crushing coefficient, C_k , and the relative curvature of the Mohr envelope at different confining pressures. The advantage of this method is that curvature effects due to

crushing alone (c.f. curvature due to sample size effects) is isolated by the independent measurement of the amount of particle breakage. A more detailed account of the procedure followed and results obtained by Datta *et al.* (1979, 1982) are given below.

Material and testing apparatus

The physical characteristics of the sands (4 calcareous, 1 quartz) are shown in Table 4.3 and particle size distributions shown in Fig. 4.6. All tests were performed at the maximum relative density, using the triaxial testing apparatus, and confining pressures ranging from 98 to 6280 kPa.

Procedure

- (i) The crushing coefficients at different confining pressures under
 - consolidation only, and
 - consolidation followed by shear
 were determined for each sand (Figs. 4.7 and 4.8). From Fig. 4.7 it can be seen that crushing during shear was substantially more than crushing during consolidation.
- (ii) The stress-strain and volume change behaviour for each sand were recorded and Mohr failure envelopes were drawn using the peak stresses at failure (Figs. 4.9 and 4.10).
- (iii) To isolate the effect of crushing on the maximum principal effective stress ratio $(\sigma_1/\sigma_3)_{\max}$, C_k was plotted against k/k_1 (Fig. 4.11)

$$k = \phi'_{\max} \text{ or } (\sigma_1/\sigma_3)_{\max} \text{ at any given } \sigma_c$$

$$k_1 = \phi'_{\max} \text{ or } (\sigma_1/\sigma_3)_{\max} \text{ corresponding to } \sigma_c \text{ at which little or no crushing occurred (98 kPa in this case)}$$

$$k/k_1 = \text{normalised principal effective stress ratio.}$$

- (iv) The relationship C_k vs k/k_1 has been plotted on a log-log scale (Fig. 4.12), reducing it to a linear form which could be expressed mathematically as :

$$k/k_1 = (C_k)^{-0.6} .$$

- (v) Defining S_c as C_k at the maximum confining pressure (6280 kPa), Datta *et al.* (1982) plotted S_c against $\Delta \phi'$, where $\Delta \phi'$ is the difference in the secant angle at the lowest and highest confining pressure (Table 4.4). This relationship for the five sands is shown in Fig. 4.13, which illustrates the tendency of the Mohr envelope to curve with increasing susceptibility to crushing.

4.4 PARTICLE SIZE DISTRIBUTION

4.4.1 Engineering Significance of Particle Size Distribution

Particle size distribution of carbonate soils can be used for two purposes :

- Index test : As for conventional soils, the engineering properties and behaviour of carbonate soils will to a large extent be influenced by the distribution of particle sizes present (e.g. void ratio, crushability, shear strength and compressibility).
- Evaluation of crushability : The amount of crushing under stress can be estimated from the change in particle size distribution before and after application of the stress. Correlation between crushing and engineering behaviour requires the accurate determination of the amount of particle breakage which occurred under the loading condition of interest.

Table 4.3 : Physical properties of the sands tested by Datta et al.

Sand	Carbonate content per cent	Particle characteristics	Particle size	C_u *
A	92.2	Plate-like shell fragments, angular to subrounded particles with large intraparticle voids	Coarse	1.50
B	93.7	Plate-like shell fragments, subangular to subrounded particles with small intraparticle voids	Coarse to medium	2.11
C	91.3	Same as Sand B	Medium to coarse	2.12
D	95.0	Subrounded coralline debris particles with small intraparticle voids	Medium to fine	1.53
E	-	Quartz, rounded particles	Medium to coarse	1.33
F	90.1	Rounded and oval nonkeletal particles	Medium to fine	1.90

* coefficient of uniformity

Table 4.4 : Decrease in secant angle of shearing resistance, ϕ_s , with increase in confining stress (Datta et al. (1979))

$\bar{\sigma}_{3c}$ (kg/cm ²) #	ϕ_s (deg)				
	Sand A	Sand B	Sand D	Sand F	Sand E
1	51.0	49.8	50.6	43.0	40.5
4	42.6	44.5	47.3	-	39.5
8	38.5	42.0	45.0	39.0	-
16	33.0	38.7	40.4	-	38.7
64	30.3	31.6	33.9	34.9	33.5
$\Delta\phi_s$ *	20.7	18.2	16.7	8.1	7.0

* $\Delta\phi_s = (\phi_s \text{ at } \bar{\sigma}_{3c} = 1 \text{ kg/cm}^2) - (\phi_s \text{ at } \bar{\sigma}_{3c} = 64 \text{ kg/cm}^2)$

$1 \text{ kg/cm}^2 = 98 \text{ kPa}$

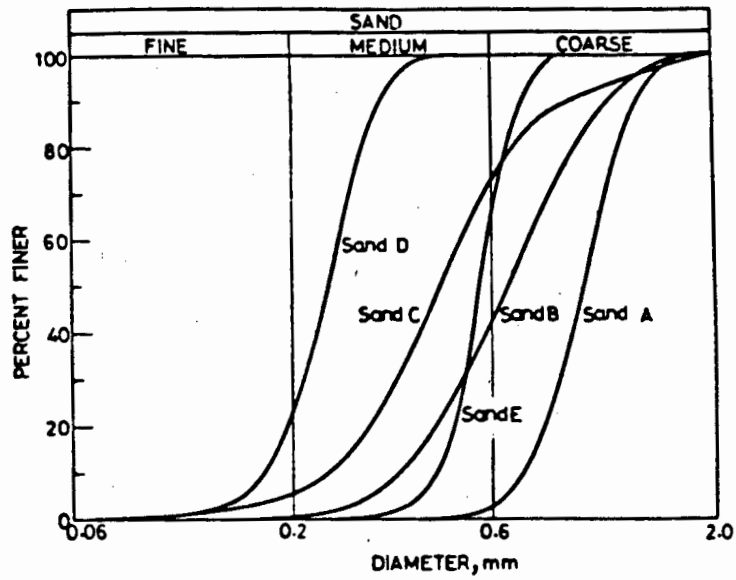


Figure 4.6 : Grain size distribution curves of the sands tested (after Datta et al, 1979)

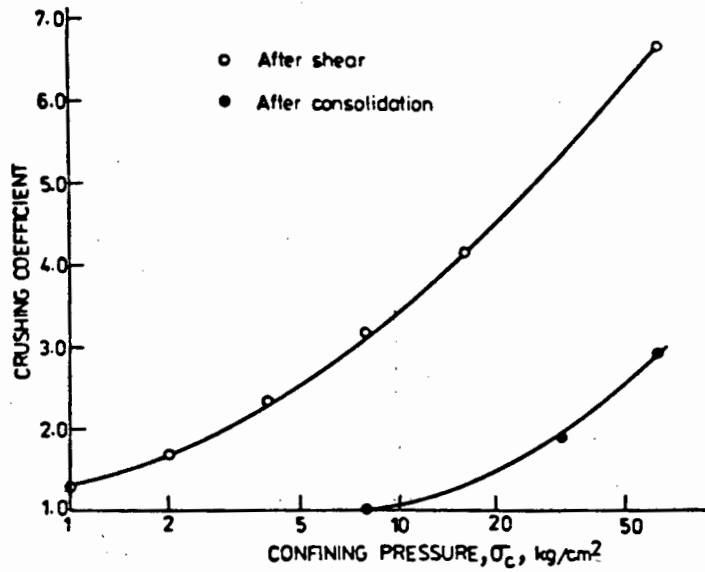


Figure 4.7 : Effect of confining pressure on crushing coefficient, after consolidation and after shearing, for sand A (after Datta et al, 1979)

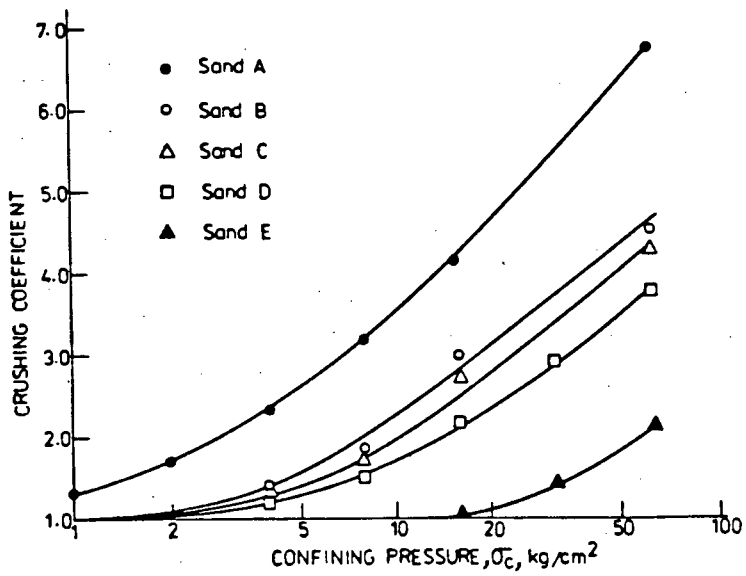


Figure 4.8 : Effect of confining pressure on the post-shear crushing coefficient (after Datta et al, 1979)

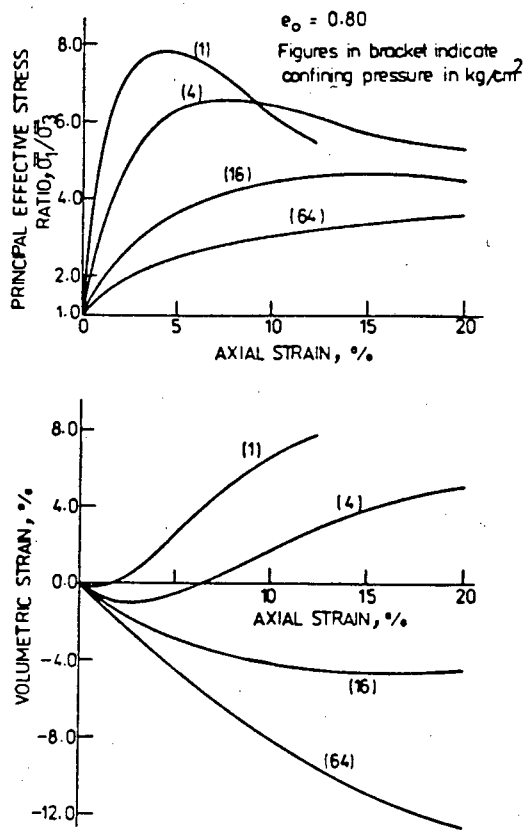


Figure 4.9 : Typical stress strain-volume change behaviour for calcareous sand (after Datta et al, 1979)

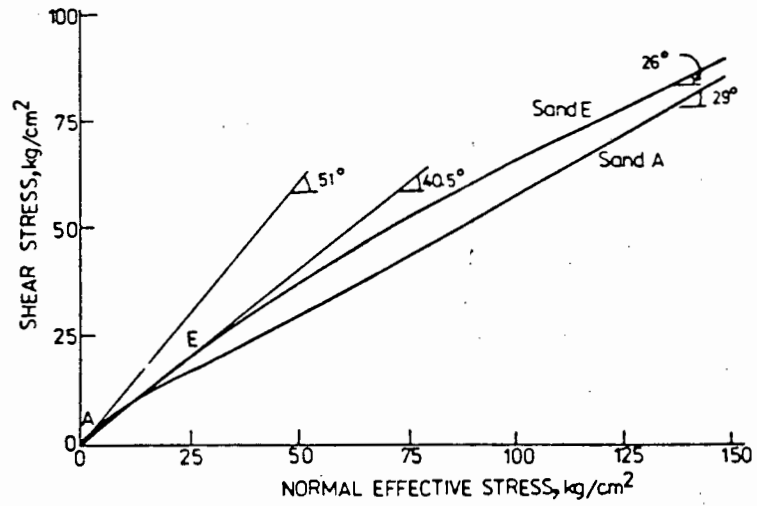


Figure 4.10 : Mohr failure envelopes for sands A and E
(after Datta et al, 1979)

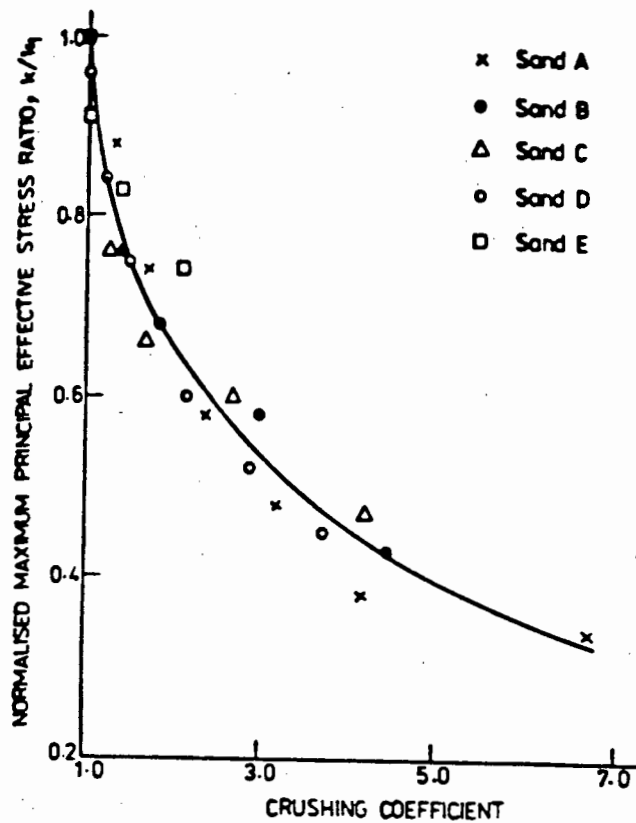


Figure 4.11 : Relation between crushing coefficient and normalised maximum principle effective stress ratio
(after Datta et al, 1979)

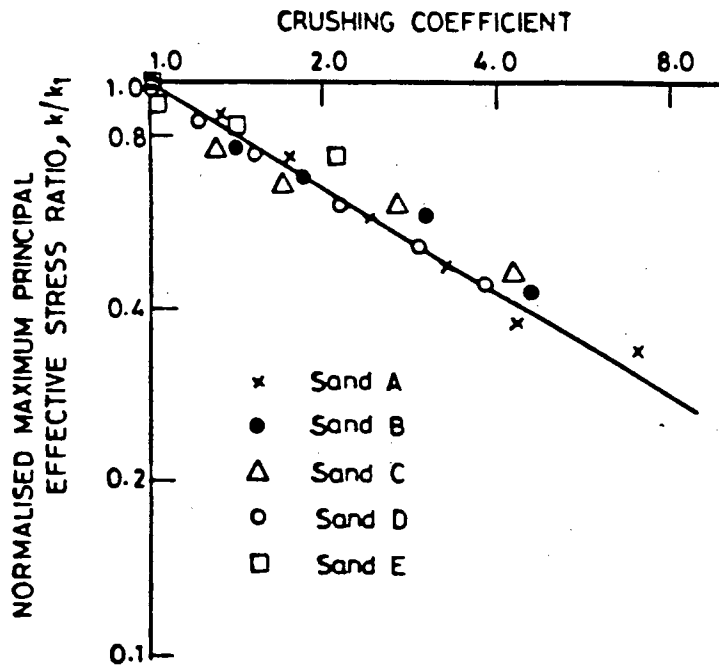


Figure 4.12 : Relationship between crushing coefficient and normalised maximum principle effective stress ratio (after Datta et al, 1979)

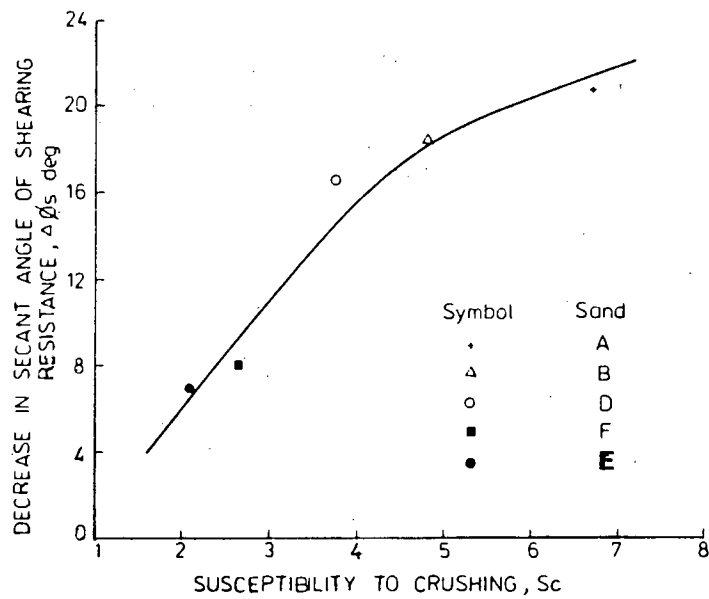


Figure 4.13 : Relationship between decrease in secant angle of shearing resistance and susceptibility for crushing (after Datta et al, 1982)

Because carbonate particles are soft and often have complex and fragile shapes, they are more sensitive to errors and difficulties of interpretation arising from the use of sieves and pipette/hydrometer techniques for determination of size distributions. The nature and sources of potential errors and variability for these techniques as applied to carbonate soils are discussed in the following section.

4.4.2 Sieve Analysis

Standard methods of sieve analysis as described in ASTM, BSS, TMH, etc., are normally used for carbonate soils. For these soils, the potential for error and variability is due to material properties, the principle of sieving and the procedure.

(i) Principle of sieving

Sieves sort grains not only according to size but also according to shape (e.g. Komar and Cui, 1984), as can be deduced from the response of spherical and elongated grains :

- the largest sphere that can pass through a sieve has a diameter equal to the mesh, whereas a needle of any length, theoretically, can pass through the sieve provided that its two smaller dimensions are less than the maximum dimension of the mesh;
- if size is defined in terms of some average diameter, sieves do not make a sharp distinction, because for non-spherical shapes the maximum length has no direct bearing on passage through the sieve.

Ludwick and Henderson (1968) studied the effect of shape variation on results obtained by sieving. They concluded that if the screen opening is

equated with particle size, the intermediate diameter of particles in each sieved fraction is frequently underestimated by 10 to 20%. Few natural soils are composed of perfectly spherical particles, so that the grading of most soils is therefore affected by particle shape. Carbonate soils however are often composed of particles with a wide range of unusual shapes not found in conventional soils. Different shape particles will be affected by sieving to a different extent, so that a size analysis can result in sorting according to mean diameter and "average particle size" in an unknown ratio.

The effect that particle shape could have on sieving results has been found to be so severe that some researchers have suggested that special sieving techniques can be used to deduce the shape distribution of particles (Section 4.5). This aspect could have important implications, both for the assessment of crushing potential from the grading curve and the measurement of the amount of crushing, since:

- flat, elongated particles (these commonly occur in carbonate soils in the form of shell fragments) could be more prone to crushing than implied by the sieve size on which they are retained;
- measurement of the amount of breakage of such particles will be affected by the relative sizes of the resulting fragments (i.e. fragments of particles with "lengths" greater than the "diameters" of the uncrushed particles will be retained on the same sieves as the uncrushed particles).

Quantification of the amount of crushing from the results of a sieve analysis (Section 4.3.3) could therefore be affected by the particle size and shape distribution before and after stressing.

Lowrison (1974) suggested that the amount of crushing can be estimated by evaluating the increase in surface area of the particles after crushing. The most widely used technique is the BET method which involves adsorption of an inert gas such as nitrogen on the carbonate surface. The disadvantage of this method is that it is a complicated, expensive and difficult procedure compared to sieving, and therefore not suitable for use on a routine basis. It could nevertheless be used to study the importance of the effects outlined earlier on crushing estimation of carbonate soils.

(ii) Experimental procedure

(a) Reproducibility and errors

The factors affecting reproducibility are the make and precision of calibration of the screens used for the sieves, the size of the sample, the type of mechanism used to shake the sieves, and the length of time of shaking (Blatt *et al.*, 1980).

Walker (1941) investigated the reproducibility of sieving sand. Repeated analysis of the same sample in the same and in different laboratories gave significant variations in results. Folk (1955) and Rogers (1965) showed that sieving is capable of yielding results of high precision, provided that standard techniques and equipment are used. (They used U.S. Standard or Tyler sieves, a sample size of about 30g and shaking time of 10 to 15 minutes on the Ro-Tap shaker).

Carbonate soils are more sensitive to variations in testing techniques than conventional soils (e.g. Netterberg, 1982), so that for standardisation of crushing measurements, the following aspects should be considered :

- Standard equipment : standard sieves with different size openings (e.g. U.S. and British) could result in different numerical values of crushing coefficients for the same sample.
- Standard procedure : the size of sample, shaking time and method of shaking.

(b) Shaking time and method

Standard recommended shaking time, normally between 10 and 20 minutes, is arrived at as a compromise aimed at avoiding wear of sieve mesh and particles, which tends to take place beyond 20 minutes, and completeness of separation, which follows a law of diminishing returns (Griffith, 1967).

For carbonate particles, being relatively soft and more prone to abrasion due to their fragile shapes, wear of particles could be significant at lesser shaking times than those for quartz based sands. On the other hand, completeness of separation for complex shaped particles will probably require increased shaking times. Hand shaking versus automatic (Ro-Tap) could be an additional source of variation.

4.4.3. Particle Size Analysis for Cemented Material

Where cementation is present, grain size distribution is believed to be an unreliable indicator of the soil composition (Agarwal *et al.*, 1977). The problem in the case of calcareous soils arises from the difficulty of removing the cementing agent without damaging the actual particles. In the case of quartz based sands, the cementing agent can be removed by dissolution with hydrochloric acid. This technique is not suitable for calcareous soils since the soil particles themselves are susceptible to dissolution in acid.

A method used by Ertec Western Inc. (1983) consists of carefully rubbing off the cementing agent by hand, taking extreme care to prevent crushing of the individual grains. After separation of the grains, specimens were soaked in distilled water for 24 hours to remove cement fines on the surfaces. The method is time consuming and unsatisfactory in respect of grain crushing. However, it does provide qualitative indication of crushing.

4.4.4 Particle Size Analysis for Silts

The standard methods of particle size analysis for silts (pipette/hydrometer methods) uses the settling velocities of the particles (as determined for example by Stoke's law) to deduce the "size" of the particles. The "size" of the particle is taken as the diameter of a solid sphere having the same settling rate as the irregular sediment grain. Many carbonate particles in the silt to clay size range (e.g. foraminefera tests) are hollow and porous in addition to being non-spherical. These particles often float in the settling tube (Valent, 1974).

The fall velocities of the particles in the tube decrease with increasing irregularity in shape, for which Valent (1974) suggested that a correction procedure might be developed. Such a procedure would however be complicated if the soil particles contain a wide variety of shapes. In addition, further errors may be introduced if the specific gravities of different size particles vary appreciably (e.g. aragonite vs kaolinite). Komar and Cui (1984) found that the presence of heavy minerals can have a significant effect on the size distribution obtained with settling tube techniques.

An alternative method which might be used is the Coulter counter. This relies on the principle of electronically counting particles which flow

through an aperture. The frequency distribution of the particles are determined in several size classes, with the size registered being proportional to the volume of the particles, rather than the shape and specific gravity, as for pipette techniques. This feature makes the method ideal for particle size and crushing analysis of carbonate silts.

In general, higher stresses are required to cause crushing as the particle size decreases. However, for silt sized hollow and porous particles, crushing may have a significant effect even at relatively low stresses. Measurement of the degree of crushing for these materials has been difficult. Microscopic techniques can be used for a qualitative assessment, but particles are liable to be crushed during sampling preparation. It appears that the Coulter counter, with sizing effects dependent on grain volume, may offer an improved technique for the study of crushing in the silt size range.

4.5 ATTERBERG LIMITS

4.5.1 Engineering Significance of Atterberg Limits

The engineering classification systems which have been proposed (Section 3.2) do not include the determination of Atterberg Limits. It is therefore of some interest to examine the significance of these indices for the classification/description of carbonate soils. For example, Atterberg Limits could possibly be used to provide an indication of the carbonate contents beyond which it ceases to have a beneficial effect (e.g. compressibility, shear strength) on calcareous clays, an aspect about which considerable uncertainty remains.

Calcrete, a carbonate material formed by chemical precipitation of CaCO_3 , is widely used as a road construction material in Southern Africa, and considerable data on its Atterberg Limits are available. A plot of such data on the LL-PI chart, as determined by Netterberg (1982) is shown in Fig. 4.14. The calcretes tend to fall on either side of the "A-line" and they generally have higher liquid limits compared to their plasticity indices than other soils of South Africa; i.e. there is a greater tendency to lie closer or below the A-line. Unfortunately, the CaCO_3 contents of materials shown in Fig. 4.14 were not given, so that it is not possible to examine the correlation between CaCO_3 content and position with respect to the A-line. However, Netterberg states that this behaviour (low PI compared to LL) can probably be attributed to particle porosity and the presence of silt size particles and diatoms, effects which tend to reduce plasticity.

Demars *et al.* (1976) determined the Atterberg Limits for calcareous clays (CaCO_3 from 10 to 90%) from offshore Britain and East Africa. Results plotted on the LL-PI charts are shown in Figs. 4.15 and 4.16. Again, results are scattered on both sides of the A-line, with most specimens with CaCO_3 content $> 40\%$ falling below the A-line. Demars *et al.* therefore used 40% carbonate content to distinguish between "high" and "low" carbonate contents (i.e. granular vs. cohesive behaviour).

Additional Atterberg Limit data for carbonate soils from both the marine environment and arid regions, together with CaCO_3 contents, are presented in Table 4.5 and plotted in Fig. 4.17.

The data presented in Table 4.5 and shown in Figs. 4.14 to 4.17 appear to indicate the following :

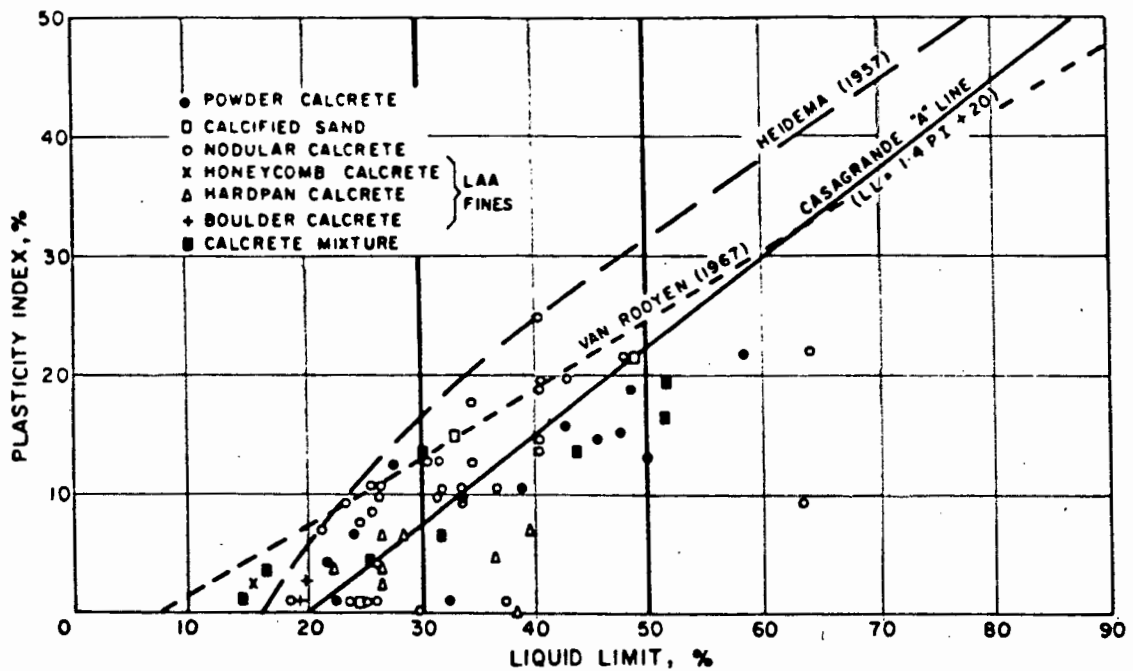


Figure 4.14 : Position of calcretes on the Casagrande plasticity chart (after Netterberg, 1982)

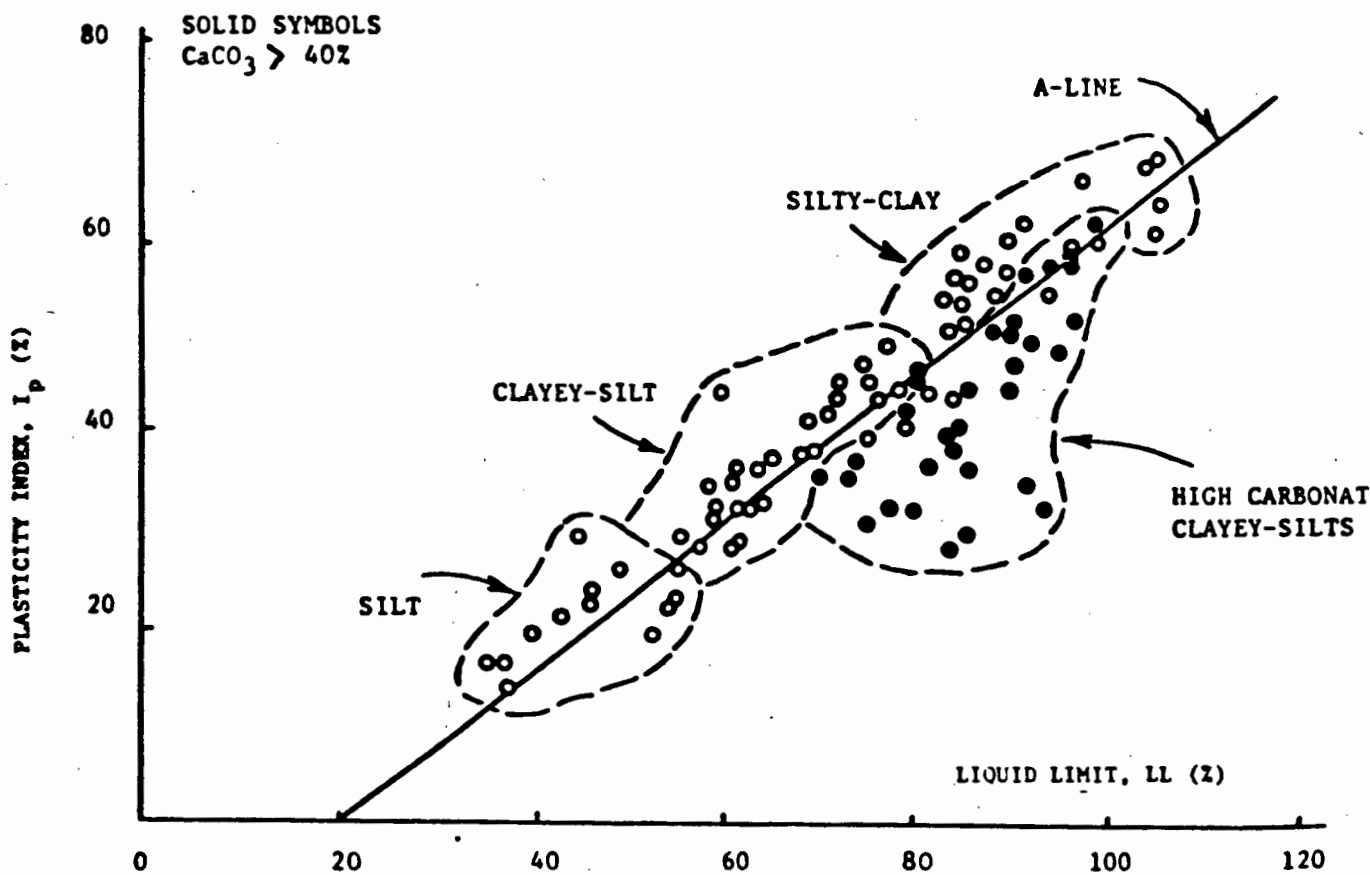


Figure 4.15 : Plasticity chart for Northeast Atlantic test specimens (after Demars et al, 1976)

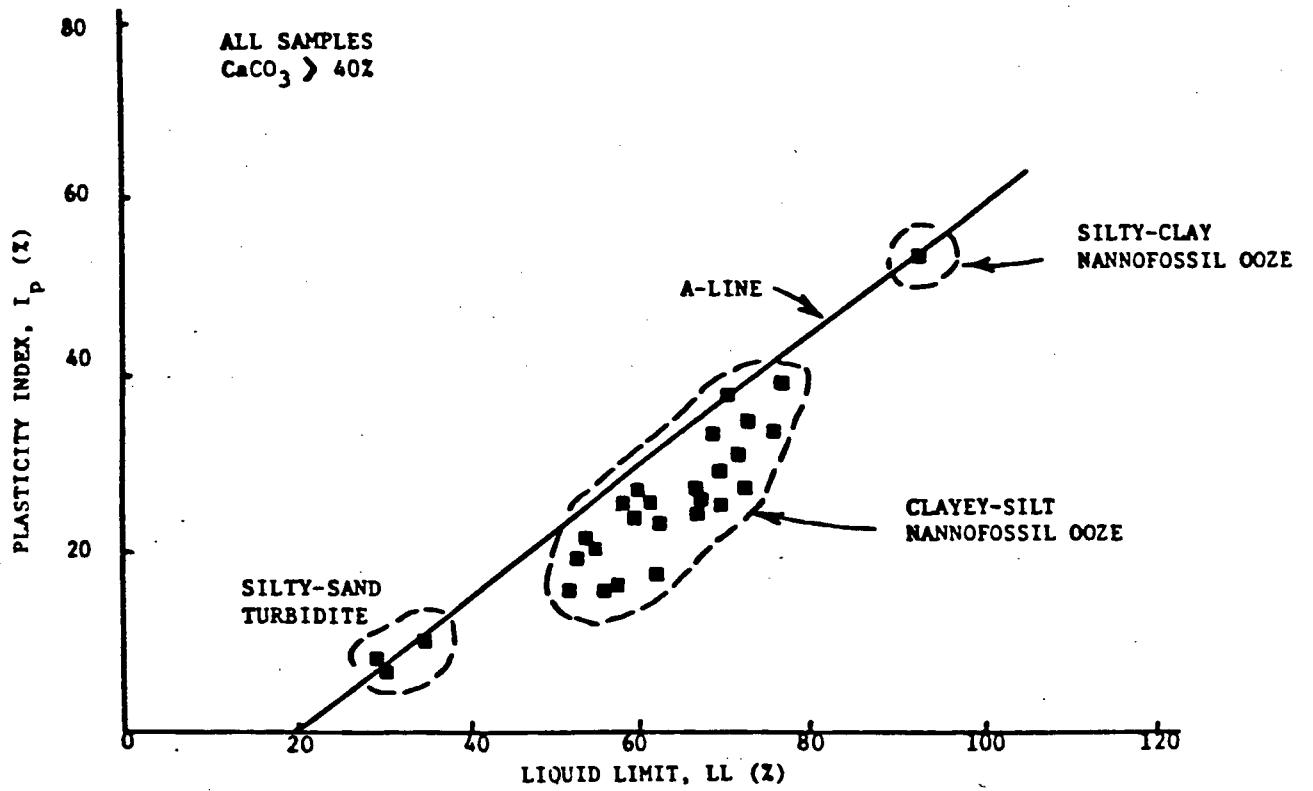


Figure 4.16 : Plasticity chart for Canary Basin test specimens (after Demars et al, 1976)

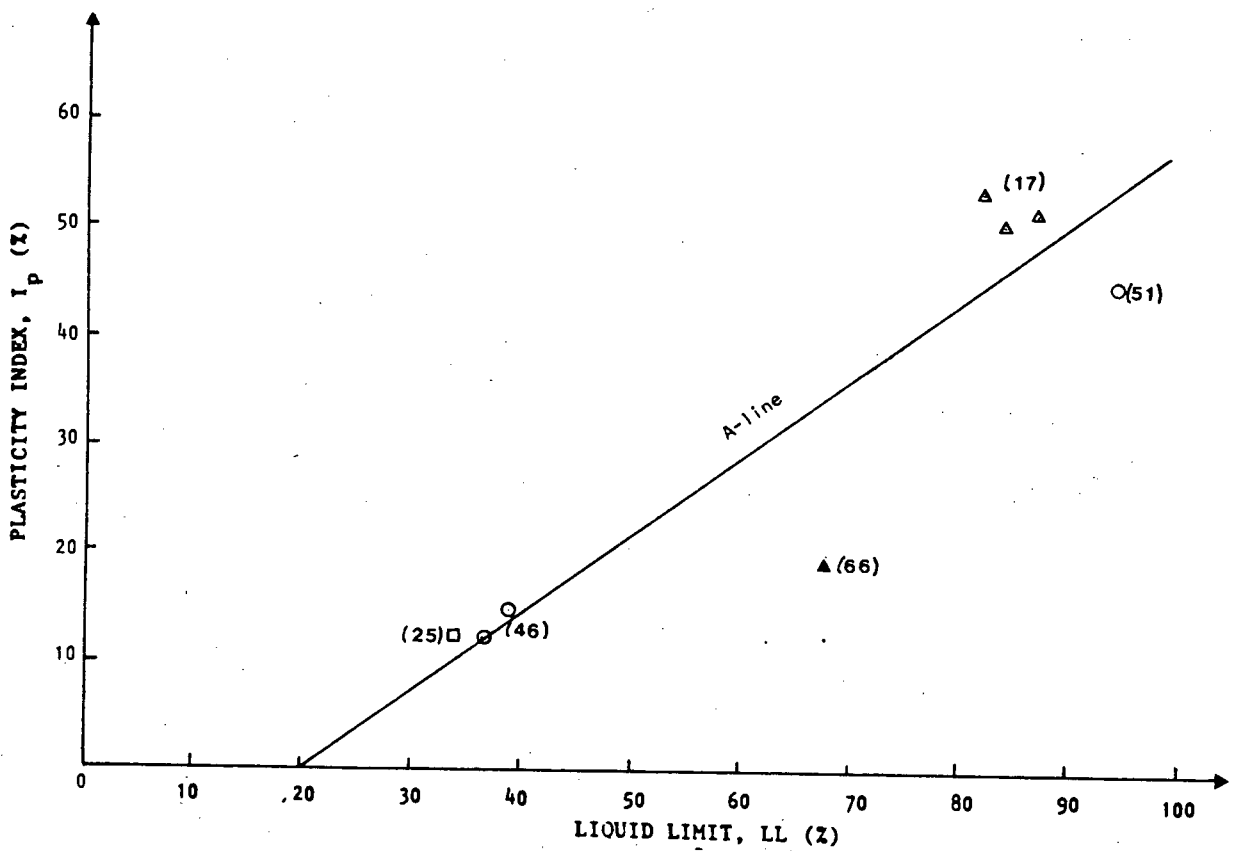


Figure 4.17 : Atterberg limits and carbonate contents (brackets) for sediments in Table 4.5.

Table 4.5 : Atterberg limits and carbonate contents for land and marine carbonate soils

Reference	LL	PL	PI	% CaCO ₃	Remarks	Symbol in Fig. 4.17
McKown and Ladd (1982)	87	36	51	16.8	Calcite cemented shale	△
	84	34	50	17.0		
	82	29	53	17.4		
Olsen and Leonard (1982)	34	34	12	± 25	Calcite-shale matrix	□
	84		50	17.0		
Datta <i>et al.</i> (1982)	94.3	49.5	44.8	51	Uncemented marine calcareous sediment	○
	36.7	24.7	12.0	46.5	Cemented marine sediment : Removal of carbonate fraction gave increased LL and PI	○
	39.0	24.5	14.5	45		
Valent <i>et al.</i> (1982)	68 (ave)	49.5 (ave)	18.5	65.5 (ave)	Uncemented calcareous ooze	▲
Bemben (1982)				20 - 25	Plot both above and below A-line, depending on whether clay or silt fraction dominates	
Clayton (1983)				86 - 98	Average of 200 tests on chalk; plot immediately above A-line, predominantly in CL region (USC)	

- Sediments with CaCO_3 contents in excess of 20% generally plot close to or below the A-line, irrespective of their origin (arid or marine).
- The relative positions on the LL-PI charts do not appear to be controlled by CaCO_3 content alone, but rather by a combination of the nature and composition of both the carbonate and non-carbonate components.

A laboratory controlled study of the effect of carbonate content on Atterberg Limits and the associated strength and stress-strain behaviour could provide valuable additional insight into the use of these indices to delineate cohesive and granular behaviour.

4.5.2 Correlation with Compressibility

Several empirical relations have been developed for correlating Atterberg indices with the compression index (C_c) of soils. For example, Terzaghi and Peck (1967) proposed the following relationship for conventional terrigenous soils :

$$C_c = 0.009 (LL-10) \quad . \quad (1)$$

Herrmann *et al.* (1972) found that for the same LL, marine clays are more compressible, and altered relationship (1) as follows :

$$C_c = 0.011 (LL-12) \quad . \quad (2)$$

There is evidence to suggest that calcareous oozes differ from terrestrial soils and other marine clays in their compression behaviour. Demars *et al.* (1982) found that oozes containing particles with intraparticle voids (e.g.

forams and nanofossils) could give misleading results. The measured water contents of such sediments for the determination of their Atterberg Limits include both the intraparticle and interparticle water contents, but it is only the interparticle water which is normally associated with the strength and compression behaviour of the soil. They recommended that compression index correlations should be based on the plasticity index instead, which nullifies the effect of intraparticle voids. They proposed the following relationship, which provided good correlation for their samples :

$$C_c = 0.024 + 0.014 \text{ PI} \quad . \quad (3)$$

A similar phenomena was noted by Valent *et al.* (1982) for Caribbean calcareous oozes. However, they found that for their samples expression (3) gave even poorer correlations than the conventional correlations based on void ratios and liquid limits.

4.5.3 Determination of Atterberg Limits

Atterberg Limits for carbonate soils are determined using the standard methods (BSS, ASTM, TMH) as for conventional soils. The four-point cone penetration method for liquid limits as set out in the new British Standard (BS 1377 test 2(a)) is also often used.

Large variability and scatter in results are common, especially for the determination of plastic limits of sediments with high carbonate contents (e.g. Bembem, 1982). Clayton (1983) found that Atterberg Limits for chalk are difficult to determine because the material contains little colloiddally active fines, and dries out rapidly during a test.

4.6 MASS PHYSICAL PROPERTIES

4.6.1 Engineering Significance

As for non-carbonate soils, the mass physical properties (bulk density, void ratio, etc.) will have a significant influence on the engineering behaviour of the soil :

- Soils with high void ratios will tend to be more compressible than soils with low void ratios.
- The shear strength and deformation moduli of a given soil could depend on its relative density.
- Although there is no definite relationship, soils with high void ratios will tend to have higher permeability. This could influence the rate of dissipation of pore water pressure.
- Particle crushing has been shown to be influenced by void ratio.
- The development of lateral pressure in driven piles is influenced by the volume change behaviour of the soil during shear, which in turn is influenced by the void ratio.

Although mass physical properties are not included in the classification systems which have been proposed, nor in the system of description recommended by Datta *et al.* (1982), they appear to be relevant, and are discussed below.

4.6.2 Definition and Measurement of Carbonate Porosity

(i) Definition of carbonate porosity

The 'total porosity' of a carbonate soil is defined, as for non-carbonate soils, as the ratio of the volume of internal open spaces to the total

volume of the sample :

$$n = V_v/V$$

where:

n = porosity

V_v = volume of voids (interparticle and intraparticle)

V = total volume.

The void ratio for carbonates is similarly expressed as the ratio of the total volume of the voids to the volume of the solids in the sample :

$$e = V_v/V_s$$

where:

e = void ratio

V_s = volume of solids.

Three types of porosity can be identified in carbonates :

- intergranular porosity : consists of openings between crystals or discrete particles; pores are interconnected;
- intragranular porosity : pores are isolated from each other;
- fracture porosity : consists of large openings through otherwise solid masses. Fracture porosity is of interest only in rock types.

In uncemented silica sands, the total porosity consists of intergranular porosity. However, in sediments composed of grains of biological origin, such as calcareous oozes, intraparticle voids can form a large fraction of the total porosity. The intratest porosity of some foraminefera has been estimated to contribute as much as 35 to 44 percentage points to the total porosity (Bachman, 1984).

The water contained in the voids created by the hollowness of these tests and shells do not participate as pore fluid reactions until crushing of the particles has occurred, upon which significant amounts of pore fluid is released. Presence of intratest water can have important implications for the determination and interpretation of some engineering index tests (Section 4.5).

(ii) Measurement of carbonate porosity

Three aspects of relevance to carbonate soils will be considered :

- laboratory measurement of total void ratio
- laboratory measurement of intraparticle voids
- in situ determination of density or void ratio.

(a) Laboratory measurement of total void ratio

As for conventional soils, the total void ratio of a sample can be calculated from a knowledge of the specific gravity of the solid particles and the mass and bulk dimensions of the sample, using the following relationship :

$$\begin{aligned}
 e &= V_v/V_s \\
 &= (V - V_s)/V_s \\
 &= (V - m_s/G_s)/(m_s/G_s)
 \end{aligned}
 \tag{1}$$

where:

- V = total sample volume
- V_s = total solids volume
- G_s = particle specific gravity
- m_s = dry mass of the sample.

The total sample volume (V) and mass (m_g) can be easily measured in the laboratory for all soil types, with a high degree of accuracy, while an assumed specific gravity of 2.65 is normally used for quartz sands with little resulting error.

The specific gravities of pure carbonate minerals are considerably higher (2.72 - 3.0) than that of quartz. Average specific gravities of natural carbonate soils varying between 2.65 (e.g. Poulos, 1984) and 2.85 (e.g. Ertec Western Inc., 1983) have been recorded. If an accurate determination of void ratio is required, an estimated specific gravity for carbonates can give appreciable error. Because the mineralogy and composition of carbonate soils can vary considerably within short distances, it could therefore be necessary to determine a representative specific gravity for each sample from a specific site.

(b) Measurement of intraparticle voids

A knowledge of the intraparticle void volume is useful for :

- correlation of sediment behaviour with engineering properties (e.g. strength, susceptibility to crushing, consolidation);
- correlation with acoustic properties (studies by Hamilton *et al.* (1982) have indicated that foraminiferal sediments react to the passage of sound waves as if they were composed of solid particles. Acoustical correlations are required for geophysical techniques of site investigation).

Routine laboratory methods of analysis determine only the total porosity. Techniques for determining the volume of isolated pores in rock (e.g. Carver, 1971) are available, but no simple technique for

loose sediments appear to be available, so that indirect techniques have to be used :

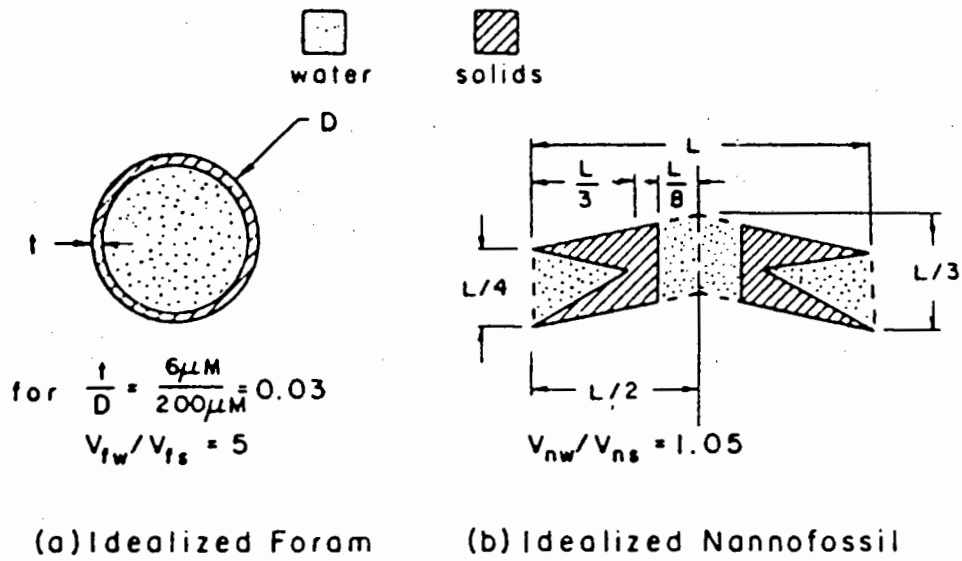
- Demars *et al.* (1982) estimated the intraparticle porosity from simplified geometrical considerations of the particle structures, which were inferred from images obtained with a Scanning Electron Microscope (Fig. 4.18).
- Bachman (1984) developed a technique which allows for the quantitative determination of inter-test porosity through indirect measurements. However, the reliability of the method appears to be dependent on structural features of the foram particles, and did not give good results with forams having porous shell-walls.

(c) In situ determination of void ratio and density

See Section 6 (Site Investigation Techniques).

4.6.3 Densities and Void Ratios of Carbonate Sediments

A characteristic of particular engineering significance displayed by carbonate sediments is the wide range of void ratios and associated bulk densities that occur. Typical void ratios and densities for some carbonate and non-carbonate sediments in the natural and remoulded states are given in Table 4.6(a) and (b). Relatively high void ratios, untypical of non-carbonate soils, are common. In addition, the range of possible void ratios (0.44 to 4.8) is much larger than would be anticipated for quartz based sands. These phenomena can be explained by considering the role of diagenetic processes and inherent material properties in the formation of pore spaces in carbonate sediments.



$V_w = 85$	$V_{bw} = 47$	BULK H ₂ O	$W_{bw} = 47$	$W_w = 85$
	$V_{nw} = 27$	NANNO H ₂ O	$W_{nw} = 27$	
	$V_{fw} = 11$	FORAM H ₂ O	$W_{fw} = 11$	
$V_s = 36.5$	$V_{bs} = 14.5$	NON CaCO ₃ BULK SOLID	$W_{bs} = 40$	$W_s = 100$
	$V_{ns} = 19.6$	NANNO CaCO ₃	$W_{ns} = 54$	
	$V_{fs} = 2.2$	FORAM CaCO ₃	$W_{fs} = 6$	

Figure 4.18 : Estimate of intraparticle porosity inferred from geometrical considerations (after Demars et al, 1982)

Table 4.6a: In situ void ratios of calcareous sediments

Reference	Sediment Type	Void Ratio
Bryant et al (1974)	Calcareous	1,7 - 3,8
Poulos (1980)	Calcareous sand	1,05 - 1,35
Parker (1972)	Deep sea carbonate sediment	1,5 - 2,0

Table 4.6b: Limiting void ratios of quartz and calcareous sands

Reference	Sediment type	Void Ratio	
		Loose	Dense
(Ertec Western Inc.) (1983)	Calcareous sand	2,0	1,4
	Quartz sand	0,75	0,60
Mcarol & Beard (1984)	Calcareous sand	1,44	1,19
	Quartz sand	0,90	0,73
Vinopal and Coogan (1978)	Anomia particles (calcareous)	4,79	3,45
Airy et al (1988)	Calcareous sand	1,48	1,12
	Calcareous sand	1,09	0,79

(i) Diagenetic factors affecting porosity

As sedimentation progresses, the fabric of a sediment is altered through compaction, reducing the primary intergranular porosity. Inhibitory factors such as early cementation preserve original grain fabric by preventing compaction. The extent to which cementation proceeds will depend on factors such as the pore water pressure and chemistry. Precipitation of cementing material can therefore also act as a primary porosity reducing agent. In some cases the cement completely fills the voids. The opposite process of dissolution can increase porosity by dissolving cement, carbonate particles, or both.

(ii) Inherent controls affecting porosity

The inherent factors controlling the porosity of a sediment are discussed by Vinopal and Coogan (1978). These factors reflect the physical and biological parameters that define the sedimentary environment, for example :

- the abundance, availability and type of contributing organism
- the growth size distribution of the organisms
- the broken size distribution of the contributing organisms.

(a) Grain size

For an assembly of perfect spheres, the void ratio can be computed from the geometry of the packing alone, and should theoretically be independent of the size of the spheres, provided that equal-sized spheres are used for each state of packing. However, data from Ellis and Lee (1919) shows that as the grain size decreases, the void ratio increases. This is believed to be a result of friction, adhesion and bridging effects which increase in finer soils as a consequence of the increasing ratio of grain surface area to grain mass, causing looser packing (Vinopal and Coogan, 1978).

(b) Grain size distribution

As the uniformity of grading decreases, the resulting void ratio also decreases, as smaller particles have the opportunity of filling voids between larger particles. Where the percentage of small sizes is large, the void ratio can be increased as larger grains are forced apart by smaller grains. The relative importance of the two effects is a function of the grain size ratio and percentage of each size component. Experiments by Fraser (1935) indicated that void ratio decrease is to an extent a function of the size sorting of sands, although he showed that it was not possible to predict void ratios merely from a knowledge of the various particle size components.

(c) Packing and orientation

Different void ratios can result, for identical particles, from differences in the spacing between individual particles. Dimensional orientation of platy and elongated grains, resulting from preferred orientation of their long axes, can have an extreme influence on the resulting interparticle void spaces. This dependence of void ratio on particle orientation gives rise to the concept of relative density (see Section 4.6.5).

(d) Particle shape

Compared to quartz based sands, carbonate particles are extremely diverse in their sphericity and roundness. In addition, many carbonate particles have porous structures and contain internal cavities. Most experimental work on the packing of materials and their void ratios has utilized compact spherical particles, while little information exists on the packing and void ratios of the irregular shaped particles found in natural carbonate soils. Such information that does exist however indicates that grain shape is the major inherent factor controlling the porosity of carbonate sands and gravels.

Vinopal and Coogan (1978) investigated the effect of particle shape on the packing of uncemented carbonate sands and gravels. This extensive study involved the measurement of grain volume (complement of porosity) for 250 single, dual and multi-component packs of varied shaped, particulate, natural and artificial sands and gravels.

Each of the random packs was assembled in two reproducible states of random packing, relative dense and loose. Particle shapes were divided into two classes on the following basis :

- simple shapes : grains which have morphologies that can be meaningfully described by means of the ratios of the long, intermediate and short axes, and
- radical shapes : particles with shapes that cannot be meaningfully described using the parameters of length, width and height.

The results obtained confirm the overriding importance of particle shape on the porosity of carbonate sediments. Some of the more important results and conclusions are summarised below.

- Measured void ratios by Vinopal and Coogan (1978) varied between the following limits :

<u>Shape</u>	<u>Loose</u>	<u>Dense</u>	<u>Shape class</u>
Anomia	4.79	3.45	Radical
8mm rods	0.53	0.44	Simple

- All particles within the void ratio range of 0.89 to 0.54 had simple shapes, while all particles with void ratios in the range of 4.8 to 1.56 had radical shapes.

- Simple shape packs showed little correlation between void ratios and relative percent difference between dense and loose packing. For simple shapes, the relative decrease in porosity from dense to loose packing was only 7 percent, while for some radical shapes this difference increased by up to 30 percent.

The void ratios that can be expected for simple and radical shape particles under different degrees of compaction are shown in Figs. 4.19(a) and (b).

4.6.4 Analysis of Particle Shape

The important influence of particle shape on the engineering properties of carbonate soils, both directly (e.g. crushing susceptibility) and indirectly (e.g. effect on mass physical properties) warrants its inclusion in an engineering description/classification system. The feasibility of such an index will however depend on the ease with which it can be measured.

Three principal grain-form parameters are in common usage (Pryor, 1971) :

- shape : defined in terms of the described spatial geometric form of grains
- roundness : describes the relative sharpness of grain corners and edges
- pivotability : a measurement based on the movement of the grains.

Analysis of grain shape is accomplished by four principal methods :

- (a) Visual description : The operator verbally describes grain morphology from visual observation of the grain or grain facsimile. This method

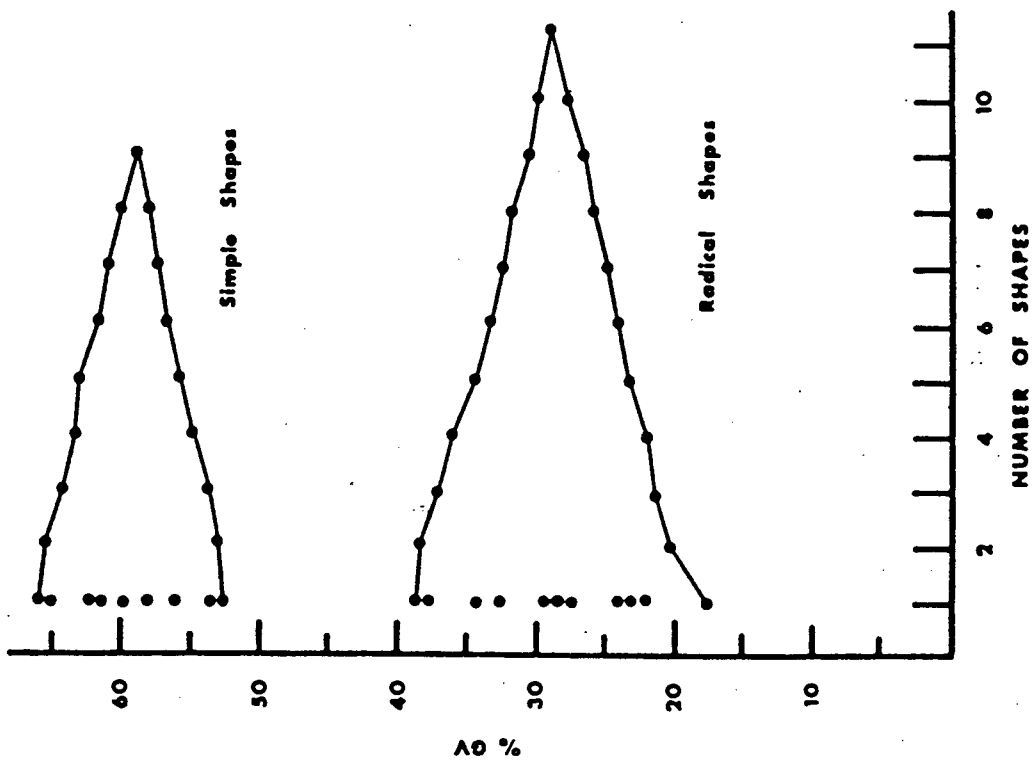


Figure 4.19 a :

Range of limiting grain volume for the set of simple shapes and set of radical shapes as determined by the number of shapes in a mixture (after Vinopal and Coogan, 1978) Note: porosity (%) = (100 - GV)

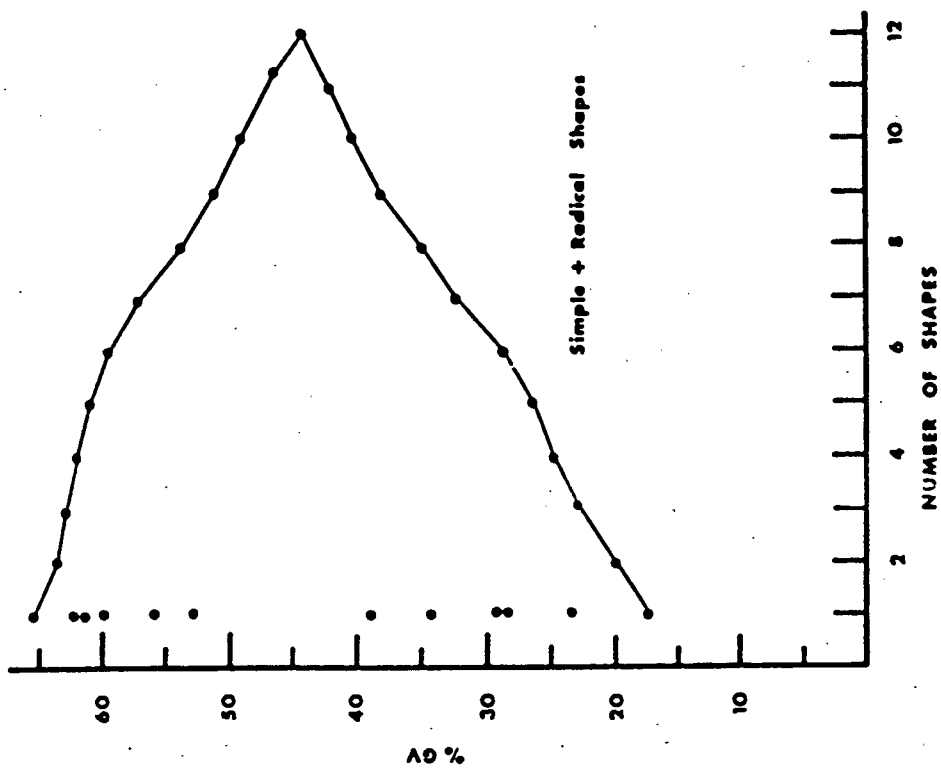


Figure 4.19 b :

Range of limiting grain volume for the set of simple and radical shapes as determined by the number of shapes in the mixture (after Vinopal and Coogan, 1978)

may give a qualitative indication of the crushing susceptibility, and is probably the most useful method for radical shaped particles in the fine sand-to-silt size range. No indication is however given of the particle shape distribution, since particle shapes as deduced from microscopic images may not be representative.

- (b) Visual comparison : The operator views the actual grain or grain facsimile and compares it to a standard reference. Charts for visual estimation of roundness and sphericity are shown in Figs. 4.20(a) and (b). These charts are only suitable for the description of "simple shape" particles.
- (c) Direct measurement : The operator makes direct measurements of the actual grain or grain facsimile. The method is time consuming and expensive and is not suitable for engineering classification-description purposes.
- (d) Response measurement : The operator observes and measures the response of the actual grain to a set of standard physical conditions. Methods which are suitable for "simple shape" particles, and which make use of the principle that sieves sort particles both by size and by shape, have been proposed.

Menke (1984) found that sieve analysis can be used to determine the size and shape distribution of a sediment sample by combining measurements with a suite of sieves with openings of different sizes and shapes. It is a tomographic process which relies on the fact that different sieves discriminate grains with different but overlapping size and shape. Optimally, this measurement requires four specially chosen elliptical sieves

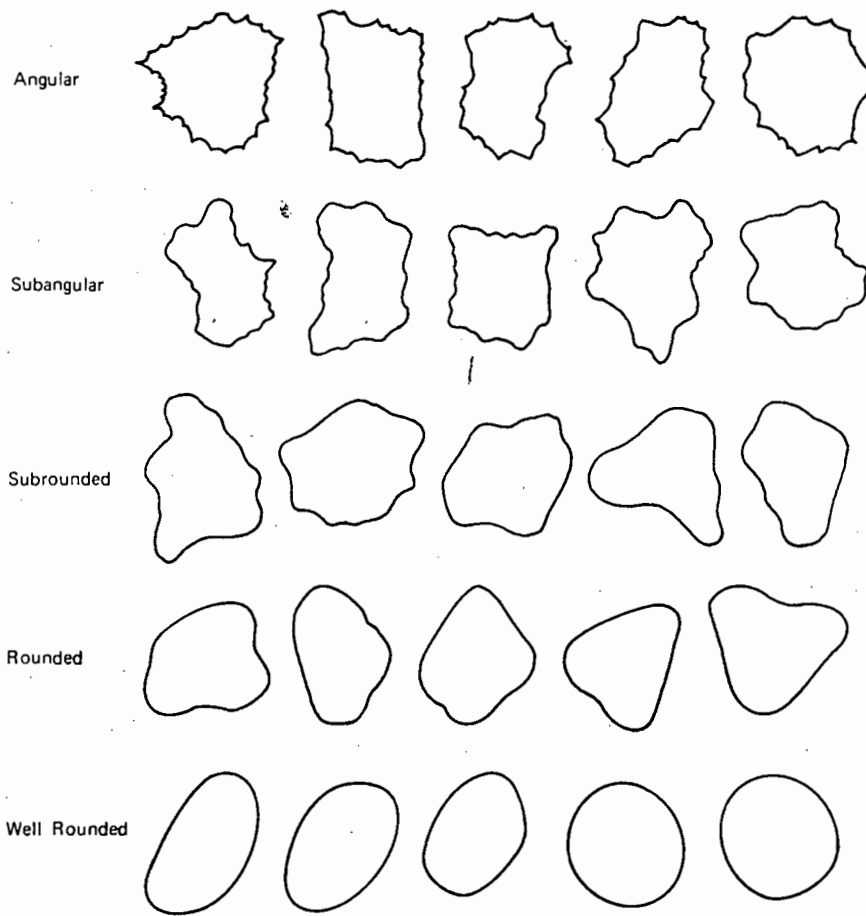


Figure 4.20 a : Roundness chart of Russel and Taylor, 1939 (after Pryor, 1971)

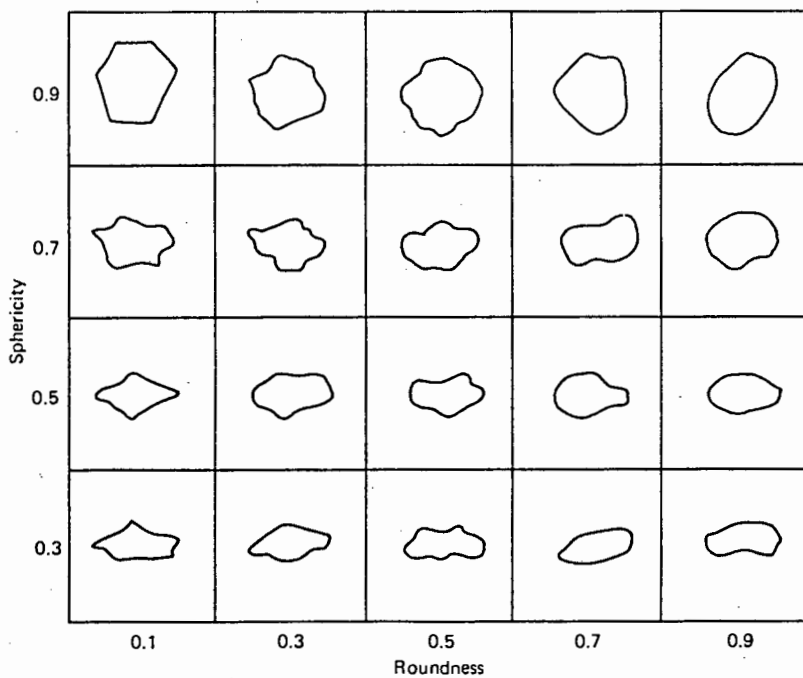


Figure 4.20 b : Chart for visual estimation of roundness and sphericity by Krumbein and Sloss, 1955 (after Pryor, 1971)

for each range, so that considerable work is required to determine the size and shape distribution of the sample. Whether such a process can be made practical has yet to be demonstrated. Menke believes that although it requires more effort than standard sieving, the process can be made faster and more routine than other methods of measuring size and shape, for example microscopic analysis.

Cascadography is a method of shape analysis which makes use of the principle that the probability of different shape particles being retained on a standard square mesh sieve varies with the shaking time and amplitude. Such a process is described by Meloy and Durney (1984). The process involves a two part sieving procedure. The first stage uses the normal sieve stack (15 minutes). Sub-samples from the first stage is then resieved through a set of sieves with identical diameters. At various time intervals the pan fraction is removed and the shape distribution is then deduced from this information, using a Fourier shape analysis equation.

The above methods can be used only with particle sizes for which sieving procedures are applicable, while carbonate particles of complex shapes are often in the fine sand to silt size range. Since this type of particle has the most pronounced effect on the mass physical properties of the sediment (Section 4.6.3), and hence indirectly on the engineering behaviour, a knowledge of the extent and distribution of these particles would be most useful. A visual examination using microscopic techniques together with particle size analysis are normally used for the engineering identification and description of these particles.

Even if there were no errors involved in particle size analysis of carbonate fines (Section 4.4.4), and the average particle sizes could be precisely

determined, such information would give little indication of the particle shapes. From Fig. 4.21 and Table 4.7 it can be seen that carbonate particles with the same average diameter (e.g. 81 μm) can have specific surface areas which differ by factors of up to 14. This is a reflection of the widely varying shapes.

The investigation of Vinopal and Coogan (1978) into the effect of particle shape on void ratio suggests an additional qualitative measure of the particle shapes present in carbonate sediments. This is possible as a result of the following findings by Vinopal and Coogan :

- that the resulting void ratios of sediments composed of "simple" and "radical" shapes fall in different ranges and that the void ratio at any given state of compactness is largely influenced by the particle shapes;
- that the possible range of void ratios between limiting states of compactness progressively increases as the complexity of particle shapes increases.

The advantage of this index is that it incorporates effects such as particle size and shape distribution into a single parameter which can be more easily linked to the engineering behaviour of the sediment. Thus, a knowledge of the limiting void ratios of the sediment together with a visual assessment of the particle shapes in the predominant size fractions should give a better qualitative assessment of the sediments' compressibility, susceptibility to crushing and volume change behaviour during shear.

Table 4.7 : BET specific surface area as a function of grain size
(after Walter and Morse, 1984)

Grain Type	Median Grain Diameter (Microns)	Observed Specific Surface Area ($m^2 g^{-1}$)	Observed/Geometrically Predicted Surface Area
Rhombic calcites	5	0.45	1.1
	81	0.03	1.2
Echinoid	81	0.14	5.7
	275	0.09	12.4
	513	0.08	20.5
Coral	51	0.23	5.9
	81	0.22	8.9
	275	0.17	23.4
	513	0.12	30.8
Halimeda	81	2.04	82.6
	215	2.10	225.8
	513	2.11	541.2

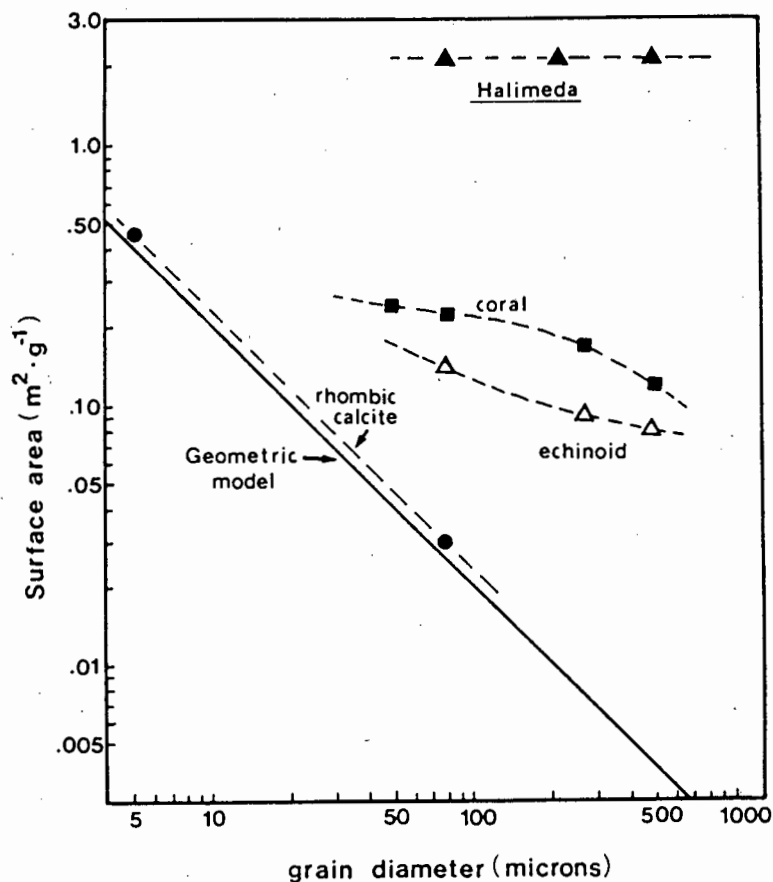


Figure 4.21 : Variation of total surface area with grain diameter for different types of carbonate particles
(after Walter and Morse, 1984)

4.6.5 Relative Density

(i) Definition of relative density

Relative density is used to express the relationship between a sample's void ratio and the limiting values of "densest" and "loosest" compaction states.

Thus, it can be expressed as :

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \cdot 100\% \quad (\text{void ratio})$$

$$\equiv \frac{\gamma_{d\max}}{\gamma_d} \cdot \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \cdot 100\% \quad (\text{dry specific weight})$$

$$\equiv \frac{1 - n_{\max}}{1 - n} \cdot \frac{n_{\min} - n}{n_{\min} - n_{\max}} \cdot 100\% \quad (\text{porosity})$$

where D_r can vary between 0 and 100 percent.

(ii) Engineering application of relative density

Some of the more important engineering applications of the relative density concept include use :

- as a reference parameter in laboratory testing and research
- as a basis for determining the in situ soil strength and settlement properties from field measurement
- as a descriptor or index property that appears to correlate susceptibility of soils to liquefaction under seismic loading (Marcuson and Franklin, 1980).

Notwithstanding its wide use, the relevance of relative density as an index property for conventional soils has been questioned by many researchers (e.g. Clayton *et al.* (1985); Tavenas and La Rochelle, (1972)). These authors have pointed out that the relative density concept should be used with extreme care. Criticism of the concept has been directed at two levels:

- the relevance of the parameter as an index of engineering behaviour
- difficulties associated with the measurement and definition of the terms contained in the relative density equation.

There appears to be no published literature on the applicability and relevance of relative density to carbonate soils, although this parameter is widely used in laboratory investigations on carbonate materials. In the following section, criticism of relative density as applied to conventional soils is examined and the likely sensitivity of carbonate soils to these factors is evaluated in the light of its unique properties.

(iii) Applicability of R.D. parameter to represent engineering behaviour

For a particular soil, it is generally found that a good correlation exists between the relative density and the shear strength as measured by the angle of internal friction. However, different soils having an identical state of compactness (i.e. different compositions but with numerically equal values of relative densities and void ratios) need not have the same engineering properties since the mechanical properties of the assemblage vary with the nature, form and statistical distribution of the dimensions of the grains of the soil. It has nevertheless been found that for non-carbonate soils, relative density offers a superior correlation with the strength of sand, compared to void ratio, presumably since it compensates for effects of

particle grading and shape which influence the values of e_{\max} and e_{\min} (Bolton, 1986).

For carbonate soils, the range of void ratios obtainable is much larger, distribution of particle sizes influence a wider range of mechanical properties (e.g. susceptibility to crushing) and the densest state of compaction obtainable is often less than the loosest state obtainable for most quartz sands. This could make the parameter unsuitable as a basis of comparison between dissimilar carbonate soils or quartz sands.

An example of how the relative density concept could lead to misinterpretation can be found in the results of model pile tests presented by Poulos and Chan (1984). They conclude that for equal relative densities, the piles in the calcareous sand had only 2/3 of the capacity of the piles in quartz sand. Such a comparison masks the fact that at equal relative densities the void ratios of the two types sands could have been substantially different, with the calcareous sand having the higher void ratio (lower limiting densities). Since pile capacity has been shown to be strongly influenced by the compressibility of the soil (e.g. Murff, 1987) and the compressibility in turn being highly dependent on the void ratio, void ratio or compressibility would probably provide a clearer basis of comparison.

(iv) Measurement and determination of RD parameters

Both for the laboratory and in situ measurement of the terms contained in the RD definition there are practical difficulties. These will be discussed in relation to :

- the definition of limiting states and the methods used for their determination

- for the in situ RD, the measurement of the in situ void ratio or density (Section 6.2).

Method of determining limiting states

The limiting void ratios for the RD definition are defined as the void ratios corresponding to the "minimum" and "maximum" compactness obtainable experimentally. More than thirty experimental techniques have been proposed, which tend to share the following basis. Tests for determining the maximum density usually involve some form of vibration. Tests used to determine minimum density usually involve pouring oven dried soil into a container.

For an assemblage of uniform spheres the limiting densities can be calculated exactly. However, in the case of natural soils, it is impossible to ensure that the limiting densities obtained by experiment correspond effectively to the maximum and minimum state of compactness for the material considered. Values of minimum and maximum density depend on the procedures used to determine them and are subject to large variability (Appendix A.2.)

A survey of carbonate soil literature appears to indicate that no standard technique is generally used. This is clearly not a satisfactory state of affairs as far as correlation of material properties is concerned. Carbonate soils are probably among those materials which are most sensitive to changes in the test method employed (Netterberg, 1982), so that variability in limiting values due to different techniques can be expected to be significantly larger for these soils. The adoption of a standardized technique for carbonate materials will help to eliminate this source of large potential variability. It appears that use of a set technique may not

be practicable for all types of carbonate testing. Thus vibration in a 5ℓ drum would be inappropriate for preparing samples for triaxial testing.

In selecting an appropriate technique for carbonate soils, two unique carbonate material properties (which will have an effect on the results obtained) must be considered. These are :

- susceptibility of particles to crush
- intraparticle void ratio.

The susceptibility of particles to crush will affect the maximum density obtainable for a given technique. Methods of compaction involving the application of impact forces (e.g. Mod AASHTO) should be avoided. Vibration techniques will probably result in the least amount of crushing. Crushability should not affect the determination of minimum densities.

As previously indicated, the presence of intraparticle voids increases the susceptibility of particles to crush and creates difficulty with the definition of "minimum obtainable" density. When this type of particle is in the 'silt-to-fine-sand' size range, artificially low densities, with little engineering significance, may be obtained. A similar trend has been noted for non-carbonate silts and fine sands (Lambe and Whitman, 1979). In this regard, the suitability of the "cylinder inversion method" which is used widely for determining the minimum density for this size range may be inappropriate.

The effect that intragranular porosity could have on the determination of limiting states can be practically illustrated with the case of limestone. Limestone, possessing both intragranular and intergranular porosity is widely used as compacted fill for embankments in subtropical regions. It

has been observed in many cases that when such fill is used, specified compaction standards are hard to meet. This situation has also been evident for calcretes in Southern Africa (Buis, 1987).

Saxena and Hendrikson (1978) investigated the effect of intragranular porosity on the maximum dry density of such compacted limestone fill. Both the intragranular porosity and specific gravity of the solids of the limestones were found to considerably influence the obtainable maximum dry densities (Fig. 4.22). To obtain high dry densities, they concluded that the aggregate must be broken into smaller fragments to "nullify the effect of the intragranular porosity".

(v) Relative density and standard penetration testing

The results of dynamic penetration tests are often used to determine the in situ characteristics of sand deposits, in particular the relative density. The results are usually interpreted by means of empirical relationships between the properties of the soil and the penetration resistance. Correlations between the relative density of soil and the standard penetration index N for conventional soils have been proposed by numerous authors, including Terzaghi and Peck (1948), Gibbs and Holtz (1957) and Schultze and Melzer (1965). Criticism of these correlations have been raised by Tavenas and La Rochelle (1972) :

- In none of these suggested correlations has the method of reproduction of the limiting states of compactness been stated. The absolute error on the RD using different techniques have been shown to be in the range of ± 10 to $\pm 34\%$, which is large enough to make the computed relative density useless.

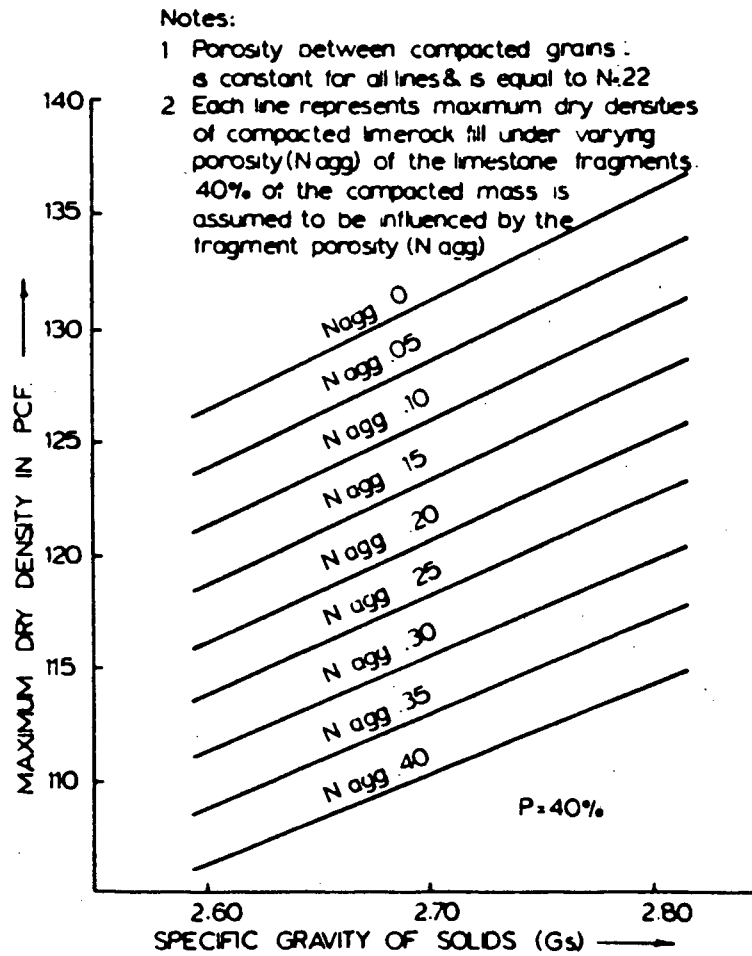


Figure 4.22 : Maximum dry densities versus specific gravities for various porosities (after Saxena and Hendrikson, 1978).

$$1 \text{ pcf} = 0.04788 \text{ kN/m}^3$$

- All the correlations have been based on tests done on a limited number of materials. The assumption that sands with different grain shapes, mineralogies and size distributions have an identical influence on both the relative density and the standard penetration index is unjustified.

Some of the soil conditions which influence the dynamic penetration resistance have been investigated by Clayton *et al.* (1985). These are listed in Table 4.8. For three of these properties, namely void ratio, particle angularity and cementation, carbonate soils can show a particularly large deviation from conventional soils. In addition, crushability, which is not listed in Table 4.8, could be expected to have a large influence on the standard penetration index of crushable carbonate soils.

The applicability of correlations derived from tests on conventional soils to carbonate soils could therefore be affected by the following factors :

- The definition and the errors involved in the determination of relative densities for carbonate soils (Section 4.6.5).
- The effect which carbonate soil properties could have on the value of the standard penetration index N .

Table 4.8 : Influence of soil conditions on dynamic penetration resistance (after Clayton *et al.* (1985))

Factor	Influence
Void ratio	Decreasing void ratio increases penetration resistance
Average particle size	Increasing particle size gives increased penetration resistance; fine-grained soils at low effective stress levels may liquefy
Coefficient of uniformity	Uniform soils exhibit lower penetration resistance
Porewater pressure	Dense fine soils dilate to increase penetration resistance; very loose fine soils may liquefy
Particle angularity	Increased angularity gives increased penetration resistance
Cementation	Cementing increases penetration resistance
Current stress levels	Increased vertical stress gives increased penetration resistance : increased horizontal stresses increase penetration resistance

5. SHEAR STRENGTH BEHAVIOUR

5.1 SHEAR STRENGTH OF NON-COHESIVE CARBONATES

In this section, the factors affecting the shear strength of cohesionless, uncemented carbonate soils are examined. For such soils, the strength under a given set of conditions can be expressed in terms of the angle of internal friction with respect to effective stress :

$$\phi' = \arctan (\tau/\sigma_n')$$

where:

- ϕ' = effective angle of internal friction
- τ = shear stress on failure plane
- σ_n' = normal effective stress on failure plane.

The following shear strength characteristics of carbonate soils, determined in the triaxial and direct shear apparatus, are well established :

- In general, friction angles are relatively high, typically above 35° and often exceeding 50° (e.g. Datta *et al.*, 1979, Murff, 1987).
- The friction angle is dependent on the confining pressure (triaxial) or normal stress (direct shear) and decreases with increasing confining pressure (Section 4.3).
- For a given soil, the peak angle of internal friction is dependent on the relative density, especially at low confining pressures (e.g. Poulos *et al.* 1984).
- Relative to quartz sands , carbonate sands tend to dilate only at low confining pressures, while at higher pressures they tend to reduce in

volume, presumably due to consolidation and crushing. A similar trend has been found for quartz sands (Vesic and Clough, 1968), but only at much higher confining pressures.

- The high friction angles of carbonate sands are not the result of a dilatant behaviour (McClelland and Beard, 1984), and the soil is softer and more compressible after grain crushing and volumetric change induced by confining or shearing stresses.

The shear behaviour of carbonate soils can be better understood by considering the "energy components" that contribute to the measured angle of internal friction. For a cohesionless soil with particle crushing, the shear strength may be expressed as follows (Lee and Seed, 1967) :

$$\begin{aligned} \text{Measured shear strength } (\sigma') &= \text{strength due to sliding friction } (\sigma_{\mu}) \\ &\quad \pm \text{ dilatancy effects} \\ &\quad + \text{ crushing and rearranging effects.} \end{aligned}$$

The angle of sliding friction, σ_{μ} , depends on the mineralogy and is normally assumed to remain constant for a particular soil, irrespective of the confining pressure. Measurement of σ_{μ} and values applicable to carbonate soils are discussed in Section 5.2. The dilatancy effect may be either positive or negative depending on whether the volume increases or decreases during shear, while crushing and rearranging of particles makes a positive contribution to the strength due to the external energy required for such crushing and rearranging.

Fig. 5.1 shows a "generalised" schematic illustration of the contribution of sliding friction, dilatancy and crushing to the measured Mohr envelope for

drained tests on sand. At relatively low normal stresses, crushing will be limited and the volume of the sample will increase, adding strength. At higher stresses, significant crushing commences. The external energy required to cause crushing increases the strength, while the decrease in volume associated with crushing reduces the measured strength. According to this model, it would appear that for carbonate soils with high compressibilities, curvature of the Mohr envelope would be more pronounced and commence at lower normal (or confining) stresses due to the negative contribution of the volume decrease of the samples to the measured strength.

5.2 ANGLE OF SLIDING FRICTION, ϕ_{μ}

Carbonate sands generally have higher angles of internal friction than quartz sands (Section 5.1), at lower normal stresses. In this section the contribution of ϕ_{μ} to the total strength (σ') and the factors influencing its values are examined.

The parameter ϕ_{μ} has been defined variously by different workers in soil mechanics, but is generally taken to be the average coefficient of kinetic friction generated when one "typical" soil particle surface is caused to slide slowly over another through significant displacement (Proctor and Barton, 1974).

Various experimental techniques have been developed to measure ϕ_{μ} :

- Sliding one smooth block of the material over another of the same material.
- Sliding several fixed particles over a smooth block.
- Sliding a mass of free particles over a smooth block.
- Sliding of one fixed particle over another.

Measurements of ϕ_{μ} for carbonate minerals, both direct and indirect (e.g. using Rowe's stress dilatancy theory) have shown that the interparticle friction angle is dependent on both mineralogy and the physico-chemical state of the surface of the grains. (Physico-chemical effects have been reported to be relevant in tests on individual grains prepared under artificial conditions, e.g. chemical cleaning, high temperature drying, etc.)

Frossard (1979) used Rowe's stress-dilatancy theory to investigate the combined effect of mineralogy and particle shape on the interparticle friction angle of quartz and calcareous grains. For quartz and calcareous particles of similar shape, the measured angle of interparticle friction (triaxial test; 50 kPa confining pressure) was consistently higher by about 9 degrees for the calcareous grains.

The frictional characteristics of minerals, including calcite, have been investigated experimentally by Horn and Deere (1961). The influence of surface moisture, surface roughness, and the rate of sliding on the frictional resistance developed between surfaces of the same mineral were considered. Coefficients of friction obtained for three conditions of surface moisture for various massive structured minerals are presented in

Table 5.1. The coefficients of friction for calcite were similar to those of quartz in the dry condition. In the saturated condition, the coefficients of friction for both minerals increased, with a proportionally larger increase for calcite. The conclusions that have been drawn from this investigation can be summarized as follows :

- The kinetic friction developed between mineral surfaces is generally equal to or slightly less than the static friction.
- As surface moisture increases, the frictional resistance developed between surfaces of massive-structured minerals (e.g. calcite, quartz) increases, whereas the reverse is true for layer lattice minerals (e.g. muscovite).
- The antilubricating action of water on surfaces of massive-structured minerals diminishes rapidly as surface roughness increases. Thus, apart from the effect of capillary forces, particle surface moisture variations have no measurable influence on the drained shearing resistance of soils composed of massive-structured minerals. The drained shearing resistance of soils composed of layer-lattice minerals decreases as the moisture on the surface of the soil particles increases.

5.3 OTHER CAUSES OF HIGH FRICTION ON ANGLES OF CARBONATES

The high angles of internal friction of carbonate soils can be partly attributed to the high mineral friction angle, ϕ_{μ} (Section 5.2).

Table 5.1 : Summary of frictional coefficients for three conditions of surface moisture
(after Horn and Deere, 1961)

Mineral	Origin	Oven-dried		Oven-dried air-equilibrated		Saturated		$\frac{\mu_s^k}{\mu_d^k}$	$\frac{\mu_s^s}{\mu_d^s}$
		Static $\frac{\mu_s^s}{\mu_d^s}$	Kinetic $\frac{\mu_k^k}{\mu_d^k}$	Static $\frac{\mu_s^s}{\mu_m^s}$	Kinetic $\frac{\mu_k^k}{\mu_m^k}$	Static $\frac{\mu_s^s}{\mu_s^s}$	Kinetic $\frac{\mu_k^k}{\mu_s^s}$		
Clear quartz	N. Carolina	0.11	0.10	0.11	0.10	0.42	0.23 *	3.82	2.30
Milky quartz	Wisconsin	0.14	0.14	0.16	0.16	0.51	0.27 *	3.64	1.91
Rose quartz	Unknown	0.13	0.11	0.13	0.11	0.45	0.26 *	3.45	2.36
Microcline feldspar	Unknown - A	0.11	0.11	0.13	0.11	0.76	0.76	6.90	6.90
Microcline feldspar	Unknown - B	0.12	0.12	0.12	0.12	0.77	0.76	6.42	6.42
Calcite (Scratching)	N. Jersey	-	-	0.21	0.21	0.60	0.60	-	-
Calcite (NS)	N. Jersey	-	-	0.12	0.12	-	-	-	-
Calcite (NS)	Kansas	0.14	0.14	0.14	0.14	0.68	0.68	4.85	4.85

- Notes :
- The above coefficients are for very smooth surfaces.
 - These coefficients are based on a rate of sliding of 0.7 in./min.
 - The coefficients refer to the friction developed between surfaces of the same mineral, e.g. quartz on quartz.
 - Relative humidity during oven-dried/air-equilibrated tests ranged between 17% and 35%.
 - The normal load ranged between 0.65 lb. and 10.2 lb.
- * Denotes approximate coefficient of kinetic friction; based on average of maximum and minimum values of frictional resistance during stick-slip movement.

For conventional soils the angle of internal shearing resistance is increased as the amount of "interlock" between particles is increased (e.g. Lambe and Whitman, 1979). Irregularity of particle shape and decreasing void ratio are factors which increase the amount of interlocking. The experiments of Frossard (1979) indicate that the angle of frictional resistance of calcareous sand particles is substantially reduced as the sphericity of the particles increases. The highly irregular and complex shapes of carbonate particles can therefore be assumed to contribute to the high shear strengths, especially at normal stresses where crushing is not significant.

5.4 MEASUREMENT OF SHEAR STRENGTH

A number of testing apparatus, for example triaxial, direct shear and simple shear can be used to measure the shearing strength of cohesionless carbonate soils. Devices such as the direct, simple and ring shear tests impose stress states on the sample which approximate to that of plane strain, while the conventional triaxial test imposes quasi-triaxial conditions.

For conventional cohesionless soils, the type of apparatus used can substantially influence the measured shear strength and stress-strain volume change behaviour. In order to apply information obtained from testing under triaxial conditions to that obtained under plane strain conditions, a knowledge of the relative magnitude of strength parameters as measured in each type of test is required. Such information would be especially useful in the case of carbonate soils, for which the availability of undisturbed samples is often limited. In addition, this information is required if an index test based on the shear strength of the sample is to be developed (e.g. Section 4.3.4).

Section 5.4.1 contains a review of the behaviour of non-carbonate cohesionless soil under plane strain and triaxial conditions, followed by a comparison of available data of carbonate sands under these conditions.

5.4.1 Triaxial and Plane Strain Conditions

Fig. 5.2 illustrates the externally applied principal stresses under triaxial conditions. In this situation, the intermediate principal stress σ_2' is equal to the minor principal stress σ_3' . In many practical problems σ_2' is greater than σ_3' and may in the limit equal σ_1' . A common special case is that of plane strain for which there is no change in length along the axis of the structure. Practical problems in which this type of situation is relevant include :

- most problems of slope stability (Bishop, 1966)
- conditions in the centre of an excavation about to fail in heaving (Bishop, 1966)
- most retaining wall problems (Lambe, 1967)
- strip footings and other shallow foundations.

Several investigators have attempted to evaluate the effect of the plane strain type of loading on the angle of friction of conventional granular soils, either through experimental measurement (Taylor, 1939), theoretical relationships (Rowe, 1969) or both. Lee (1970) summarised the results obtained by various researchers from direct measurements. A comparison of these measurements are presented in Table 5.2, on the basis of $\phi_p' - \phi_t'$, where :

- ϕ_p' = peak effective angle of internal friction in a plane strain test
- ϕ_t' = peak effective angle of internal friction in triaxial test.

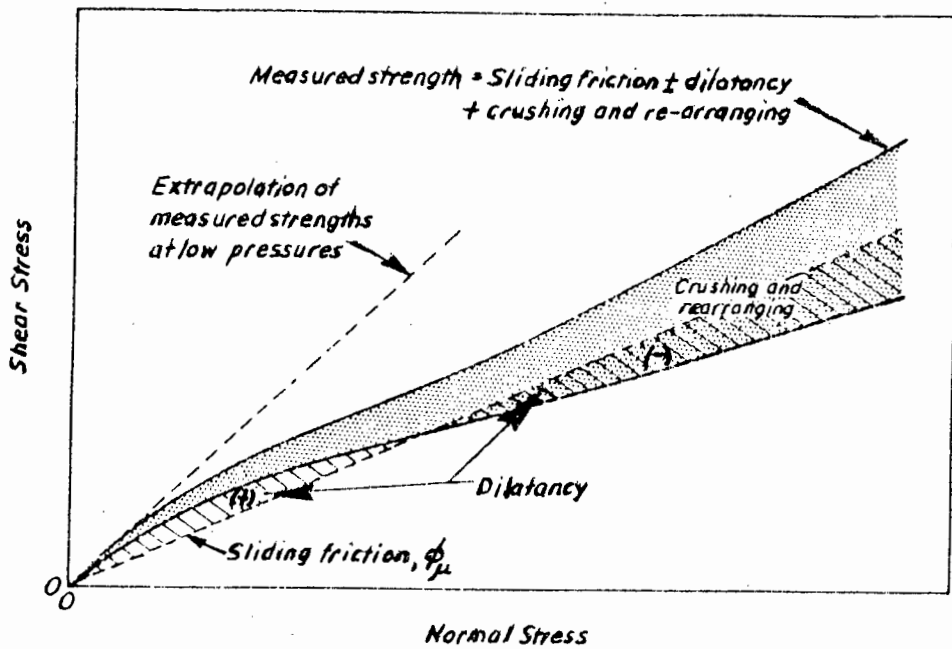


Figure 5.1 : Schematic illustration of contribution of sliding friction, dilatancy and crushing to the measured Mohr envelope for drained tests on sand (after Lee, 1967)

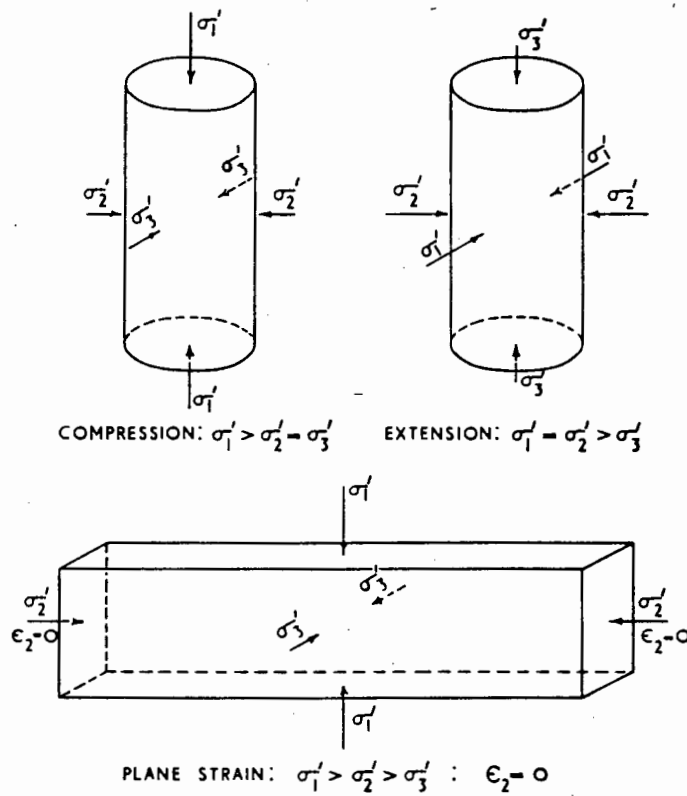


Figure 5.2 : Principal stresses in compression, extension and plane strain tests (after Bishop, 1966)

Table 5.2 : Comparison of results of plane strain and triaxial tests on sand by various researchers (after Lee, 1970)

Soil	$\phi_p - \phi_t$, deg.	Plane strain apparatus and comments
Sand	+ 8	Cube - vary all three stresses as desired
Dense sand	+ 4	Direct shear; critical void ratio is higher in plane strain
Loose sand	- 1	
Sand	+ 5	Direct shear
Dense sand	+ 4	Direct shear
Loose sand	- 2	Direct shear
Sand, gravel and lead shot	+ 2 to + 7	Direct shear
Sand	+ 8	Direct shear
Sand	+ 2	Hollow cylinder; failed by increasing internal radial pressure
Compacted clay	+ 2 to + 4 effective-stress basis	2 x 4 x 16 in plain strain apparatus, effective-stress basis; plane strain gives lower strain at failure and higher modulus; plane strain gives higher pore pressure at failure
Dense sand	+ 4	Plane strain apparatus
Loose sand	+ 0	Drained tests
Sand	+ 4 to + 5	Active earth pressure on model retaining wall
Ottawa sand	+ 4 to + 5	Bearing capacity of model strip footings
Sand, gravel	+ 3 to + 4	Vacuum compression on long rectangular specimens; modulus greater and strain to failure less in plane strain
Ottawa sand	+ 6	Hollow cylinders failed by increasing outside radial pressure
Ottawa sand	+ 5	Hollow cylinders failed by increasing outside or axial stress while measuring the other two stresses
Ottawa sand	- 4 to - 6	Torsion tests on very thin annular rings of soil at various rates of strain
Compacted clay	+ 2 to + 4 effective-stress basis	Rectangular plane strain apparatus; effective-stress basis modulus greater and strain failure less in plane strain
Glass spheres	$\phi_p > \phi_t$	Rectangular plane strain apparatus
Dense sand	+ 5	Bishop plane strain apparatus Plane strain gives lower strain to failure and less dilatant volume change at failure
Loose sand	+ 3	
Saturated NC silty clay	+ $3\frac{1}{2}$ effective stresses	Plane strain apparatus; $S_{u'}$ greater for plane strain
Saturated clay	+ 1 effective-stress basis	Bishop plane strain apparatus, ACU tests; pore pressure at failure approximately the same
Ottawa dense	+ 3	Vacuum plane strain and vacuum triaxial
loose	+ 1	
Monterey sand dense	+ 3	
loose	+ $\frac{1}{2}$	
Dense fine sand low pressure	+ 2	Direct shear
elevated pres.	+ 0	

The results indicate that the angle of friction in plane strain is generally greater by 0 to 8° for sands in the dense state. The difference is greatest at low confining pressures. For loose sands, plane strain tests generally give slightly lower angles of friction.

Cornforth (1964) proposed an explanation for this behaviour based on the restricted movement of individual sand grains due to asymmetry of external stresses. He concluded that for the same density, the plane strain condition will mobilize the maximum shearing resistance in the sand, and symmetric strain conditions will mobilize the minimum. The strain condition during shear is therefore a major factor contributing to the strength of the sand.

The test performed by Cornforth was conducted in a specially designed plane strain apparatus (Fig. 5.3) which allowed the accurate measurement of parameters such as volume change behaviour and the intermediate principal stress. The comparison of plane strain and triaxial failure characteristics is shown in Figs. 5.4(a) to (c). The following trends were observed :

- The angle of friction was always higher for plane strain, the difference decreasing as the sand became less dense (Fig. 5.4(a)).
- The triaxial specimens had a greater positive volume change at failure, but the volume change at failure for the two types of test tended to converge at looser densities (Fig. 5.4(b)).
- The axial strains at failure in the triaxial test were almost three times greater than those of plane strain specimens at the same density (Fig. 5.4(c)).

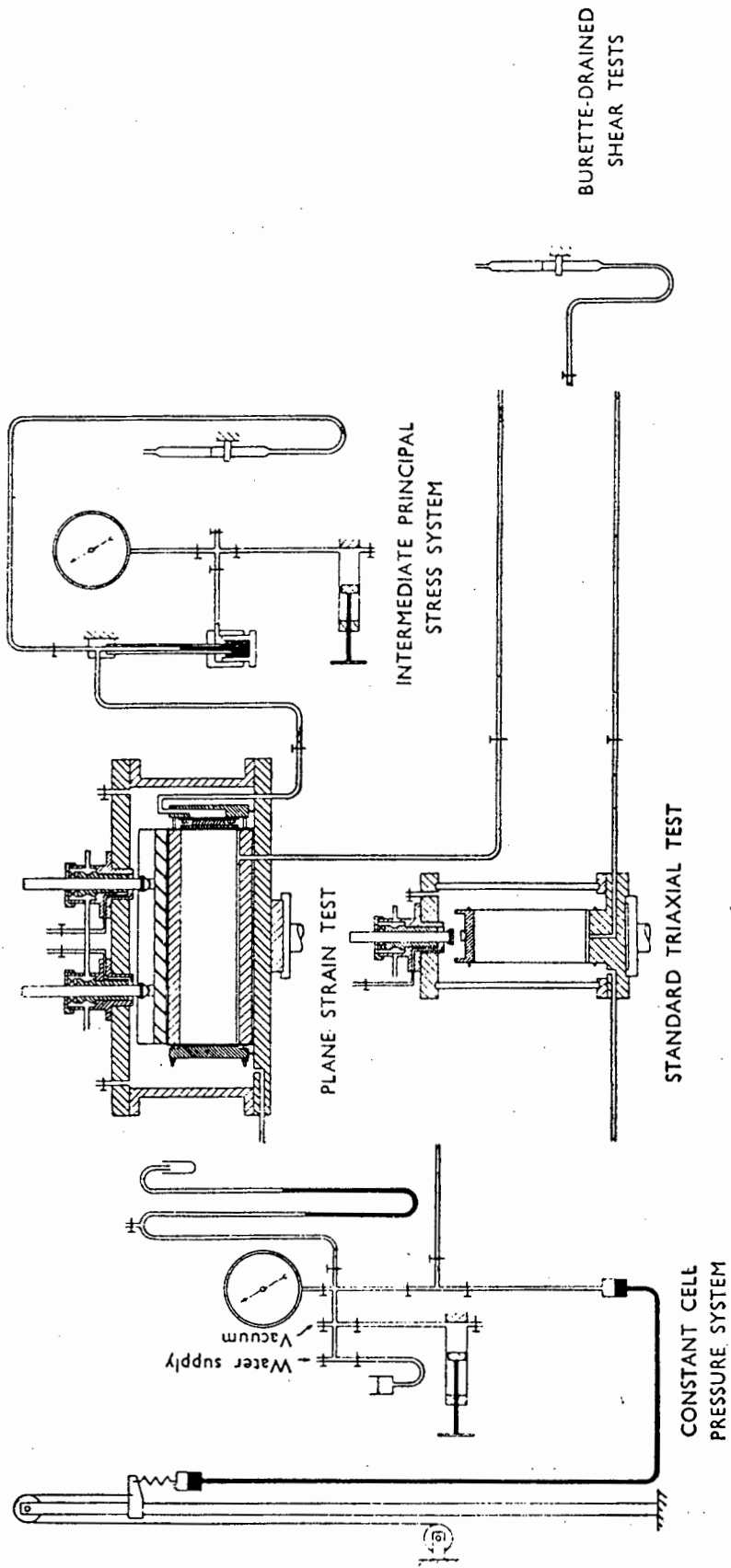


Figure 5.3 : Imperial college plane strain testing apparatus
(after Cornforth, 1964)

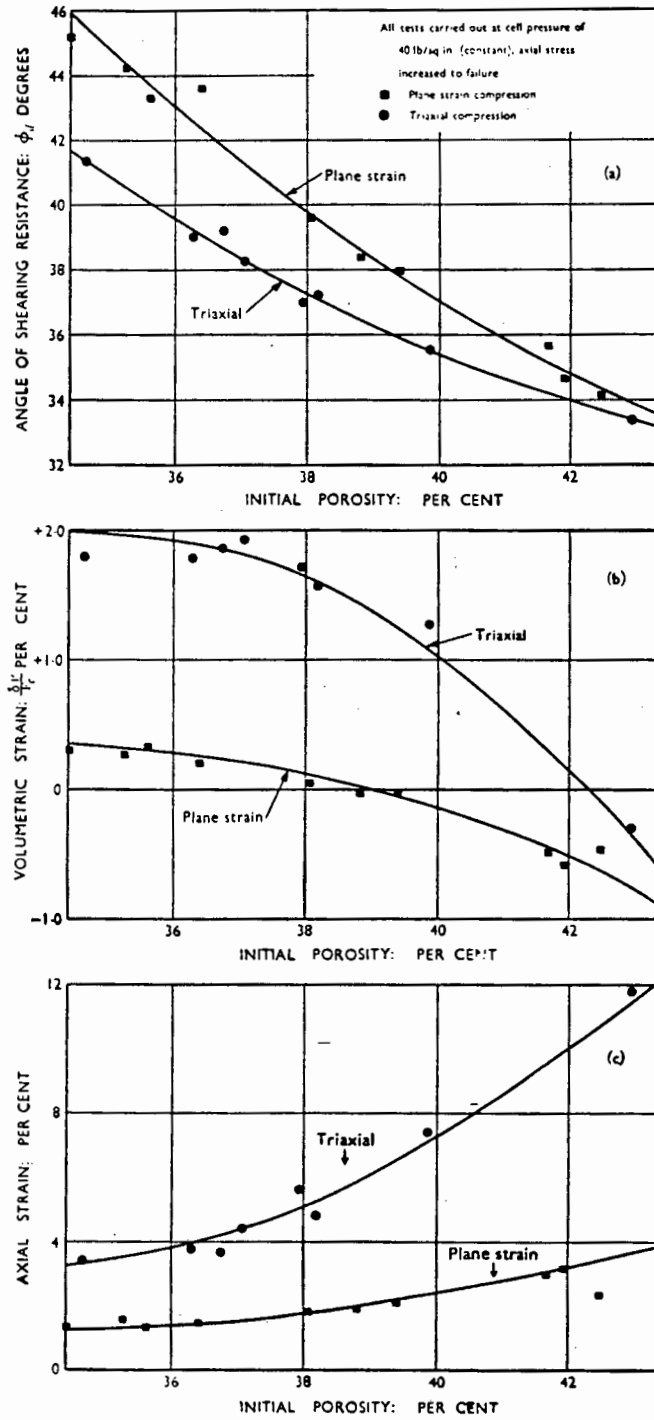


Figure 5.4 a,b,c : Comparison of plane strain and triaxial failure characteristics (after Cornforth, 1964)

- The ultimate strengths measured in plane strain compression tests were constant, irrespective of placement density, and had approximately the same values as those measured in triaxial compression tests.

Little published data on angles of internal friction for carbonate sands in plane strain and triaxial tests is available :

- Poulos (1982) : values of ϕ' (direct shear) for loose crushable sand were significantly lower than the values obtained in triaxial tests. For dense sands, similar values were obtained in both tests.
- Datta et al. (1980) : values of ϕ' determined from direct shear and triaxial tests corresponded well for a number of dense, crushable sands tested under various pressures (1 - 8 kg/cm²).

From this limited data, it would appear that for carbonate sands, the correspondence between ϕ'_t and ϕ'_p is closest in the dense state of packing, (i.e. opposite to conventional sands), with $\phi'_p - \phi'_t$ decreasing as the packing becomes looser. This aspect requires further investigation.

5.5 INTERFACE PROPERTIES OF CARBONATE SANDS

The low load bearing capacities of piles in carbonate sands are not readily apparent from the relatively high shear strengths often determined in laboratory tests. The low shaft resistance of driven piles could be a result of a combination of factors, which can be examined in relation to equation (1) :

$$\begin{aligned} Q_s &= f A_s \\ &= (k \sigma'_f \tan \delta) A_s \quad , \end{aligned} \quad (1)$$

where : Q_s = shaft resistance
 f = unit skin friction capacity
 σ_f' = effective stress on pile-soil interface
 δ = angle of friction between pile and soil
 k = coefficient of lateral earth pressure
 A_s = surface area of side of pile.

The total shaft resistance is directly proportional to both σ_f' and δ , so that low capacities could be due to either :

- insignificant increase in soil effective stresses from pile driving due to collapse of cemented soil structure or crushing of individual particles;
- possible low magnitudes for coefficient of friction developed between carbonate sediments and most construction materials.

Soil-interface testing provides a means of estimating the soil-pile friction angle and identifying its relative contribution to the observed low values of pile skin resistance.

Most interface tests on carbonate soils have been conducted using modified direct shear apparatus. The building material for which the coefficient of friction is to be measured is substituted for the lower portion of the shear box (e.g. Valent, 1979; Agarwal, 1977; Poulos, 1984). Other plane strain devices which can be used for interface testing, together with the relative advantages and disadvantages, are shown in Table 5.3. The friction coefficient has also been measured by placing a rigid, flat steel plate on the surface of the sand (McCarel and Beard, 1984). After the normal stress

is applied, the plate is subjected to shear loads in small increments until sliding occurs.

Information obtained from such interface tests can provide answers to the following questions : How does the angle of friction between calcareous sand and various building materials compare to those of quartz based sands? What is the effect of soil density, cyclic loading and crushability?

(i) Angle of skin friction for calcareous and quartz-type sands

Tests by some researchers have indicated that the coefficients of skin friction developed in calcareous sediments and quartzitic sands are superficially very similar.

Valent (1977) performed direct shear friction tests against mild steel and concrete, each in a smooth and rough finish. Three calcareous sands with differing properties and one quartzitic sand was used. The residual angle of internal friction (sand-sand) for the four sands were very similar in magnitude (i.e. between 28 and 31 degrees). For these materials, friction forces mobilized against "smooth" steel were about 1/3 of those mobilized in internal shear of the sand. Coefficients of skin friction developed on "rough" steel, "smooth" concrete and "rough" concrete were almost equal to those developed in the respective sands in internal shear.

McCarel and Beard (1984) also found that the friction angle between calcareous sands tested and steel were comparable to those of quartz sand and steel. Soil-steel friction angles for all their samples fell in a range of 18 to 23 degrees.

These tests suggest that the low friction forces that are developed between piles and calcareous soils are probably not the result of low coefficients of friction ($\mu = \tan \phi$) between calcareous sediments and building materials. Frictional stresses of up to 160 MPa was measured in some of the laboratory interface tests performed by Valent, whereas a limit of only 20 kPa is recommended by McClelland (1974), whose recommendation was based on the field performance of piles in calcareous sands.

If one were to assume that the samples tested by McCarel and Beard and Valent were somewhat similar in composition, then the range of the results obtained and the difference in their means would appear to be unacceptably large (i.e. 9°), and it would seem likely that the following factors have led to the differences :

- the method of testing, i.e. direct shear vs. "sliding plate" test
- smoothness of the steel interface surface
- different methods of sample preparation and applied normal loads.

(ii) Effect of relative density

For the calcareous sands tested by McCarel and Beard (1984), the soil-steel friction angle appeared to be essentially independent of the relative density. A similar trend was noted (by these authors) for Ottawa sand.

Poulos (1984) evaluated the soil-aluminium friction angle for a crushable calcareous sand obtained from the Bass Strait, Australia. He found that the initial density had a marked effect on the internal angle of friction (sand-sand in direct shear) but relatively little effect on the friction angle between the sand and aluminium (see Table 5.4).

Table 5.3 : Advantages and disadvantages of interface testing apparatus (after Kishida and Uesugi, 1987)

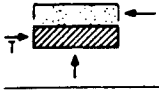

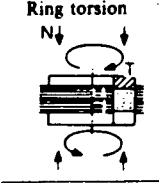
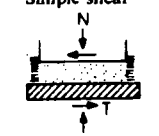
Type	Example	Advantages	Disadvantages
<p>Direct shear</p> 	<p>Potyondy (1961) Desai, Drumm & Zaman (1985)</p>	<p>Commonly available device Simple system Simple preparation Simple procedure</p>	<p>Displacement factors unable to be separated (sliding displacement and displacement due to sand deformation) Interface area reduced with increase in displacement</p>
<p>Annular stress</p> 	<p>Brumund & Leonards (1973) Miyamoto, Kishida & Kobayashi (1975)</p>	<p>Geometrically similar to skin friction of piles and friction of steel reinforcements</p>	<p>Normal stress on interface unknown Stress concentration at ends</p>
<p>Ring torsion</p> 	<p>Yoshimi & Kishida (1981)</p>	<p>Endless ring interface No stress concentration at ends Constant interface area Displacement factors observed by X-ray photography (sliding displacement and displacement due to shear deformation of sand)</p>	<p>Complicated system and procedure Difficult to prepare uniform sand mass in a ring shape Difficult to finish surface roughness of metal ring uniformly</p>
<p>Simple shear</p> 	<p>This work</p>	<p>Constant interface area Simple preparation Simple procedure Displacement factors measured separately (sliding displacement and displacement due to shear deformation of sand)</p>	<p>Stress concentration at ends</p>

Table 5.4 : Summary of direct shear sand-sand and sand-aluminium testing (after Poulos *et al.*, 1982)

Case	Friction angle (degrees)	
	Peak	Residual
Loose sand	35	34
Dense sand	50	48
Loose sand/aluminium	29	29
Dense sand/aluminium	31	31

(iii) Effect of cycling

Beringen *et al.* (1982) performed reverse loading large displacement direct shear tests on specimens of calcareous silt to measure the residual soil-steel friction angle. Five complete loading cycles caused no significant reduction of the soil-steel friction angle. Results obtained by Poulos *et al.* (1984) for calcareous sand-aluminium showed a small reduction in the friction angle with continuous cycling. This was also evident to a slightly greater degree for the soil to soil case, particularly for dense samples.

(iv) Stress-strain volume change behaviour

The stress-strain and volume change behaviour of sands against rough steel, tested by Valent (1977) is illustrated in Fig. 5.5. For all sands composed of solid grains, there was a very slight decrease in volume before reaching the peak friction angle, followed by a general volume increase. For the foramineferal sand-silt, composed of hollow crushable calcareous particles, there was a continuous and large decrease in volume throughout the test. No peak friction angle was observed and the residual friction angle at the end of the test was higher than that for the other soils. This type of behaviour is believed to be one possible cause of low developed pile capacities in some calcareous soils. The frictional force developed over a material surface is a function of both the coefficient of friction of soil against the material and the effective normal stress acting between the soil and that surface. As a pile is inserted in soil, densification with an accompanying increase in normal stresses results. Penetration of a pile in a foram sediment would thus result in densification of the sediment through crushing of the hollow shells, but such densification might not result in increased effective stresses in the soil. The soil mass may just have been transformed from a loose arrangement of hollow shells to that of shell

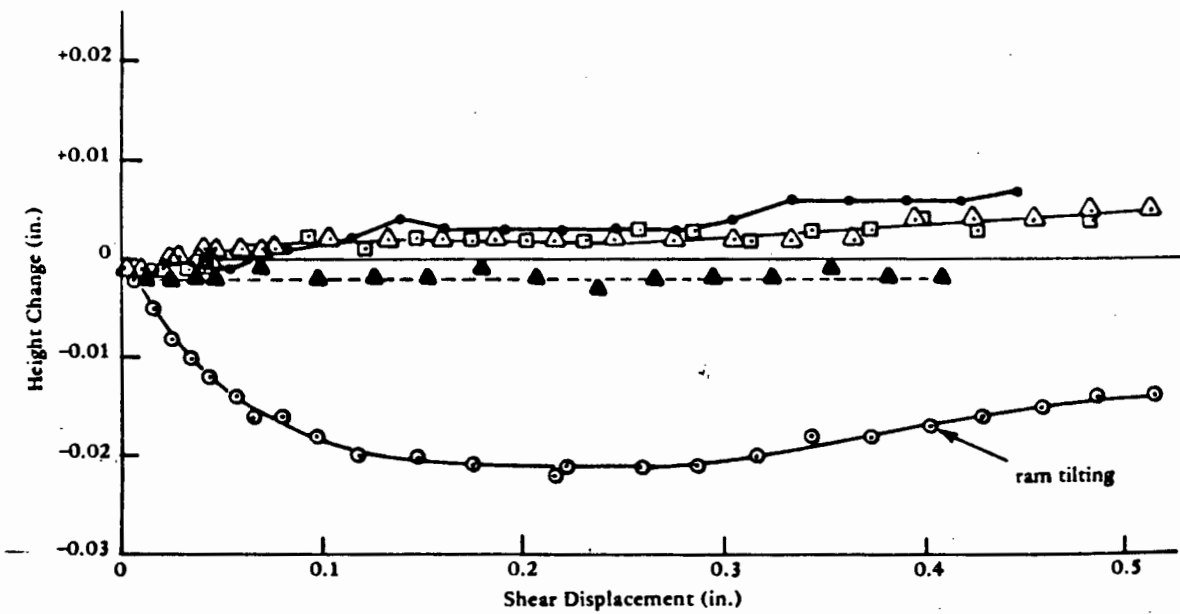
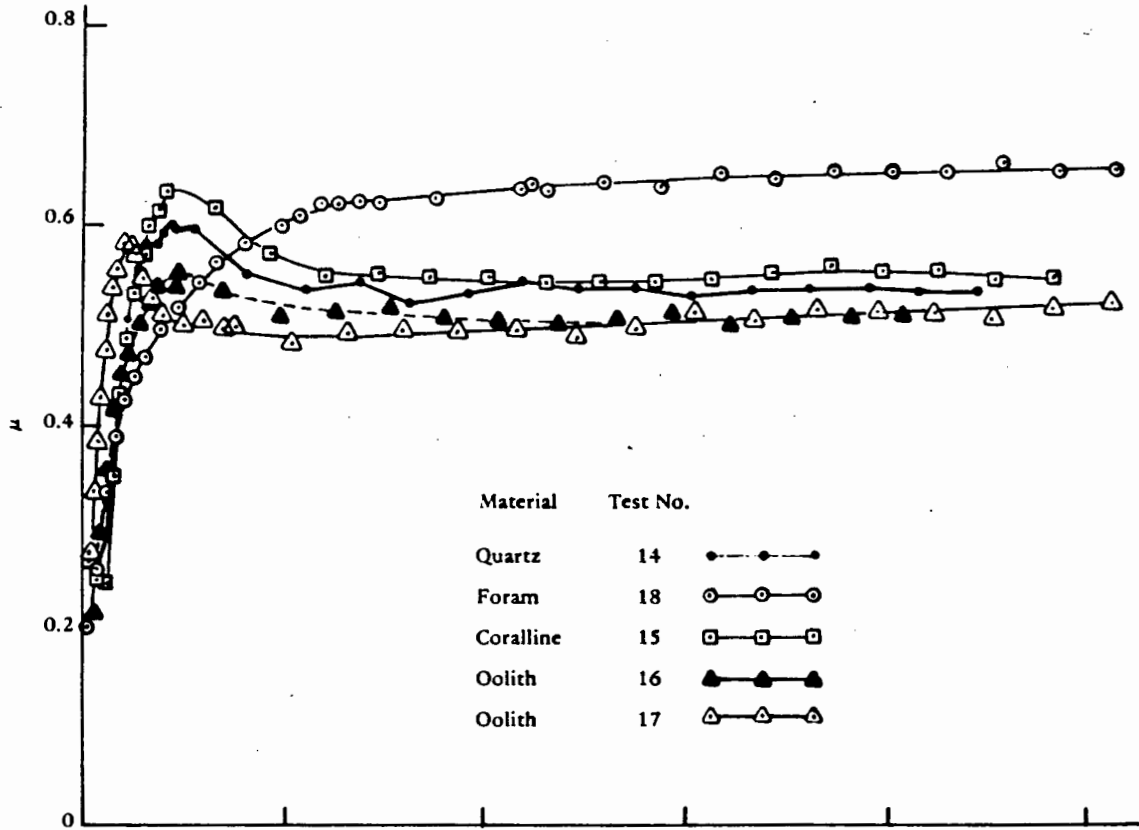


Figure 5.5 : Friction tests of soil samples on rough steel
(after Valent, 1977)

μ = coefficient of sliding friction

fragments (Valent, 1977). A similar mechanism is believed to be the cause of low pile capacities in lightly cemented soils.

The importance of volume change behaviour during cyclic loading has also been noted by Poulos *et al.* (1982). Cyclic loading led to small decreases in the angle of friction between soil and aluminium, but for both loose and dense packing arrangements each cycle of loading caused a reduction in sample volume. The inference that may be drawn is that sands subjected to cyclic shear will tend to experience a reduction in both friction angle and volume, so that, for a constant normal stress, the skin friction between the pile and sand will tend to decrease under cyclic loading.

(v) Effect of crushing and normal stress

Valent (1977) and McCarel and Beard (1984) concluded that crushing of particles had little influence on the angle of skin friction developed between the soil and the interface.

Research by Datta *et al.* (1979) has revealed that crushing could have an influence on the magnitude of the angle of skin friction. The apparent contradiction is probably a result of the fact that the tests conducted by Valent, McCarel and Poulos were performed at a constant, relatively low level of normal stress. Datta, however, performed direct shear interface tests with slightly roughened steel, using materials with different susceptibilities to crushing, and normal stresses as high as 784 kPa. These results are presented in Table 5.5. As the level of normal stress was increased, the measured angle of skin friction decreased for all soils. Sands with a larger propensity to crush showed larger reductions in skin

friction with increasing stress. It was furthermore noted that crushing for sand-to-steel shear was much less than crushing for sand-to-sand shear.

(vi) The relation between angle of skin friction and angle of shearing resistance

To determine the relation between the angle of skin friction and the angle of internal shearing resistance, Datta *et al.* performed sand-to-sand direct shear tests. Their results are presented in Table 5.6. In these tests the ratio

$$\frac{\tan \delta}{\tan \phi} = \frac{\tan (\text{angle of skin friction})}{\tan (\text{angle of internal shearing resistance})}$$

was found to increase as the susceptibility of particles to crush diminished. The value of the ratio remained constant at 0.66 for the quartz sand. This corresponds to the results obtained for quartz sands by other researchers (e.g. Potyondy, 1961).

For the crushable carbonate soils, the ratio was found to be less than 0.65, dependent on the level of normal stress, and showed variation on account of differences in susceptibility to crushing. The conclusion reached by Datta *et al.* is that the steel-to-sand friction behaviour of calcareous sand is similar to quartz sands, only when the degree of crushing in calcareous sands is limited. However, the relative contribution of decreasing skin friction angles to low pile capacities in carbonate sands remains uncertain.

Table 5.5 : Decrease in angle of skin friction, δ ,
with increase in normal stress (after Datta et al. (1979))

σ_n (kg/cm ²) [#]	δ (deg)				
	Sand A	Sand B	Sand D	Sand F	Sand E
1	25.8	27.3	29.2	26.8	29.5
4	23.9	25.7	29.1	-	27.5
8	22.1	23.3	26.8	25.1	27.5
$\Delta\delta$ *	3.7	4.0	2.5	1.7	2.0

* $\Delta\delta = (\delta \text{ at } \sigma_n = 1 \text{ kg/cm}^2) - (\delta \text{ at } \sigma_n = 8 \text{ kg/cm}^2)$

$1 \text{ kg/cm}^2 = 98 \text{ kPa}$

Table 5.6 : $\tan \delta / \tan \phi_d$ values (after Datta et al. (1979))

σ_n (kg/cm ²) [#]	$\tan \delta / \tan \phi_d$ *				
	Sand A	Sand B	Sand D	Sand F	Sand E
1	0.37	0.41	0.46	0.53	0.66
4	0.43	0.46	0.51	-	0.66
8	0.45	0.49	0.55	0.60	0.68

$1 \text{ kg/cm}^2 = 98 \text{ kPa}$

* ϕ_d = angle of internal friction (sand-to-sand) as determined in direct shear

Sand A, B, D, F : carbonate sands ($C_aCO_3 > 90\%$)

Sand E : quartz sand

6. CLASSIFICATION USING IN SITU TESTS

Ideally, a classification of carbonate soils should be achievable, as far as possible, using index tests which can be performed on disturbed soil samples. This is due to :

- the difficulty of obtaining undisturbed samples in carbonate sediments, especially in the marine environment (e.g. Lee, 1979);
- uncertainty in interpretation of in situ and laboratory tests as a result of the unique material properties (e.g. Anon., Underwater Technology, 1987).

These aspects have led many authors (e.g. Beringen *et al.*, 1982) to conclude that realistic design criteria, especially for piles, can only be obtained by means of in situ testing. However, data from some in situ tests can also be used to improve soil description and classification.

6.1 DYNAMIC PENETROMETERS

The SPT sampler is the most frequently used driven sampler for marine investigations (Angemeer and McNiellan, 1982). However, extensive calibration between the dynamic resistive force and design strength is required before use can be made of the data. This is a serious shortcoming because :

- (a) little study relating SPT blow count with carbonate properties has been performed;
- (b) correlations often require laboratory test results for which it is difficult to obtain undisturbed samples;

- (c) the cost and practical problems of carrying out tests in the marine environment often does not allow for sufficient tests to establish trends;
- (d) the variability and heterogeneous nature of the deposits can make correlation of results between different sites impossible.

The test is nevertheless useful for providing a relative resistance value and a qualitative indication of material variability, together with the advantage of providing a disturbed sample for visual examination and classification analyses. Expensive and time consuming drilling operations and equipment to advance the borehole between sample intervals are involved. Dynamic penetration testing is reasonably adaptable to floating base operations so that some advantage over static methods is gained in terms of time and complexity of equipment required.

Further disadvantages associated with the nature of dynamic penetrometer testing are especially relevant to carbonate deposits :

- (a) data is developed at intermittent depths only, so that the presence of lenses of cemented material of variable thickness and hardness can be overlooked. An advantage is that the requirement of drilling capability does allow penetration of very hard cemented layers;
- (b) the percussive action of dynamic penetration can destroy inter-particle cementation and cause the soil to densify (Beringen *et al.*, 1982). It has been noted, through onshore experience, that driven samplers can cause hard layers or even soft rock to go undetected.

6.2 STATIC PENETROMETERS

As for dynamic penetrometers, little data for correlation of static cone penetrometer data (e.g. "Dutch cone") in carbonate environments is available. The method nevertheless holds considerable promise for the testing of marine carbonate soils. Beringen *et al.* (1982) noted the following advantages :

- (a) One of the major difficulties involved in testing of carbonate soils is to quantitatively determine the degree of cementation (see Section 4.2). By comparing soil profiles based on drilling and percussion sampling with a cone resistance profile at the same site, Beringen *et al.* (1982) showed that the CPT test provides a significant insight into the degree of in situ cementation (Fig. 6.1). Cemented layers which showed up clearly in the cone resistance values went unnoticed due to sampling disturbance in the percussion profile. In addition to providing an indication of the degree of cementation, a continuous quantification of the soil profile is obtained.
- (b) For pile design, the static cone can be considered to function as a miniature "model test", as the measured cone resistance and sleeve friction incorporate the combined effect of factors such as soil density and in situ stress which affect pile capacity. This has an advantage over methods that define pile shaft friction as a function of in situ lateral stress and internal angle of friction, since the in situ lateral stress is hardly ever known and is not relevant to the stress condition in a pile approaching failure. For this reason, static cone tests are expected to provide useful predictions in

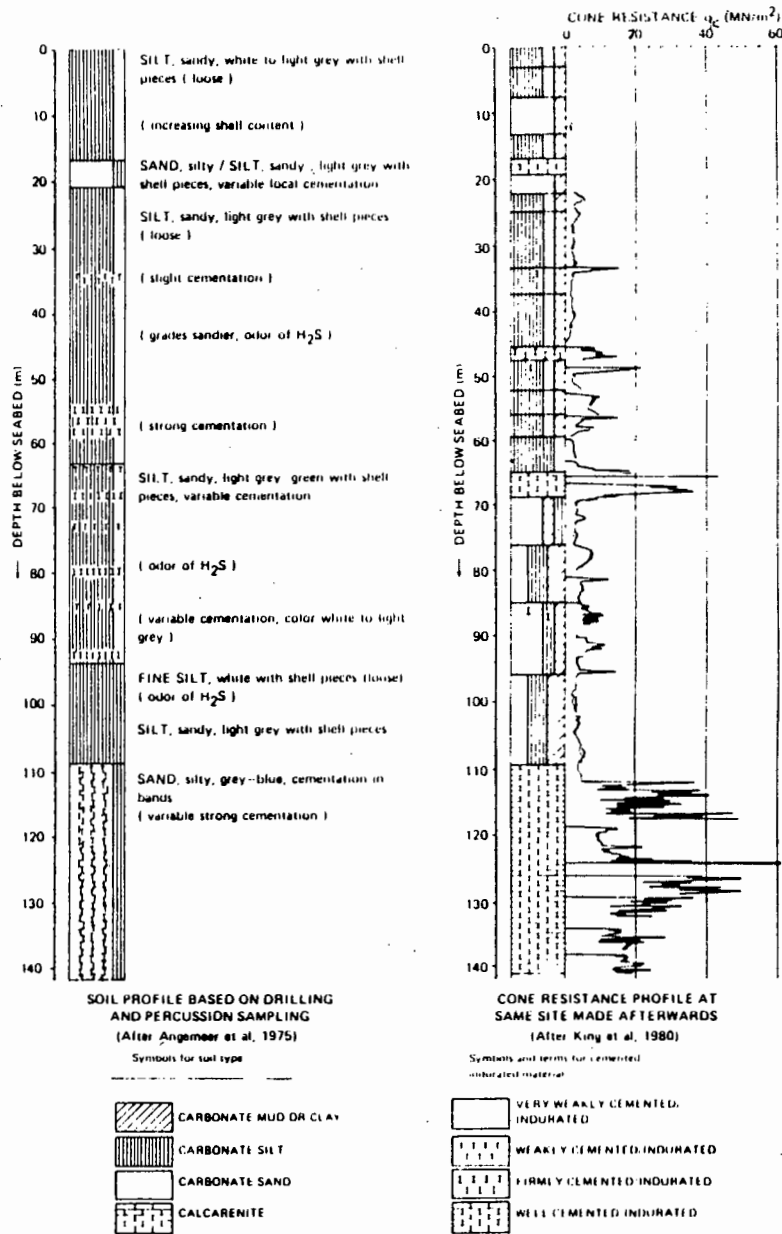


Figure 6.1 : Improvement in soil classification using CPT data
(after Beringen et al, 1982)

calcareous sands, where theoretical concepts are even less applicable than usual.

An additional advantage is that systems are available for incorporating well logging tools inside penetrometers. The most widely used sensor is the nuclear density probe which measures soil density in situ (Underwater Technology, 1987).

The test can be performed either from the seabed using the 'Seacalf' (Fig. 6.2), or from the bottom of a borehole with a wireline cone penetrometer such as the 'Wison' (Fig. 6.3) (King *et al.*, 1980). CPT testing is not suitable for floating platform operations, and calm seas are therefore a requirement if the latter method is used.

The 'Seacalf' is a remote-controlled underwater rig which can perform cone penetration tests from the seabed in water depths up to 300 m. A hydraulic jacking system provides the maximum reaction force of 200 kN. The 'Wison' is a downhole CPT device which can be used between drilling operations. The reaction force is obtained either from the weight of the drill collars or from the borehole walls by means of a roughhole packer. Intermittent profiles up to 3 m in length can be obtained. Interpretation of data and the influence of operational procedures for these two methods are discussed by Van der Zwaag and Sunderland (1982).

6.3 VANE SHEAR TESTS

The vane shear apparatus can be used to determine the undrained shear strength of saturated clays. Standard field vane apparatus can be used but it must be operated from a fixed platform. In order to overcome the latter

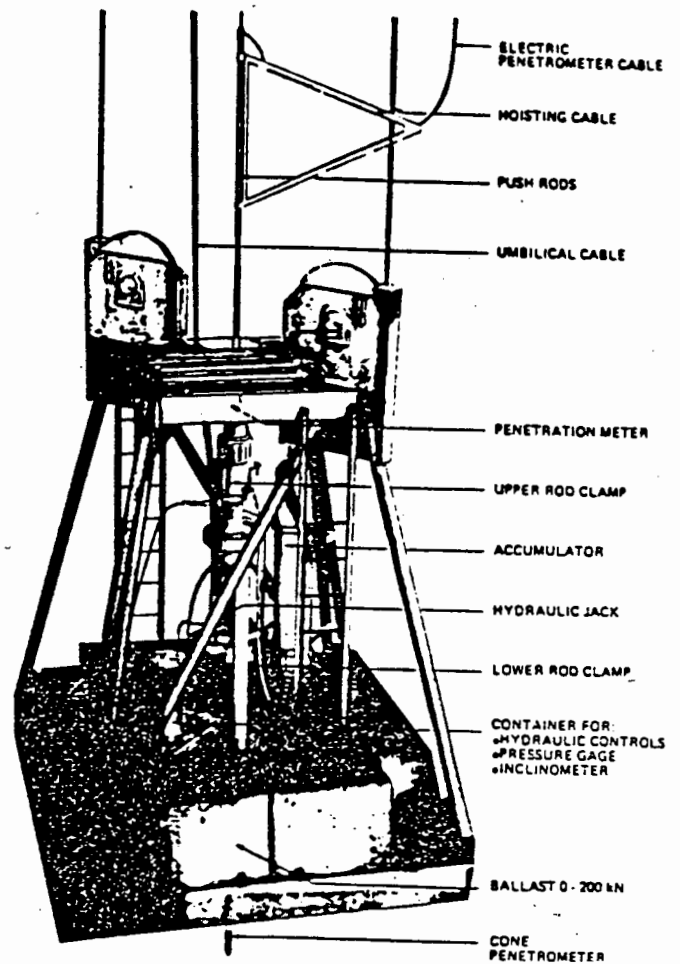


Figure 6.2 : Seacalf-seabed cone penetrometer rig
(after King et al, 1980)

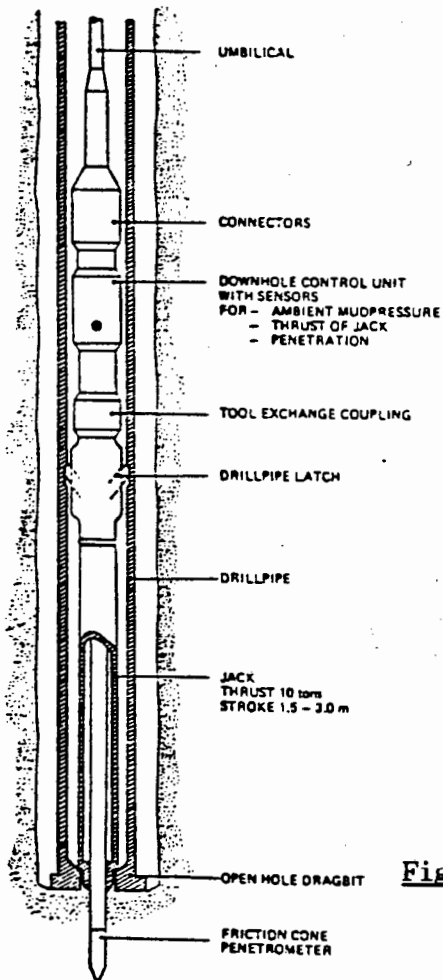


Figure 6.3 : Wireline cone penetrometer
(after King et al, 1980)

restraint, a special remote-controlled version has been developed (McClelland, 1971). The remote shear vane is an adapted version of the standard field vane which allows direct measurement of undrained shear strength of clays and convenient operation from a floating base. A pilot model described by McClelland consists of a remotely controlled probe designed for wireline operations in a marine soil boring. Onshore trials indicated that results comparable to high quality sampling techniques can be obtained with this device.

6.4 PRESSUREMETER

Prior to 1980, no proven pressuremeters for offshore use appear to have been available. A crude adaption, used for offshore investigations on the North West Shelf of Australia, is described by King *et al.* (1980). It consists of a packer built into the drillstring. After drilling was completed to test depth, the packer was inflated with oil supplied via an umbilical control cable from the vessel. Soil shear moduli and limit pressures can be deduced from the test.

Three types of pressuremeters for offshore use have subsequently been developed; the "self-boring", the "push-in" and the "full displacement". These tools have the potential of determining the in situ stiffness, strength and K_0 values of carbonate soils, but research work on the interpretation of results is still required.

A "push-in" pressuremeter developed in the United Kingdom has been used for site investigations in carbonate deposits at the North Rankin site, Australia. In spite of problems of interpreting tests, the results were extensively used in analysing lateral pile response (Fahey, 1988). Although

extreme care was taken to eliminate drilling disturbance, pressure-expansion plots obtained from the tests showed evidence of drilling disturbance. Fahey concluded that it may not be possible to avoid drilling disturbance in this type of soil, and hence to measure the in situ horizontal stress using the pressuremeter.

6.5 RECORDING OF DRILLING PARAMETERS

The method, also referred to as "instrumented rotary drilling" (IRD), basically consists in continuously measuring and recording one of several parameters as a function of depth during boring progress. Parameters which can be recorded include :

- instantaneous rate of penetration
- thrust on the drillstring
- torque on the drillstring
- pressure of the circulating fluid.

Appropriate parameters are selected as a function of their sensitivity to the properties of the anticipated material and drilling facilities. Recording of parameters requires to be strictly independent of tide variations and wave actions, which is automatically satisfied when the investigation is conducted from a fixed structure.

Bécue *et al.* (1988) found that IRD can be used to improve site investigations in carbonate soils in the following respects :

- to provide a continuous log which is representative of the vertical heterogeneity of the soil

- as a sound basis to balance mechanical characteristics as determined from laboratory tests
- in areas where CPT reaches refusal.

Bock *et al.* (1988) have shown that the drilling penetration rate in coral reef deposits can serve as a sensitive indicator of material variability. Fig. 6.4 shows the inverse of penetration rate versus depth of penetration at such a site, while Fig. 6.5 shows the results of a heavy dynamic probing at the same site.

6.6 GEOPHYSICAL METHODS

Geophysical methods such as resistivity measurements has a long and successful history in exploration for petroleum and other minerals (Hulbert *et al.*, 1982), but little data on its use in carbonate soils are available. As for many other in situ methods, extensive calibrations and correlations with physical properties are required before the results of geophysical methods can be sensibly interpreted. This necessitates laboratory testing of undisturbed samples, which is a considerable disadvantage.

Delfache *et al.* (1971) established empirical relationships between void ratios, coefficients of compressibility and seismic velocities by comparing laboratory consolidation tests with simultaneous compressional wave velocities for various clay muds from the Gulf of Mexico. Charts developed from the relationships enable a rapid and economical determination of sediment compressibilities from in situ velocity measurements. Even though the clay was non-calcareous, the method seems to hold promise for application in some clayey carbonate sediments which do not display a high degree of compositional variability.

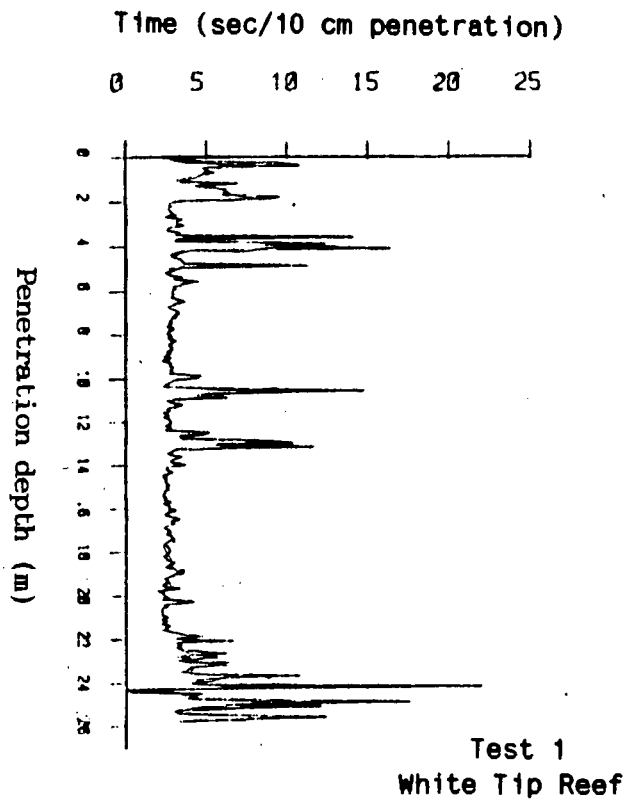


Figure 6.4 : Example of instrumented rotary drilling results from White Tip Reef site (after Bock et al, 1988)

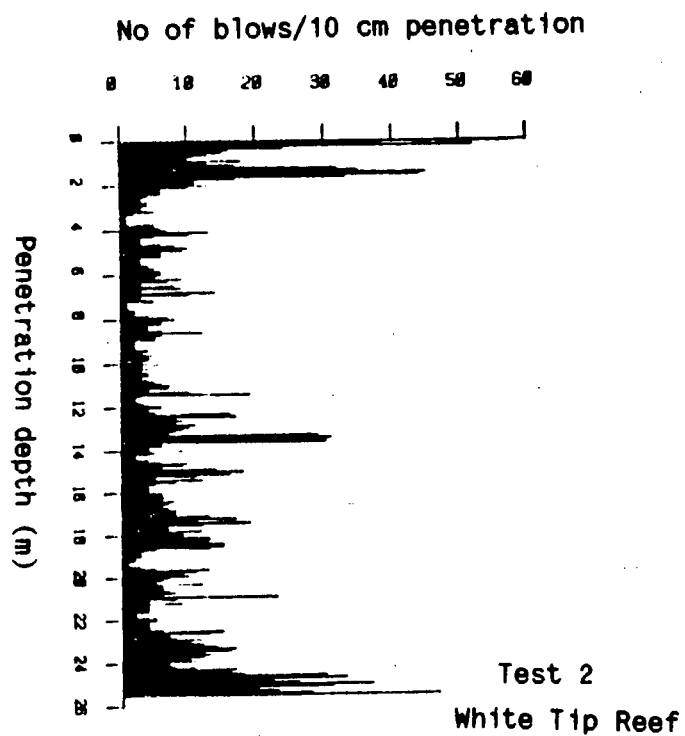


Figure 6.5 : Example of heavy dynamic probing results from White Tip Reef site (after Bock et al, 1988)

Erchul and Gulharte (1982) performed laboratory electrical resistivity measurements on carbonate materials ranging from lithified corals to oozes. The results showed that void ratios correlated well with the electrical conductivities. This provides the possibility of developing void ratio conductivity relationship for a specific sediment and indirect determination of certain index properties without extensive sampling.

An in situ resistivity probe used for differentiating layers in a soil profile was developed by Hulbert *et al.* (1982). The unit was employed by vibratory driving in calcareous sediments off the Southeast coast of Florida. The sediments in the study area did not show great variability in electrical conductivity, but lower values of resistivity were measured in materials that showed stronger seismic reflection.

7. DESIGN CONSIDERATIONS

7.1 SHALLOW FOUNDATIONS

7.1.1 Foundation Types

Shallow foundations are in contact with and are supported directly on the underlying subsurface materials, being embedded at shallow depths or bearing directly on the soil surface. Foundations of this type are found in the following situations :

- (i) Gravity-type concrete platforms and tanks placed directly on the seabed without the use of piles. These structures are pre-fabricated and assembled onshore and floated out to site, with the advantage that construction and assembly under difficult conditions is eliminated. Fig. 7.1 shows five such gravity-type production platforms and an offshore oil storage tank. (Some twenty of this type of platform were constructed during the 1970s). Foundation performance, design criteria and required soil information for this type of structure is discussed by Bjerrum (1973).
- (ii) Temporary supports for jack-up drilling rigs used in offshore site investigation and oil exploration. Typical footing dimensions and shapes for such structures are illustrated in Fig. 7.2. A mobile jack-up drilling rig faces a much greater accident exposure than most engineering structures. McClelland *et al.* (1982) estimate that more than one third of mobile rig accidents are caused by foundation failure. Design considerations for these structures are discussed by McClelland *et al.* (1982).

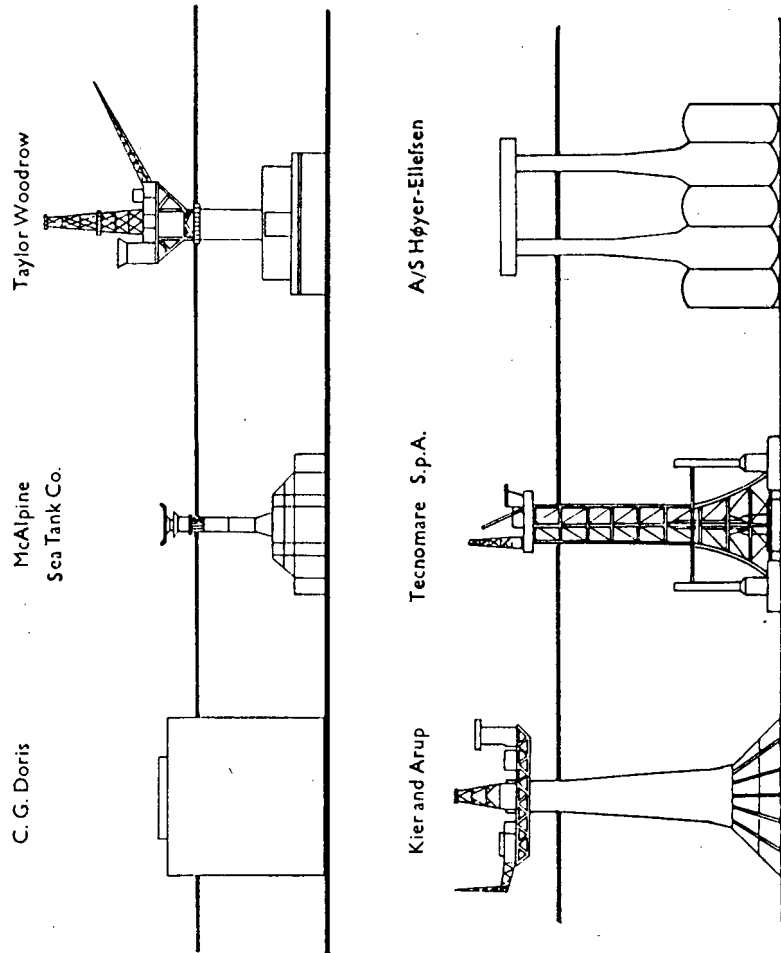


Figure 7.1 : Different types of structure designed to be placed directly on the sea floor (after Bjerrum, 1973)

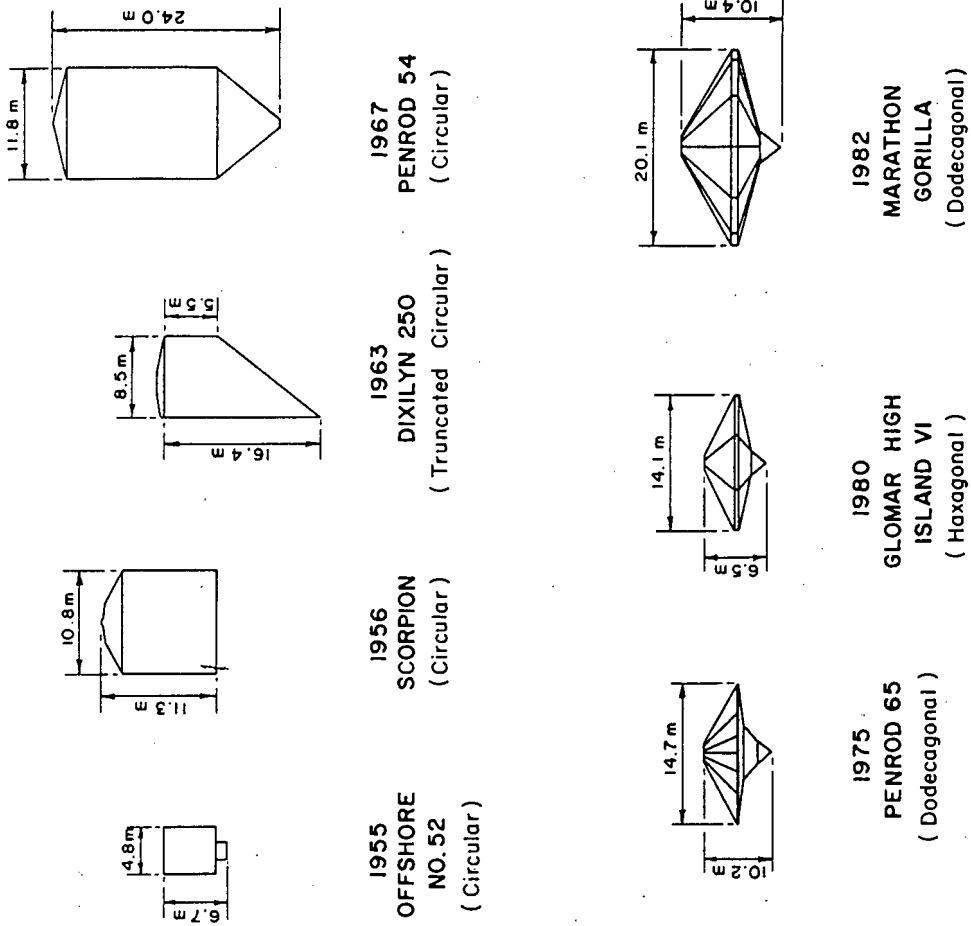


Figure 7.2 : Typical dimensions of jack-up drilling rig footings (after McClelland et al, 1982)

- (iii) Nearshore structures supported on conventional shallow foundations, including strip footings, mats, pavements, tank pads, etc.

7.1.2 Design Experience and Recommendations

There appear to be little published full-scale performance data for shallow foundations on marine carbonate sands, but information from field load tests, jack-up rig footing failures (McClelland *et al.*, 1982) and model laboratory studies (Poulos and Chua, 1985; Poulos *et al.*, 1984) give some indication of the performance of shallow foundations on marine carbonate soils.

Angemeer and McNielann (1982) performed plate loading and field CBR tests on reefoidal carbonate deposits at Diego Garcia. For moderate loads, mobilization of the internal shear strength in a bearing mode was comparable with conventional terrigenous granular soils. They concluded that, provided the subsurface is reasonably compact and continuous without large voids, conventional procedures can be used for design of lightly loaded foundations.

Where cemented rock-like carbonate layers (high bearing capacity) overlying weak calcareous silt are encountered (e.g. Arabian Gulf), punch-through failures of drilling-rig footings during preload have occurred (McClelland *et al.*, 1982). Geological conditions that produce such hazards are often found in carbonate soils, so that a primary intent of soil investigations for shallow foundations should include the determination of the subsurface continuity of the soil.

A series of laboratory model footing tests on calcareous and quartz sand was conducted by Poulos and Chua (1985) :

- For a given surcharge pressure (103 kPa), the measured drained bearing capacity for both types of soil was found to increase with increasing relative density.
- Bearing capacities on the calcareous sands were considerably less than for the quartz sands. Poulos and Chua attributed this to the volume reduction during shear experienced by the sand (Fig. 7.3).
- Bearing capacities for the calcareous sand predicted by three conventional theories were compared with measured values. These are illustrated in Fig. 7.4.
- Terzaghi bearing capacity theory grossly overestimated the bearing capacities.
- Modification of Terzaghi bearing capacity theory by reducing the friction angle (ϕ) to take account of "local shear" still overestimated the bearing capacities.
- Incorporation of the volume reduction during shear using "cavity expansion theory" developed by Vesic (1972) provided the best estimate of the actual bearing capacities.

The predicted settlement of these footings was evaluated by Poulos *et al.* (1984). Three methods of calculation were used to predict theoretical settlements :

- Conventional one-dimensional theory : substantially overpredicted settlements at the lower stress level.

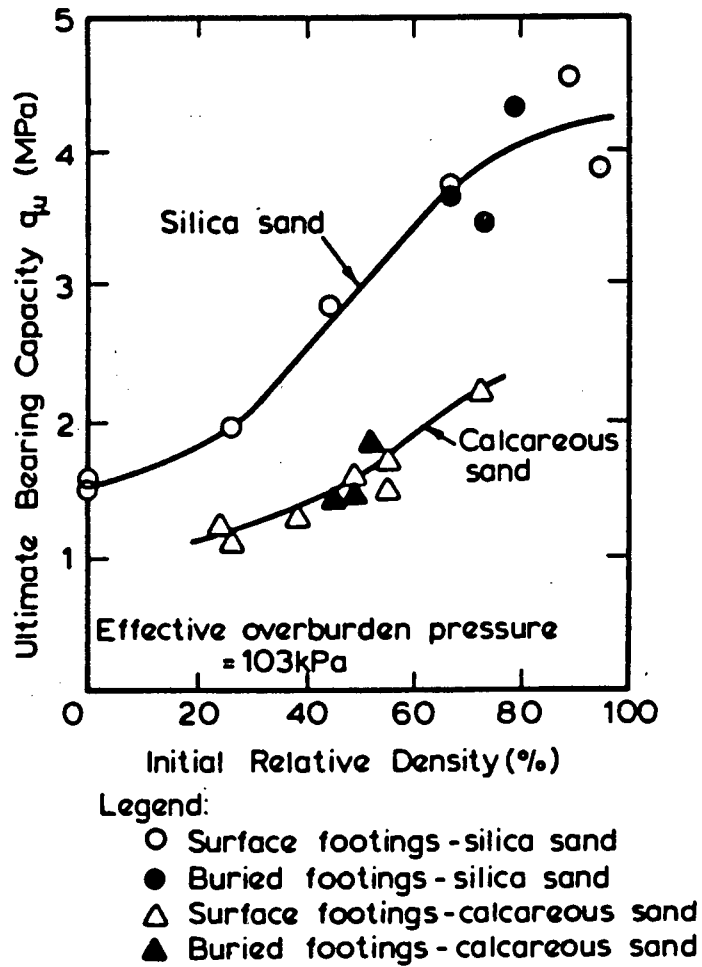


Figure 7.3 : Influence of initial relative density and footing embedment on ultimate bearing capacity (after Poulos and Chua, 1985)

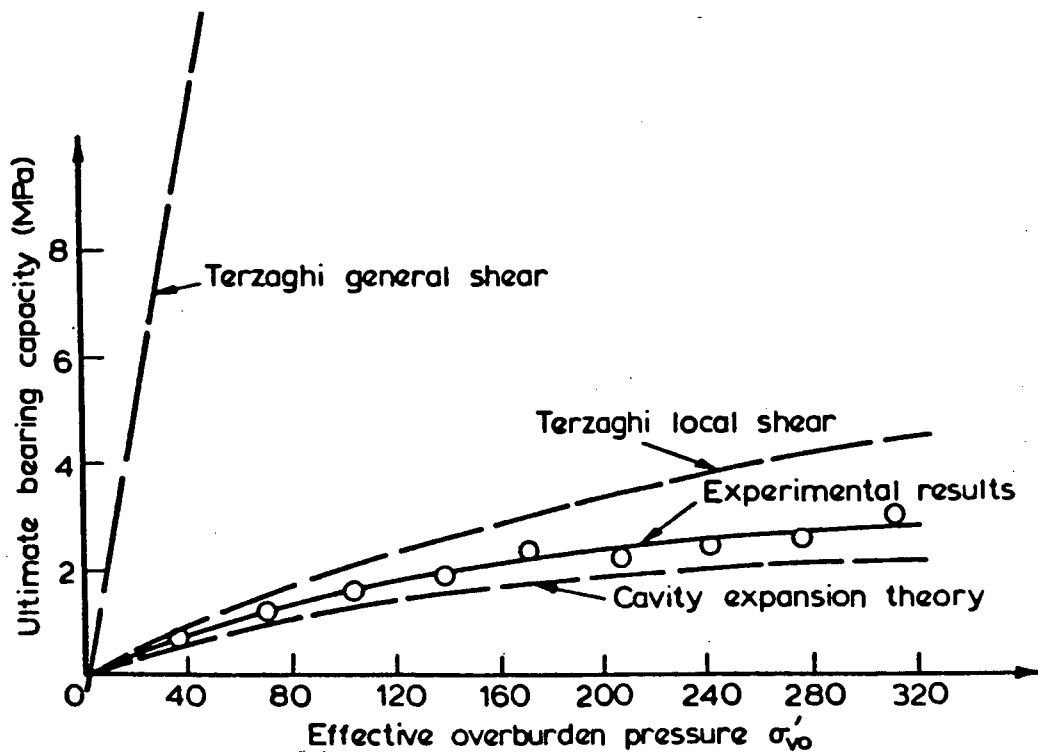


Figure 7.4 : Comparison of experimental and computed bearing capacity of footings on calcareous sand (after Poulos and Chua, 1985)

- Three-dimensional elastic displacement theory : gave acceptable settlement predictions, provided that soil moduli for the appropriate stress level relative to failure were used.
- Non-linear finite element analysis : yielded good predictions. The method is unfortunately time-consuming and expensive and not suitable for routine settlement calculations.

Dutt and Ingram (1988) evaluated the accuracy of various bearing capacity equations by comparison with full scale field performance data of jack-up rig foundations in calcareous sand and silt. They concluded that the high compressibility of these soils is the key characteristic that influences the behaviour of the footings. The "cavity expansion theory" overpredicted the bearing capacities, but to a lesser extent than those theories which do not take account of the soil volume change characteristics. The "cavity expansion theory" requires the accurate determination of soil properties such as E (Young's modulus) and ν (Poisson's ratio). Values for these parameters as determined from triaxial testing were found to be unreliable with wide scatter and poor repeatability, while more consistent results were obtained for the angle of internal friction. They therefore recommended that a reduced friction angle be used instead for a better estimation of the actual bearing capacity :

$$\phi^* = \tan^{-1} (0.33 \tan \phi) \quad ,$$

where :

ϕ = angle of internal shearing resistance of soil

ϕ^* = reduced angle of internal shearing resistance to account for soil compressibility and scale effects.

Application of this expression to a number of jack-up sites in the Middle East and Florida Coast has provided improved prediction of bearing capacity.

7.2 DEEP PENETRATION PILING

7.2.1 File Installation Techniques

Both the type of pile and the technique of installation affect pile load carrying capacity (open ended steel pipe piles appear to be the commonly used type of pile for calcareous soils). The installation techniques in order of frequency of use has been listed by McCarel and Beard (1984) :

- (a) Driving with impact hammer. This is the simplest technique and also the most troublesome in carbonate soils with respect to development of adequate pile capacity.
- (b) Drilling and grouting. In this approach an oversized hole is drilled, the pile is placed in the hole, and the annulus between the pile and soil is filled with grout. The technique is time-consuming and expensive relative to driven piling. However, large improvements in pile capacity have resulted where this technique was used in carbonate sands, with typical shaft resistance values often exceeding those for driven piles in quartz sand (Murff, 1987).
- (c) Driving, drilling and grouting combination. Used where rock layers or highly cemented strata are present so that the pile cannot be driven to its predetermined design depth.
- (d) Drilling an enlarged base, then grouting : Belled pile foundations have been used to take advantage of the high bearing resistance of carbonate rock and highly cemented calcareous soil in Saudi Arabia (Burt and Harris, 1980).

- (e) Driving with vibratory hammers : Installation with high capacity, low frequency vibratory hammers appears promising, but little data is available to evaluate the effect of this installation method on the load carrying capacity of piles in calcareous soils (McCarel and Beard, 1984).

More recently two other methods of piling installation for calcareous soils have been proposed :

- (f) Driven piles with provision for grouting : This method has been used at the Rankin Field in Western Australia. It appears to be the method likely to be used in extensions to this oil field.
- (g) "Expansion" piles in which an expanding mandril is dropped down the inside of a slit steel pipe pile, after driving (Burland, 1987).

7.2.2 Load Tests

In the absence of reliable empirical/theoretical relationships for the load carrying capacity of driven and grouted piles in carbonate sands, data obtained from load tests can aid in the identification of factors causing low capacities and selection of design parameters. This type of data can be divided into three categories :

Full-scale field tests : arbitrarily defined (Murff, 1987) as tests involving piles with a diameter greater than 15cm. These piles closely simulate the actual behaviour of the prototype pile, but the data obtained is highly site-specific.

Model field tests : performed on small scale (less than 15cm diameter) piles at a specific site. Validity of extrapolating results to full-size piles is uncertain.

Laboratory model tests : extrapolation of results to actual field piles is uncertain, but the carefully controlled conditions possible in such tests make them useful to identify factors causing low capacities.

The results obtained from these tests for driven and grouted piles and their interpretation are discussed in the following section.

7.2.2.1 Driven piles

(a) Full-scale field tests

Murff (1987) summarised the results of field load tests obtained by a number of researchers. Test details and information on soil type are presented in Table 7.1. These tests share the following common characteristics :

- the soils have high carbonate contents
- they are generally granular, although often cemented
- the reported pile capacities are lower than is typical for quartzitic sands.

Some important results from these tests are outlined below :

- For frictional materials, the value of skin resistance is considered to increase linearly with depth up to a limiting value, depending on the soil type. For calcareous soils, it appears that this limiting value

Table 7.1 : Summary of driven pile field loading tests (after Murff (1987))

References	Location	Pile description	Pile diameter D(m)	Pile embedment E(m)	Width of side for square pile, S(m)	Number of tension tests	Number of compression tests	Soil description	Remarks
Angemeer <i>et al.</i> (1973)	Bass Strait, Australia	Steel pipe, open end	0.407-0.050	45.1-102.4	-	6	8	Silty calcareous sand un cemented to highly cemented	Special compression tests to determine end bearing alone
Angemeer <i>et al.</i> (1975)	Northwest Shelf Australia	Steel casing, open end	0.34	4.9-11.0	-	2	-	Calcareous sandy silt with layers of silty sand	Test piles driven through sleeve at 39m and 73m below mudline
Stevenson and Thompson (1978)	Barbados	Square, prestressed concrete piles	-	18.6-21.9	0.406	-	8	Coral and coral sand of varying density	One test with oversize plate; five proof tests
Fuller (1979); Engling (1980)	Arabian Gulf near Ju'aymah Saudi Arabia	Circular, prestressed concrete cylinder piles	-	10.6-19.6	1.37	7	6	Thin caprock overlying well cemented carbonate sand	Piles driven in predrilled hole; majority proof tested
Hagenaar and Van den Berg (1981)	Red Sea coast of Saudi Arabia	Octagonal prestressed concrete pile with hollow core	0.60	18.0	-	1	1	Coral, carbonate sands and gravels, weakly cemented limestone rock, thin alluvial layers	
Hagenaar <i>et al.</i> (1982;1985)	Red Sea coast of Saudi Arabia	Steel pipe, open and closed end	0.614-1.42	10.3-46.0	-	4	24	Carbonate sands, coral, buried coral, coral detritus, and alluvial sediments intermixed	
Puyvelo, <i>et al.</i> (1983)	Philippines	Steel pipe, partially closed and closed end	0.762	10.0-42.0	-	-	6	Coral sands and gravels with cemented coral lenses	
Dutt and Cheng (1984)	Gulf of Suez	Steel pipe, open end	0.609	7.62-30.49	-	12	-	Carbonate sand, weakly to moderately cemented	Each pile tested at least twice at different depths
Dutt <i>et al.</i> (1985)	Philippines	Steel pipe, open end	1.06	40.5-67.0	-	4	-	Coralline limestone and coral gravel; carbonate silt and siltstone	
Gilchrist (1985)	Red Sea coast of Saudi Arabia	Steel pipe, open end	1.422	11.0-44.0	-	5	5	Honeycomb coral, coralline silty sand	Two piles tested in compression and tension at several penetrations
Nauroy and Le-Tirant (1985)	Western France	Steel pipe, open end	0.30	23.0	-	1	-	Uncemented carbonate sands with thin cemented seams	

is relatively small and occurs at shallow depths (e.g. Datta *et al.*, 1980).

- Based on the assumption that the shaft resistance increases linearly to a depth of 10m and then remains constant, Murff (1987) calculated values of limiting skin friction for some of the tests shown in Table 7.1. These results are presented in Table 7.2. Values for f (limiting skin friction) were typically only 20 to 25% of values used for silica sands. [c.f. American Petroleum Institute (API) Recommendation (1984); $f = 80$ kPa for medium dense silica sand.]
- Field tests by Angemeer *et al.* (1973) indicated that cyclic loading can cause reduced stiffness and lowered peak shaft resistance.
- Soil "setup" (increased resistance after a delay in driving) in calcareous sand have been observed (Angemeer *et al.*, 1973, Nauroy and Le Tirant, 1985) and have been attributed to both pore water dissipation and recementation (Murff, 1987).
- Tests conducted by Gilchrist (1985), Angemeer *et al.*, (1973) and Hagenaar *et al.* (1985) allowed determinations of end bearing capacities to be made. The tests suggest a very wide range of end bearing values [1.3 to 36 MPa, c.f. 5 MPa API recommended (1984) for medium dense silica sand].

(b) Model Field and Laboratory Tests

Poulos and Chan (1984) obtained laboratory data on the skin friction characteristics of model piles in calcareous sand. It was demonstrated that the ultimate skin friction was generally reduced following cyclic loading and that the reduction depended primarily on the amplitude of cyclic displacement. Significant cyclic degradation of skin friction commenced when the cyclic displacement amplitude reached the static displacement required to mobilize full skin friction. Other factors which influenced

cyclic degradation were :

- method of pile installation
- relative density of the soil
- the overconsolidation ratio.

Yuan and Poulos (1986) investigated the effect of loading rate on pile skin friction in reconstituted quartz and carbonate sands. The tests showed that the ultimate skin friction of the pile increased as the loading rate increased in carbonate sands, but that there was no such effect in quartz sands.

7.2.2.2 Drilled and grouted piles

(a) Full-scale field tests

Details of some reported field load tests on drilled and grouted piles are presented in Table 7.3. Driven piles at these sites were found to have negligible shaft resistance. From the results in Table 7.3 it can be seen that drilling and grouting considerably increases pile capacities in calcareous sands. The use of drilling mud appears to have a large deteriorating effect on bearing capacity (Angemeer *et al.*, 1973).

(b) Model field and laboratory tests

Grouted section tests carried out in the laboratory as well as on small model piles in the field (Nauroy and Le Tirant, 1985) have confirmed that relatively large shaft resistance values may be developed by grouting.

7.2.3 Design Practice for Driven Piles

In his state-of-the-art review on piling in calcareous sands, Murff (1987) concluded that the prediction of pile capacity remains a highly speculative

Table 7.2 : Limiting skin friction based on pile test interpretation (after Murff (1987))

Reference	Number of tests analyzed	Range of peak values (kPa)	Mean peak value (kPa)	Standard deviation (kPa)	Coefficient of variation
Angemeer, <i>et al.</i> (1973)	7	9.2-18.3	13.4	3.3	0.25
Angemeer, <i>et al.</i> (1975)	1	33.1	-	-	-
Hagenaar, <i>et al.</i> (1981; 1982)	5	16.7-22.5	20.3	2.2	0.11
Dutt and Cheng (1984)	12	9.8-18.2	13.3	2.5	0.19
Dutt <i>et al.</i> (1985)	4	9.5-17.3	-	-	-
Gilchrist (1985)	4	11.5-21.0	17.0	4.3	0.25
Nauroy and Le Tirant (1985)	1	1.0	-	-	-

Table 7.3 : Full scale load test data for drilled and grouted piles (after Murff (1987))

Reference	Average shaft capacity (kPa)	Comment
Wees and Chamberlin (1971)	85	Piling for underwater oil storage tank
Angemeer <i>et al.</i> (1973)	10 70	Bass Strait Holes drilled with mud Holes drilled with seawater
Angemeer <i>et al.</i> (1975)	81 95	Bass Strait : Grouted section Tests at 90-135 m
Naurey and Le Tirant (1985)	100	Driven piles at same site had negligible capacity

exercise. "The range of design parameters is so large (shaft resistance and end bearing) that use of a single procedure is likely to be either unconservative or uneconomical".

Design practice for determining the ultimate axial capacity of piles in calcareous sediments can be divided into four categories (McCarel and Beard, 1984) :

(a) Conventional theory with modifications to account for certain aspects of calcareous soils

With this approach, it has first to be established whether the potential for low bearing capacity exists. No specified procedures for quantitatively determining low potential capacities from conventional laboratory or in situ testing techniques have yet been developed, but the following characteristics appear to be good indicators :

- Carbonate content : There is widespread agreement that sands with carbonate contents in excess of 80% have the potential for abnormally low capacities, but there is however no agreement as to the lower limit.
- Compressibility : Nauroy and Le Tirant (1985) have collected data on piles in calcareous sands in laboratory and field tests that indicate a good inverse correlation between compressibility and shaft resistance (see Fig. 7.5 and Table 7.4).
- Nature of particles : Datta *et al.* (1982) suggested that carbonates that contain few intraparticle voids (and are less crushable) approach the behaviour of non-carbonates.
- Cementation and void ratio : It has been argued that uniform, well cemented calcareous sands give high shaft resistances and that weak,

Table 7.4 : Limiting unit shaft resistance versus soil compressibility index (after Nauroy and Le Tirant (1985))

Sand	Soil description		
	Particle Characteristics	Carbonate content	Particle size
C1	Shelly sand	90.0	Uniform, coarse
C2	Coralline and shelley	83.0	Well graded coarse
C3	Coralline and shelley	70.0	Uniform, medium
S	Quartz	0	Uniform, medium

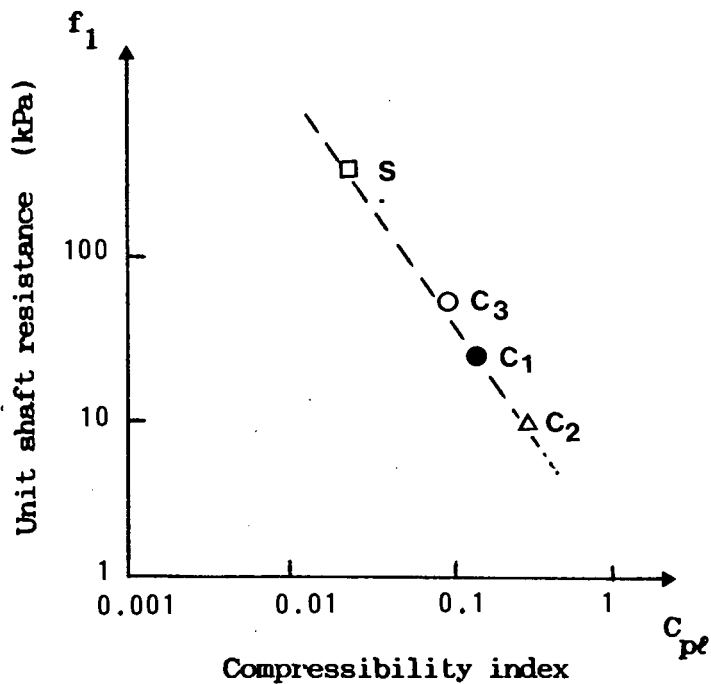


Figure 7.5 : Limiting unit shaft resistance versus soil compressibility index (after Nauvroy and Le Tirant, 1985)

C_{pc} is defined as the slope of the $e-f_n(p)$ curve for $p = 800$ kPa, in an isotropic triaxial test.

partial or irregular cementation is a cause of low shaft resistance (Angemeer *et al.*, 1975; Beringen *et al.*, 1982), although a combination of low void ratio and cementation is probably more significant than the degree of cementation alone (Murff, 1987).

Limiting design parameters have been suggested by a number of authors :

McClelland (1974)	:	20 kPa (shaft resistance)
Agarwal <i>et al.</i> (1977)	:	28 kPa (carbonate soils)
	:	32 kPa (soils grading between carbonate and non carbonate)
Datta <i>et al.</i> (1980)	:	15 kPa (uncemented calcareous sands)
	:	55 kPa (well cemented materials).

(b) Empirical correlation with resistance to driving

This approach correlates the driving resistance to the axial pile capacity of piles using empirical formulae. Results of work conducted at Diego Garcia indicated that driving resistance is not a good indication for predicting pile capacity in calcareous soils (McCarel and Beard, 1984).

(c) Correlation with in situ penetration tests

Standard penetration resistance have been used for calculating pile capacity in chalk, but the correlation yields a wide scatter in results. The use of the Dutch cone penetrometer test to obtain strength parameters for piles in calcareous soils is discussed by Beringen *et al.* (1982).

(d) Full-scale pile load tests

Due to the difficulties associated with design and conventional testing of calcareous soils, this method is still regarded by many authors as the only

reliable method for taking site-specific parameters into consideration. This approach is unfortunately very expensive.

7.3 ALTERNATIVE PILE CONCEPTS

The poor development of skin resistance in open-ended pipe piles in calcareous soils has led to interest in the development of piles which do not suffer loss of capacity on account of its unsuitability for this soil type.

Lack of adequate lateral pressure due to a combination of grain crushing and volumetric contraction having been identified as the major cause of such low capacities, improved piling systems must incorporate a combination of the following features :

- must increase the effective lateral stress on the pile shaft
- must force the pile to transfer load to the zone of soils where degradation due to grain crushing is minimal
- should eliminate or reduce the effect of grain crushing and associated volumetric contraction as well as soil arching
- should increase contact area between pile and sediment.

The 'Burland' type of expanding pile and grouted pile technique are methods which endeavour to exploit these features.

8. RECOMMENDATIONS FOR THE BASIS OF AN IMPROVED CLASSIFICATION SYSTEM

8.1 RELEVANT INDEX PROPERTIES AND DESCRIPTIONS

The relevance of calcium carbonate content and the degree of cementation, recommended as index properties in the classification systems proposed by Beringen *et al.* (1982), Clark and Walker (1977) and Fookes and Higgenbotham (1975) have been examined in Sections 4.1 and 4.2. The effect of crushability, particle size distribution and Atterberg Limits, as recommended by Datta *et al.* (1982) were examined in Sections 4.3 to 4.5. It was found that all of these properties can exert a strong influence on the engineering behaviour of the soil, and that their inclusion in an engineering classification or description system is therefore warranted.

In addition, the importance of mass physical properties, which have not previously been considered as an index in the published literature, was examined in Section 4.6. It is suggested that, due to the dominant influence which the constituent particle characteristics have on the mass physical properties, the limiting densities can be used as a qualitative indicator of the likely engineering behaviour (Section 4.6.4.). This conclusion has also been independently reached by recent authors in "Proceedings, Engineering for Calcareous Sediments, Perth, 1988".

- The importance of mass physical properties (e.g. density, void ratios) on engineering behaviour of carbonate soils is now widely recognised. Many authors include some measure of "limiting" densities in their routine soil descriptions, while some (e.g. Airey *et al.*, 1988) have also suggested that the maximum obtainable void ratio should form part of the classification system.

- Researchers are now in agreement that the soil compressibility is the key parameter which influences the magnitude of skin friction and bearing capacity which can be mobilized by carbonate soils. Because compressibility data can be directly applied to the practical design of piles (e.g. Peuch *et al.*, 1988) it has been suggested that some form of compressibility parameter should be included in the classification system.

It appears that, at this stage, the index properties discussed above would provide an improved engineering classification or description system, although it is recognised that other parameters such as permeability might also be of importance.

8.2 QUANTIFICATION OF INDEX PROPERTIES AND STANDARDISATION

With the exception of the degree of cementation, and to an extent, particle crushability, all the suggested indices can be quantified with the aid of standard laboratory equipment used in routine soil investigations. One of the major requirements of index testing is therefore satisfied.

Further research on the quantification of cementation is required. A qualitative indication of the degree of cementation can be obtained in situ with the aid of the static cone penetrometer (Chapter 6). When undisturbed samples are available, the presence of cementing bonds might be deduced from the apparent overconsolidation behaviour of the sample in standard laboratory tests (Section 4.2.1).

Particle crushability, limiting densities and compressibility can be quantified using a wide variety of equipment and procedures. Results

obtained depend on the equipment and procedure used, operator error and definition of parameters. Since much of the research up to now has centred on the identification of relevant index properties, little attention has been given to standardisation of testing. That this situation is still prevalent is confirmed by papers in "Proceedings of Engineering for Calcareous Sediments (March 1988)".

If the primary function of a classification system is to be satisfied, i.e. to serve as a rational basis for comparison and delineation of different engineering properties and behaviour, then clearly urgent attention should now be given to standardisation of carbonate index tests and other parameters. Such a system would have practical significance, since it would provide a clearer indication of the extent to which site-specific data can be extrapolated to other soil conditions.

8.3 FURTHER RESEARCH

Mass physical properties, compressibility and crushing of particles are to a certain extent interrelated. If these parameters are included in a classification system, it would be of interest to know how the interdependence between these parameters are affected by variables such as particle size and loading condition. For example, the shearing of a coarse carbonate sand could result in extensive crushing with relatively minor accompanying volume decrease, while shearing of a calcareous ooze (e.g. Valent *et al.*, 1982) could result in large volume decrease with little associated evidence of particle crushing. It would therefore appear that description of the soil by a single parameter expressing the compressibility would not be sufficient.

Most researchers have investigated soil crushability using the triaxial apparatus and "natural" soils with varying particle sizes and compositions within each soil type. The exaggerated response of artificially constituted samples in which it is possible to control the variables such as particle shape and size will help to identify the relative sensitivity of the index properties to changes in particle composition and other characteristics. Consequently a limited experimental program was undertaken using the direct shear testing equipment and artificially constituted samples in order to :

- evaluate the effectiveness of the direct shear test for soil classification (e.g. crushability, angle of friction, volume change and stress-strain behaviour);
- evaluate the dependence of the "maximum obtainable void ratio" on a number of variables (crushability, angle of friction, particle size).

9. DIRECT SHEAR TEST

9.1 DIRECT SHEAR EQUIPMENT

The direct shear test, undertaken using a conventional "shearbox", can be used to determine the shear strength of both cohesive and non-cohesive soils, in terms of total stress. Effective stress conditions cannot be determined because the pore pressure cannot be established accurately. The basic concept of the direct shear test is the direct measurement of the shear strength of the soil by causing failure along a predetermined horizontal plane whilst subjecting the sample to a confining load applied normal to that plane.

The test parameters are determined as follows :

- (a) A sample of known cross-sectional area and height is confined within two identical shearbox halves (Fig. 9.1).
- (b) A normal confining load (N) is applied to the upper face of the specimen, through a loading pad (Fig. 9.2).
- (c) With the bottom section held stationary, a horizontal force is applied to the upper half of the split box by a worm-driven ram, causing a relative horizontal displacement between the two halves.
- (d) The relative horizontal displacement of the two halves results in shearing of the sample approximately along the dividing plane of the box.

During the test, 3 parameters can be continuously determined : the relative displacement of the two halves of the shearbox, the horizontal shearing force and the vertical displacement of the loading cap. The measured

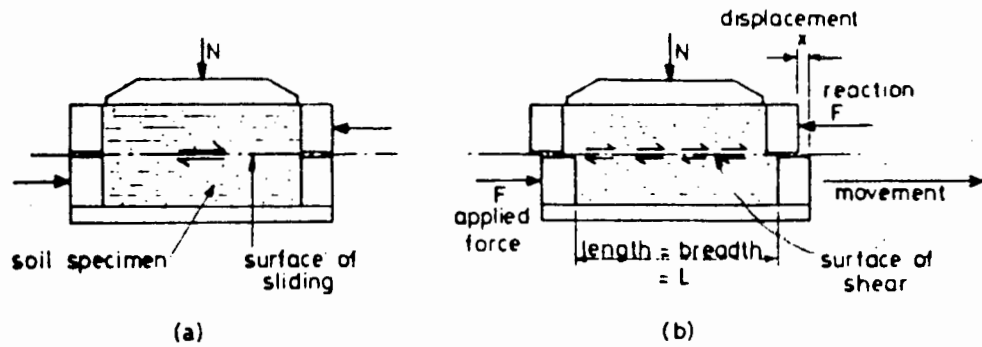


Figure 9.1 : Principle of shearbox test : (a) start of test,
 (b) during relative displacement (after Head, 1982)

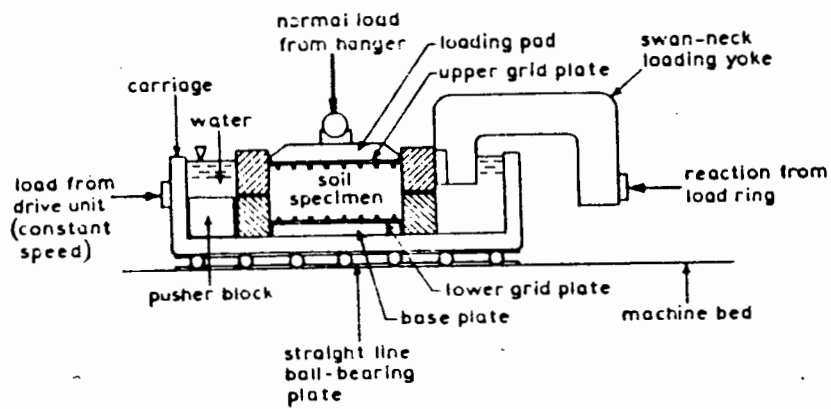


Figure 9.2 : Arrangement of conventional 60 mm shearbox apparatus
 with displacement control (after Head, 1982)

vertical and horizontal displacements may be used to obtain a qualitative indication of the applied "strain" and the volume change characteristics of the sample.

Experimental details of the test apparatus and procedures used are given in detail in Head (Vol. 2, 1982).

9.2 CALIBRATION OF EQUIPMENT

(i) Sample volume

In order to determine the void ratio of a sample, an accurate estimate of the sample's volume is required. The components of the shearbox assembly are shown in Figs. 9.3 and 9.4, and the dimensions required to calculate the sample height (and hence the volume) are given in Fig. 9.5. These dimensions, determined to the nearest 0.01mm using vernier calipers, are listed in Table 9.1.

Table 9.1 : Measured quantities for sample height and volume determination

Shearbox size	t_1	q	p	r	n	L'	t_2	t_3	B	L
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
60mm x 60mm	6.48	1	1.8	1.54	15	59.45	2.19	6.46	50.80	59.9

The height of the sample can be determined using the following expression :

$$H = B - (t_1 + n_2 t_2 + n_3 t_3 + x') \quad (1)$$

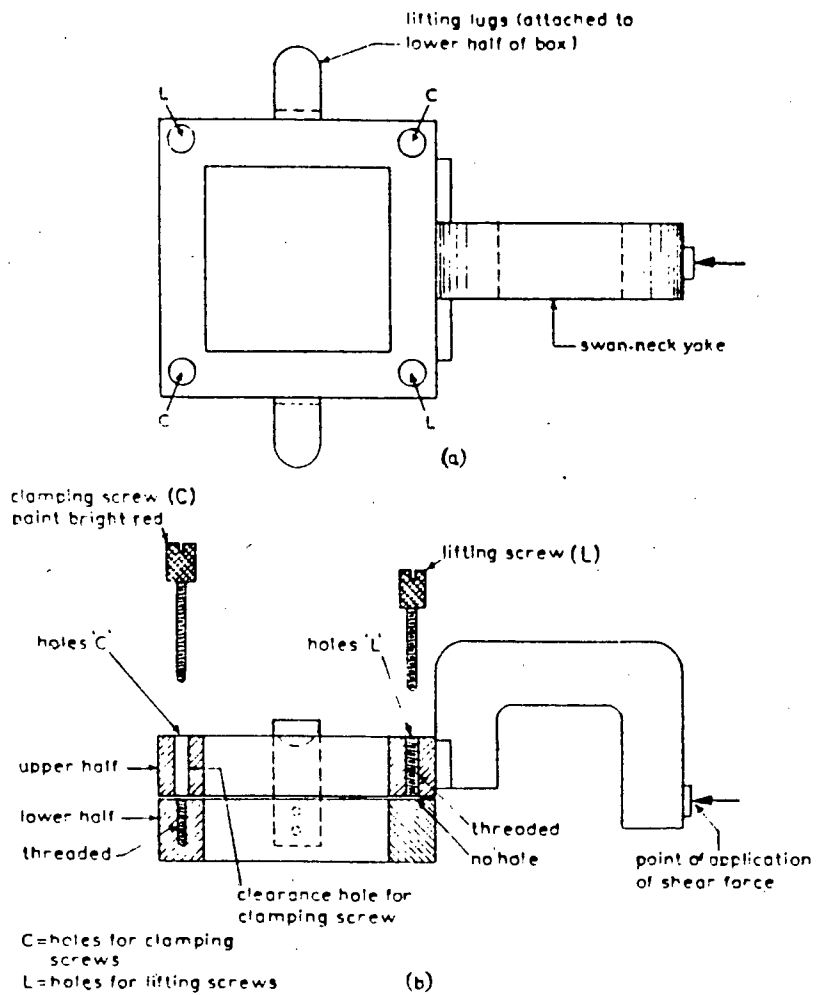


Figure 9.3 : Details of 60 mm shearbox : (a) plan view
(b) section through box (after Head, 1982)

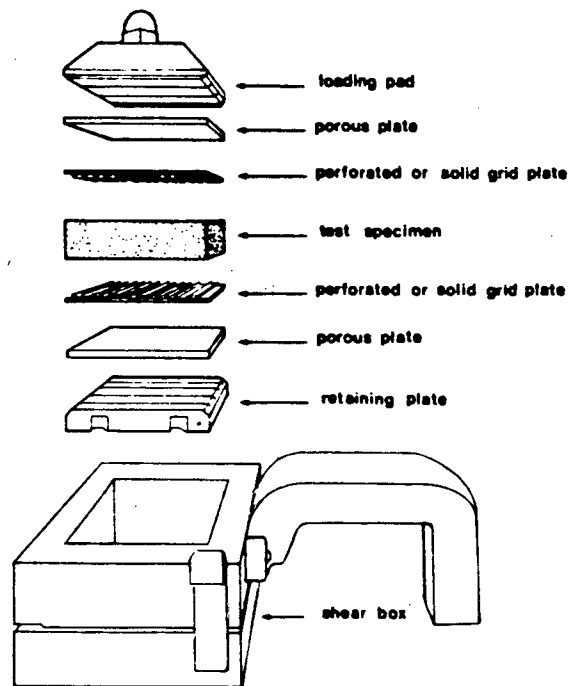


Figure 9.4 : Assembly of 60 mm shearbox (after Head, 1982)

where:

H = height of sample

B = depth of two shearbox halves

t_1 = thickness of base plate

t_2 = thickness of grid plate

t_3 = thickness of porous stone

n_2 = number of grid plates

n_3 = number of porous stones

x' = distance from top of shearbox to top of grid plate or porous stone.

For the nominally 60mm x 60mm Wykeham Farrance shearbox used, this expression reduced to :

$$H = 44.32 - 2.19 n_2 - 6.46 n_3 - x' \quad (\text{mm}) \quad . \quad (2)$$

The sample volume can be calculated as :

$$V = H.A$$

where

$$A = L \times L = \text{plan cross-sectional area of shearbox.}$$

(ii) Normal stress

The arrangement of the levers for applying normal vertical confining pressure to the shearbox specimen is shown in Fig. 9.6. The normal stresses were determined using the following relationship :

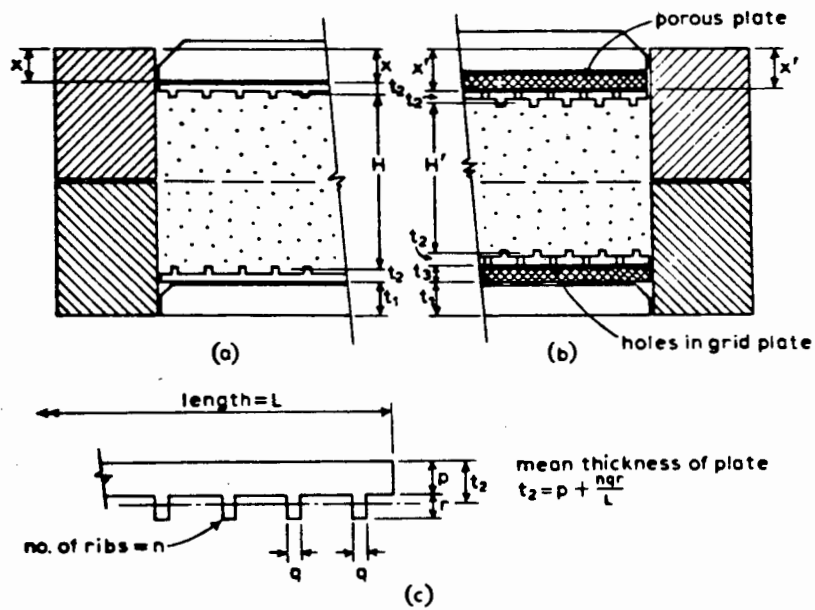


Figure 9.5 : Grid plates in shearbox : (a) solid grid plates, (b) perforated plates with porous plates, (c) details of grid plate (after Head, 1982)

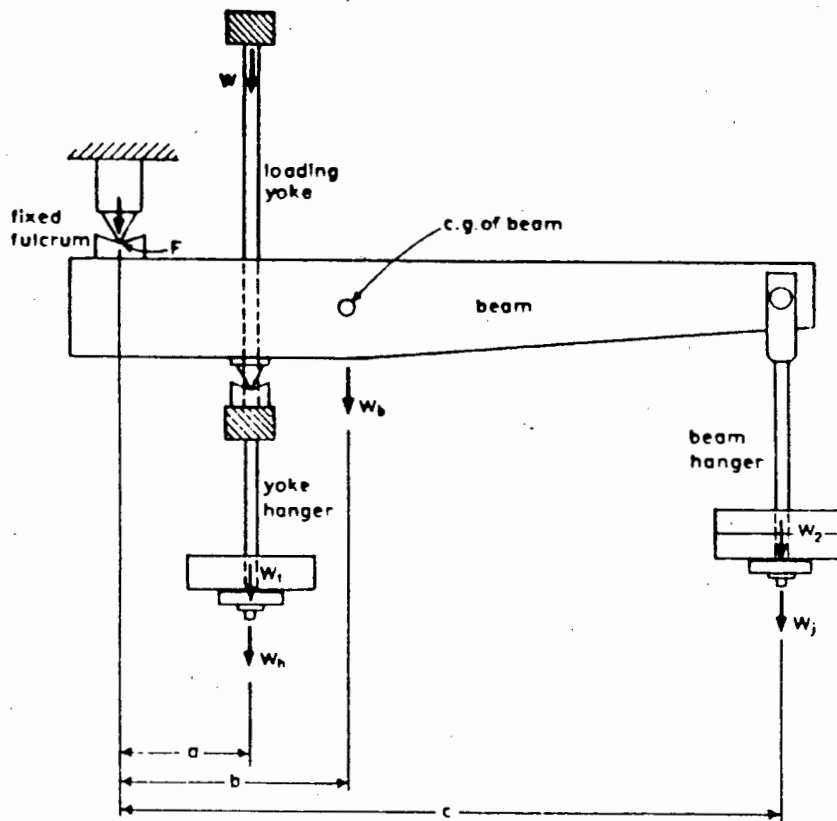


Figure 9.6 : Details of lever arm for applying high normal pressures to shearbox specimen (after Head, 1982)

$$\begin{aligned}\sigma_n &= \frac{9.81 W}{L^2} \cdot 1000 \\ &= \frac{(9.81)1000}{L^2} \left[w_h + w_1 + w_b \frac{b}{a} + (w_j + w_2) \frac{c}{a} \right]\end{aligned}\quad (3)$$

where

σ_n = normal stress (kN/m²)

L = plan dimension of square shearbox (mm)

w = effective load (kg)

w_h = mass of loading yoke and hanger (kg)

w_b = mass of lever arm beam (kg)

w_j = mass of beam hanger (kg)

w_1 = load on hanger (kg)

w_2 = load on beam hanger (kg)

a, b, c = distance of load centre of gravities from fixed fulcrum.

The masses were determined to the nearest 0.001 kg, by weighing on an electronic weighing scale, and the distances a, b and c were measured to the nearest 0.1mm. The measurements are listed in Table 9.2.

Table 9.2 : Measured quantities for normal load determination

Shearbox size	w_h (kg)	w_b (kg)	w_j (kg)	a mm	b mm	c mm
60mm x 60mm	4.368	3.322	0.962	50.8	120	253

Substituting the values given in Table 9.2 into equation (3), the following expression for normal stress is obtained for the 60mm x 60mm shearbox used :

$$\sigma_n = 46.549 + 2.734 w_1 + 13.671 w_2 \quad (4)$$

The accuracy of computing the normal stress using this expression was checked by directly measuring the loads transferred to the top loading pad of the shearbox, using an electronic weighing scale (Teraoka Seiko, Model S-Bk). This calibration data is listed in Table 9.3, from which it can be seen that the calculated loads exceed the measured loads. The error is likely to be due to friction and measurement inaccuracy. To compensate for this error, equation (4) was adjusted to :

$$\sigma_n = 40.05 + 2.734 w_1 + 13.671 w_2 \quad . \quad (5)$$

Figure 9.7 shows the Mohr failure envelopes for a standard cohesionless sand obtained by using equations (4) and (5) to calculate the normal stresses. The zero intercept resulting from the use of equation (5) is an indication of the validity of the adjustment.

9.3 INSTRUMENTATION

The overall instrumentation for the direct shear apparatus is shown in Figure 9.8. Proving ring deflections and the vertical displacement of the samples were measured with linear variable differential transformers (LVDT's). An analogue plotter provides a plot of shear stress versus horizontal strain while the test is running.

9.4 DATA RECORDING AND ANALYSIS

The shear force and vertical displacement was logged electronically by computer using the program, SAMPLING. This program, consisting of 600 programming lines in BASIC, performs the following functions :

Table 9.3 : Calibration data for normal loads

		Measured (kg)	Calculated using eqn. (4) (kg)	Calculated using eqn. (5) (kg)
<u>Loads :</u>				
w_h		3.99		
$w_h + w_b$		9.42		
$w_h + w_b + w_j$		14.18	14.55	
<u>Additional loading :</u>				
<u>w_1 (kg)</u>	<u>w_2 (kg)</u>			
0	1.137	19.74	22.71	20.33
4.542	1.137	24.26	27.25	24.87
4.542	5.511	46.00	49.12	46.75
5.542	5.511	46.98	50.12	47.75
6.542	5.511	46.96	51.12	48.75
4.542	5.511	46.00	49.12	46.75
4.542	7.511	55.96	59.12	56.75

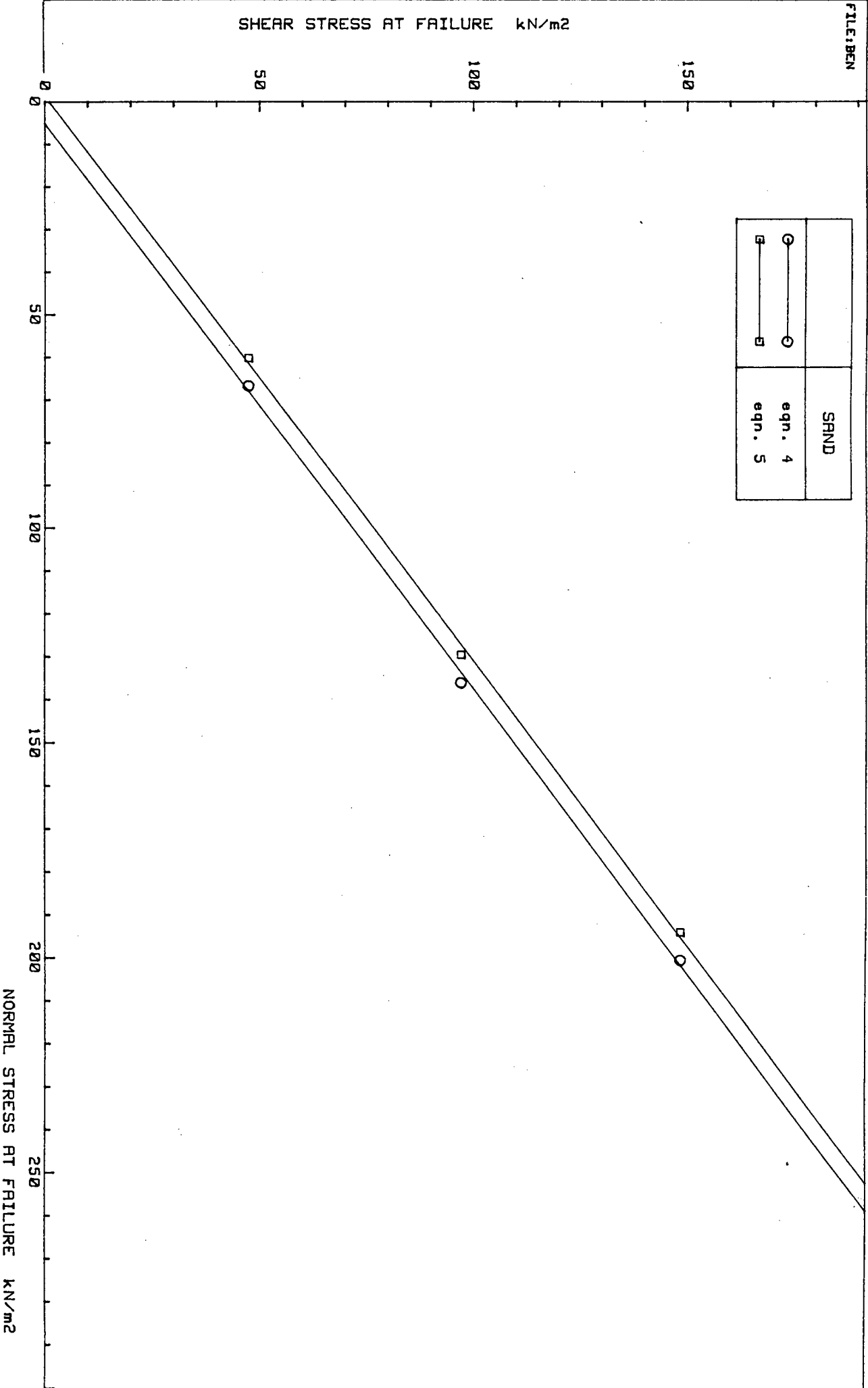


Figure 9.7 : Mohr failure envelopes for a standard cohesionless sand resulting from the use of equations (4) and (5) to calculate normal stresses

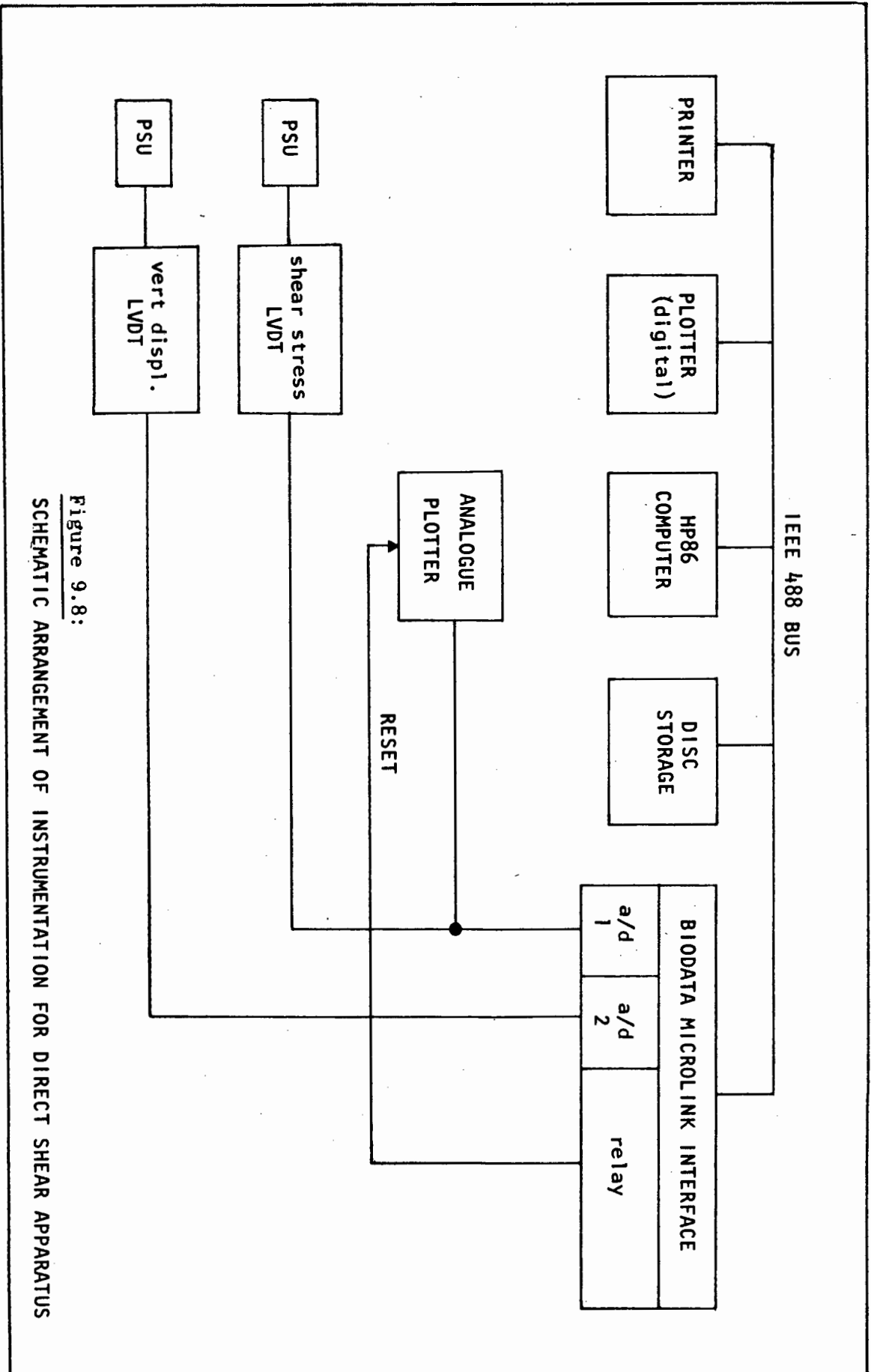


Figure 9.8:
 SCHEMATIC ARRANGEMENT OF INSTRUMENTATION FOR DIRECT SHEAR APPARATUS

- converts voltage signals from the two LVDT's to digital values of equivalent shear stress and vertical displacement;
- stores data;
- draws plots of shear stress and vertical displacement versus horizontal strain.

Mass physical properties, sample heights and friction angles were calculated using the program, SHEARBOX. Proving ring deflections obtained independently from dial gauge readings were used as input in this program to calculate shear stresses and friction angles. A listing of the program is given in Appendix A.3.

10. CONTROL TESTS ON REPEATABILITY AND FRICTIONAL PROPERTIES

10.1 OBJECTIVES

Control tests were conducted using the 60mm x 60mm direct shear apparatus in order to :

- Establish the variability in measured void ratios and friction angles for standardised samples repeatedly tested under "identical" conditions;
- Determine the effect of sample size on repeatability;
- Compare the shear strength behaviour of these materials under a fixed, constant normal pressure;
- Evaluate the effect of particle shape and mineralogy on compressibility and frictional properties.

10.2 SAMPLE PROPERTIES

A range of samples comprising a wide spectrum of average particle sizes, shapes and mineralogies were used for the control tests. These included a well rounded quartz sand (SR), two finer grade quartz sands (HP, SI), diatomaceous earth (DE), two crushed limestones (K2, K10) and crushed shell fragments (SH). The carbonate content, mean particle size and shape of these samples are listed in Table 10.1.

10.3 SAMPLE PREPARATION AND TESTING

The silt size control samples (K2, K10, DE) were placed in the shear mould in three layers. Each layer was firmly compacted by applying a uniform

Table 10.1 : Carbonate content, particle size and shape characteristics of control samples

Sample	Code	Carbonate content (%)	Ave. particle size (μm)	Shape
Quartz sand	SR	0	750	Well rounded
Quartz sand	HP	0	250 ($C_u = 1.73$)	Rounded
Quartz sand	SI	0	250 ($C_u = 1.21$)	Rounded
Diatomaceous earth	DE	0	50 * (silt)	"Complex" (see Plate 1)
Powdered limestone	K2	99	2.25 (silt)	Rhombohedral
Powdered limestone	K10	99	10	Rhombohedral
Crushed shell	SH	82	1400	Platey, angular

* estimated from electron microscopy given in Plate 1.

pressure with a smooth, flat perspex instrument. The sand size materials were placed in the mould in a single layer by pouring the sand through a 7mm diameter funnel from a constant height of 100mm. No vibration was applied to these samples.

The samples were placed in the shear box carriage, the initial heights were measured (to determine the unconsolidated void ratios, e_u) and then a normal stress of 193 kPa was applied to consolidate the sample. Sample heights were again measured after the consolidation stage (to determine the void ratio at the start of the test, e_o) and then sheared at a constant rate of strain of 1.2mm/min. All samples were sheared in an air dried condition.

Data was recorded electronically using an analogue-to-digital converter and computer. Stress-strain curves were drawn using the program, SAMPLING. Plots of shear stress and vertical displacement versus horizontal shear strain for these tests are shown in Figures 10.1(a) to (e). The associated void ratios and peak and "ultimate" friction angles listed in Table 10.2 were calculated using the program, SHEARBOX.

10.4 DISCUSSION OF RESULTS

(i) Repeatability

The standard deviations in measured values of void ratio and friction angle of the control samples are listed in Table 10.2. The largest standard deviations were recorded for samples of diatomaceous earth (DE), which also exhibited the highest void ratios. The normalised standard deviations (s/\bar{x}), however, were similar to those of the other samples, as shown in Table 10.2.

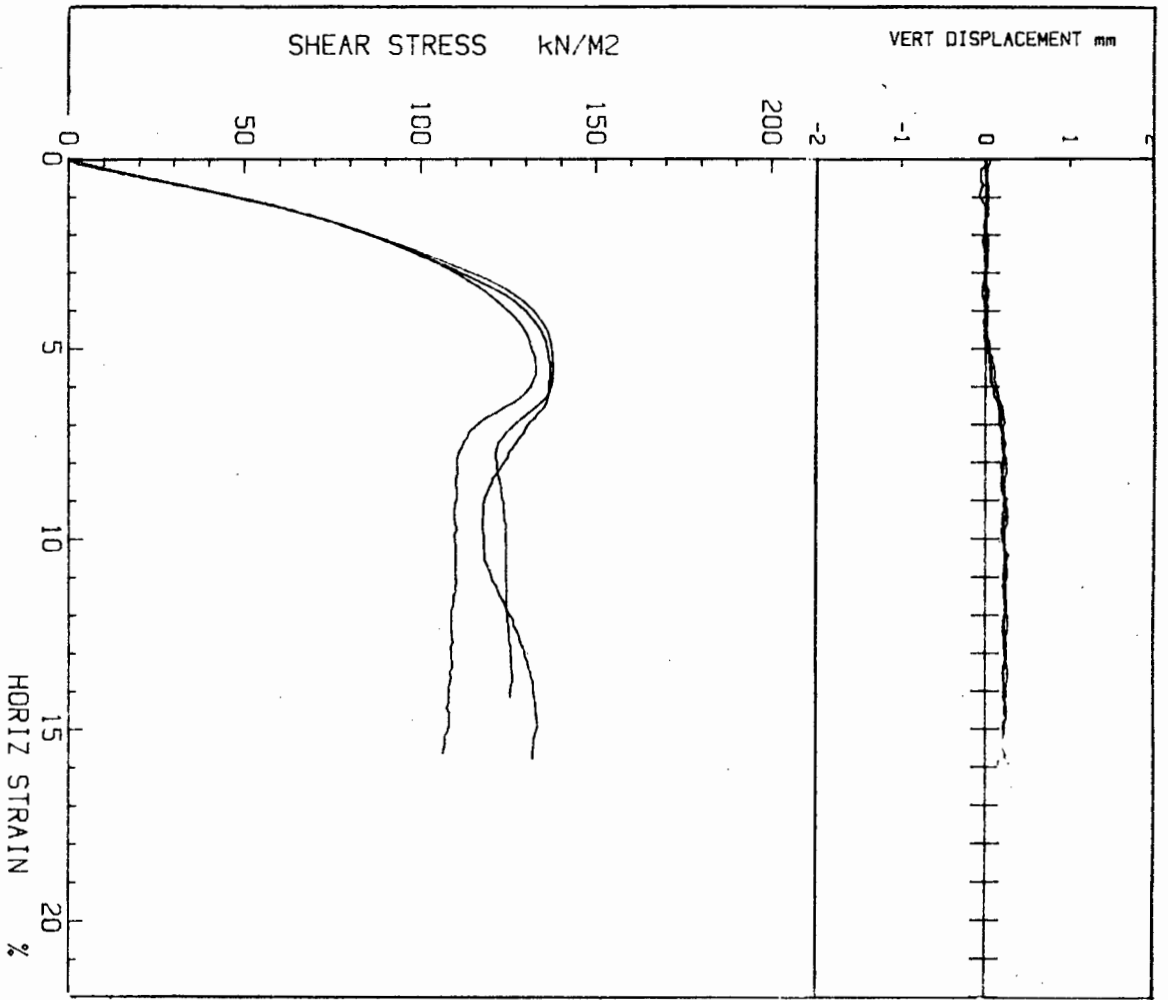


Figure 10.1 (a) : Stress-strain curves for quartz sand (HP)

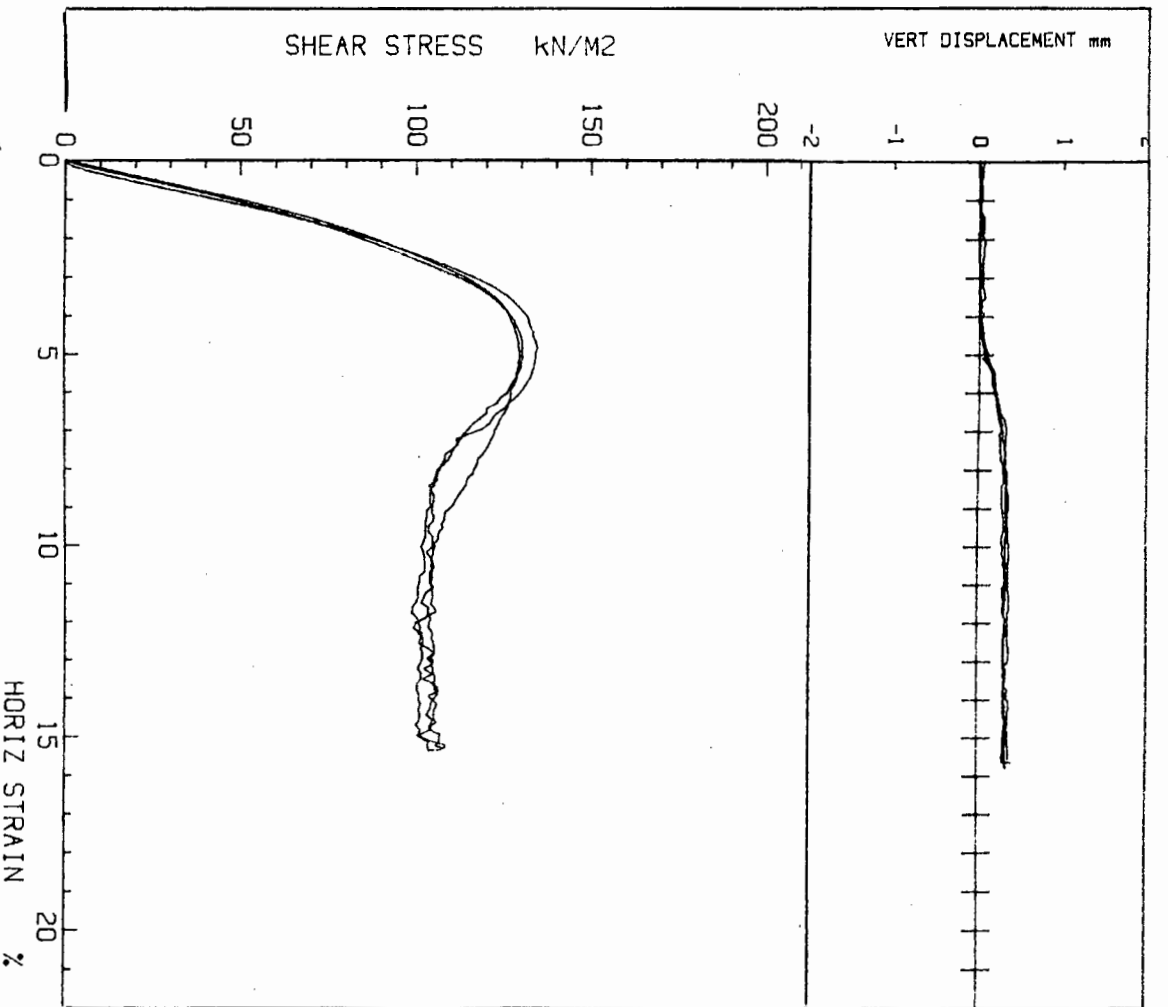


Figure 10.1 (b) : Stress-strain curves for quartz sand (s)

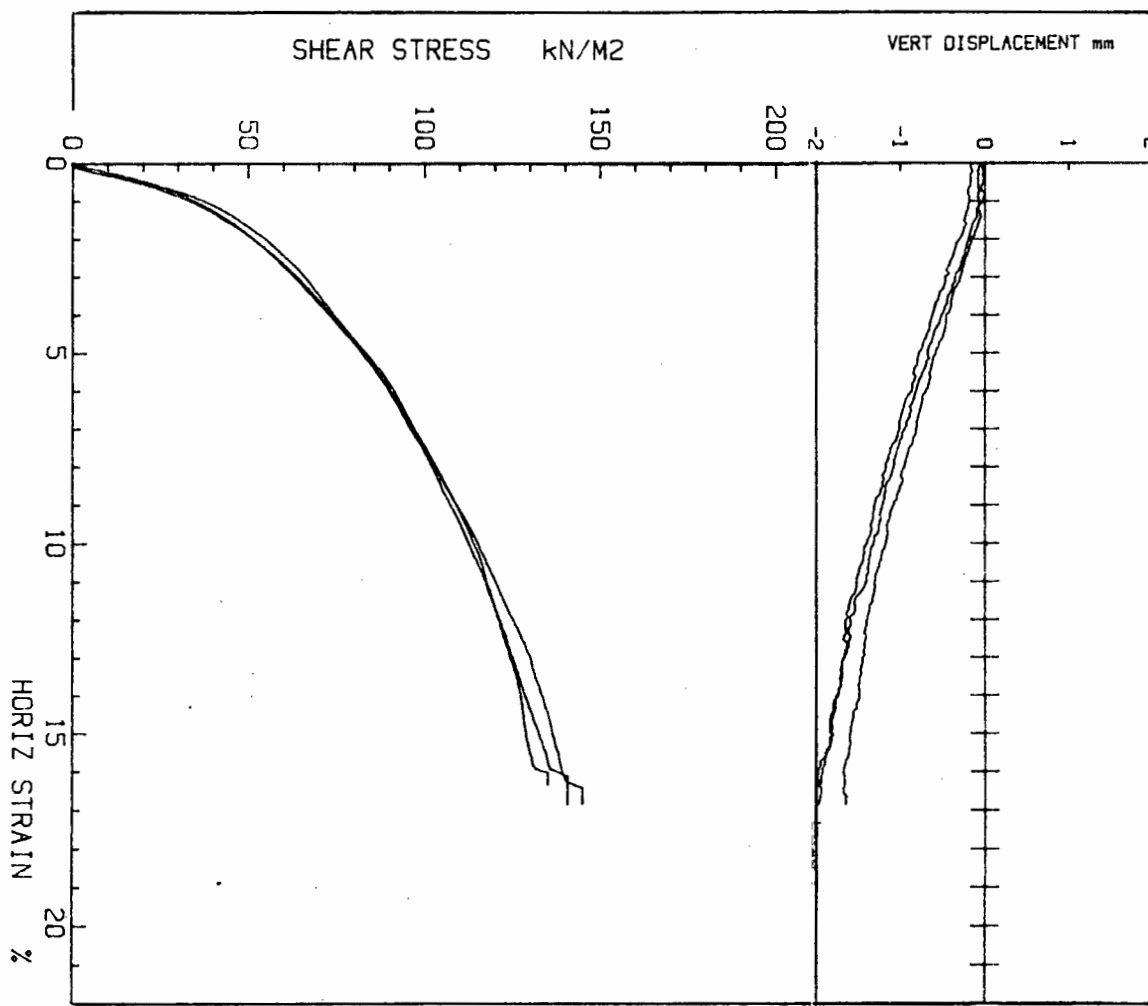


Figure 10.1 (c) : Stress-strain curves for diatomaceous earth (DE)

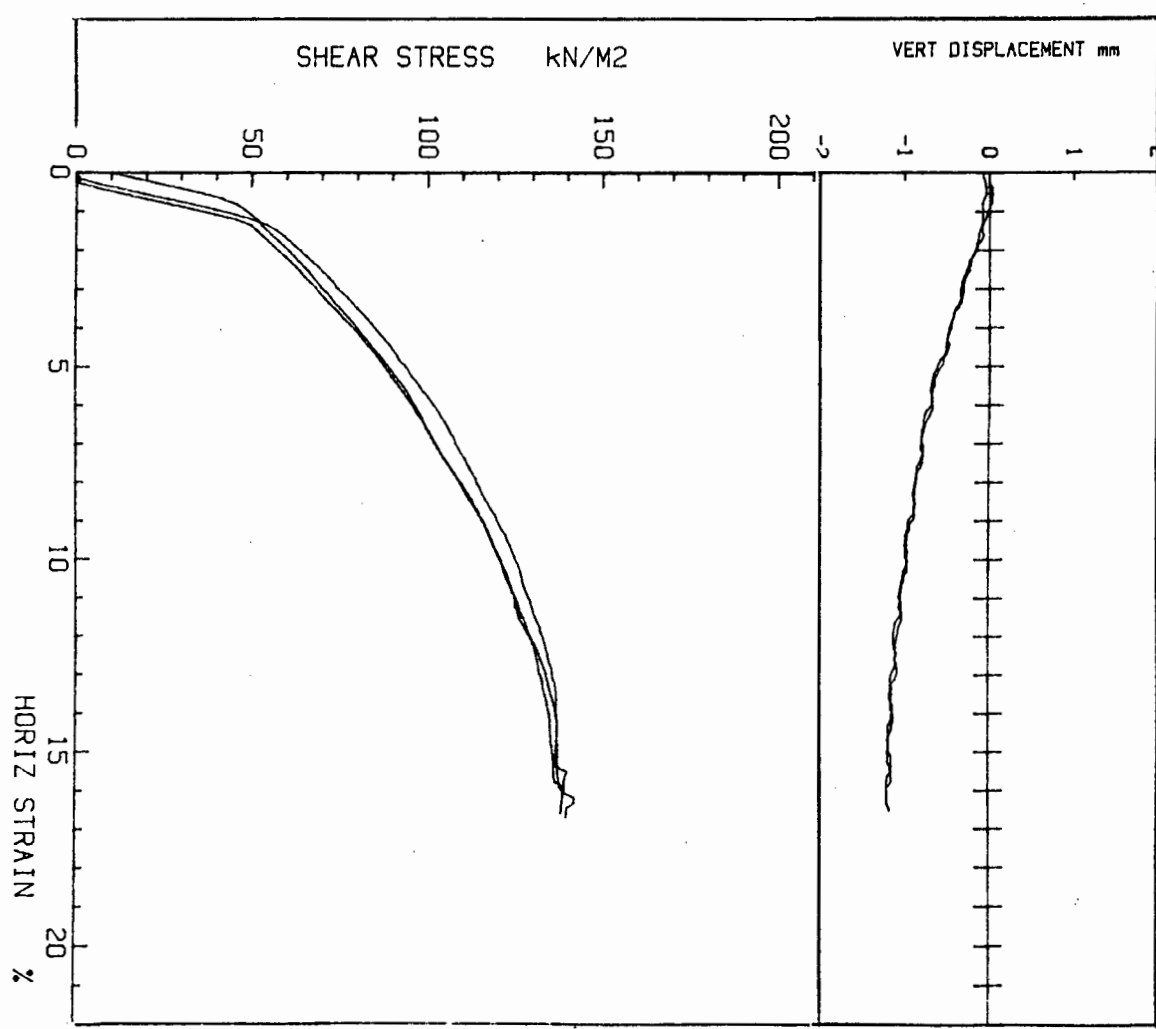


Figure 10.1 (d) : Stress-strain curves for crushed limestone (K2)

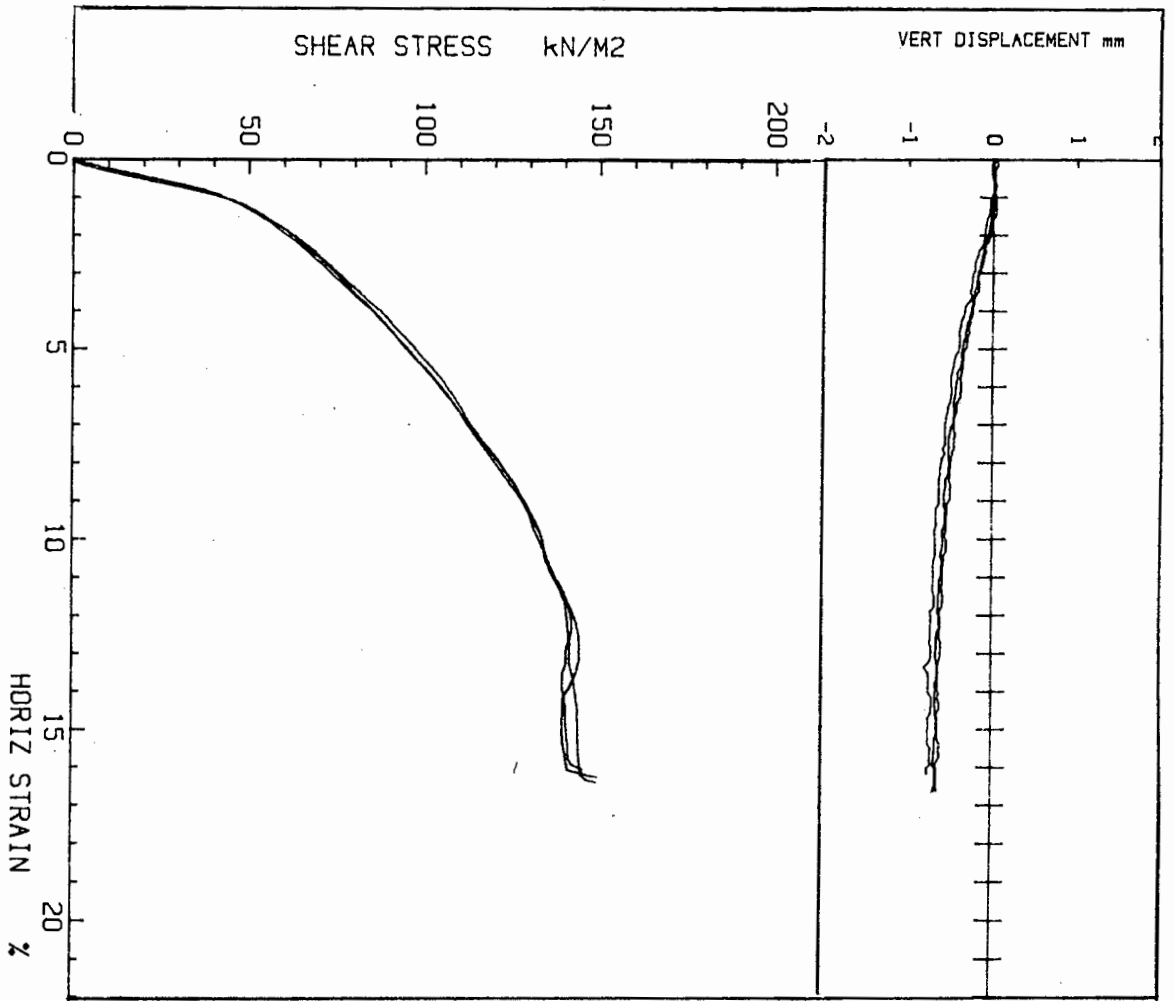


Figure 10.1 (e) : Stress-strain curves for crushed limestone (K10)

Table 10.2 : Variability in measured void ratios and friction angles of control samples

Material	Test No.	e_u	e_o	$\phi_{max} (^{\circ})$	$\phi_{ult} (^{\circ})$	σ_n (kPa)
SR	SR - 1	0.574	0.556	36.16	27.85	193
SR	SR - 2	0.573	0.563	36.61	28.53	193
SR	SR - 3	0.555	0.554	37.05	27.85	193
M E A N		0.567	0.558	36.61	28.08	
STANDARD DEVIATION		0.011	0.005	0.445	0.393	
s/\bar{x}		0.019	0.009	0.012	0.014	
HP	HP - 1	0.685	0.671	35.47		193
HP	HP - 2	0.685	0.667	35.01		193
HP	HP - 3	0.685	0.670	35.24		193
M E A N		0.685	0.669	35.24		
STANDARD DEVIATION		0	0.002	0.23		
s/\bar{x}		0	0.003	0.007		
SI	SI - 1	0.640	0.621	33.87	27.17	193
SI	SI - 2	0.619	0.609	34.90	28.08	193
SI	SI - 3	0.633	0.616	33.48	28.30	193
M E A N		0.631	0.615	34.08	27.76	
STANDARD DEVIATION		0.011	0.006	0.734	0.512	
s/\bar{x}		0.017	0.010	0.022	0.018	
DE	DE - 1	6.060	5.735	34.00		193
DE	DE - 2	6.178	5.807	35.12		193
DE	DE - 3	5.897	5.742	35.86		193
M E A N		6.045	5.761	34.99		
STANDARD DEVIATION		0.141	0.040	0.936		
s/\bar{x}		0.023	0.007	0.027		
K2	K2 - 1	1.276	1.167	35.07		193
K2	K2 - 2	1.273	1.129	35.21		193
K2	K2 - 3	1.203	1.138	34.98		193
M E A N		1.251	1.145	35.09		
STANDARD DEVIATION		0.041		0.116		
s/\bar{x}		0.033	0.017	0.003		
K10	K10 - 1	0.896	0.808	36.23		193
K10	K10 - 2	0.881	0.827	35.89		193
K10	K10 - 3	0.914	0.792	36.72		193
M E A N		0.897	0.809	36.28		
STANDARD DEVIATION		0.017	0.018	0.417		
s/\bar{x}		0.019	0.022	0.011		

Assuming the maximum standard deviations, the variability in the measurements can be expressed as :

Unconsolidated void ratio	:	$e_u \pm 0.02$
"Consolidated" void ratio	:	$e_o \pm 0.02$
Maximum friction angle	:	$\phi_m \pm 1^\circ$
'Ultimate' friction angle	:	$\phi_u \pm 0.5^\circ$.

The variation in void ratio and friction angle for similar control samples of differing initial sample size (obtained by using different initial sample heights) are listed in Table 10.3. Comparison with Table 10.2 shows that variation in sample height results in significantly larger standard deviations for the measured void ratios and 'ultimate' friction angles, while the peak friction angles appear to be relatively unaffected.

(ii) Shear strength behaviour

From the data in Table 10.2 and Figures 10.1(a) to (e), the following observations can be made with respect to shear strength behaviour :

- Notwithstanding the large variation in average particle size, shape and carbonate content of the samples, the maximum angles of internal friction (ϕ_m) fall within a relatively narrow range (34.1° to 36.6°).
- The stress-strain curves together with the measured void ratios provide a better indication of the sample properties than the measured friction angles. The tendency of samples to decrease in volume during shear is increased as the initial void ratios (e_o) becomes larger.

Table 10.3 : Effect of variation in sample height on measured void ratios and friction angles

Material	Test		e_u	e_o	$\phi_{max} (^{\circ})$	$\phi_{ult} (^{\circ})$	σ_n (kN/m ²)
SR	SR-4	a	0.570	0.545	37.20	28.97	193
SR	SR-5	b	0.523	0.514	36.68	28.30	193
SR	SR-6	c	0.542	0.518	36.49	29.63	193
MEAN			0.545	0.526	36.79	28.97	
STANDARD DEVIATION			0.024	0.017	0.368	0.665	
s/\bar{x}			0.043	0.032	0.010	0.023	
SI	SI-4	a	0.733	0.693	30.28	29.45	193
SI	SI-5	b	0.661	0.643	30.06	27.72	193
SI	SI-6	c	0.706	0.683	31.47	27.40	193
MEAN			0.700	0.673	30.60	28.19	
STANDARD DEVIATION			0.036	0.026	0.759	1.103	
s/\bar{x}			0.052	0.039	0.025	0.039	
SH	SH-4	a	0.858	0.846	41.92	39.51	193
SH	SH-5	b	0.797	0.790	43.26	39.33	193
SH	SH-6	c	0.875	0.863	43.57	39.71	193
MEAN			0.843	0.833	42.92	39.54	
STANDARD DEVIATION			0.041	0.038	0.877	0.192	
s/\bar{x}			0.049	0.046	0.020	0.005	

Average initial sample height: a=37mm b=29mm c=24mm

(iii) Effect of particle shape

Plate 1 is a Scanning Electron Micrograph (SEM) showing the similar level of shape complexity of foraminefera and diatomaceous earth particles. These shapes result in an average initial void ratio (e_0) of 5.76, even under a consolidation pressure of 193 kPa. The crushed limestones (K2, K10), with carbonate contents of 99%, consist of solid rhombohedral particles and have average initial void ratios of 1.15 and 0.81 respectively, for the same consolidation pressure.

Figure 10.2 shows the change in vertical displacement for the two samples of powdered limestone and one sample of diatomaceous earth, each sheared under a normal pressure of 193 kPa. The diatomaceous earth which is a siliceous ooze with zero carbonate content and exhibiting the maximum initial void ratio, has the highest relative compressibility. Thus, particle shape, as opposed to mineralogy, clearly has a greater effect on compressibility in this case, which also illustrates the limited use of carbonate content alone as an index delineating engineering behaviour.

To evaluate the effect of initial void ratio (e_0) on shear strength, samples of the diatomaceous earth were overconsolidated by applying normal pressures of up to 800 kPa and sheared under normal stress of 193 kPa. Results are listed in Table 10.4 and stress-strain plots shown in Figure 10.3. As the initial void ratio is decreased (increasing overconsolidation ratio), the samples become less compressible and start to show volume decrease during

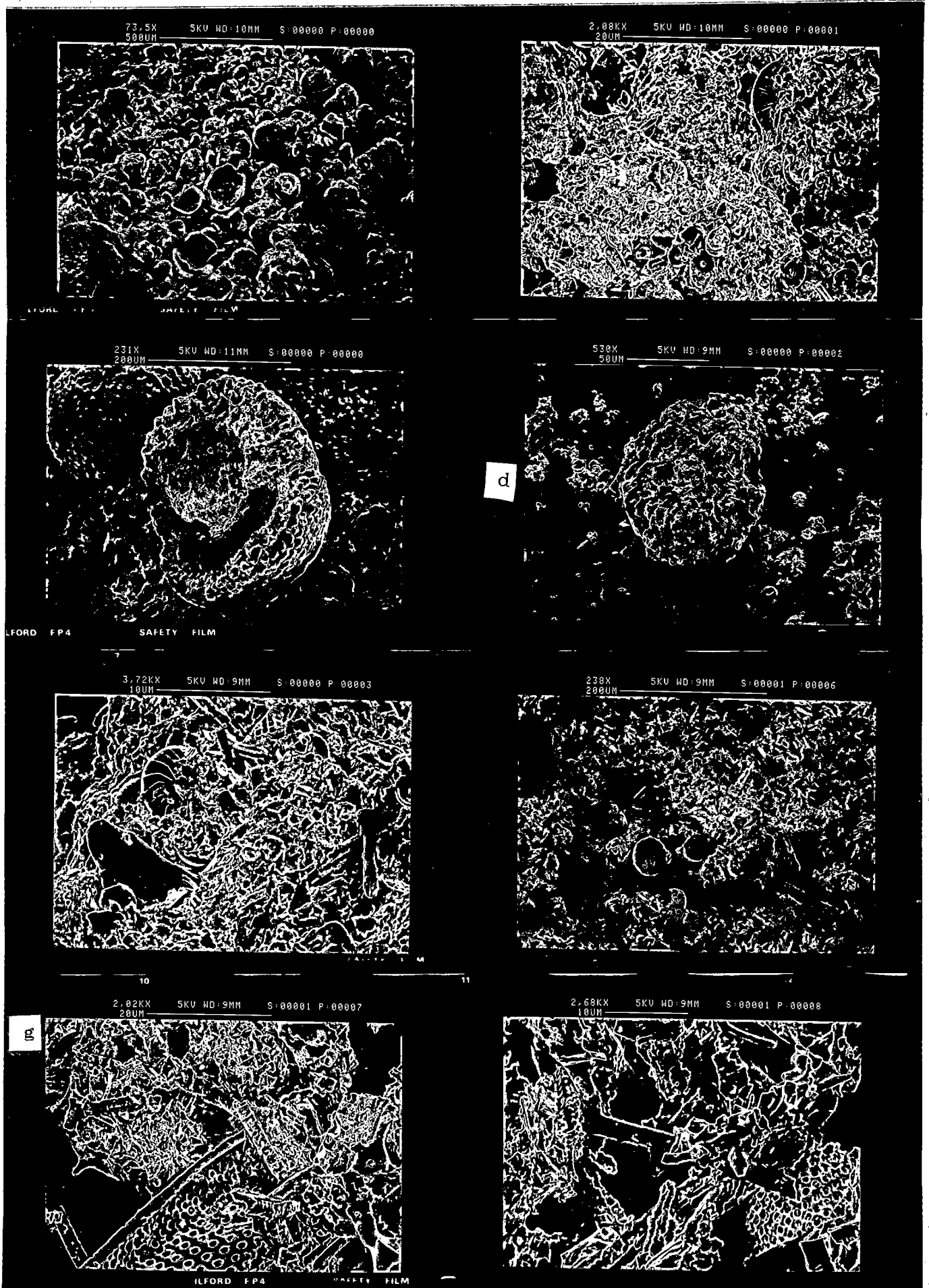


Plate 1: Scanning Electron Micrograph of foraminifera and diatomaceous earth particles; (a) to (e): foraminifera; (f) to (h): diatomaceous earth.

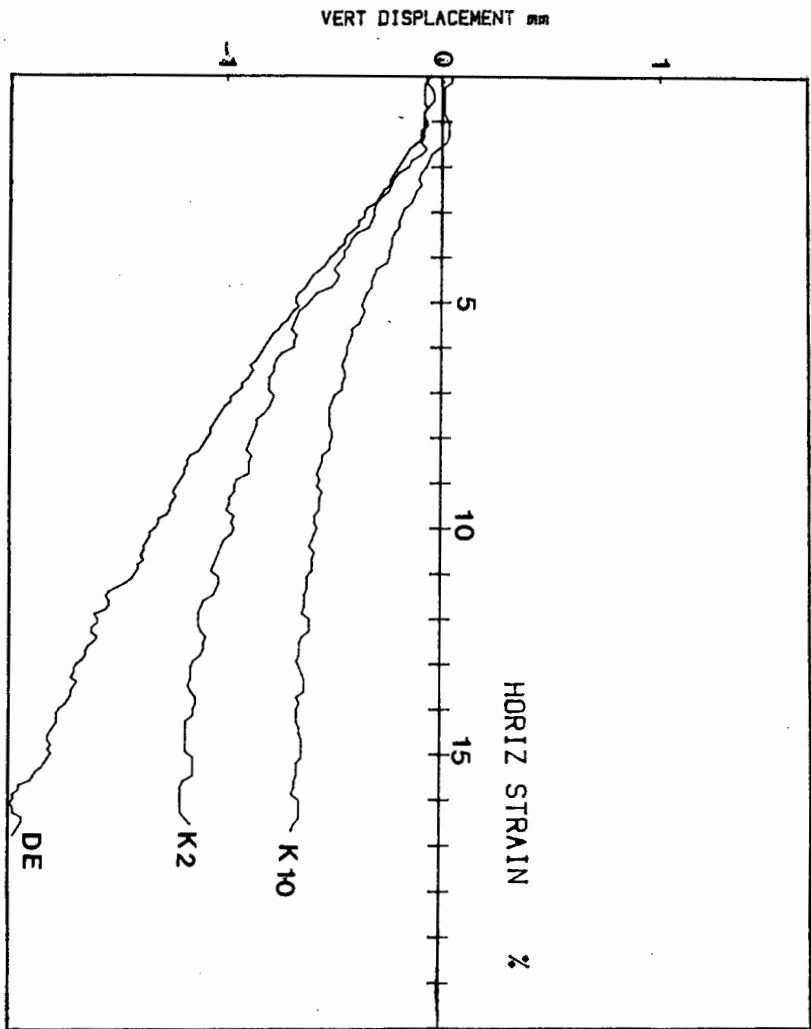


Figure 10.2 :
Change in sample height during shear under a normal stress of 193 kPa for samples of crushed limestone (K2, K10) and diatomaceous earth (DE)

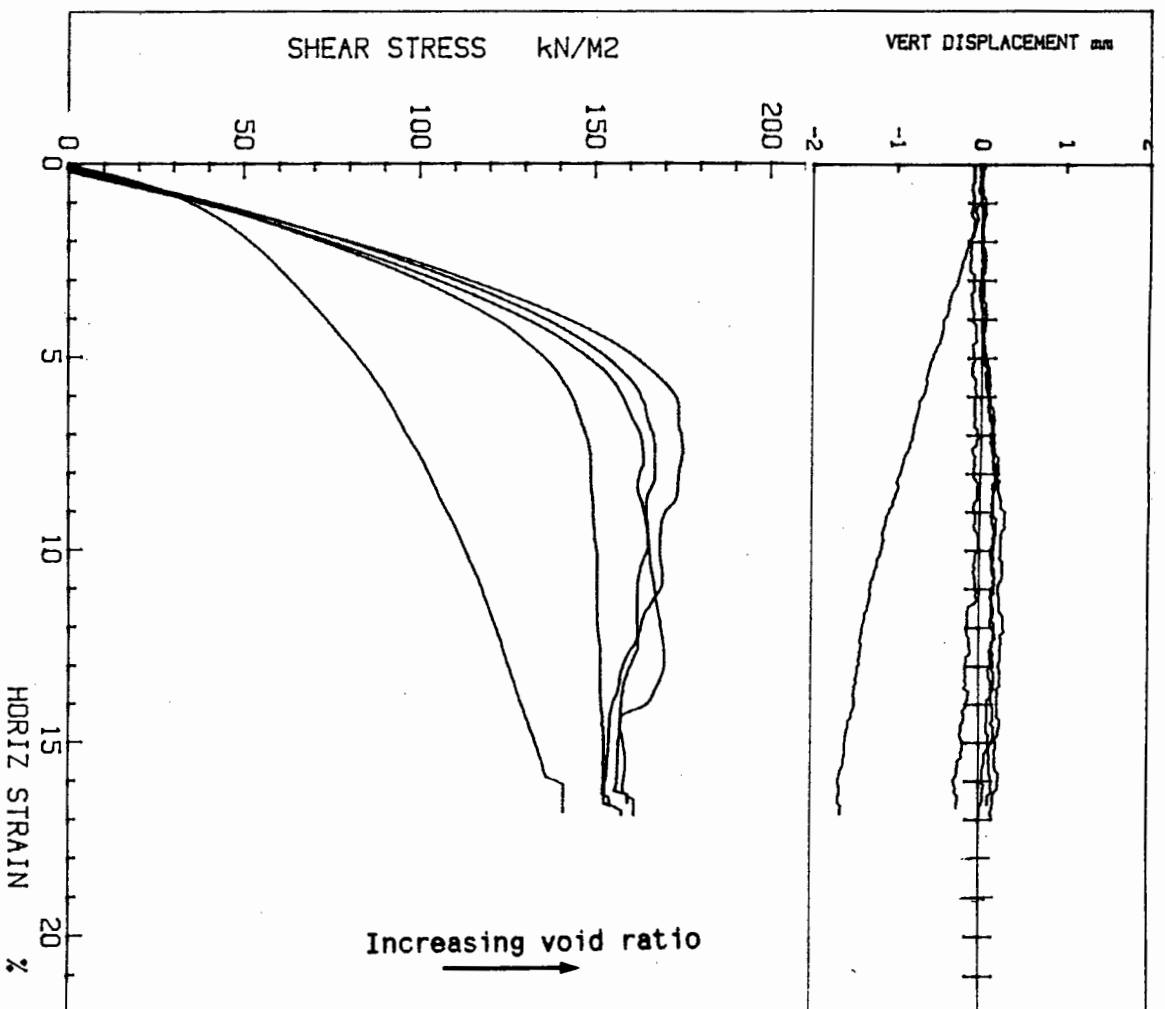


Figure 10.3 : Stress-strain curves for samples of diatomaceous earth sheared under a normal stress of 193 kPa and increasing initial void ratios

Table 10.4 : Frictional properties for diatomaceous earth sheared at different initial void ratios

Test no.	e_o	σ_n (kPa)	$\phi_{max} (^{\circ})$	$\phi_{ult} (^{\circ})$
1	5.81	193	35.12	35.12
2	5.74	193	35.86	35.86
3	5.06	193	38.44	38.44
4	4.32	193	41.21	39.12
5	4.05	193	42.14	37.96

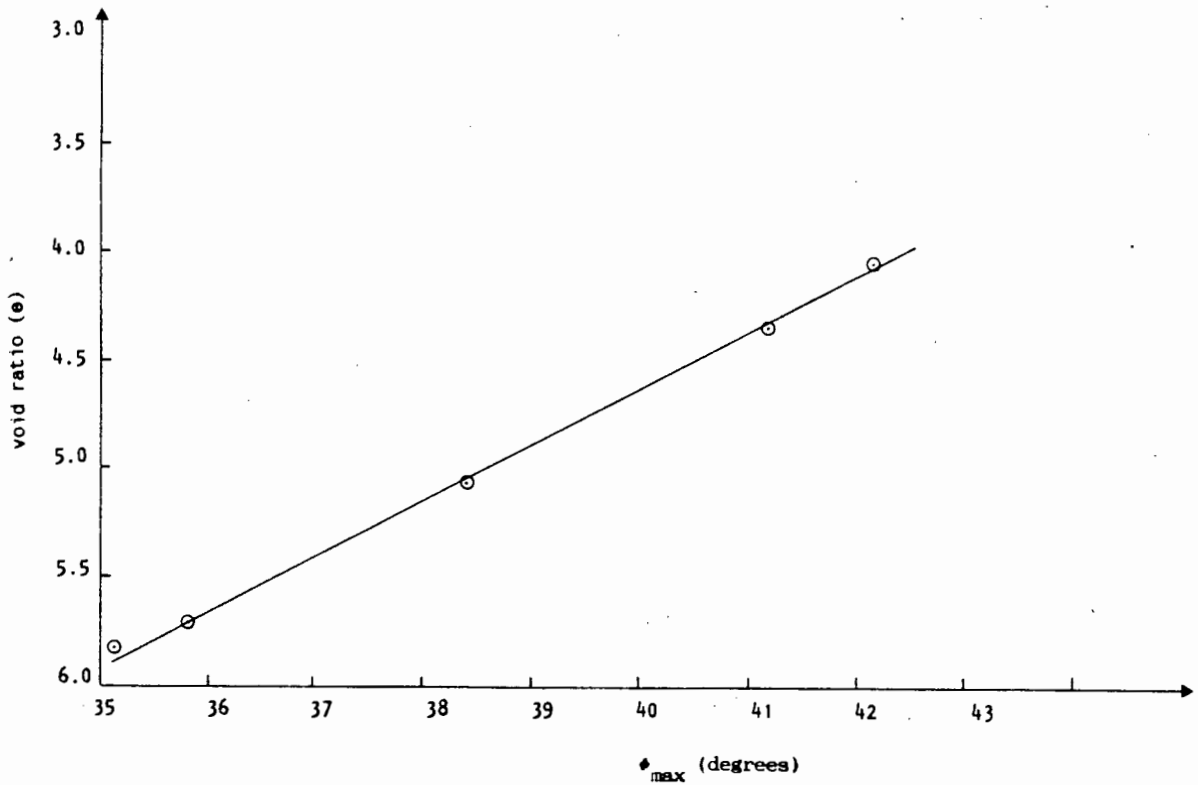


Figure 10.4 :

Effect of initial void ratio on the maximum angle of internal friction for samples of diatomaceous earth (DE) sheared under a normal stress of 193 kPa

shear, as reflected in the increasing maximum friction angles. This effect is illustrated in Figure 10.4.

The sensitivity of shear strength parameters to the initial density for silty, crushable materials such as the diatomaceous earth and crushed limestone suggests that an engineering classification based on a parameter related to compressibility could be more appropriate than one based on frictional properties or crushability. For these materials, relatively small normal pressures could result in large settlements and associated crushing. It would be extremely difficult to measure such crushing directly (c.f. Section 4.4.4). That significant crushing occurs at relatively small normal pressures can be deduced from Figure 10.5.

In Fig. 10.5 an "unused" sample of diatomaceous earth was placed in the shear mould using "standardised" initial compaction (three layers, pressed by hand) and consolidated under a sequence of increasing normal loads. The sample was then removed from the mould, remoulded into a loose powder, recompacted and consolidated as before. The spacing between the two compression curves indicates that a smaller average particle size was present in the second stage of the test. This comminution is a clear indication of crushing.

10.5 CONCLUSIONS ON CONTROL TESTS

- (a) Provided that a "standardised" initial placement density and sample size is used, good repeatability of measured void ratios and frictional properties can be achieved with the direct shear apparatus.

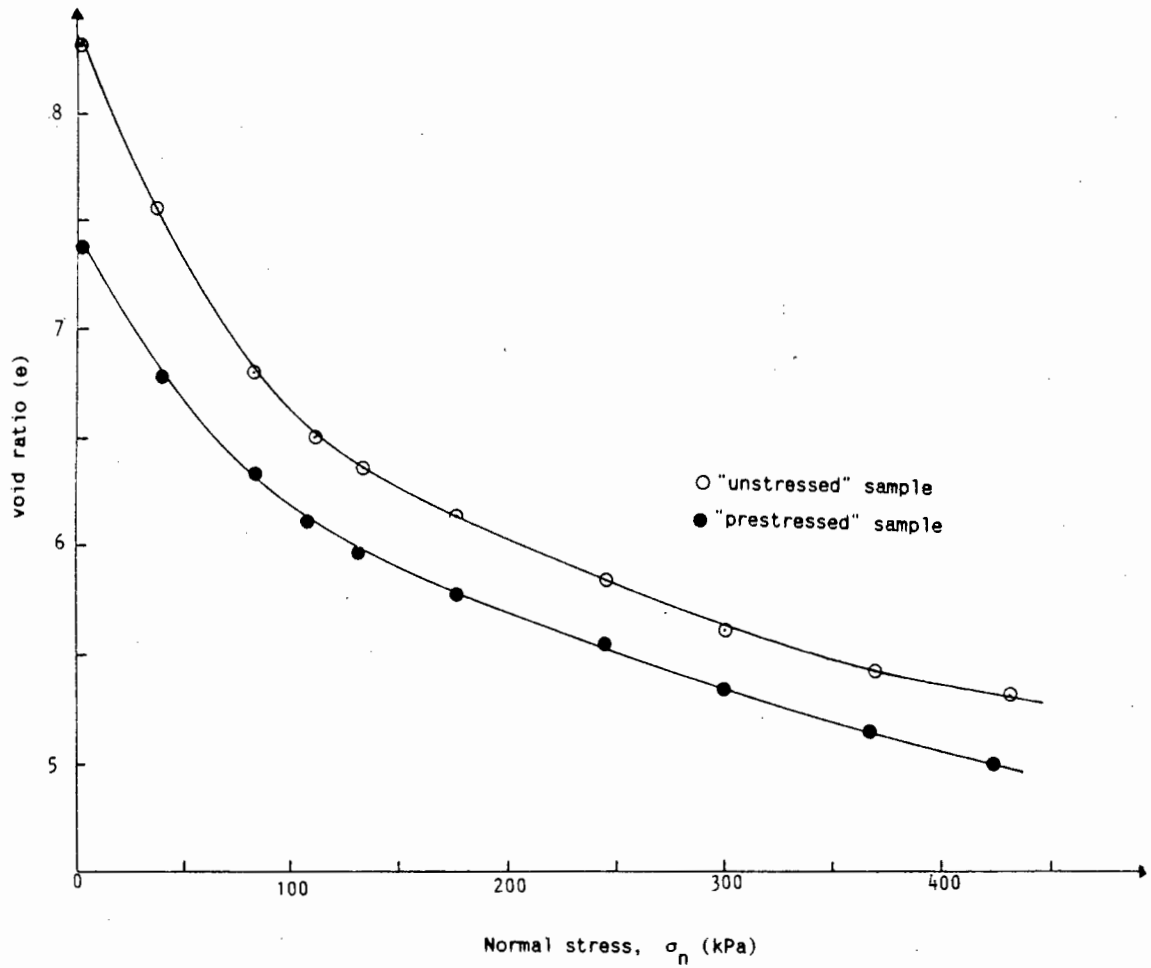


Figure 10.5 : Effect of "prestressing" on the measured void ratio of a sample of diatomaceous earth consolidated in the direct shear apparatus

- (b) Friction angles measured at an arbitrary, single normal pressure fail to discriminate between materials on a basis of particle size, shape and mineralogy, especially in the fine sand to silt size range.
- (c) Although some silt particles are highly crushable, classification or description of silt materials in terms of crushability does not appear to be a suitable parameter. Mass physical properties, particle shape and compressibility would provide a more useful indication of the likely engineering behaviour.

11. CRUSHABILITY TESTS

11.1 OBJECTIVES

The crushing of carbonate particles at relatively low stresses has been identified as one of the factors causing anomalous engineering behaviour. Thus it has been suggested (e.g. Datta *et al.*, 1979) that some measure of the crushing susceptibility should be included in an engineering classification or description system for carbonate soils. Although several indices have been suggested, their use is limited by a lack of a simple and standardised procedure for their quantification.

In this investigation, two carbonate sands (composed of small shell fragments) with differing mass physical properties were subjected to direct shear tests in order to undertake a preliminary evaluation of :

- The suitability of the direct shear apparatus to measure crushability.
- The effect of particle shape and intraparticle voids, as reflected in the maximum obtainable void ratio, on the amount of crushing.
- The effect of particle size on crushing.
- The effect of sample thickness on crushing.
- The relative magnitude of crushing under normal stress alone, followed by shearing under the same normal stress.

11.2 SAMPLE DESCRIPTION AND PREPARATION

Two types of shell, which will be referred to as "Killa" and "Table Bay" shell, were used in the investigation. A coarse quartzitic sand was used as

control. The prefixes K, T and S have been used to identify the samples of these materials, respectively.

The Killa shell (K) was purchased from a supplier of building materials and was received at the laboratory in 50 kg bags of coarse shell fragments. Approximately 80% by mass of these particles were in the fine to medium gravel size range.

The Table Bay shell (T) was obtained offshore of Cape Town on the UCT research vessel, the R/V Thomas B Davie. A near-surface disturbed sample of approximately 20 kg was retrieved with a box corer. Most of the particles were in the fine to coarse sand size range.

Although the external shapes of the Killa and Table Bay shells appear to be superficially similar (platey and angular), they display some textural and structural differences. The Killa shell consists of solid particles with a glossy lustre, while the Table Bay particles are porous and have a matt appearance. These particles are shown in Plates 2 and 3.

In order to investigate the effect of particle size on crushability, the shell fragments were separated into 3 size ranges, the largest of which (1180 - 2000 μm) was selected to comply with a 2mm maximum particle size restriction for the 60mm x 60mm direct shear test (Head, Vol. 2, (1982)). As the bulk of the Killa shell fragments were coarser than 2mm, a considerable amount of sieving was required to obtain sufficient amounts of finer samples. Unfortunately particles obtained by artificial crushing differed markedly in shape and texture from those obtained by sieving the "uncrushed" samples; artificial crushing could therefore not be used.

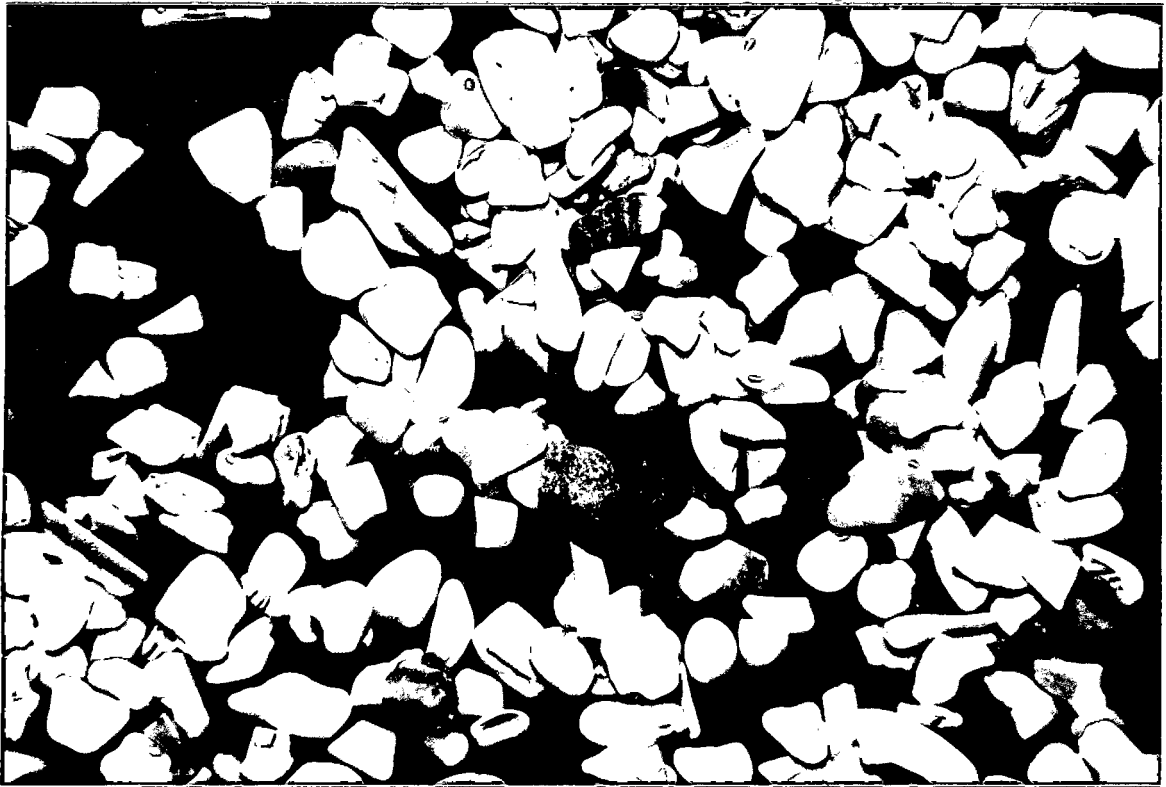


Plate 2 : Coarse fraction (> 2mm) of Killa shell.



Plate 3 : Coarse fraction (> 2mm) of Table Bay shell.

Samples of both Killa and Table Bay shell were therefore prepared using the following procedure :

- the coarsest fraction of the air dried samples was first removed by discarding all fragments retained on a 3000 μm sieve;
- the samples were washed on a 425 μm sieve in order to remove fines and contaminants;
- the oven dried samples were carefully hand sieved and separated into 3 size ranges :

Size A	2000 - 1180 μm
Size B	1180 - 850 μm
Size C	850 - 600 μm .

11.3 PARTICLE SIZE DISTRIBUTION

In order to assess the particle size distribution, the bulk samples were riffled and particle size analysis performed on representative quantities of material in each size range. A standard dry sieving procedure (BS 1377 : 1975, Test 7(B)) with mechanical shaker and shaking time of 10 minutes was used. Repeated analysis of the same samples showed excellent repeatability of results with only minor particle degradation due to sieving.

The particle size distributions for the 3 size ranges of Killa and Table Bay shell, as well as the quartz sand are shown in Figures 11.1(a) to (c) and the associated uniformity coefficients are given in Table 11.1. The coefficients of uniformity (D_{60}/D_{10}) for all samples were considerably less than 2. The samples may consequently be regarded to be of "uniform" grading distribution (Lambe and Whitman, 1979).

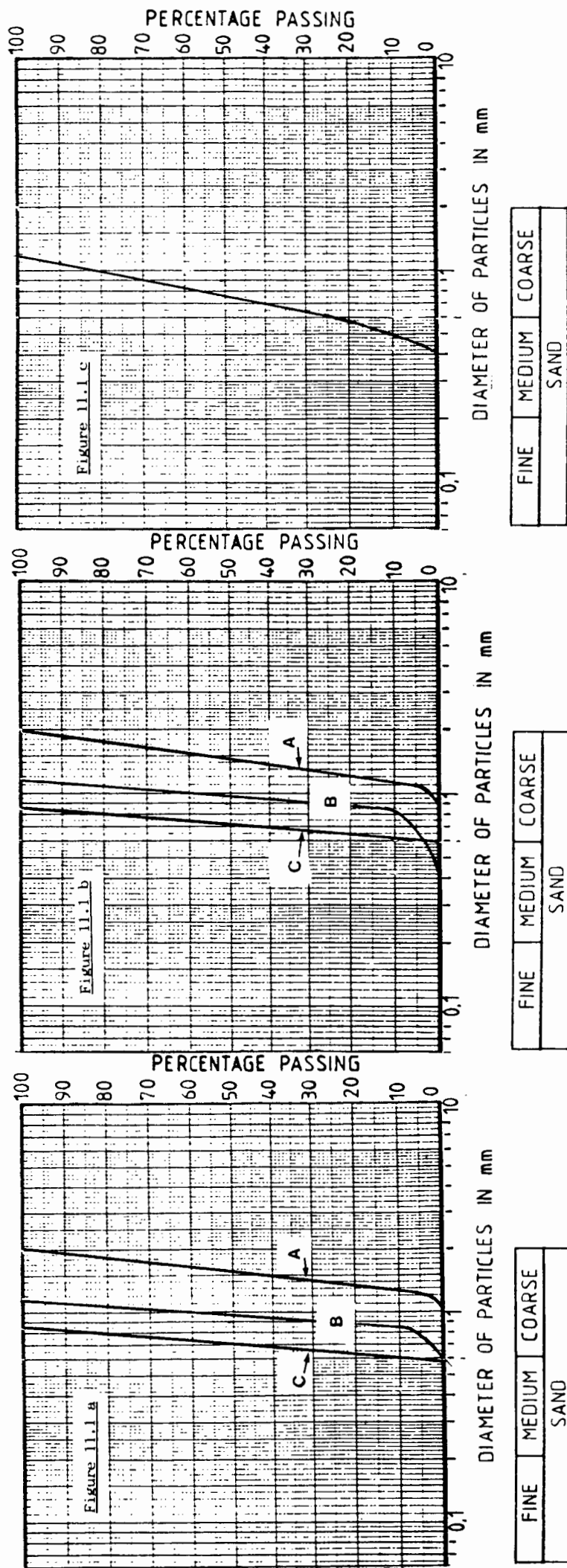


Figure 11.1 : Particle size distribution curves for samples used in crushability tests;

- 11.1 (a) Killa shell
- 11.1 (b) Table Bay shell
- 11.1 (c) Quartz sand

11.4 CARBONATE CONTENT

Carbonate contents were determined using hydrochloric acid reaction and pressure measurement as in the "Karbonat-Bombe" technique (Muller and Gastner, 1971). The average carbonate content of the Killa samples was 82% of dry weight, and the Table Bay samples 83% of dry weight.

11.5 SPECIFIC GRAVITY

Particle specific gravities (relative mass densities) were determined using the "density bottle method" (BS 1377 : 1975, Test 6(6)). Both the Killa and Table Bay particles had average specific gravities of 2.71.

11.6 LIMITING DENSITIES AND VOID RATIOS

The minimum dry densities of the samples were determined using the method devised by Kolbuszewski (1949) and described by Head, Vol. 1, (1984). This is an extremely simple test which requires pouring a predetermined mass of oven dried sample into a graduated cylinder, quickly inverting the cylinder and using the maximum volume reading obtained to calculate the minimum dry density.

The minimum dry densities obtained are listed in Table 11.2. The corresponding maximum void ratios were calculated using the following relationship :

$$e_{\max} = (G_s \rho_w / \rho_{d_{\min}} - 1)$$

Table 11.1 : Particle size characteristics of samples

Size	Killa			Table Bay			Quartz
	A	B	C	A	B	C	
D_{60} (mm)	1.6	1.0	0.73	1.55	1.0	0.74	0.80
D_{10} (mm)	1.2	0.86	0.61	1.18	0.85	0.61	0.52
$U=D_{60}/D_{10}$	1.33	1.16	1.20	1.31	1.18	1.21	1.54

A : 1180 - 2000 μm
 B : 850 - 1180 μm
 C : 600 - 850 μm

Table 11.2 : Limiting densities and void ratios

	Particle size range (μm)		
	2000 - 1180	1180 - 850	850 - 600
	A	B	C
<u>Killa</u>			
$\rho_{D_{\min}}$ (kg/m^3), 10^3	1.25	1.25	1.30
e_{\max}	1.17	1.17	1.09
* e_{\min}	0.70	0.82	0.69
<u>Table Bay</u>			
$\rho_{D_{\min}}$ (kg/m^3), 10^3	0.93	0.99	1.11
e_{\max}	1.93	1.74	1.44
* e_{\min}	1.43	1.23	1.11
<u>Silica</u>			
$\rho_{D_{\min}}$ (kg/m^3), 10^3		1.52	
e_{\max}		0.75	
* e_{\min}		0.44	

* Estimated from results of direct shear tests

where :

- G_s = particle specific gravity
 ρ_w = density of water
 $\rho_{d_{min}}$ = minimum dry density.

The minimum void ratios listed in Table 11.2 were deduced from the results of direct shear tests. It was arbitrarily defined as the minimum which could be obtained by vibrating the sample under normal stress without appreciable crushing.

11.7 TESTING PROCEDURES TO INVESTIGATE CRUSHING

A series of tests to investigate crushing, as described under (i), (ii) and (iii) below, was conducted with the 60mm x 60mm direct shear apparatus.

(i) Effect of normal stress on crushing

Samples were subjected to normal stresses of 700 kPa and 1200 kPa and particle size analysis performed on the unsheared samples.

The air dried samples were placed in the shear mould in three layers. Each layer was lightly tamped in an endeavour to obtain the maximum possible density without causing particles to crush. The mould was placed in the shearbox carriage and the sample height (h_u) was measured in order to determine the "unconsolidated" void ratios (e_u). After application of the appropriate normal stress, the samples were vibrated with a specially adapted vibrating engraving tool for approximately 1 minute. The normal loads were removed and the sample heights (h_o) measured to determine the "consolidated" void ratios (e_o). Sample masses were recorded after the

tests and particle size analysis was performed using the identical procedure to that described under Section 11.3.

(ii) Effect of shear stress on crushing

Samples were sheared under normal stresses of 100, 400, 700 and 1200 kPa and particle size analysis performed on the sheared samples.

Initial preparation of samples followed the procedure described in Section 11.7(i). After measurement of the "consolidated" sample heights, the normal loads were re-applied and the samples sheared at a constant rate of strain of 1.2mm/min. Data of shear stress and vertical displacement versus horizontal displacement were recorded electronically, using the program SAMPLING. Maximum and "residual" shear stresses were calculated directly from proving ring dial gauge readings. The masses of the sheared samples were recorded and changes in particle size distributions analysed.

The results of these tests are tabulated in Table 11.3 and the recorded stress-strain curves shown in Figures 11.2(a) to (u).

(iii) Effect of sample height on crushing

The average sample height of 24mm was increased to 37mm and tests similar to those described under (i) and (ii) were performed, using an applied normal stress of 1200 kPa.

11.8 DISCUSSION OF RESULTS OF CRUSHABILITY TESTS

(i) Selection of crushing coefficients

As noted in Section 4.3.3 of the literature review, not all measures of particle breakage are numerically equally sensitive. In order to compare

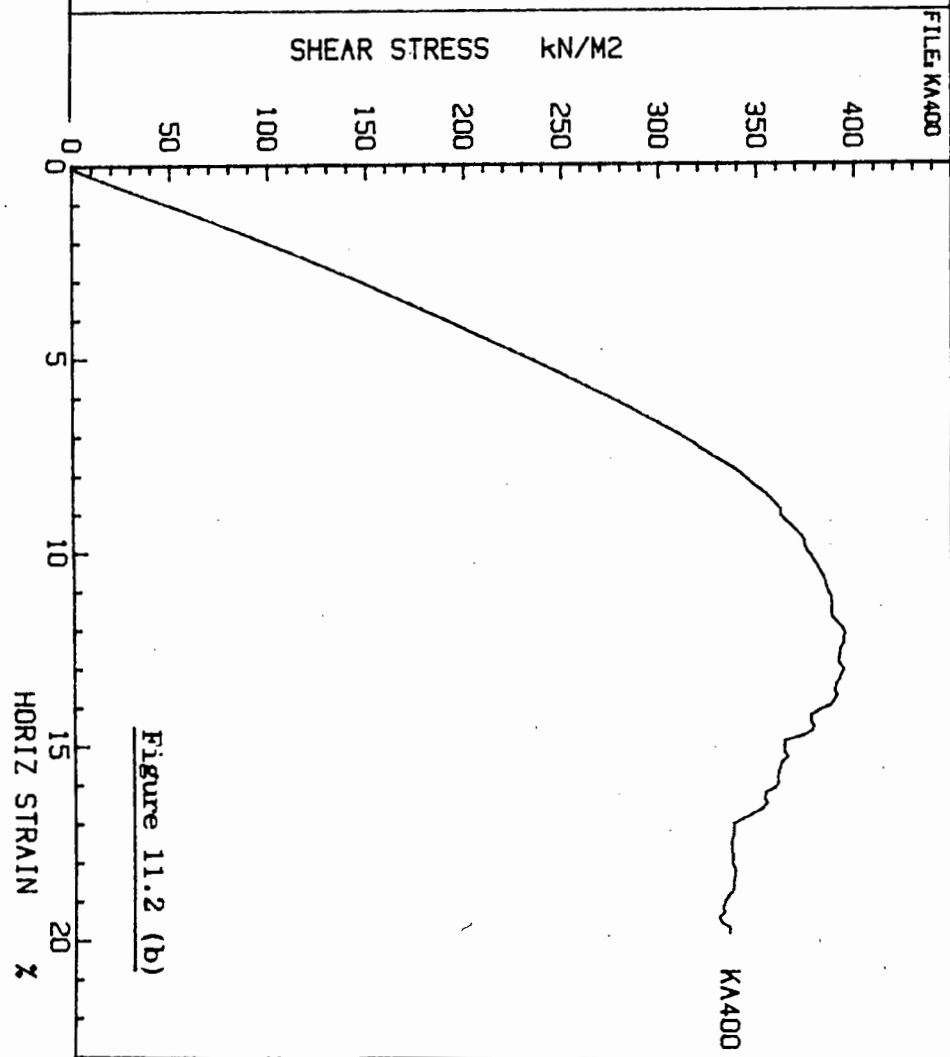
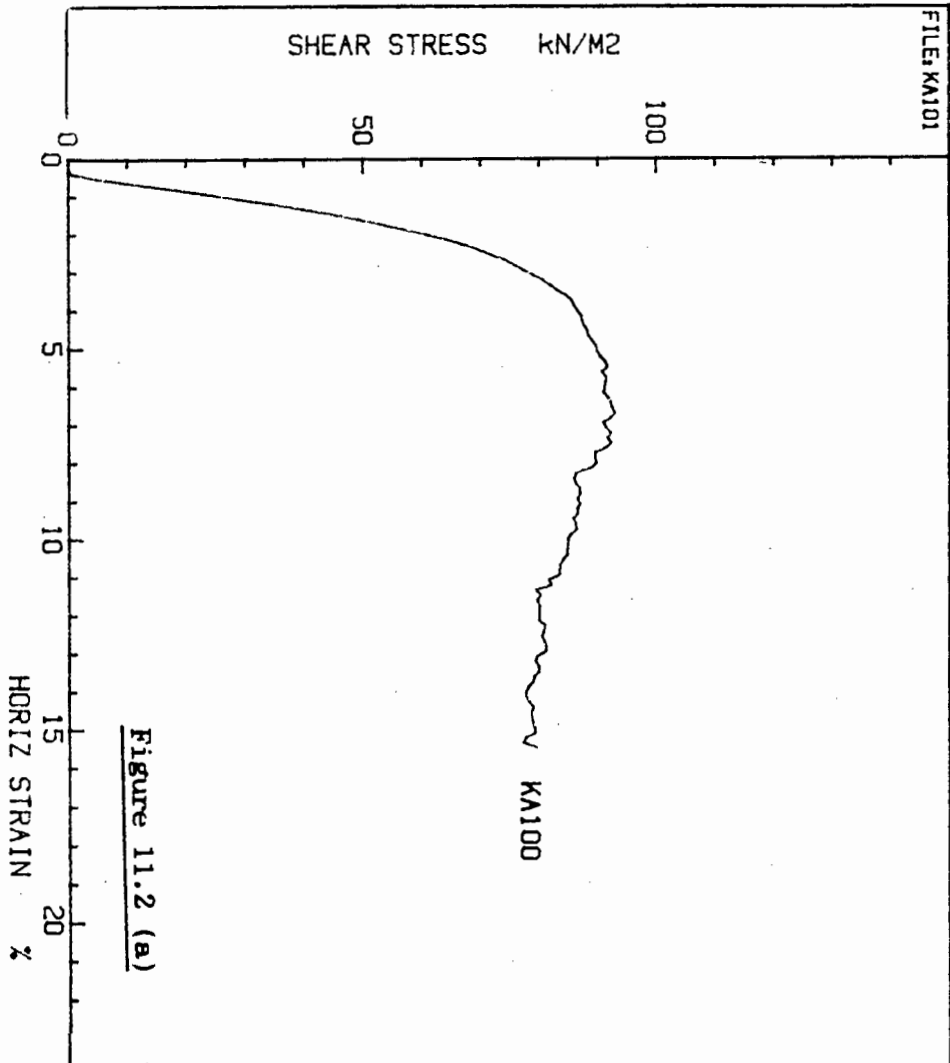
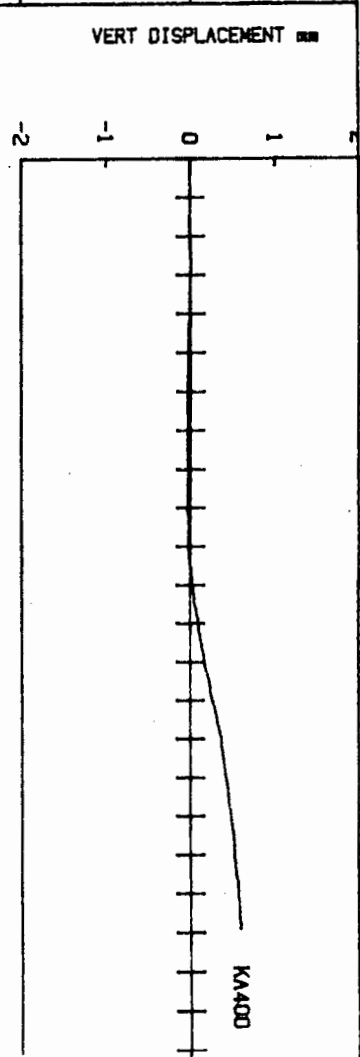
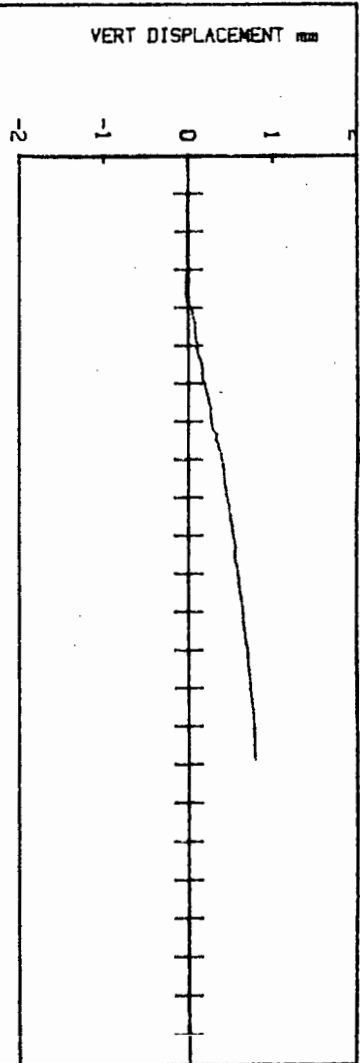
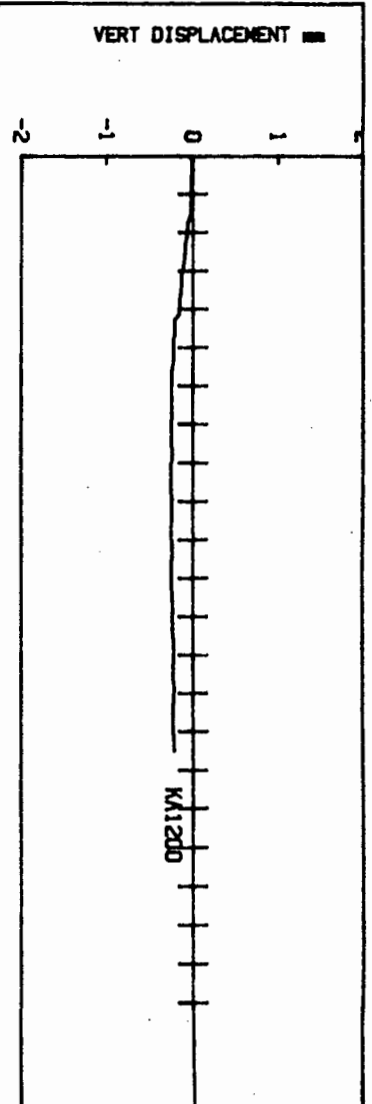


Figure 11.2 (a)

Figure 11.2 (b)



FILE: KA1200

SHEAR STRESS kN/M2

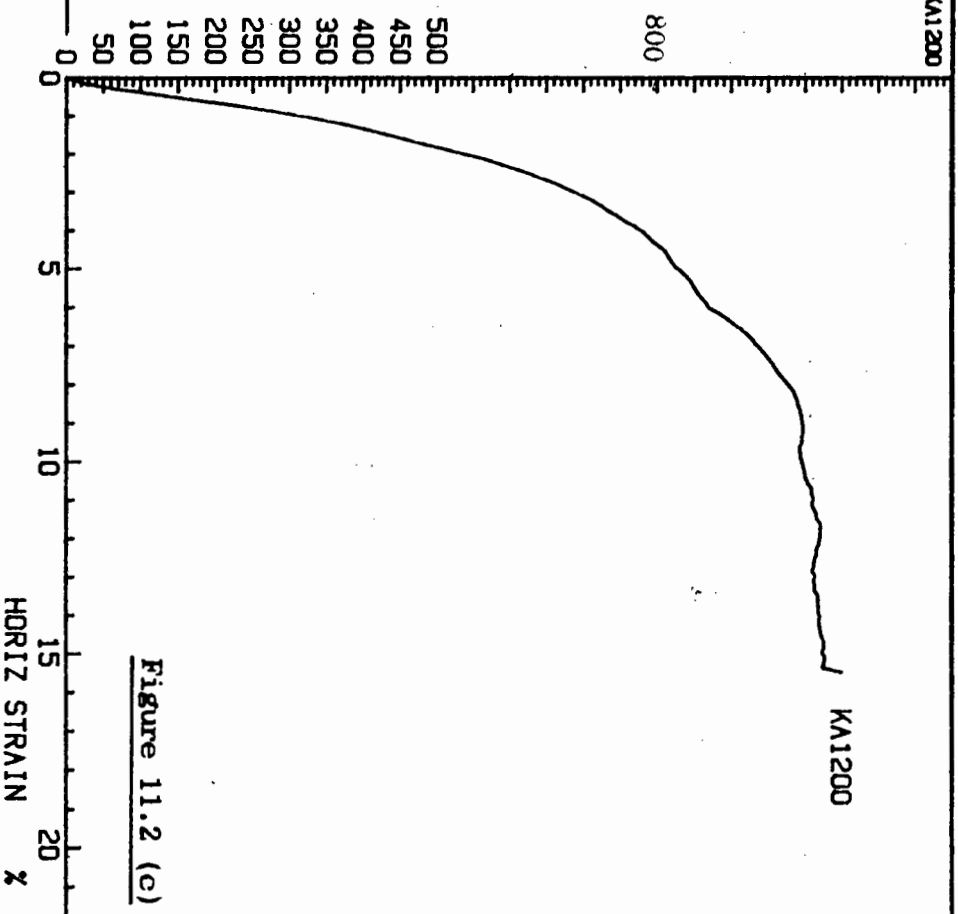
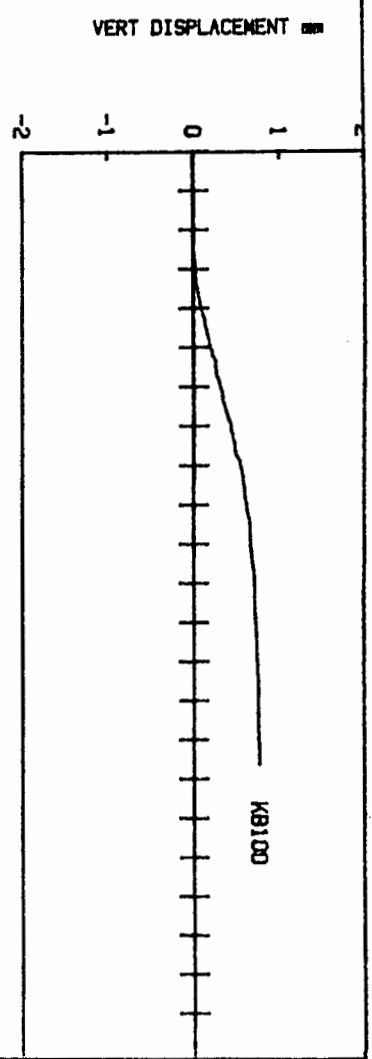


Figure 11.2 (c)



FILE: KB100

SHEAR STRESS kN/M2

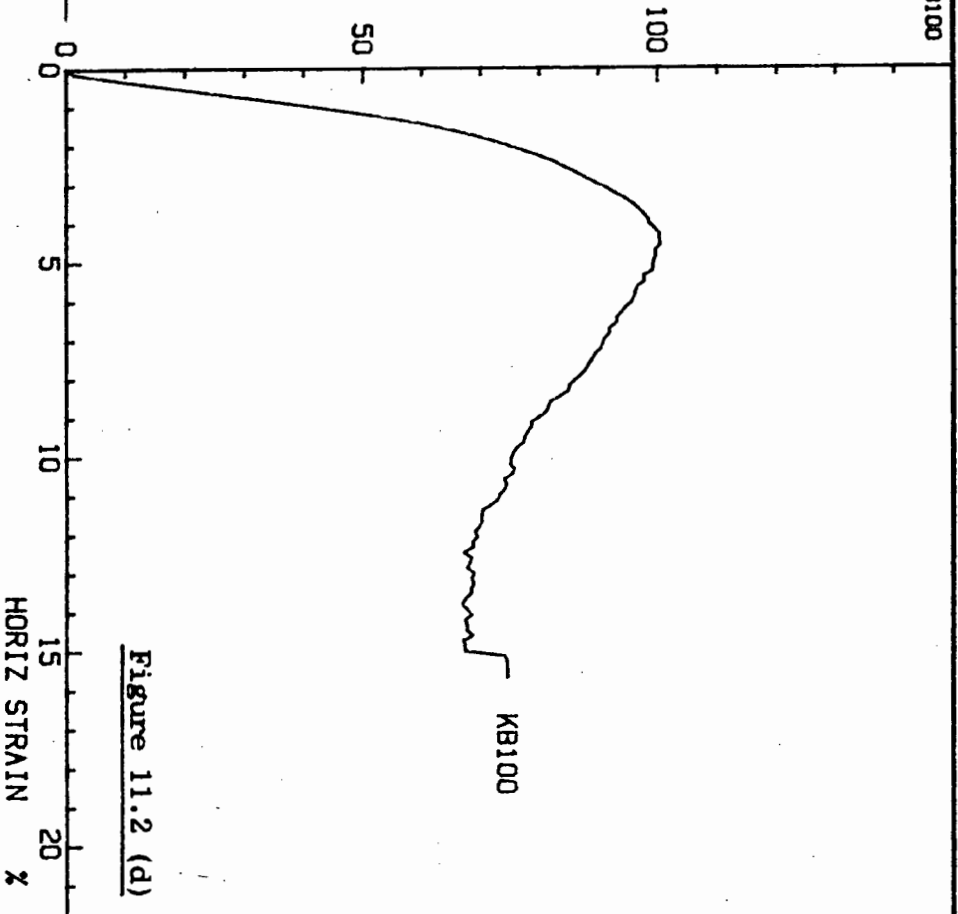
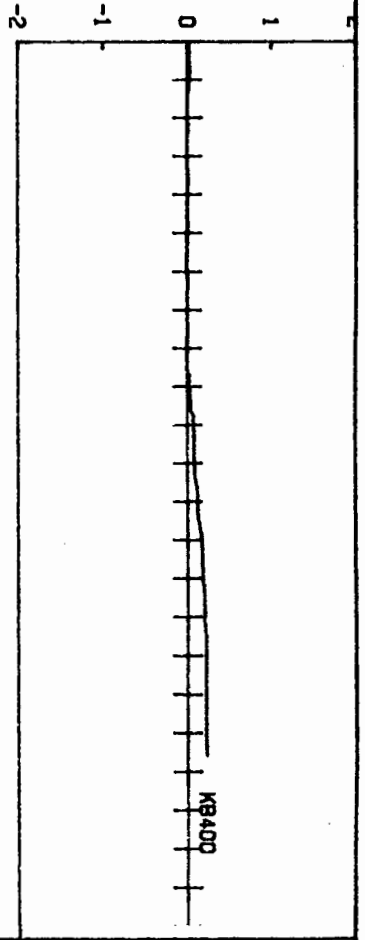


Figure 11.2 (d)

VERT DISPLACEMENT



FILE KB400

SHEAR STRESS KN/M2

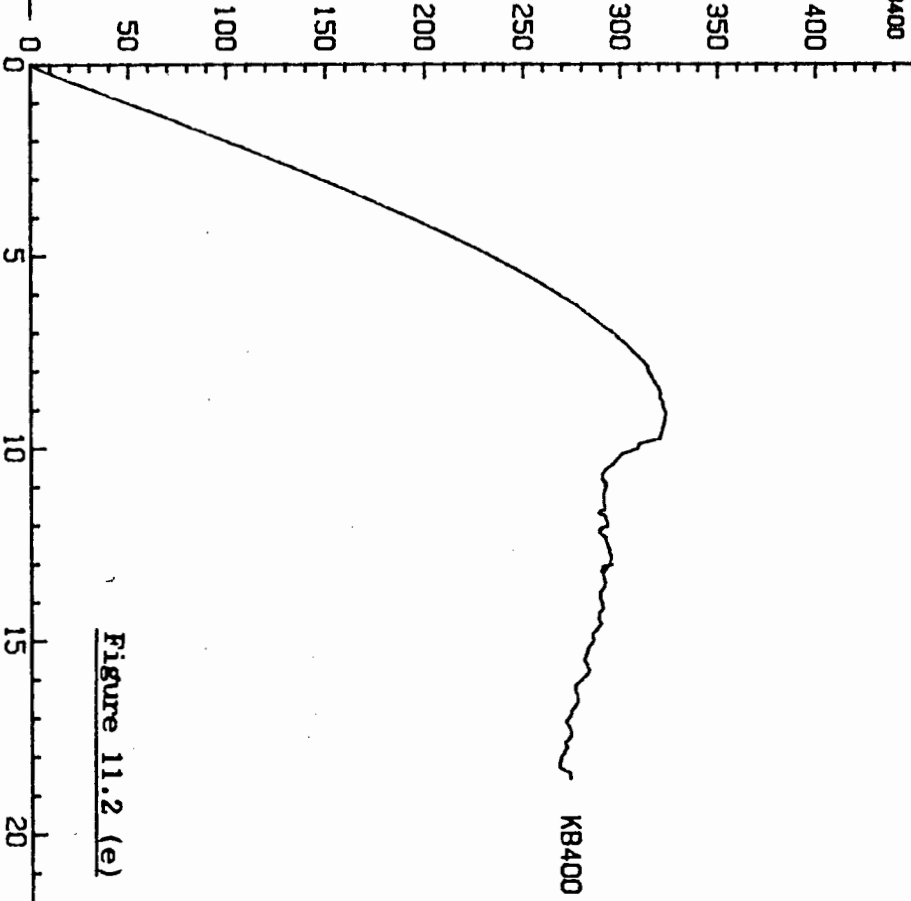
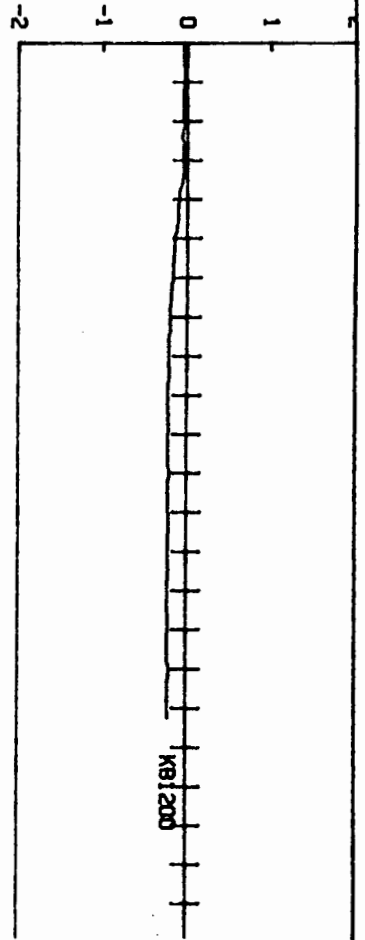


Figure 11.2 (e)

VERT DISPLACEMENT



FILE KB1200

SHEAR STRESS KN/M2

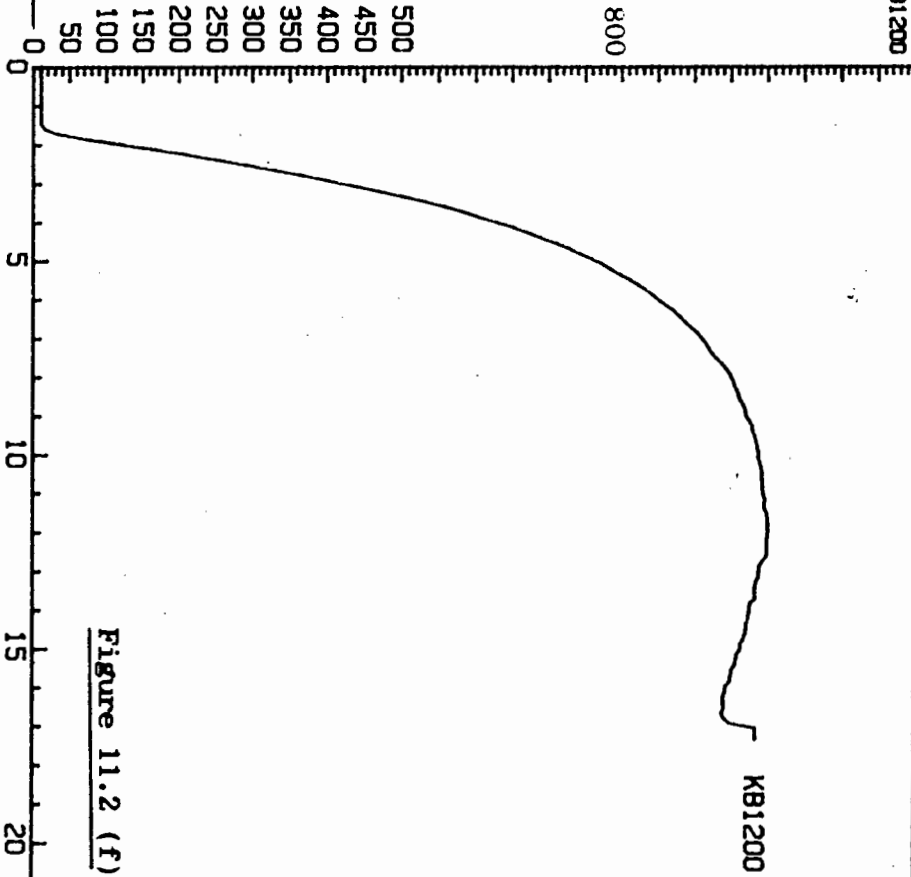
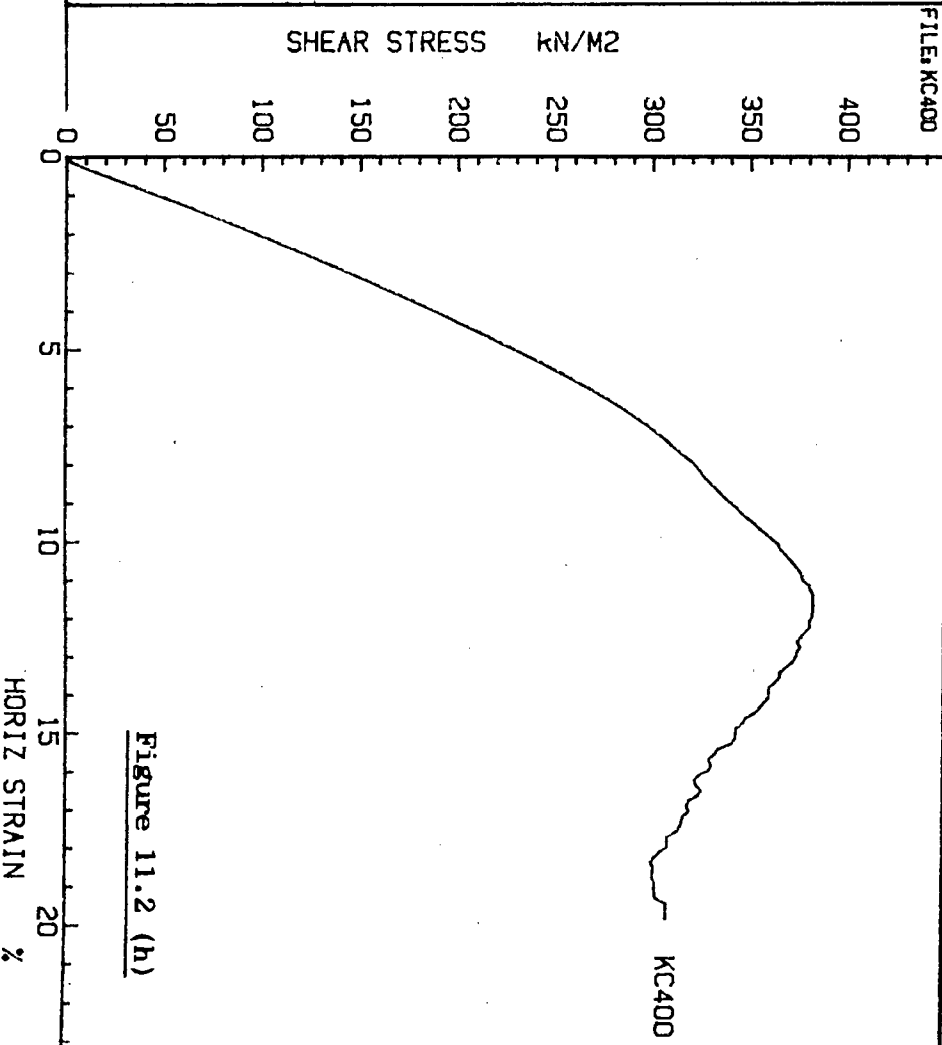
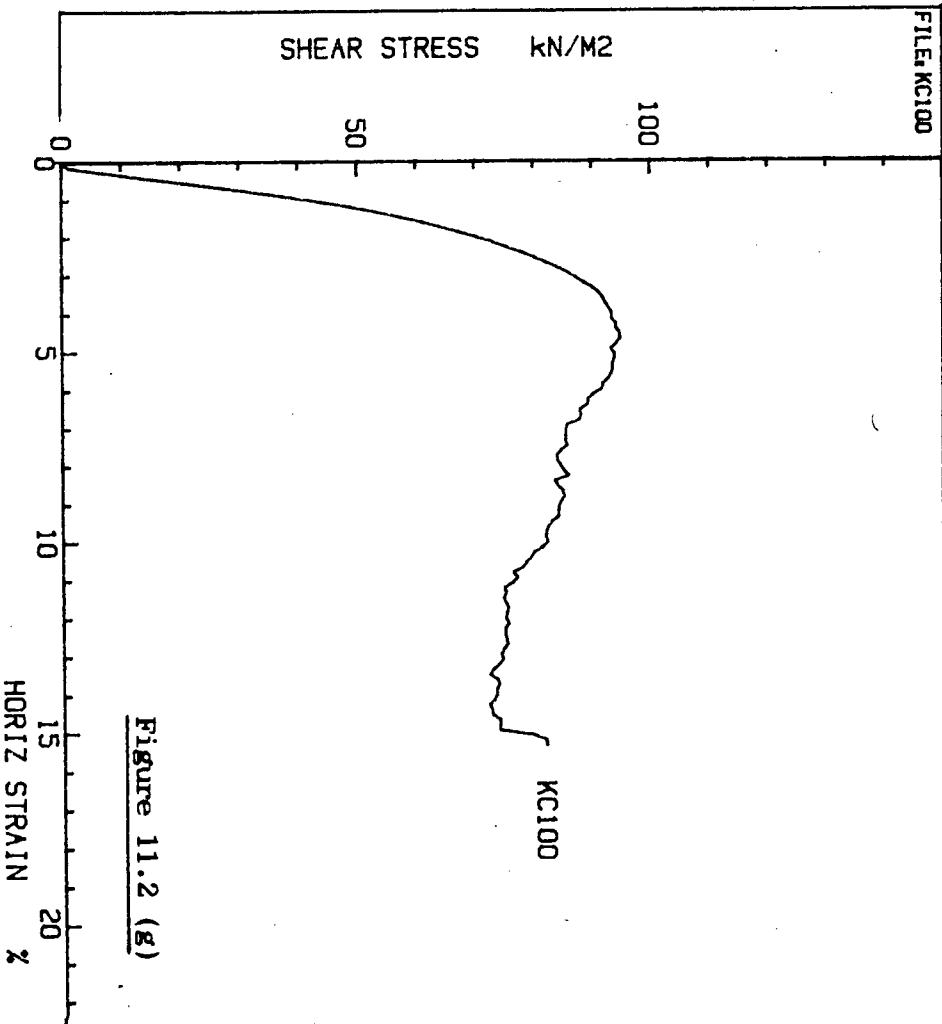
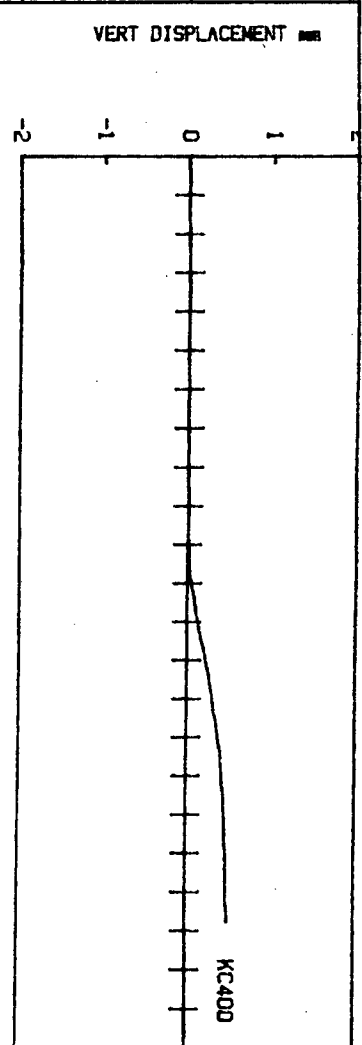
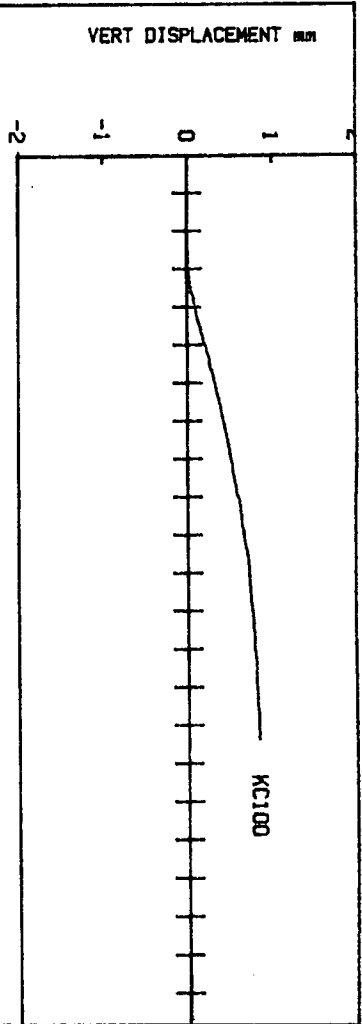
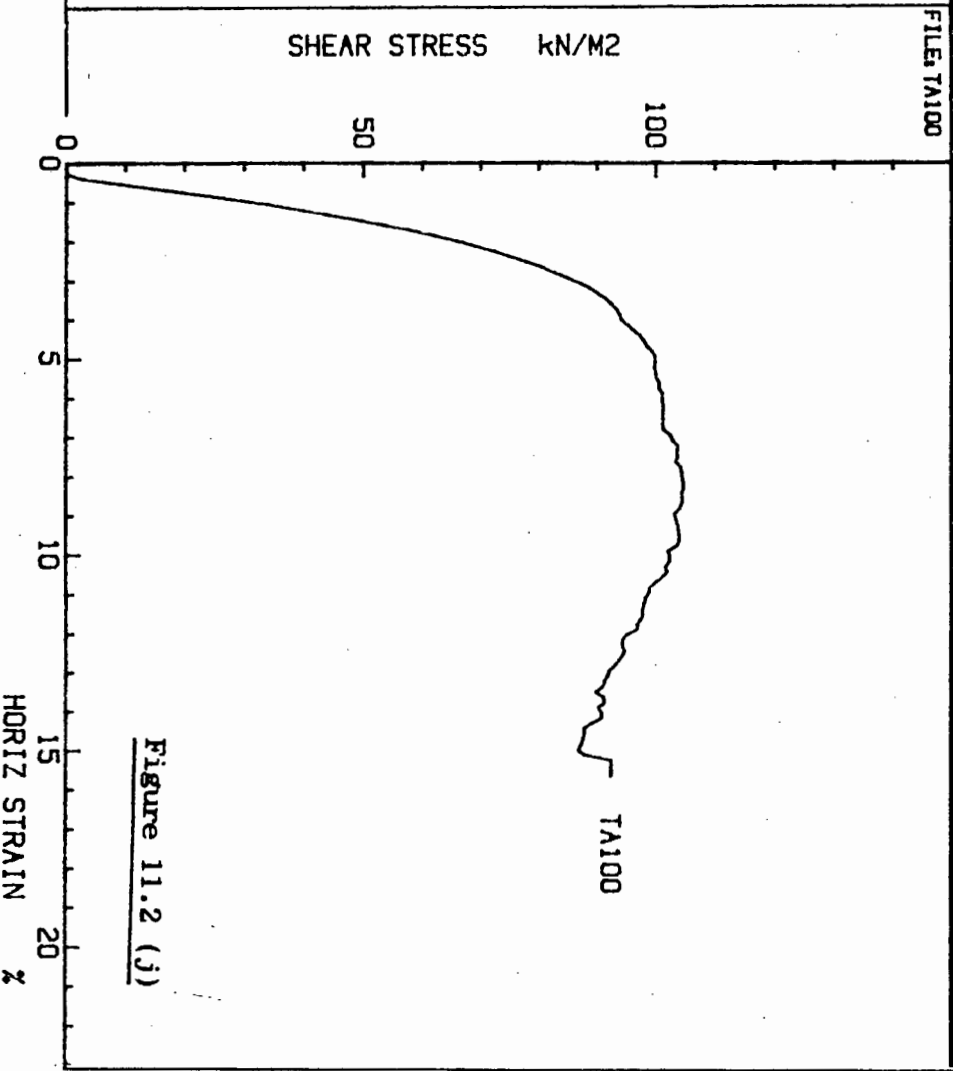
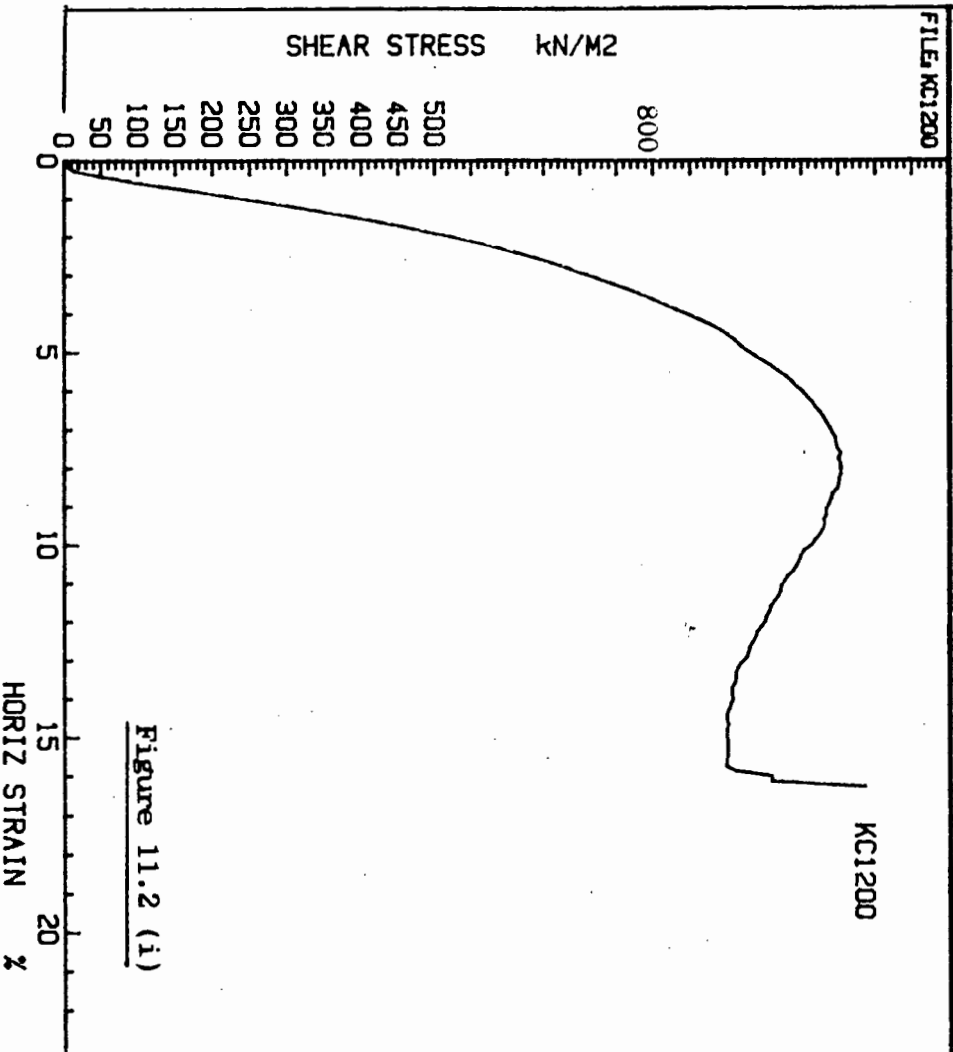
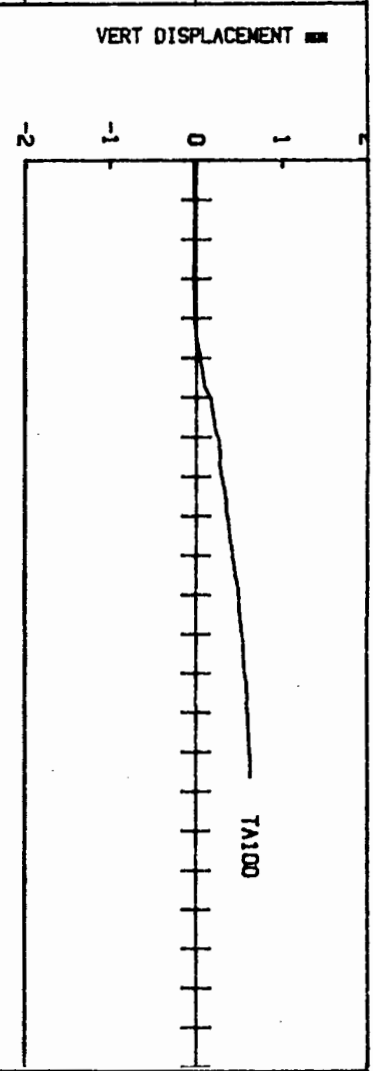
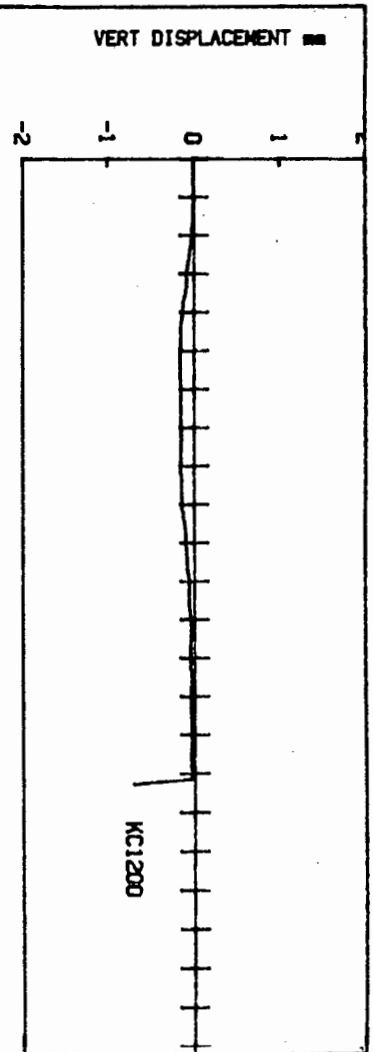


Figure 11.2 (f)





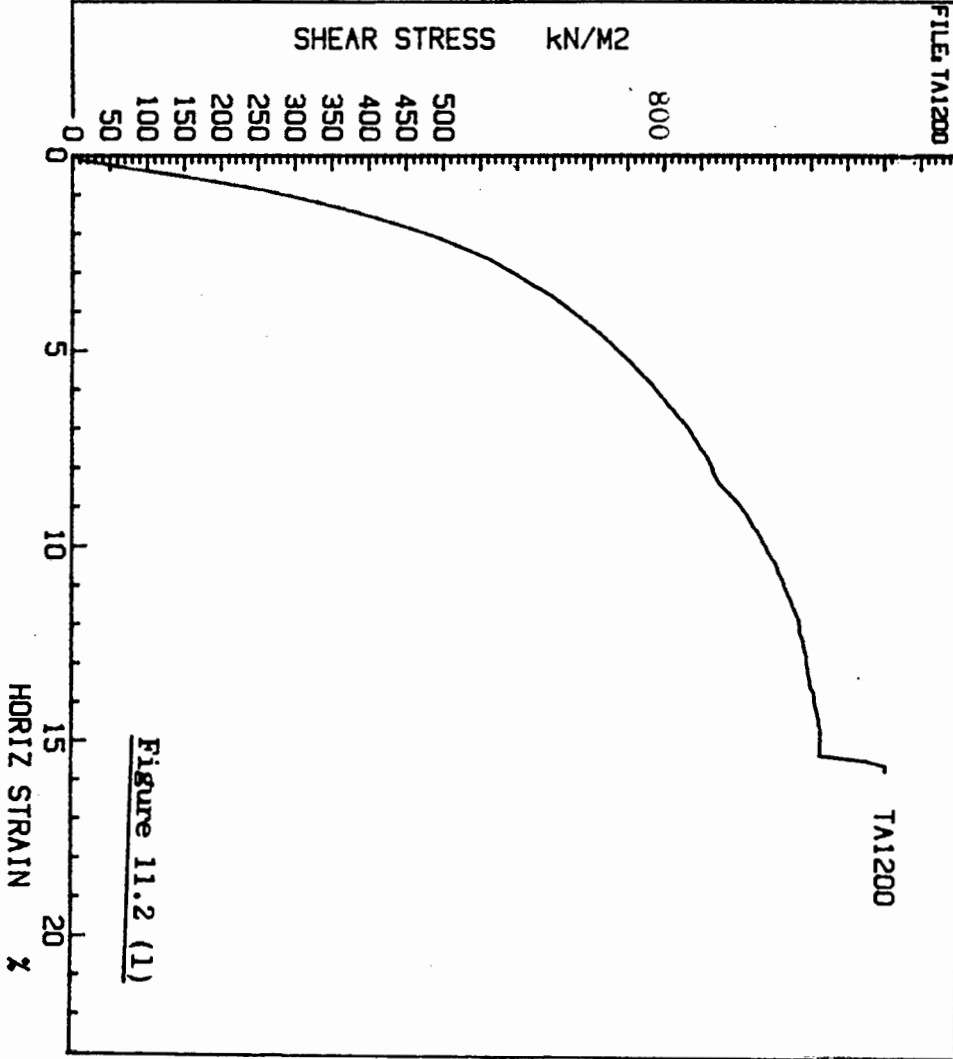
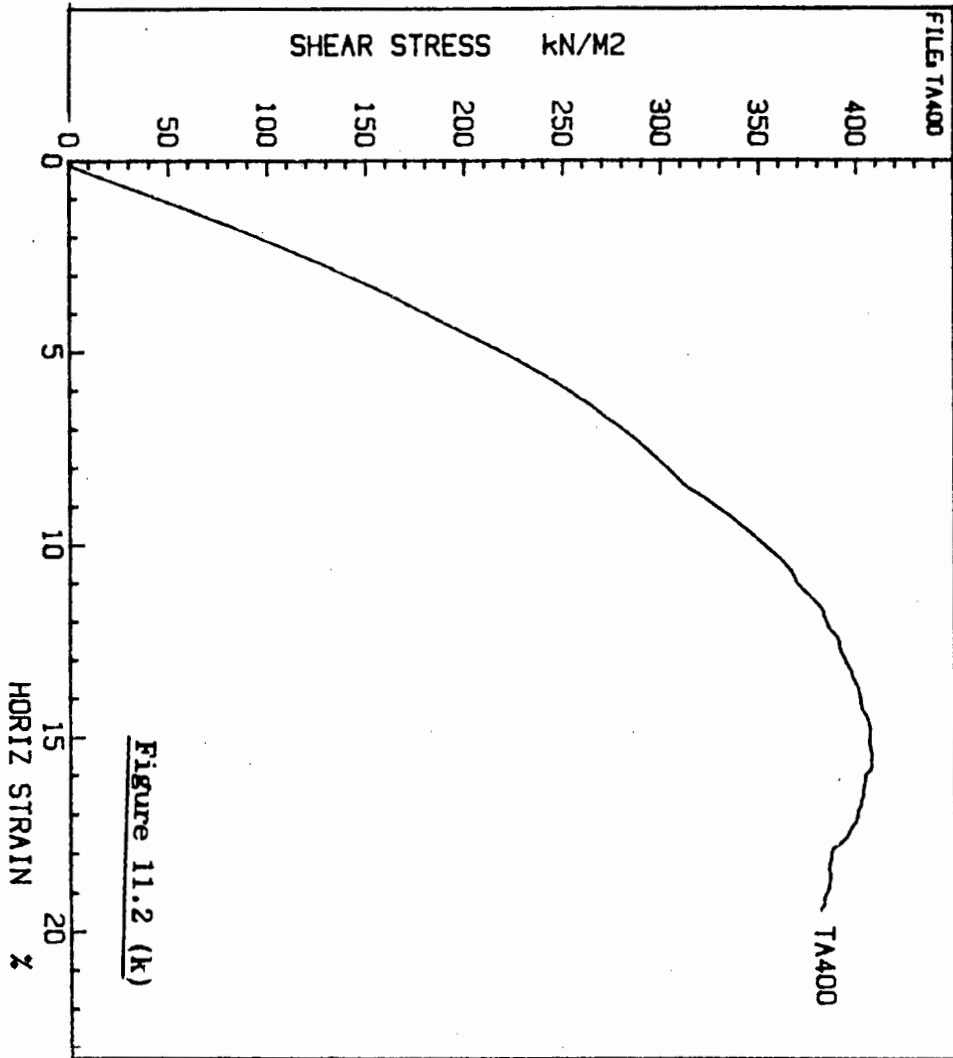
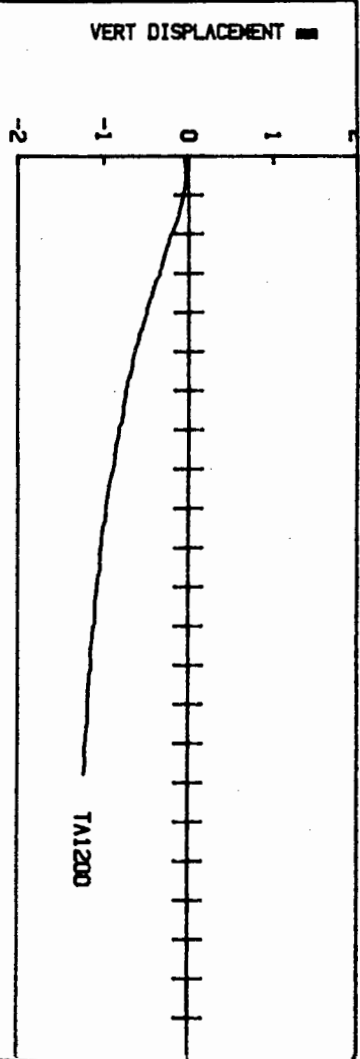
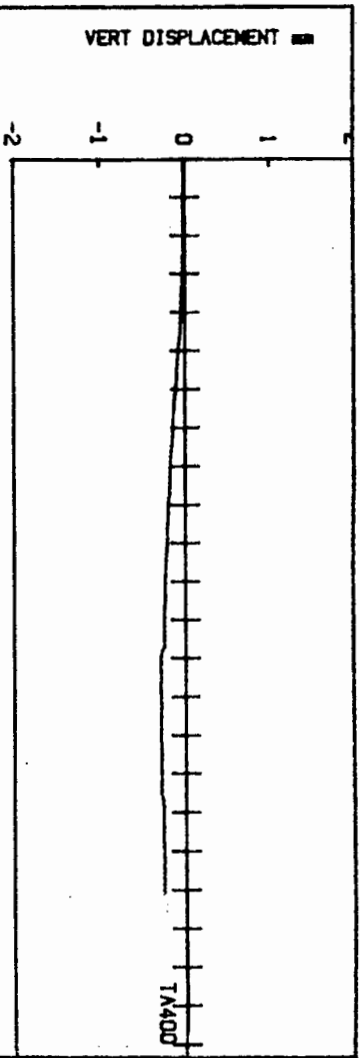


Figure 11.2 (k)

Figure 11.2 (l)

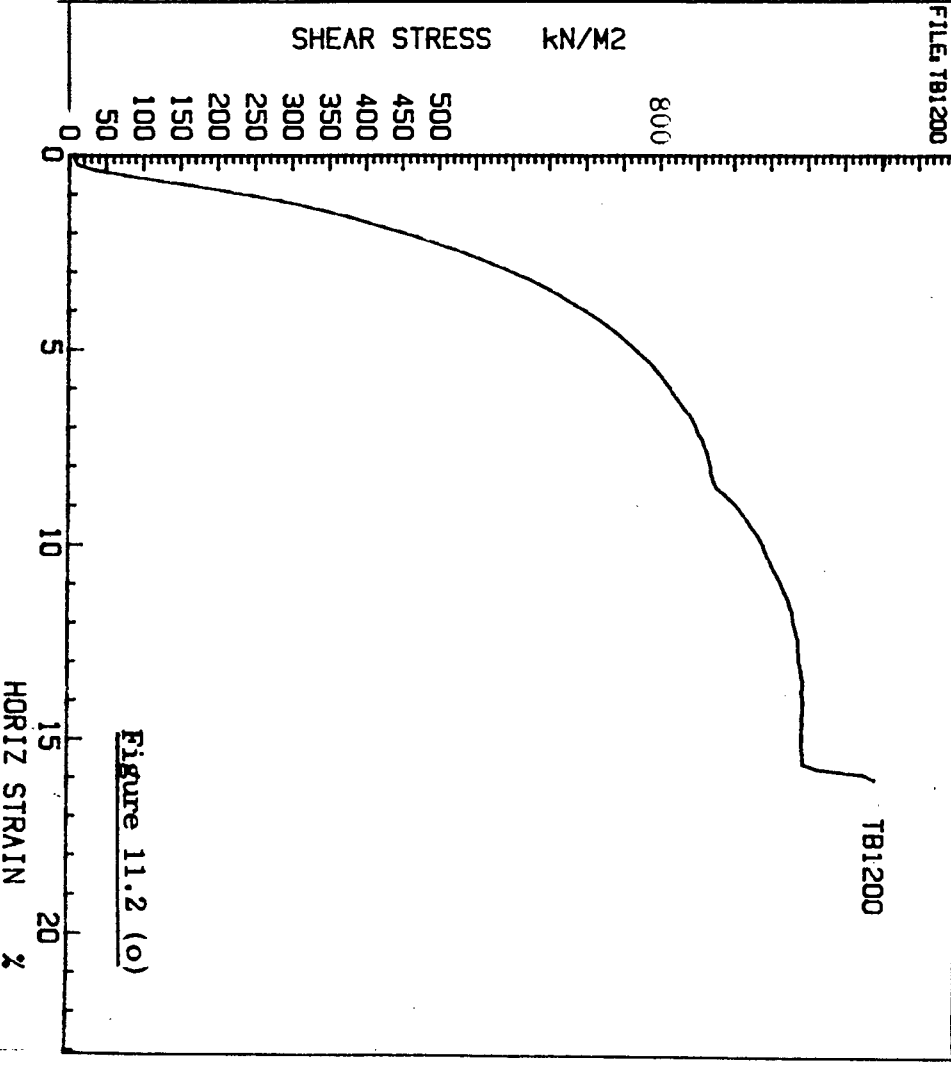
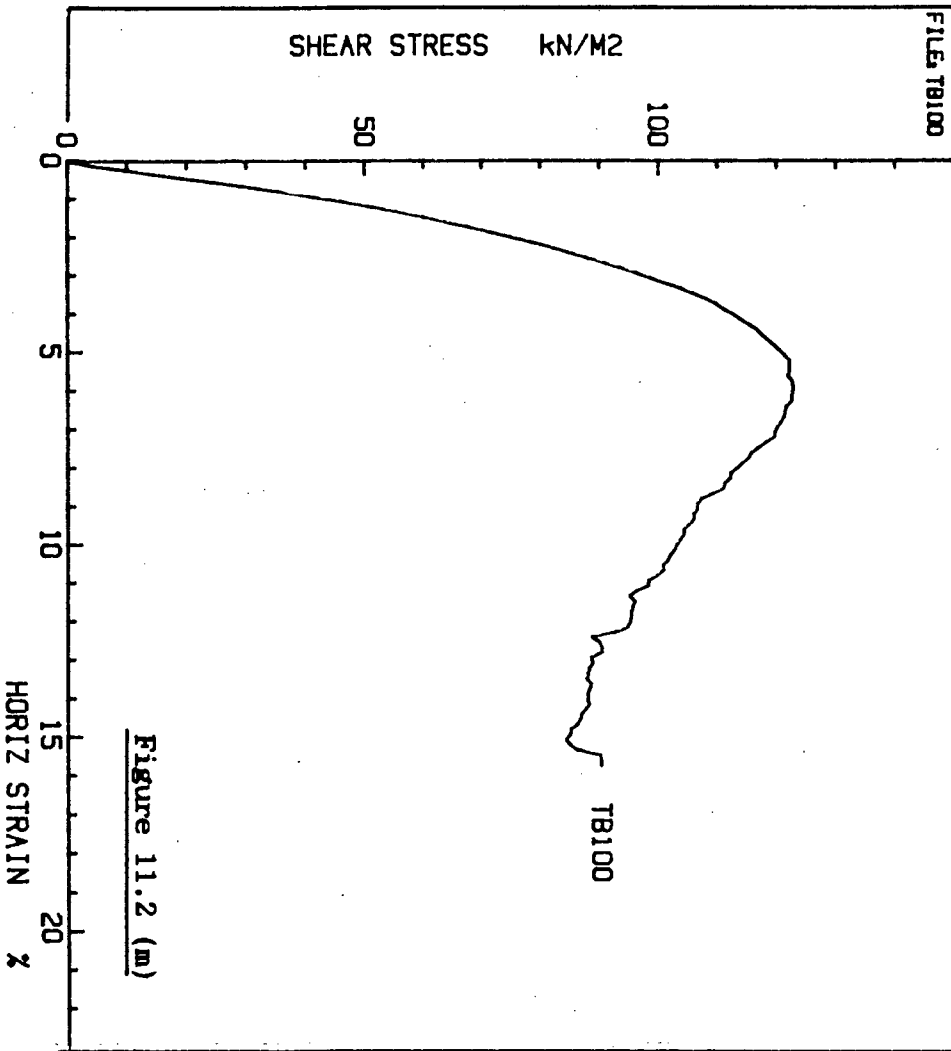
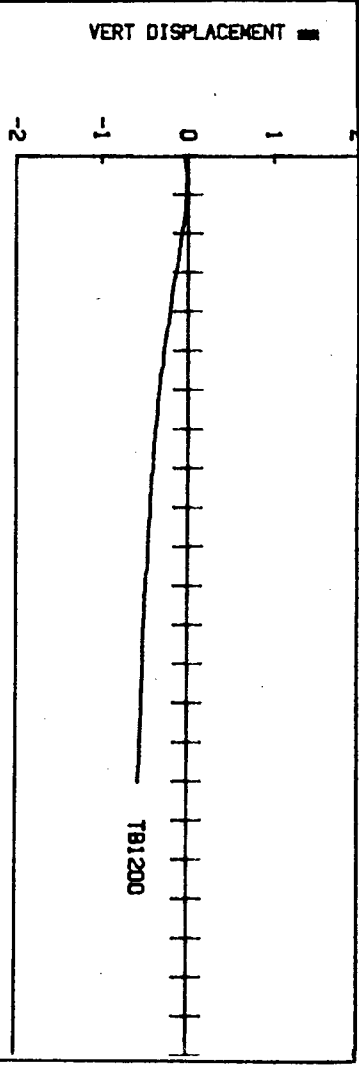
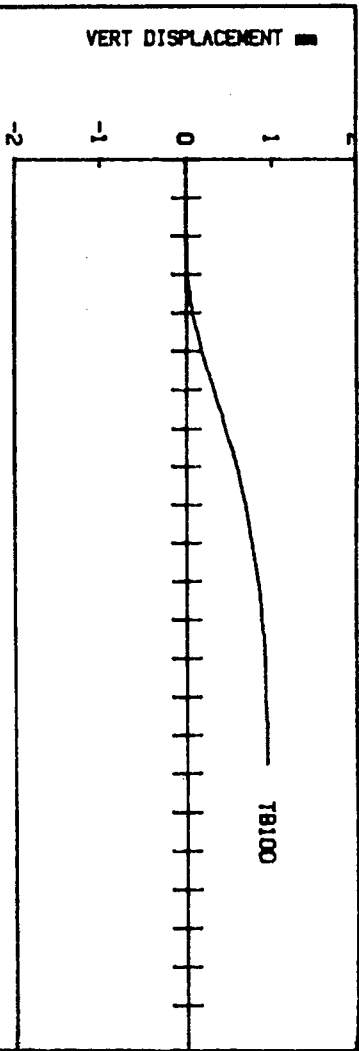
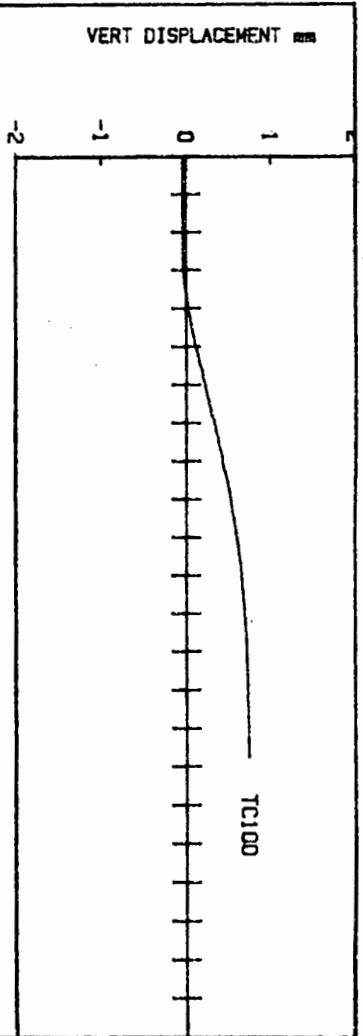


Figure 11.2 (m)

Figure 11.2 (o)



FILE: TC100

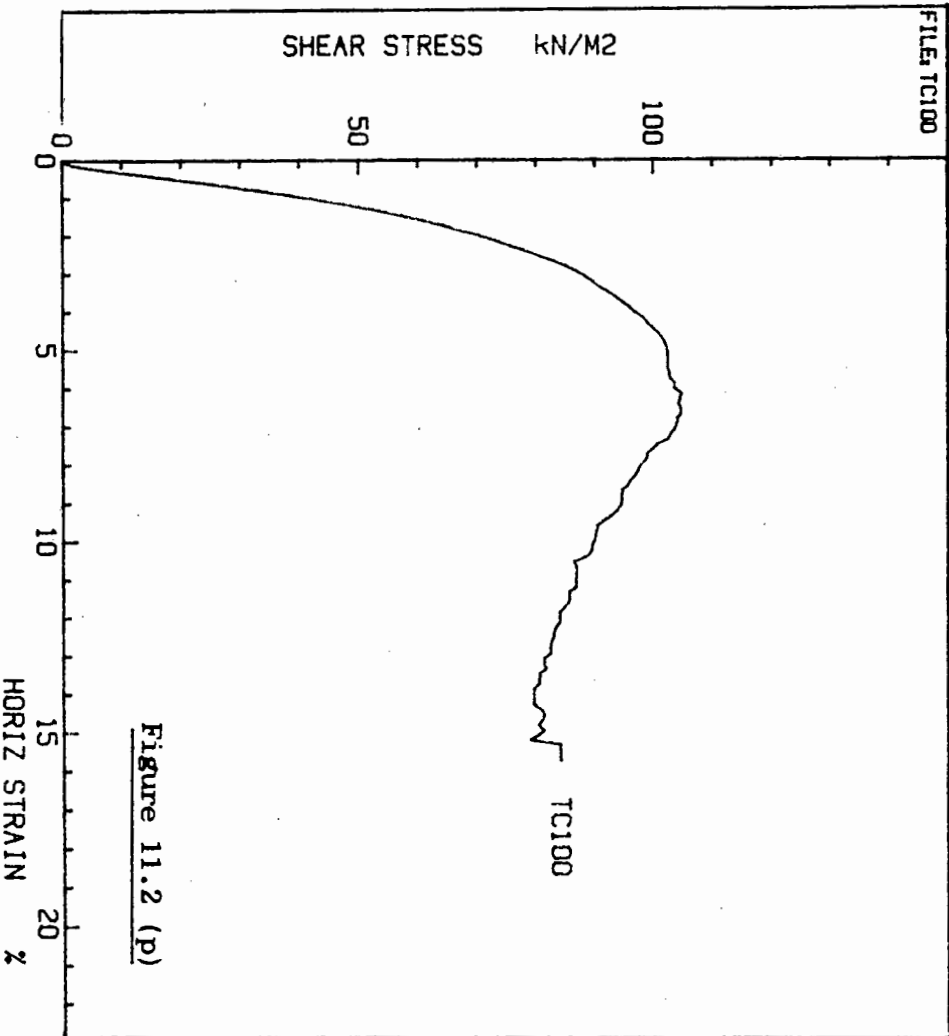
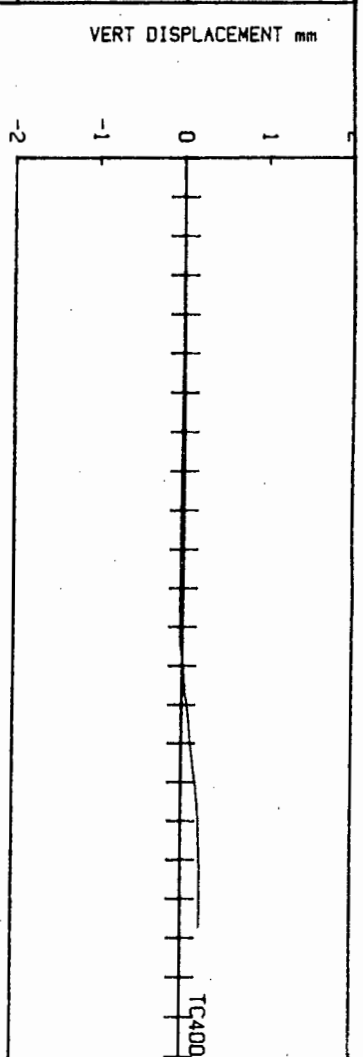


Figure 11.2 (p)



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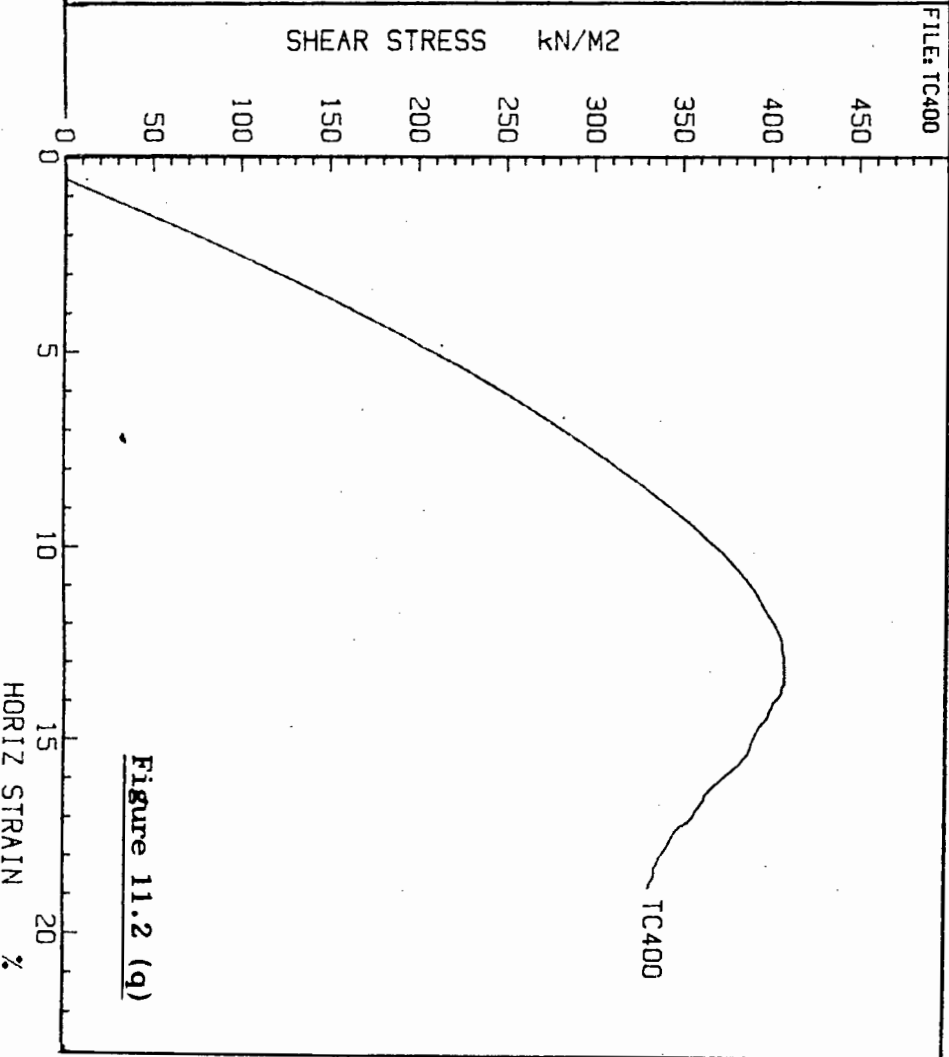


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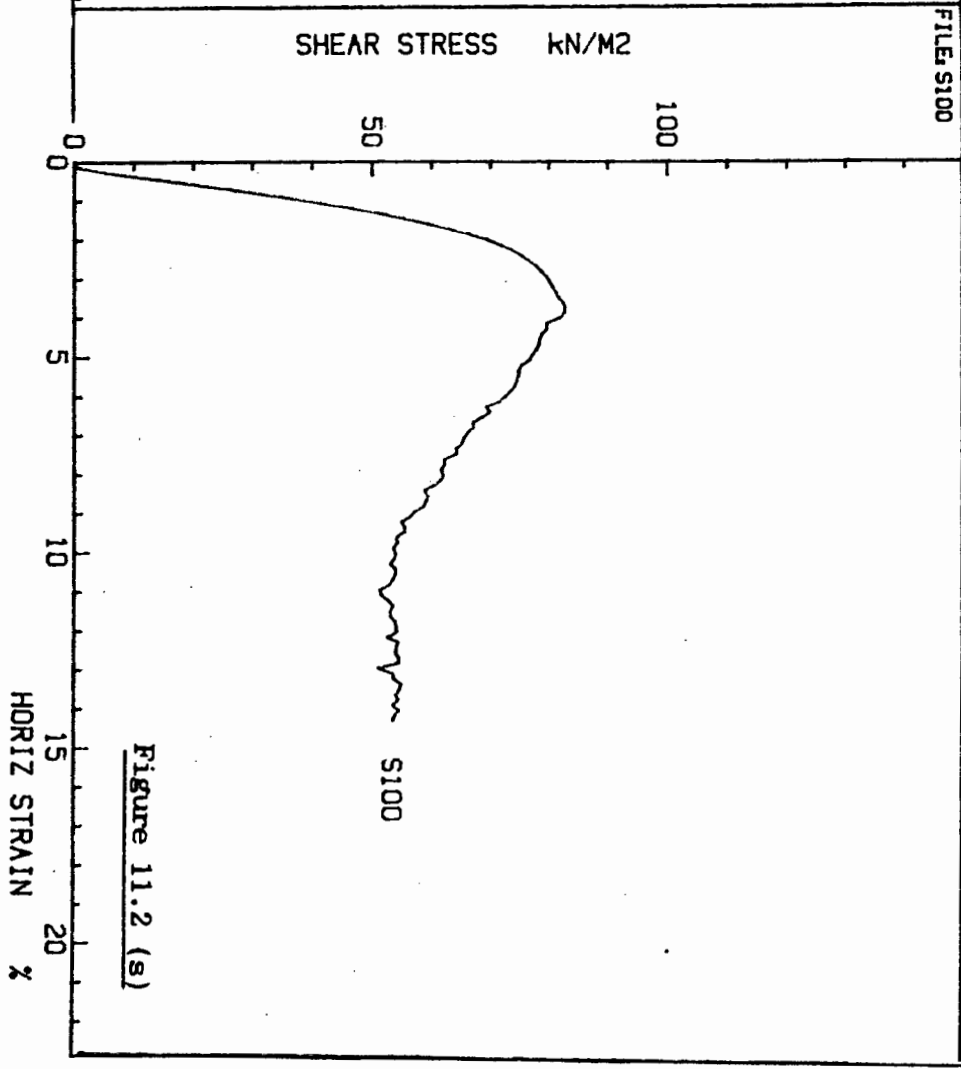
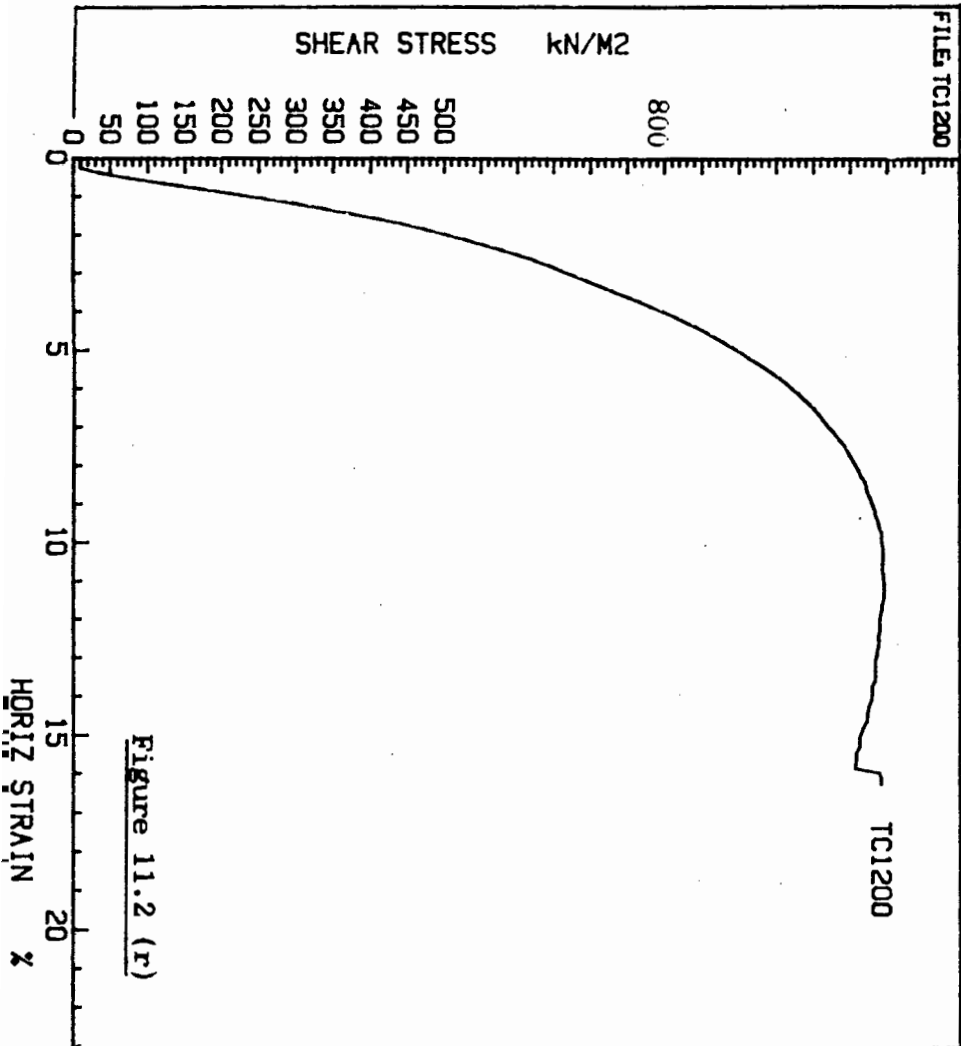
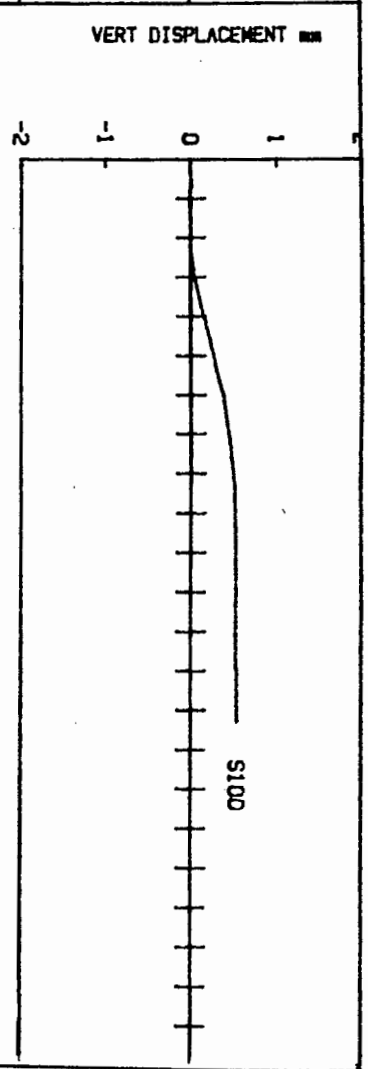
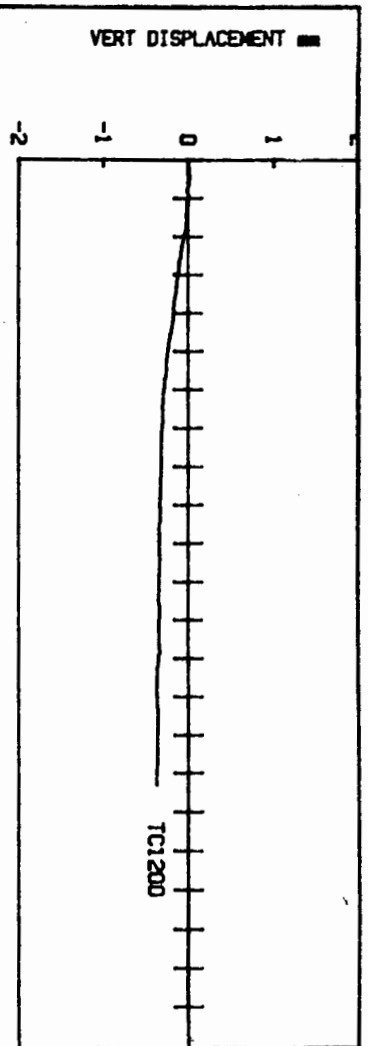


Figure 11.2 (r)

Figure 11.2 (s)

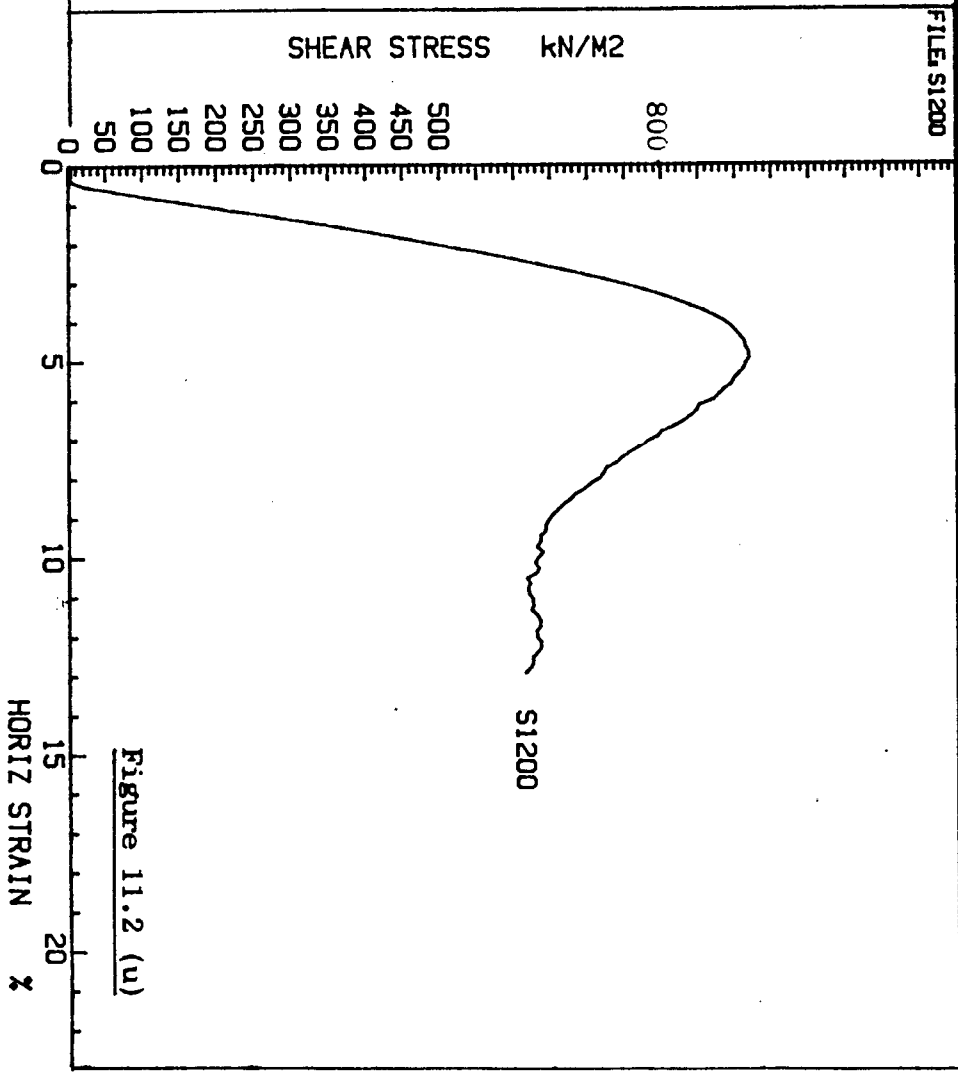
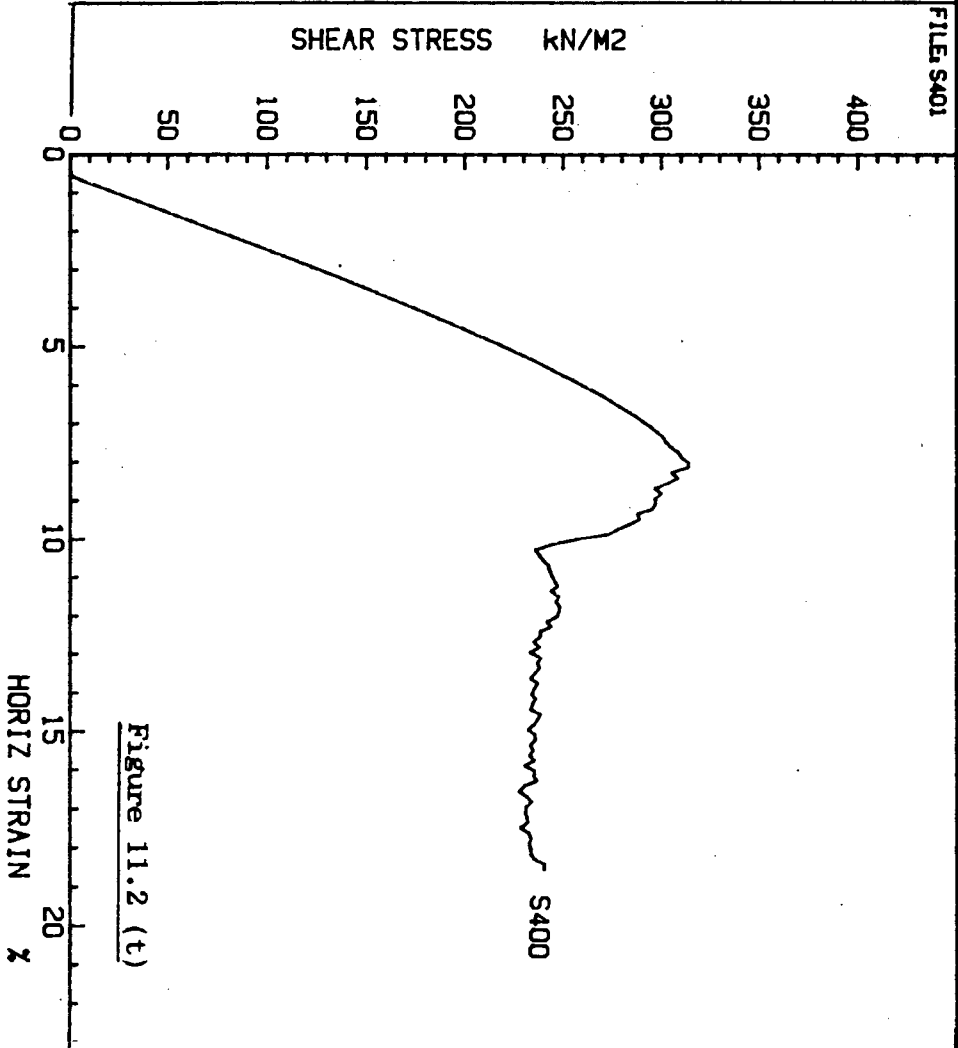
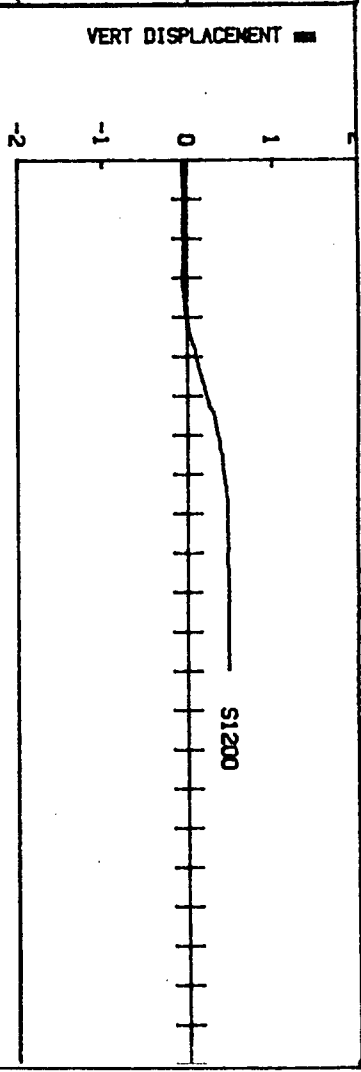
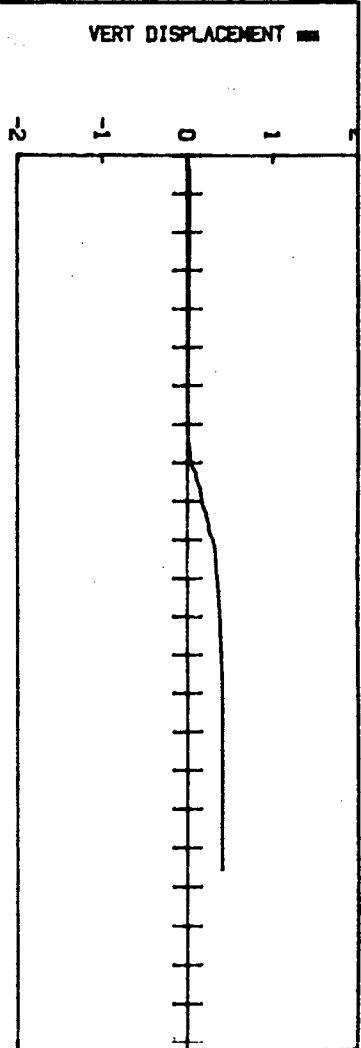


Figure 11.2 (t)

Figure 11.2 (u)

Table 11.3 : Measured friction angles and void ratios for crushability tests

Test No.	Size range (μm)	Normal stress (kPa)	e_u	e_o	$\phi_{\text{max}}(^{\circ})$	$\phi_{\text{ult}}(^{\circ})$	Δh_g (mm)	Stress strain curve
Killa								
K(A) 100	2000 - 1180	100	0.90	0.88	44.0	40	+ 0.85	Figure 11.2(a)
K(A) 400		400	-	0.81	44.0	38.9	+ 0.55	Figure 11.2(b)
K(A) 700		700	0.87	0.84	-	-	-	-
K(A) 1200		1200	0.79	0.75	39.9	39.9	- 0.25	Figure 11.2(c)
K(B) 100	1180 - 850	100	0.83	0.82	43.2	35.5	+ 0.75	Figure 11.2(d)
K(B) 400		400	0.86	0.85	43.7	36.6	+ 0.25	Figure 11.2(e)
K(B) 1200		1200	0.90	0.86	39.8	37.7	- 0.25	Figure 11.2(f)
K(C) 100	850 - 600	100	-	0.69	45.0	33.6	+ 0.85	Figure 11.2(g)
K(C) 400		400	0.77	0.75	42.5	35.3	+ 0.50	Figure 11.2(h)
K(C) 1200		1200	0.78	0.74	41.2	36.2	- 0.15	Figure 11.2(i)
Table Bay								
T(A) 100	2000 - 1180	100	1.48	1.48	46.2	40.9	+ 0.65	Figure 11.2(j)
T(A) 400		400	1.43	1.40	45.2	42.7	- 0.25	Figure 11.2(k)
T(A) 700		700	1.51	1.42	-	-	-	-
T(A) 1200		1200	1.43	1.25	40.3	40.3	- 1.2	Figure 11.2(l)
T(B) 100	1180 - 850	100	1.24	1.23	50.5	39.5	+ 0.9	Figure 11.2(m)
T(B) 400		400	1.25	1.23	45.7	40.0	-	Figure 11.2(n)
T(B) 1200		1200	1.26	1.18	40.1	40.1	- 0.85	Figure 11.2(o)
T(C) 100	850 - 600	100	1.14	1.08	46.2	37.7	+ 0.75	Figure 11.2(p)
T(C) 400		400	1.15	1.18	45.3	39.1	+ 0.25	Figure 11.2(q)
T(C) 1200		1200	1.11	1.07	42.1	41.0	- 0.35	Figure 11.2(r)
Quartz								
S 100		100	0.44	0.44	39.7	27.9	+ 0.55	Figure 11.2(s)
S 400		400	0.45	0.44	38.2	29.4	+ 0.50	Figure 11.2(t)
S 1200		1200	0.49	0.46	37.6	27.3	+ 0.45	Figure 11.2(u)

the amount of crushing under different conditions (e.g. variation in particle size), a numerical expression which is compatible with the particle size characteristics of the original sample is required.

Table 11.4 lists the crushing coefficients as defined by Datta *et al.* (1979), Lee and Fahroomand (1967) and Marsal (1965) for samples sheared under a normal stress of 1200 kPa. For the samples in which crushing was severe, the coefficient defined by Datta *et al.* (1979) was generally the most sensitive. This is consistent with the findings of Beringen *et al.* (1982). However, Datta's coefficient has a poor sensitivity for samples in which moderate but measurable crushing occurred. For example, it indicates no crushing for the Killa size A sample (K(A)400) sheared under a normal stress of 400 kPa.

The Marsal coefficient is more sensitive for samples which experienced little crushing (crushing coefficient of 4.1% for the sample K(A)400). This is due to the high degree of uniformity in original particle sizes, in particular the fact that for all samples more than 90% of the original mass had been retained on a single sieve size. Thus the coefficient (based on the Marsal definition) was calculated as :

$$\% \text{ crushing} = \frac{\% \text{ retained originally} - \% \text{ retained after crushing}}{\% \text{ retained original sample}}$$

where the percentage crushing can vary between 0 and 100%. Coefficients for all tests calculated in this way are listed in Table 11.5. These coefficients are used as a basis for comparison in subsequent discussions.

Table 11.4 : Crushing coefficients (C_k) for samples sheared under a normal stress of 1200 kPa

Shell type	Killa			Table Bay		
	A	B	C	A	B	C
Lee and Fahroomand (1967)	1.2	1.0	1.0	2.4	1.6	1.2
Datta <i>et al.</i> (1979)	2.2	1.5	1.1	5.3	4.1	2.3
Marsal (1965) (%)	19.0	8.6	1.4	49.4	34.2	17.8

A : 2000 - 1180 μm

B : 1180 - 850 μm

C : 850 - 600 μm

Table 11.5 : Measured crushing coefficients based on Marsal definition (C_k %) for samples subjected to crushing tests

σ_n (kPa)	Killa			Table Bay		
	A	B	C	A	B	C
100 #	0.6	-	-	2.0	-	-
400 #	4.1	-	-	20.9	-	-
700 #	6.3	-	-	32.4	-	-
1200 #	19.0	8.6	1.4	49.4	34.2	17.8
700 *	1.3	-	-	5.3	-	-
1200 *	3.0	-	-	16.4	-	-

sheared under normal stress

* normal stress only applied

(ii) Shear strength behaviour during crushing tests

The data presented in Table 11.3 and Figures 11.2(a) to (u) indicates that the shear strength behaviour of the samples was generally consistent with those of similar materials tested by other researchers in that :

- All samples, including the quartz sand, showed a decrease in the maximum angle of internal friction as the normal stress was increased from 100 kPa to 1200 kPa. However, the decrease for the quartz 'control' sand was relatively small compared with that of the carbonate sands. This implies a greater degree of curvature for the Mohr failure envelopes of the carbonate sands.
- At the lower normal stresses the carbonate sand samples showed 'brittle' behaviour and a volume increase during shear. At the higher normal stresses behaviour gradually became more 'plastic' and sample volumes decreased during shear (cf. Figures 11.2(a) and (c)). The 'brittle' and 'plastic' behaviour is evidenced by the decreasing differences between the maximum and "ultimate" friction angles at increasing normal stresses (i.e. no "peaking"). In contrast, the quartz sand dilated strongly even at the maximum normal stress (Figure 11.2(u)).

The decrease in the angle of maximum shearing resistance corresponding to an increase in applied normal stress from 100 kPa to 1200 kPa ($\Delta\phi_m$), together with the crushing coefficients at 1200 kPa are listed in Table 11.6. The average decrease in angle of maximum shearing resistance is 4.8° . Sample TB1200 suffered the maximum decrease (10.4°) and the quartz sand, for which there was no measurable crushing, experienced the minimum decrease (2.1°). However, for the carbonate sands, there is no clear correlation between $\Delta\phi_m$ and the amount of crushing.

Datta *et al.* (1979) conducted triaxial experiments on near-pure carbonate sands and a quartz sand falling in a size range similar to the materials used in this investigation (see Section 4.3.4). Secant angles of maximum internal resistance at the lowest confining pressure (98 kPa) by Datta *et al.* show a good correspondence with the results obtained at 100 kPa in the present study. Datta *et al.* also found a good correlation between crushability and decreasing friction angles at increasing confining pressures. However, these tests were conducted at confining pressures of up to 6272 kPa. Values of $\Delta\phi_m$ ranging from a minimum of 7° to 20.7° and crushing coefficients of up to 8 were recorded.

The limitation on the maximum normal pressure available in the direct shear apparatus (i.e. $1300 \pm$ kPa) unfortunately did not allow for the generation of $\Delta\phi_m$ values sufficiently large to establish a clear relationship between this parameter and crushability.

(iii) Effect of particle shape on crushability and compressibility

In Section 4.6.4 of the literature review it was suggested that mass physical properties, in particular the maximum obtainable void ratio (e_{\max}), could serve as a useful index for crushability and compressibility.

The values of e_{\max} (Table 11.2) for the samples tested ranged from 0.75 for the quartz sand, (indicating "simple shape" particles according to the classification of Vinopal and Coogan (1978)), to 1.93 for the 1180 - 200 μm Table Bay sand (indicating particles of "radical shape").

Figure 11.3 shows a plot of e_{\max} versus C_k for the samples sheared at 1200 kPa. From this plot a clear correlation between increasing particle size, e_{\max} and crushability is evident. The slopes of the lines drawn through

points of equal particle size indicate that, within the range of material properties used in this investigation, an increase in e_{\max} of 0.1 units leads to an increase in C_k of approximately 5 percentage points, irrespective of particle size.

Figure 11.4 shows a plot of Δe_{\max} versus ΔC_k by which the effect of particle size in Fig. 11.3 is eliminated :

Δe_{\max} = difference in e_{\max} of samples with similar particle size

ΔC_k = difference in C_k of samples with similar particle size.

A point at the origin of such a plot would imply that particles of carbonate sand with the same average particle size and e_{\max} have equal crushing susceptibility. The close intercept of the linear regression line with the origin of this plot confirms the validity of the assumption for the particular materials tested.

The compressibility of samples cannot be determined accurately with the direct shear apparatus, but two measurements can be used as a qualitative indicator; the initial "consolidation" settlements and the relative changes in sample height during shear.

Table 11.7 lists the sample heights directly after placement (h_u), sample heights after consolidation under a normal stress of 1200 kPa (h_o), the resultant settlement (Δh) and e_{\max} values. The Table Bay samples with larger e_{\max} experience the largest Δh , the Killa samples with similar e_{\max} all have similar values of Δh while the quartz sand with the smallest e_{\max} show the least settlement. However, no clear trend can be established as

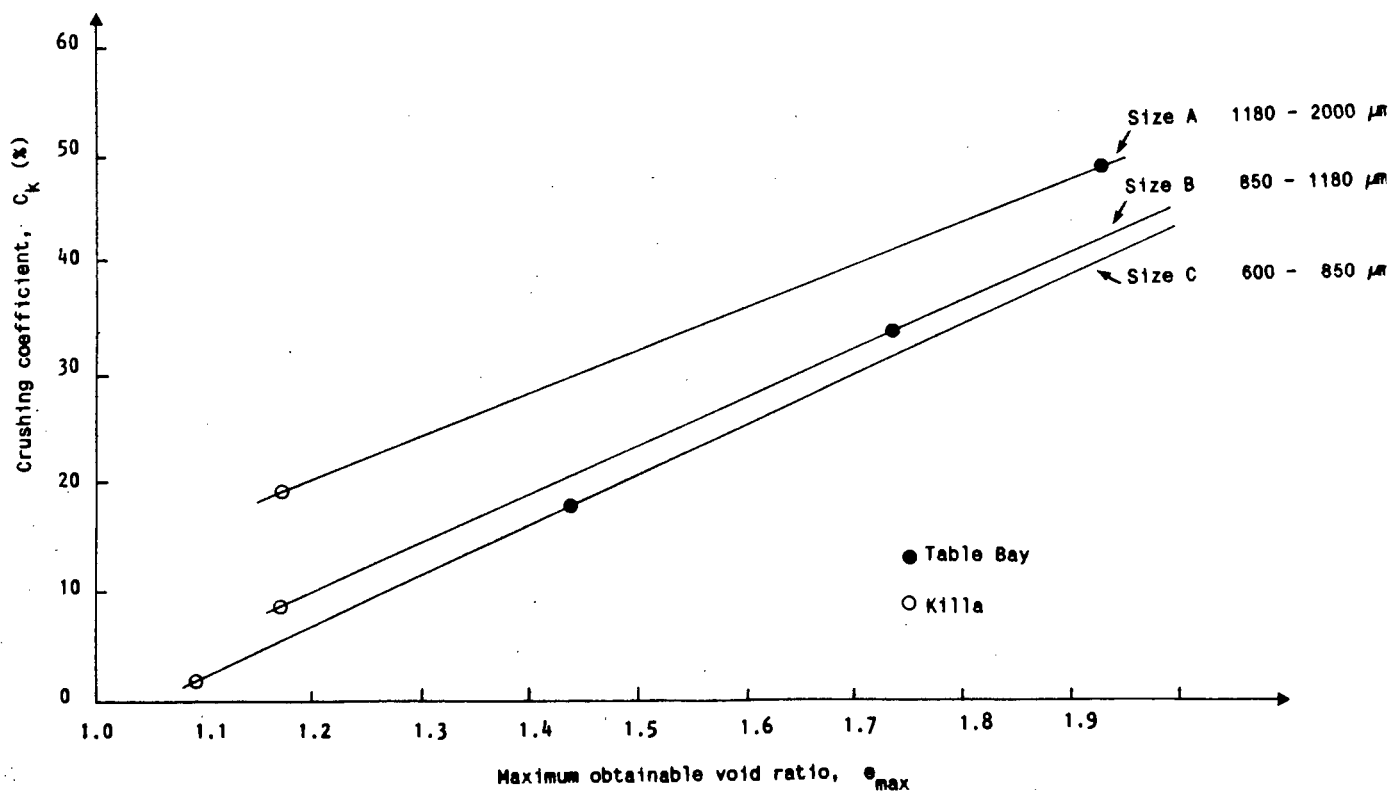


Figure 11.3 : Crushing coefficients versus maximum obtainable void ratios for samples sheared under a normal stress of 1 200 kPa

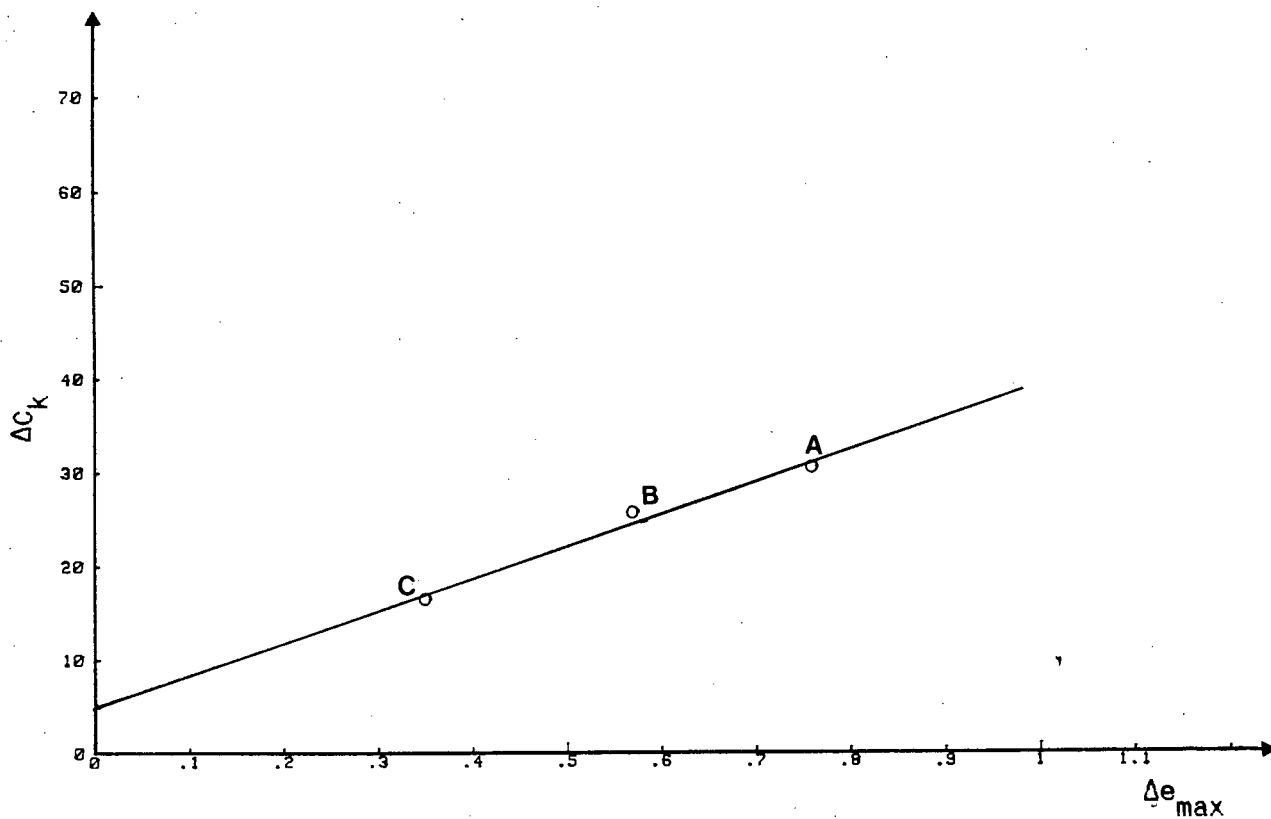


Figure 11.4 : Δe_{max} versus ΔC_k for carbonate sands sheared under a normal stress of 1200 kPa

Table 11.6 : Difference between maximum friction angles at minimum and maximum applied stress

Sand type	Killa			Table Bay			Quartz
	A	B	C	A	B	C	
$\Delta\phi_m$ (°) *	4.1	3.4	3.8	5.9	10.4	4.1	2.1
$C_{k_{max}}$ (%)	19.0	8.6	1.4	49.4	34.2	17.8	0

* $\Delta\phi_m = (\phi_{max} \text{ at } \sigma_n = 100 \text{ kPa}) - (\phi_{max} \text{ at } \sigma_n = 1200 \text{ kPa})$

$C_{k_{max}}$ = crushing coefficient (Marsal definition) at $\sigma_n = 1200 \text{ kPa}$

A : 2000 - 1180 μm

B : 1180 - 850 μm

C : 850 - 600 μm

Table 11.7 : Consolidation settlements for applied normal stress of 1200 kPa

Sand type	Killa			Table Bay			Quartz
	A	B	C	A	B	C	
h_u (mm)	22.99	23.99	22.37	24.07	23.36	22.98	23.60
h_o (mm)	22.49	23.56	22.89	22.27	22.46	22.50	23.22
Δh (mm)	- 0.50	- 0.43	- 0.52	- 1.79	- 0.90	- 0.48	- 0.38

this measurement is relatively sensitive to variability in the initial placement densities.

The change in sample heights during shear (Δh_g) under increased normal loads also provides an indication of the relative compressibilities. Maximum recorded values for each test are listed in Table 11.3. Curves of vertical displacement versus horizontal strain for applied normal stresses of 100 and 1200 kPa are shown in Figures 11.5(a) and (b). At the lower stress of 100 kPa, all samples show dilating behaviour, but there is no correlation with e_{\max} values. At the maximum applied stress of 1200 kPa, only the quartz sand continued to dilate, while all other samples decreased in volume with the largest volume reductions corresponding to the largest e_{\max} values.

The maximum changes in sample heights during shear under an applied stress of 1200 kPa, together with corresponding e_{\max} values, are plotted in Figure 11.6. This plot shows that if the maximum change in sample height at 1200 kPa is taken as a measure of the sample "compressibilities", then a remarkably good correlation between e_{\max} and relative "compressibility" applies.

(iv) Effect of particle size

The relative spacing between lines through points of equal particle size in Figure 11.3 indicates the sensitivity of crushing susceptibility to particle size. The spacing between these lines is roughly proportional to the ratio of the average particle size (D_{60}) represented by each line :

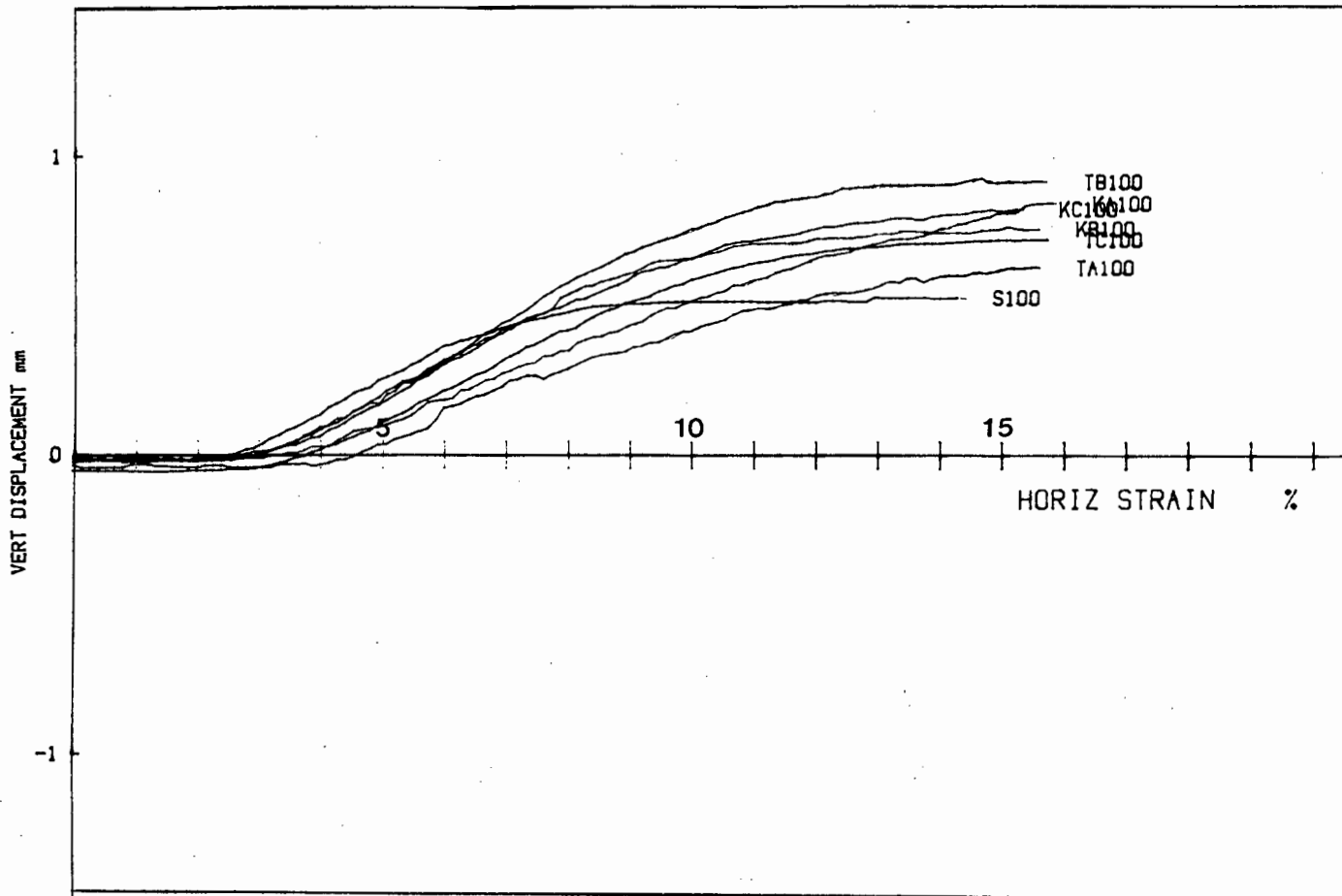


Figure 11.5 a : Change in sample height for sands sheared under a normal stress of 100 kPa

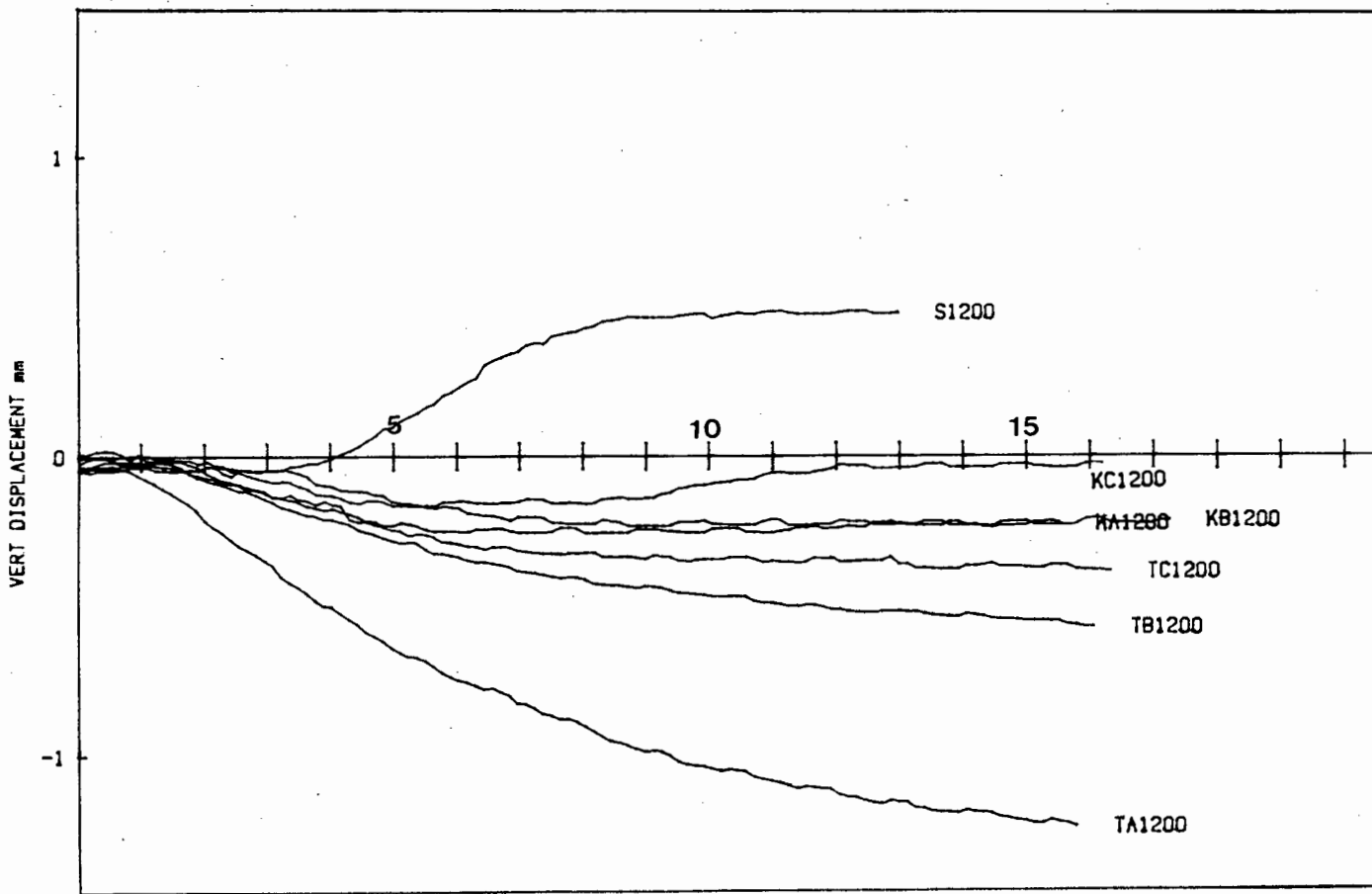


Figure 11.5 b : Change in sample height for sands sheared under a normal stress of 1200 kPa

<u>line</u> :	A	B	C
Average D_{60} (mm)	1.575	1.0	0.735

and using line C as a base;

$\frac{D_{60}}{D_{60c}}$	2.14	1.36	1.0
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Average particle size (D_{60}) versus crushing coefficients (C_k) are shown in Figure 11.7, which also illustrates the strong influence of particle size on crushing susceptibility. The effect of size is partially obscured for the Table Bay samples, since the three size ranges also have widely variable and decreasing e_{max} , implying decreasing crushing susceptibility. However, a near-linear relationship between particle size and crushability is indicated for the Killa shell, for which the three size ranges have very similar e_{max} values.

Particle size appears to have a much more dominant influence on the crushability than on the compressibility. Figure 11.8(a) shows the vertical displacement versus horizontal strain for two size ranges of Killa shell with identical e_{max} , sheared at 100 kPa and 1200 kPa. For comparison, the curves for the two Killa shell samples are plotted against those of the Table Bay samples with identical particle size but differing e_{max} in Figures 11.8(b) and (c) respectively.

(v) Effect of sample size

Results for the 1180 - 2000 μm Killa and Table Bay samples subjected to normal and shear stresses of 1200 kPa, at different sample heights, are listed in Table 11.8. The following observations can be made :

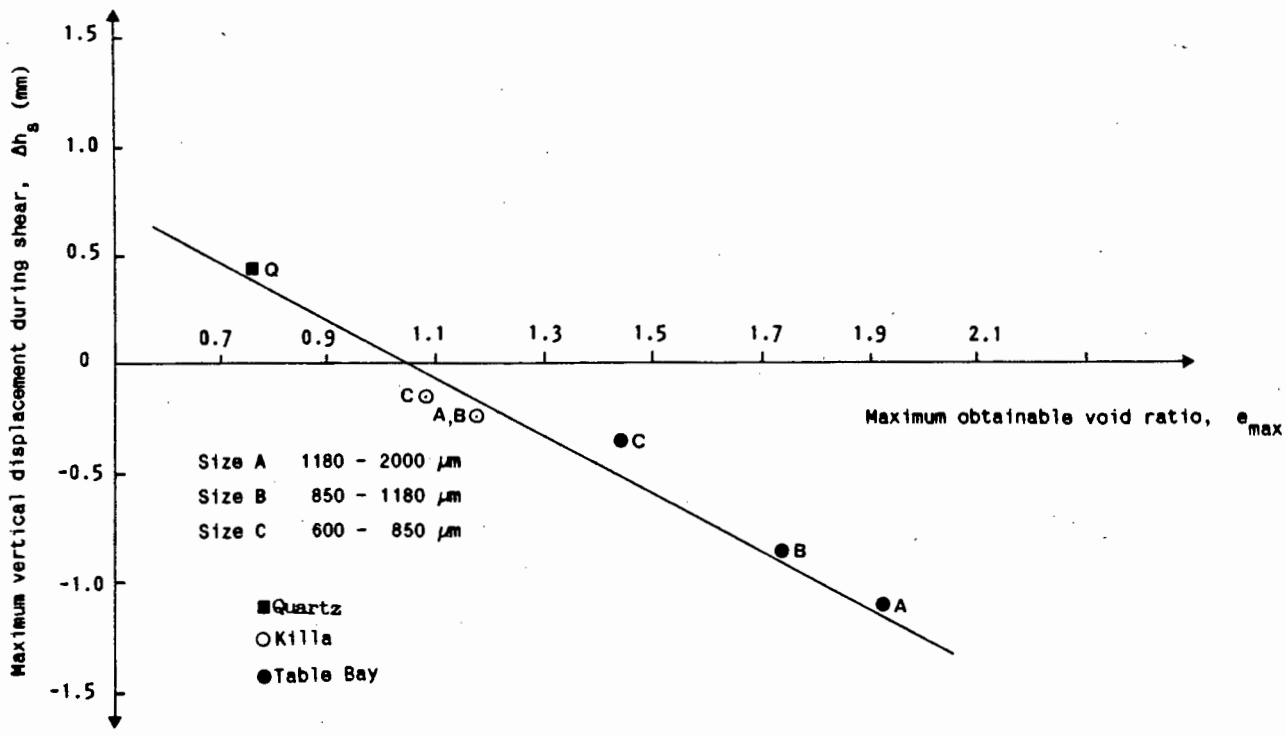


Figure 11.6 :

e_{max} versus maximum change in sample height (Δh_s) for sands sheared under a normal stress of 1200 kPa

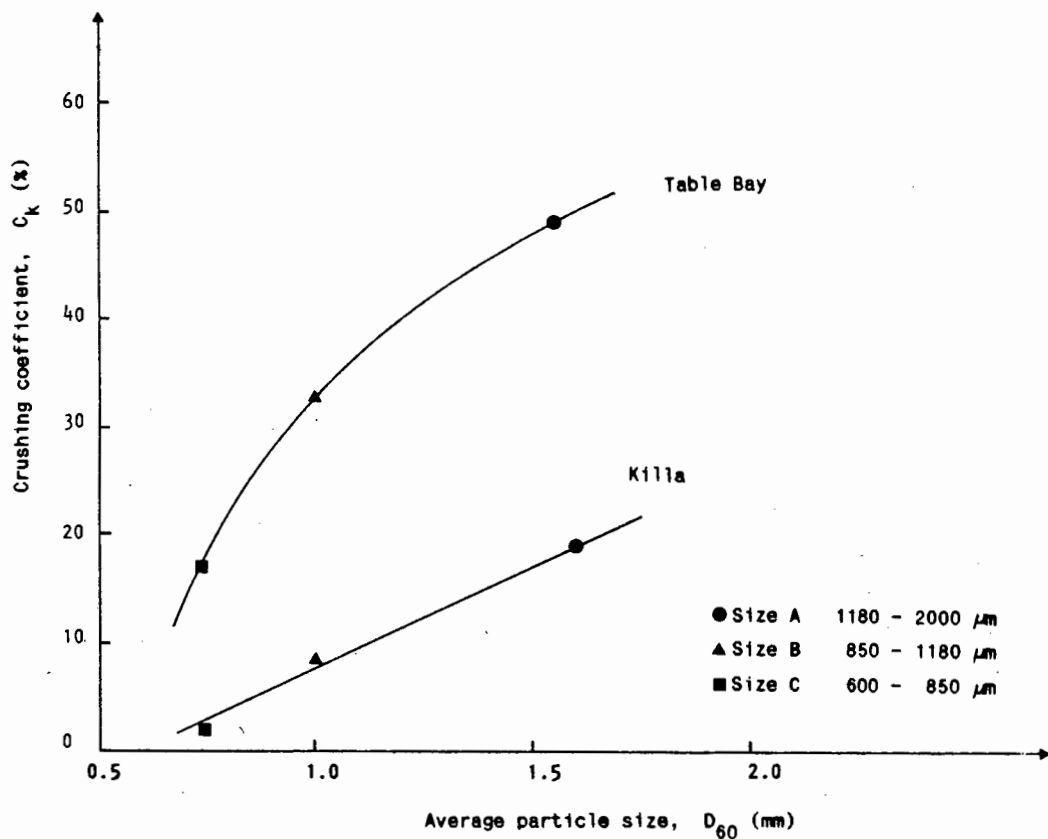


Figure 11.7 :

Average particle size (D_{60}) versus crushability (C_k) for Killa and Table Bay sands

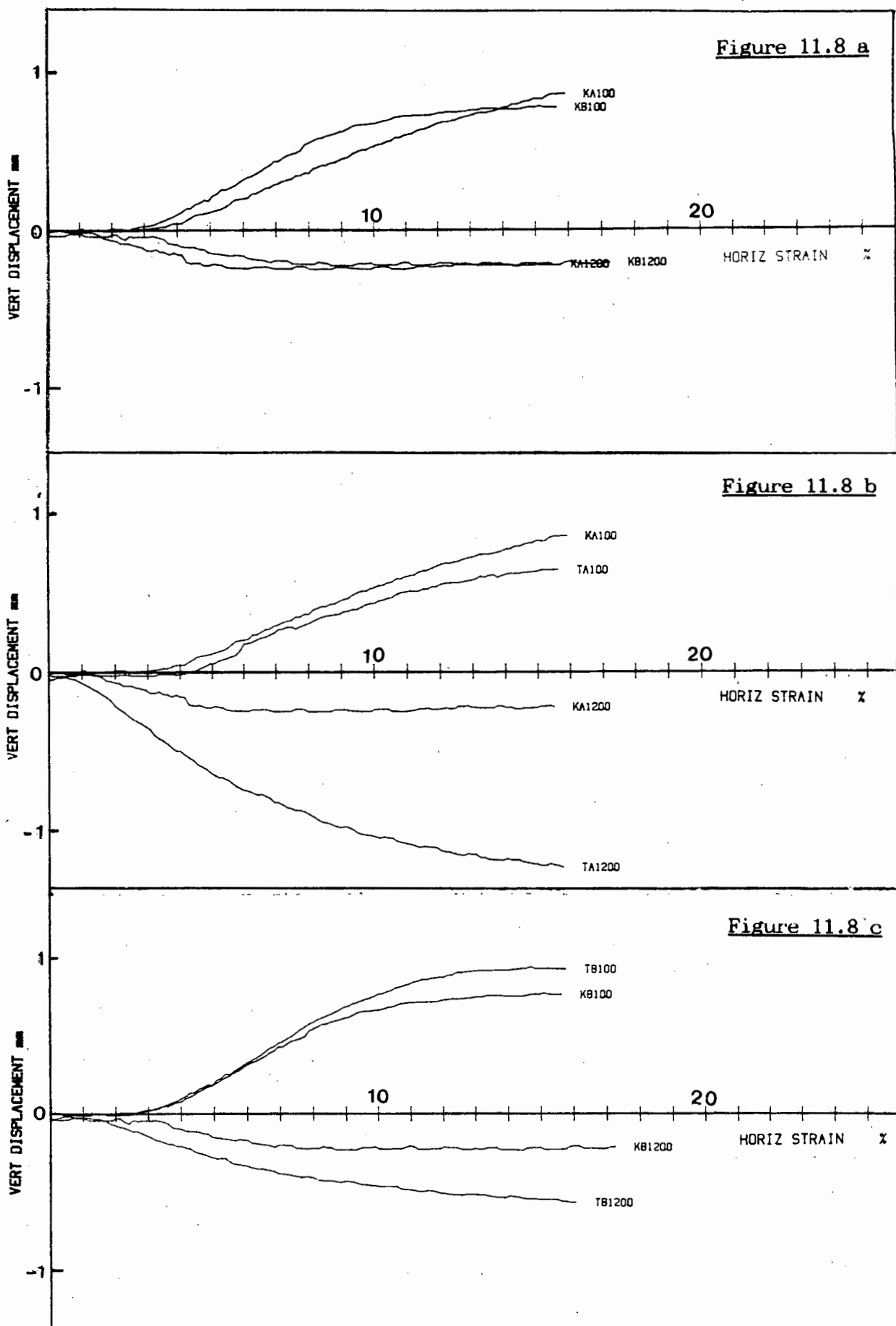


Figure 11.8 : Vertical displacement versus horizontal strain for samples sheared under normal stresses of 100 kPa and 1 200 kPa;

11.8 (a) equal e_{max} ; differing D_{60}

11.8 (b) equal D_{60} ; differing e_{max}

11.8 (c) equal D_{60} ; differing e_{max}

Table 11.8 : Effect of variable sample height on crushability and "consolidation" settlement

Test No.	Applied Stress	ϕ_m (°)	C_k (%)	h_u (mm)	h_o (mm)	Δh (mm)	$\Delta h/h_u$
<u>Killa</u>							
KA 1200(i)	Normal		3.0	23.60	23.22	- 0.38	- 0.016
KA 1200(ii)	Normal and shear	39.9	19.0	22.99	22.49	- 0.50	- 0.022
KA 1200(iii)	Normal and shear	38.6	13.7	37.64	37.02	- 0.62	- 0.016
<u>Table Bay</u>							
TA 1200(i)	Normal		16.4	23.99	22.68	- 1.31	- 0.055
TA 1200(ii)	Normal		15.7	37.61	35.54	- 2.07	- 0.055
TA 1200(iii)	Normal and shear	40.3	49.4	24.06	22.27	- 1.79	- 0.074
TA 1200(iv)	Normal and shear	39.2	39.4	37.40	35.08	- 2.32	- 0.062

KA : Killa 1180 - 2000 μ m
 TA : Table Bay 1180 - 2000 μ m
 Applied normal stress of 1200 kPa used for all samples

- Increasing the sample height by a factor of approximately 1.6 results in a marginal decrease in the peak angle of internal friction for both the Killa and Table Bay samples;
- The amount of crushing under application of normal stress alone is influenced negligibly by an increase in sample height;
- The measured amount of crushing after application of shear stress is influenced by the sample height for both the Killa and Table Bay shell. Increasing sample heights result in a relatively large decrease of the crushing coefficients.

It was observed that for the sheared samples, particle degradation was severest on the shear plane, with most large particles reduced to a fine powder. The development of a zone of severest crushing in proximity to the shear plane could explain the greater sensitivity of crushing coefficients to variation in overall sample dimension. The extent of such a zone would depend, amongst other factors, on the average particle size and the overall dimensions of the sample. As the sample height is increased, a proportionally larger percentage of the particles will lie outside the zone of severest crushing. Since particle size distributions are calculated on a basis of "percent finer by mass" of the total sample, it follows that sample dimension will have a greater influence on the crushing coefficients as the distribution of stress within the sample becomes less uniform (i.e. shear stress vs. 1-D consolidation). This hypothesis is further supported by the fact that for both the Killa and Table Bay samples the total mass of fines ($< 425 \mu\text{m}$) remained virtually constant, irrespective of sample height.

From Table 11.8, it can also be seen that an increase in the initial unconsolidated sample height (h_u) leads to an increased consolidation settlement (Δh). The normalised settlement ($\Delta h/h_u$) however appears to

decrease as the sample height is increased. This implies that crushing under application of normal stress alone could also be affected by the initial sample height, although insufficient tests have been performed to verify this.

(vi) Effect of normal and shear stress on crushability

Crushing coefficients for the 1180 - 200 μm Killa and Table Bay samples after application of the normal stress alone and shearing under the same normal stress are listed in Table 11.5. The following observations can be made :

- For both shell types, the amount of crushing under normal stress alone is considerably less than the amount of crushing after shear;
- At a particular level of applied normal stress (e.g. 1200 kPa), the ratio $C_k(\text{normal stress})/C_k(\text{shear stress})$ is not the same for different shell types. For the Killa shell the ratio is less than 1/6 while for the Table Bay shell it is approximately 1/3. It appears that the difference in the amount of crushing under normal and shear stress decreases as the crushing susceptibility of the particles increases;
- For a given shell type and size, the ratio $C_k(\text{normal stress})/C_k(\text{shear stress})$ does not remain constant as the level of applied stress is varied.

This strong dependance of crushability on particle structure and the level and type of stress implies that crushing coefficients measured in different laboratories under non-standard conditions, as is presently the practice, has little or no purpose for classification and description.

11.9 PRACTICAL IMPLICATIONS OF RESULTS AND RECOMMENDATIONS FOR FURTHER WORK

- (a) A classification of carbonate soils for engineering purposes based on the relation between crushability and the change in friction angle determined in any shear test (e.g. Datta *et al.*, 1979) does not appear to be satisfactory since shear tests are not "simple" tests. (Datta's test requires triaxial apparatus which, although available in many geotechnical laboratories, needs adaptation or may not be capable of providing large enough confining pressures. Consequently, their method is not suitable for routine testing but it might be useful for research).
- (b) Considering Fig. 11.3; there appears to be a clear relation between e_{\max} , D_{60} and the crushing coefficient C_k as defined by Marsal. The measurement of 2 parameters only, i.e. e_{\max} and D_{60} , may uniquely fix the position of the soil on such a "crushability chart", particularly if the value of e_{\max} is independent of particle size. This appears to be a reasonable assumption for sand-size particles of uniform grading distribution, as were used in the present investigation. Further research is required into the effect of non-uniform size distributions on the relation between e_{\max} , D_{60} and C_k . (For silts, the value of e_{\max} will depend on average particle size (Section 4.6.5 (iv))).
- (c) For sand-size particles, both e_{\max} and D_{60} can be determined using simple, inexpensive tests which are available in most geotechnical testing laboratories. The simplicity requirement of an index test for classification is satisfied.

- (d) If a "standardised crushing" chart can be developed, measurement of e_{\max} and D_{60} may provide an additional classification parameter (C_k) which could be quantitatively determined, with little experimental variability. Such a chart could be constructed by performing a large number of tests in a single laboratory, using one apparatus and a wide variety of soils with differing e_{\max} and D_{60} . The use of arbitrary but equal sample size and stress paths will avoid variability in the numerical value of C_k which arise from these factors (Section 11.8 (v) and (vi)).
- (e) The tests undertaken suggest that although crushability is sensitive to particle size, "compressibility" seems to be relatively unaffected. However, a good correlation was found between e_{\max} and crushability and e_{\max} and "compressibility". Nauroy and Le Tirant (1985) found a good correlation between pile load carrying capacity and an arbitrarily defined compressibility parameter (C_{PL}) measured in the triaxial test. This suggests that the value of e_{\max} might be additionally correlated with pile load capacity. However, additional research is required.
- (f) An investigation into the effect of intraparticle voids on the engineering properties would be purposeful.

12. CONCLUSIONS

- (a) The maximum obtainable void ratio (e_{\max}) has been shown to provide a useful index property suitable for classifying carbonate sands.
- (b) For carbonate sands of uniform grading distribution, e_{\max} correlated well with crushability and "compressibility", as measured in the direct shear testing apparatus.
- (c) The method proposed by Kolbuszewski (1948) for measuring e_{\max} is recommended as the standard for carbonate sands.
- (d) Average particle size appears to correlate with crushability and use of D_{60} is proposed as an index for carbonate sands and silts.
- (e) For carbonate silts, measurement of crushability is difficult and an alternative measurement such as the sample's compressibility would be more useful. A parameter based on void ratio would provide an indication of the particle shape and, as a consequence, the relative compressibility and crushability. Because the method for determining e_{\max} for sands is not applicable to silts, the use of an alternative measure of silt void ratio should be investigated.
- (f) The preparation of a "standard crushability chart" (e_{\max} versus crushability) for different particle sizes and materials is recommended for further research. Similar charts (for sands and silts) based on compressibility would also be purposeful.

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Appendix A.1

Physical properties of carbonate minerals

<u>Property</u>	<u>Calcite</u>	<u>Aragonite</u>
Common crystal form	Rhombohedron	Prism
Cleavage	Rhombohedral perfect	Prismatic distinct
Hardness	3	3,5 - 4
Colour	Transparent, coloured	Transparent coloured
Streak	white	white
Specific gravity	2,71	2,94
Special property	Dilute HCl causes effervescence	Dilute HCl causes effervescence

APPENDIX A.2.

METHODS FOR DETERMINING LIMITING VOID RATIOS

Two approaches are generally used to determine the relative densities for all soils (Tavenas and La Rochelle, 1972) :

- (a) Define and measure the real maximum and minimum densities of a given material by experimenting with several techniques. In this case relative densities will be measured which will depend essentially on the testing technique and will therefore vary from one experiment to the other. Kolbuszewski (1948) investigated variations due to different techniques and showed that the method used has a significant influence on the value of the measured characteristics. The maximum and minimum densities for different materials were not always obtained using the same method. Thus, the main objective of relative density, to bring the engineering properties of different soils together on a common basis, is not fulfilled.
- (b) Adopt arbitrarily one technique to reproduce some particular states of compactness which will be called "limiting", even if they are not necessarily the maximum or minimum in absolute terms. In the USA, a recognised Standard (ASTM-D-208-64T) exists. Tavenas and La Rochelle evaluated the accuracy of this ASTM Standard for a large series of tests conducted by the same technician. The resulting errors were small, but not negligible, and were shown to apply only to results yielded by the "standard technique", undertaken with a given equipment by a given technician. When these factors were changed, errors similar in magnitude to those resulting from different experimental methods resulted. The research of Tavenas and La Rochelle (non-carbonate soils) indicated that a variability of at least $\pm 10-22\%$ result from such relative density determinations.

Appendix A.3

Listing of "SHEARBOX" program.

```
10 ! SHEARBOX - USED FOR REDUCTION OF DIRECT SHEAR RESULTS
15 ! B. STERIANOS - 19 SEPTEMBER 1988
20 ! *****

30 CLEAR
35 DISP "SHEARBOX"
40 DISP "TEST NO";
50 INPUT T1$
60 REM DIMENSIONS
70 L=59.9
80 DISP "NO. POROUS STONES";
90 INPUT N
95 DISP "NO. GRID PLATES";
96 INPUT P
97 DISP "UNCONSOLIDATED X";
98 INPUT U1,U2
100 DISP "X1,X2,Y1,Y2";
110 INPUT X1,X2,Y1,Y2
120 REM Z=INST ZERO
130 Z=13.83
131 U=(U1+U2)/2
140 X=(X1+X2)/2
150 Y=(Y1+Y2)/2
159 H0=44.32-(Z-U)-N*6.48-P*2.19
160 H1=44.32-(Z-X)-N*6.48-P*2.19
170 H2=44.32-(Z-Y)-N*6.48-P*2.19
180 REM S0,S1,S2=SETTLEMENT
190 S0=X-U
191 S1=Y-X
192 S2=Y-U
200 REM MASSES
210 DISP "MASS CONTAINER ,g";
220 INPUT M5
230 DISP "MASS CON.+WET SAMP ,g";
240 INPUT M6
250 DISP "MASS CON.+DRY SAMP ,g";
260 INPUT M7
261 ! M8=WET MASS,M9=DRY MASS
270 M8=M6-M5
280 M9=M7-M5
290 REM VOLUMES
299 V0=L^2*H0/1000
300 V1=L^2*H1/1000
310 V2=L^2*H2/1000
320 DISP "SPECIFIC GRAVITY";
330 INPUT G
339 ! V5=SOLIDS VOLUME
340 V5=M9/G
350 DISP "NORMAL LOADS "
360 DISP "W1(DIRECT),W2(OFFSET)";
370 INPUT W1,W2
380 N0=39.795+2.734*W1+13.671*W2
390 REM SHEAR STRESSES
400 C=348.59907
410 Q=.002
420 DISP "MAX. DEFL,ULT. DEFL";
```

```

430 INPUT D1,D2
440 T1=C*Q*D1*1000/L^2
450 T2=C*Q*D2*1000/L^2
451 ! TANMAX=G1 TANRES=G2
452 G1=T1/N0
453 G2=T2/N0
460 REM MOIST CONT
470 W0=(M8-M9)*100/M9
480 REM DENSITIES,P0=INIT BULK,P1=DRY,P3=FIN BULK,P7=UNC BULK,P8=UNC DRY
481 P7=M8*9.81/V0
482 P8=100*P7/(100+W0)
490 P0=M8*9.81/V1
500 P1=100*P0/(100+W0)
510 P3=H1*P0/(H1-S1)
520 REM Voids ratio,E0=INIT,E2=FINAL,E7=UNC.
529 E7=(V0-V5)/V5
530 E0=(V1-V5)/V5
540 E2=(V2-V5)/V5
550 IMAGE DDD.DD,XX,20A
560 IMAGE DDD.DDD,X,20A
570 DISP USING 550 ; N0,"NORMAL STRESS"
580 DISP USING 550 ; T1,"MAX SHEAR STRESS"
590 DISP USING 550 ; T2,"ULT SHEAR STRESS"
591 DISP USING 550 ; G1,"TAN MAX"
592 DISP USING 550 ; G2,"TAN ULT"
600 DISP USING 550 ; W0,"MOISTURE CONTENT"
607 DISP USING 550 ; P7,"UNC.BULK WEIGHT"
608 DISP USING 550 ; P8,"UNC DRYWEIGHT"
610 DISP USING 550 ; P0,"BULK WEIGHT"
620 DISP USING 550 ; P1,"DRY WEIGHT"
630 DISP USING 550 ; P3,"FINAL BULK WEIGHT"
638 DISP USING 560 ; E7,"UNC.VOID RATIO"
640 DISP USING 560 ; E0,"INITIAL VOID RATIO"
650 DISP USING 560 ; E2,"FINAL VOID RATIO"
651 DISP USING 560 ; S0,"UNC.TO CONS SET"
652 DISP USING 560 ; S1,"FIN TO CON SET"
653 DISP USING 560 ; S2,"FIN TO UNC SET"
654 DISP USING 560 ; H0,"UNC.SAMLE HEIGHT"
655 DISP USING 560 ; H1,"CONS. SAMPLE HEIGHT"
656 DISP USING 560 ; H2,"FIN.SAMPLE HEIGHT"
660 PRINTER IS 701,80
670 ! *****
680 PRINT
690 PRINT
700 PRINT "TEST NO ";T1$
710 PRINT "NO. POROUS STONES=";N
760 PRINT "ASSUMED SPECIFIC GRAVITY";G
770 DISP "NORMAL STRESSES ;W1,W2";W1,W2
790 PRINT
800 PRINT
810 PRINT USING 550 ; N0,"NORMAL STRESS"
820 PRINT USING 550 ; T1,"MAX SHEAR STRESS"
830 PRINT USING 550 ; T2,"ULT SHEAR STRESS"
831 PRINT USING 550 ; G1,"TAN MAX"
832 PRINT USING 550 ; G2,"TAN ULT"
840 PRINT USING 550 ; W0,"MOISTURE CONTENT"
841 PRINT USING 550 ; P7,"UNC. BULK WEIGHT"
842 PRINT USING 550 ; P8,"UNC. DRY WEIGHT"
850 PRINT USING 550 ; P0,"BULK WEIGHT"

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860 PRINT USING 550 ; P1,"DRY WEIGHT"  
870 PRINT USING 550 ; P3,"FINAL BULK WEIGHT"  
879 PRINT USING 560 ; E7,"UNC. VOID RATIO"  
880 PRINT USING 560 ; E0,"CONS. VOID RATIO"  
890 PRINT USING 560 ; E2,"FINAL VOID RATIO"  
891 PRINT  
892 PRINT USING 560 ; S0,"UNC.TOCONS SET,mm"  
893 PRINT USING 560 ; S1,"FIN.TOCONS SET,mm"  
894 PRINT USING 560 ; S2,"FIN.TOUNC.SET,mm"  
895 PRINT USING 560 ; H0,"UNC.SAMPLE HEIGHT"  
896 PRINT USING 560 ; H1,"CONS.SAMPLE HEIGHT"  
897 PRINT USING 560 ; H2,"FIN.SAMPLE HEIGHT"  
900 PRINT  
910 PRINT  
920 PRINTER IS 1  
930 END
```

Appendix A.4

Postgraduate courses completed by the author

<u>Course No.</u>	<u>Course</u>	<u>Credits</u>
FE 512	Engineering Economy and Optimisation	5
CE 5C1	Aquatic Chemistry	5
CE 5C3	Advanced Aquatic Chemistry	5
CE 5C4	Wastewater Treatment I	5
CE 5C6	Low Cost Sanitation	5
CE 5D2	Coastal Hydraulics	5
CE 5D6	Coastal Engineering Practice	5
CE 5E5	Geotechnical Site Investigation	5
	TOTAL	40

22 FEB 1989