CHAPTER 2

PREVIOUS STUDY ON SHALE ROCKS

§2.1. INTRODUCTION

In this thesis a series of tests on intact and reconstituted Wianamatta group shales is reported. One of the purposes of this chapter is to present an overview of previous studies on the nature and classification of argillaceous rocks and to examine the general characteristics of shale material that can have consistencies that range from stiff clay to rock. Another purpose of this chapter is to investigate the engineering behaviour of argillaceous rock with particular emphasis on shale properties.

There is considerable confusion in the terminology used to describe shale materials. This review is mainly concerned with what will be called “clay shales”. These are defined as stiff shales with more than 50% clay particles by weight that are highly susceptible to significant deterioration as a result of interaction with water. They have been referred to in the literature as “stiff”, “fissile”, “intact”, “compacted”, or “brittle” shale as well as “soil-like shale”.

The nature of cementation, the index properties and the mechanical behaviour of clay shales are reviewed. Data from previous studies on shales including various laboratory
tests for determining the strength and stiffness of rock are considered. The main aim is to determine tests that are appropriate for the investigation of Bringelly shale.

The main features of the critical state concept and its application to soft rocks are presented and the suitability of the application of this concept to clay shales rocks is considered.

The post-depositional history of the Wianamatta group is discussed. The role that the geology has played in establishing characteristic features of the Bringelly shale, and insights that this can provide about the likely engineering response are also considered.

§2.2. BASIC GEOLOGICAL FEATURES IN THE SYDNEY BASIN

2.2.1 Basic geology of Wianamatta group

The Wianamatta group ranges in age from early to middle Triassic. No late Triassic sediments are known to have been deposited in the Sydney Basin (Bembrick et al., 1980). This agrees with Herbert, (1979) who suggested that Wianamatta group was deposited in the middle Triassic during a single overall regressive episode after subsidence of the Hawkesbury sandstone. Moreover, since Triassic times the surface of the Sydney basin has been above sea level and consequently any further deposition of sediments will have been terrestrial.

The Sydney metropolitan area is founded on three major rock units, the Wianamatta group, the Hawkesbury sandstone and the Narrabeen group. These three rock units are overlain locally by Quaternary / Tertiary alluvium. The Wianamatta group rocks (up to 304m thick), and their weathering products are of engineering importance as they form the foundations for most of the residential, and industrial districts to the west of the city of Sydney, as shown by the distribution of Wianamatta group rocks (Fig. 2.1). Both Bringelly and Ashfield shales represent important resources of brick clay in Sydney. However, the few remaining Ashfield shale pits have mostly been engulfed by suburban
development and have little remaining accessible reserves. In the western suburbs of Sydney, Bringelly shale is frequently encountered during construction.

The Wianamatta group is believed to have been deposited during a single overall regressive episode (Helby, 1973) and is an abundant geologic sequence in the Sydney basin. It is dominated by argillaceous rocks. Shale comprises the upper rock layer for the majority of suburban Sydney, covering a total area of approximately 1125 km². The residual soil layer is typically only a few metres thick. Two geologically distinct shale types are found that are known as Ashfield shale and Bringelly shale.

The geological group is composed of prodelta and delta front shale (Ashfield shale), barrier and barrier bar sandstone (Minchinbury sandstone), and an alluvial coastal plain sequence (Bringelly shale). According to Herbert (1979, 1980b) the continuous supply of sediment into the Sydney basin at the time of deposition caused the shoreline to build out seawards with a vertical sequence of deposits. These deposits upgrade from lacustrine to brackish or shallow marine deposits at the base (Ashfield shale), through a shoreline sand (Minchinbury sandstone) and finally into alluvial sediments (Bringelly shale). This interpretation was consistent with studies carried out by Chesnut (1983). No large-scale sedimentary breaks have been recorded by previous geological studies on the Wianamatta group.

The Bringelly shale was first described by Lovering (1954) then redefined by Herbert (1979). It is interpreted as a coastal alluvial plain sequence which grades up from a lagoonal-coastal marsh sequence at the base to increasingly more terrestrial, alluvial plain sediments towards the top of the formation. Lithologically, it comprises sequences that can be listed in order of decreasing volumetric significance as (1) claystone and siltstone (2) laminite (3) sandstone (4) coal and highly carbonaceous claystone. Good outcrop is uncommon in Bringelly shale due to soil cover developed in situ as a result of the ongoing weathering of the parent rock.
Figure 2.1  Distribution of Wianamatta group in the west of Sydney
(after Herbert, 1979)
2.2.2 Evolution of Sydney basin

The Sydney basin came into existence as a result of two major evolutional events. The first major event occurred during late Paleozoic while the second major event occurred during Late Permian to Middle Triassic period which represents the age of Wianamatta group. The latter event has resulted in earth movement particularly during the middle Permian (270 mya) and for the next 70 million years, Permian-Triassic sediments were subjected to periodic episodes of marine transgressions and regressions that alternately inundated and exposed the developing basin (Herbert, 1980). During this period, the basin received large amounts of sediment from both land-based and marine sources.

Sedimentation episodes are believed by some researchers to have largely ceased by the end of the Triassic (205 mya) still the subject of argument by many researchers (details will be given in section 2.2.3.2). This was followed by episodes of volcanism, weathering, soil formation, subsidence and uplift, all of which have resulted in the formation of the present topography. Moreover, the formation of the basin was previously thought to be the result of rifting (Branagan et al., 1976).

However, a recent interpretation of the structural history of the basin (Stewart et al., 1995) suggests that compression has played a more dominant role whereby many major depositional cycles may have been initiated during foreland loading. These cycles of deposition caused a compression that led to folding (indicated by the New England fold belt) and subsequently to the establishment of the northern Sydney Basin. Geologically and structurally, the study area lies within the major geological feature known as the Sydney basin. It is bounded in the east by the coastline and extends between Batemans Bay in the south, Port Stephens in the north and out to Illawara and Lithgow in the west (Fig. 2.2). The Basin is approximately 350 kilometres long and an average of 100 kilometres wide. The total onshore area of the basin is approximately 44,000 square kilometres with an offshore component of about 5000 square kilometres which extends to the edge of the continental shelf. Details of the geology of the Sydney Basin with
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Figure 2.2 Main structural features of Sydney basin
(after Branagan, 1983)

emphasis to Wianamatta group were given by Herbert, 1980; Herbert, 1979, and Lovering, et al., 1969).

§2.2.3 Depositional and post-depositional events of Wianamatta group

§2.2.3.1 Depositional events

The depositional environment has a significant influence on the evolution of fabric and structure of the material. This was evident from the sequential variations of Bringelly shale which were interpreted as a coastal alluvial plain sequence which grades up from a lagoonal i coastal marsh sequence at the base to be more alluvial at the top (Lovering,
1954). These deposits were described as extensive swamplands cut by meandering estuarine and alluvial channels (Macgregor, 1985). The flow of these channels has resulted in a deposition of uniform soil with very fine particles. This episode has occurred in a single major marine regression. Lagoon, levee, peat marsh, and flood plain deposits were the major features throughout this period.

This agrees with a description by Chesunt (1998) who suggested that the preserved Bringelly shale of fine silt / clay particles was deposited (200mya) during periods of rising sea level (Fig. 2.3) which he believed to be the last depositional stage in the history of the basin (210mya) during the Jurassic period (more details are given in the next section).

However, Helby (1973) held a different view as he suggested that the last stage of deposition that occurred during the mid-late Triassic has affected the sedimentary structure of Wianamatta group whereby major geological events such as diagenesis, tectonism, weathering, and erosion are believed to be the prime contributors to the variation in some physical properties such as porosity. During the formation of deposits, compression due to overburden stress may have reduced the porosity of these deposits.

2.2.3.2 Post-depositional events

The episodes of deposition that occurred in lake and swamp environments have been followed by post-depositional events that have led to changes in structure, lithology, and mineralogy. In the Wianamatta group, most of the chemical and mineralogical changes are believed to have taken place at the sediment-water interface. The chemical changes for instance have had an impact on the chemistry of the depositional environment of this geological group and subsequently on the physio-lithological properties of its shale formations. This is evident from the presence of fossil rootlets (Retallack,1980) and a mottled texture of the light-grey claystone of Bringelly shale where the leaching of iron and calcium is evident. The carbonaceous type of this claystone was probably deposited in a swamp environment, while the non-carbonaceous type was probably deposited in...
flood basin lakes that were too deep to support standing plants (Herbert, 1991). Thick laminite at the base of Bringelly shale was probably deposited in a coastal lagoon behind the Minchinbury sandstone beach and barrier bar system while claystone-siltstone sequences of a uniform mid-grey colour are probably of lacustrine origin.

Mineralogically, changes in the chemistry of the depositional environment can also have a great impact on the silicate minerals which are the main constituents of clay. They
become unstable under the influence of weathering processes that can cause a
dissociation where a proportion of silicate minerals are dissolved incongruently, leaving
behind sheet silicates and iron oxides. During weathering, rock deteriorates back to clay,
some leaching occurs, roots and worms increase the porosity, moisture is taken back into
the clay structure and the mass per volume of the rock (density) decreases. Structural
features such as bedding, partings, and joints can be sufficient to initiate different
processes of weathering. These processes are carried out by various agents that are
capable of creating a gentle rounded topography (except where thick sandstone units-old
beach or sand dunes are present).

The originally deposited soil of Bringelly shale that was described as uniform with very
fine particles and low void ratio would have decreased porosity even more due to the
subsequent lithification (by compression). Subsequently, the ongoing process of erosion
of overlying material leads to a state of unloading. This state allowed discontinuities in
the rock masses such as bedding planes, partings and joints to open up and to enhance
weathering processes, particularly chemical ones. Chemical weathering is more effective
in environments where lithification of rocks is not complete. This condition will have
assisted in further reduction to some rock minerals such as Fe, Ca, Na, Mg, and some
silicate groups.

This post-deposition geological history has left remarkable features that distinguish
Bringelly shale, and have made it easy for engineers and scientists to identify them.
These features and their relation to mode of deposition can be summarized in the
following points:

- void ratio and porosity is very low due to the dominancy of fine material
  constituting the rock.

- moisture content is very low, due to depth of burial and associated compression
  which has caused an expulsion of water and reduction in void ratios

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fossil rootlets, worm casts, siderite nodules and a faintly mottled texture are present. These features might have led to an increase in the void ratio prior to the process of soil lithification.

Beds of carbonaceous claystone-siltstone in the upper part of the Bringelly shale show evidence of plant debris and / or fossil roots. This is an indication that the environment of deposition consisted of flood basins and lakes which were reasonably shallow and able to support standing vegetation.

Bedding planes and partings which distinguish Bringelly shale are probably due to unloading caused by erosion, which has in turn allowed chemical weathering to take place. These are evident from the presence of mica sheets that are often found between two distinctive layers i.e. claystone and siltstone.

Erosion of an upper-most layer of the Wianamatta group is suggested by the significant difference in the thickness of Bringelly shale at the western side and that at the eastern side of the formation. On the western side, the preserved sediment is 257m at the Razorback range (Camden-Picton district) compared to 60m thick at the eastern side of the formation (Wright, 1970). The erosion processes affecting the Wianamatta group have been the focus of investigation by many researchers since the early seventies. These studies were aimed at finding a connection between the missing thickness of sediments from the post-Triassic period and the formation of the passive eastern margin in the floor of the Tasman sea.

From the engineering perspective, knowledge of the geological processes would enable the depth of sediment to be determined and allow the role of sedimentation compaction in controlling the engineering properties to be assessed. Limited engineering investigations have aimed to relate the post-depositional events that followed the formation of the Permo-Triassic deposits to the characterization of Wianamatta shale properties such as cementation, mineralogy, stiffness, and low porosity. Prior to the current study these investigations were largely confined to Ashfield shale (e.g. Ghafoori, 1994).
The outcomes of the geological researches have led to a wide range of published figures for the thickness of the deposits and the associated geological conditions. For brevity, the results of the studies including the estimated thickness and techniques used in the investigation are summarised in Table 2.1.

Table 2.1 Estimated cover thickness on the top of the Wianamatta group rocks

<table>
<thead>
<tr>
<th>Source</th>
<th>Estimated Thickness (km)</th>
<th>Techniques</th>
</tr>
</thead>
<tbody>
<tr>
<td>Helby and Morgan (1979)</td>
<td>0.5</td>
<td>Spore assemblage</td>
</tr>
<tr>
<td>Middleton &amp; Bennett, 1980</td>
<td>1-2</td>
<td>Vitrinite reflectance</td>
</tr>
<tr>
<td>Crawford et al., 1980</td>
<td>1.4</td>
<td>Erosion of Diatreme</td>
</tr>
<tr>
<td>Falvey &amp; Middleton, 1981</td>
<td>0.6</td>
<td>Subsidence mechanisms</td>
</tr>
<tr>
<td>Middleton &amp; Schmidt, 1982</td>
<td>&gt; 1.0</td>
<td>Magnetic overprinting</td>
</tr>
<tr>
<td>Branagan, 1983b</td>
<td>0.65</td>
<td>Volume of deposits</td>
</tr>
<tr>
<td>Faiz &amp; Hutton, 1993</td>
<td>1-2.4</td>
<td>WinBury thermal modelling</td>
</tr>
<tr>
<td>Stewart &amp; Alder, 1995</td>
<td>1-4</td>
<td>Surface maturity data</td>
</tr>
<tr>
<td>Chesnut, 1997</td>
<td>0.1</td>
<td>?</td>
</tr>
<tr>
<td>Keene, 1991, Bai et al., 2001</td>
<td>1.5-2.1</td>
<td>Fluid inclusion &amp; stable isotope</td>
</tr>
</tbody>
</table>

Other researchers have based their investigation on the geological events without involvement in laboratory examinations. The outcomes of their investigations were in agreement with the concept of a missing thick cover, but disagreed about the quantification of the overall vanished layer (Shibaoka et al., 1973; Markham, 1980; Hamilton et al., 1984).

Geologists have a range of opinions because there is no agreement in answer to questions such as:
Based on regional comparisons and surface maturity data, recent studies by Stewart & Alder (1995) and Keene (2001) have suggested that the deposition of at least 1-2 km, and possibly up to 4 km of Jurassic to Cretaceous sedimentation including small bodies of magma are believed to have occurred before being eroded during tectonism associated with Tasman sea rifting and / or underplating of the eastern continental margin.

The products of erosion of these thick sediments that can be estimated as ~10500 km$^3$ are believed to be deposited on the Tasman sea floor. These sediments will have influenced the lithology and structure of the shale in the Wianamatta group due to the loading and unloading and stress relief. For instance, it is believed that the low void ratio of both shales (Ashfield and Bringelly) that were deposited in Late Triassic can be explained by deep burial. Ashfield shale, for instance has a range of porosity between 5% to 12% (Ghafoori, 1994). The existing sediment has a maximum depth of 257 m (Herbert, 1979) which implies a significant missing cover that was surely thicker than the currently existing layers.

In summary, most researchers agree on the reality of a now non-existent overlying material, but argue about the thickness of such cover. However, given geological uncertainty, it is difficult to estimate accurately the depth of burial that occurred during the Triassic age.
2.3 CLASSIFICATION AND GENERAL NATURE

2.3.1. Classification schemes

In general, argillaceous rocks such as shale, mudstone, claystone, siltstone, and clay shale are characterized by wide variations both in their engineering properties and composition. The common characteristics of this group of rocks are that all members are fine-grained and composed predominantly of clay and silt sized materials.

The term shale has been used by some authors for all argillaceous rocks, including claystone, siltstone and mudstone (Ingram, 1953; Krumbein et al., 1963). Others have specified the large group as the mudstone group and classified shale as a member of this group (Twenhofel, 1937; Muller, 1967). Terzaghi (1946) had a different opinion in defining shale. He claimed that the material should be called shale when it displayed a clear ring upon striking by a hammer, and showed no change in volume when it was immersed in water.

Many classifications used for argillaceous rocks are geological and depend on such properties as quartz content, grain size, colour, and the degree of compaction. Although these provide important information regarding the geological history of these materials, such classifications can be misleading when concerned with engineering behaviour. This is particularly evident when evaluating the behaviour of clay shales.

The general characteristics of clay shales include (1) highly overconsolidated, (2) commonly small scale fissured, (3) strong diagenetic bonding, (4) tendency to slake when rewetted after drying, (5) high swelling pressure in the presence of water, and (6) significant disintegration as a result of interaction with water.

Beyond this general description of clay shales, the classification of these materials has become complicated and confusing. Numerous classification schemes for argillaceous materials have been proposed, and have reviewed by Shamburger, Patrick, and Cutten.

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(1975), Deen (1981), and others. A summary of these reviews concentrating on issues relevant to the present study is given below.

2.3.1.1. Geological classification of clay shale

The major objective of geological classifications is the determination of the geological history of deposits. Initially classification (Wentworth, 1922) was based primarily on grain size and arbitrarily set the boundary between argillaceous material and the remaining sedimentary rocks. Ingram (1953) took the classification one step further, he subdivided all clayey materials based on percentages of silt and clay components, and on their breaking characteristics. Ingram used the term fissility which is the fine scale fracturing in the shale surface to distinguish shale from stone, while the prefixes řclayô, řsiltô, or řmudô are derived from the relative percentages of the grain size components. Thereafter, such terms as claystone, siltstone, and clay shale began to be used in the literature.

In an attempt to distinguish between compacted and cemented shale, Philbrick (1950) performed a simple weathering test that was based on five cycles of drying and wetting. He suggested that the shales that reduced to grain sized particles be termed compacted shales and those that were unaffected be termed cemented shale. This approach followed earlier classification by Mead (1936) who classified shales according to their cementation into two broad groups, the first is compacted shales that have been consolidated under stress by the overlying sediment without intergranular cement, and the second is cemented shales that could have a cementing agent (calcareous, siliceous, or ferruginous) or a bonding material formed by recrystallisation of clay minerals.

A similar division by Underwood (1967) introduced new terms, “soil-likeô shale for compacted shale and řrock-likeô shale or bonded shale for cemented shale. Although the classification was aimed to serve geological purposes (Fig. 2.4), the division between these two groups is poorly defined. This shortcoming motivated Folk (1968) to clarify Ingramôs scheme by refining řmudstoneô as argillaceous materials with sub-equal
amounts of clay and silt. This was further modified by Gamble (1972) who introduced a classification scheme that was essentially the same as Ingram’s except that the terms clay shale and silt shale have been changed into “clayey shale” and “silty shale”. Although this change may seem insignificant, the term clayey shale does help to distinguish a clay rich shale from a clay shale which, in engineering usage, implies certain engineering behaviour and not simply a fissile rock which is rich in clay content.

Based on stress history, Bjerrum (1967b) classified shales as overconsolidated plastic clays with strongly developed diagenetic bonds and clay-shales as overconsolidated
plastic clay with poorly developed diagenetic bonds. Similarly, Skempton and Hutchinson (1969) attempted to crudely relate geological origin of materials to their potential engineering behaviour. However, the usefulness of their scheme for purposes other than for providing a general understanding of possible relationships is quite limited.

Although these geological classification schemes can provide some useful information for engineers, they are generally inadequate for evaluating potential engineering behaviour of clay shale. Nevertheless, the above review indicates the use of the term "clay shale" in the geological sense to generally describe a fissile rock, rich in clay-sized components. However, the use of the term clay shale does not carry the same meaning when it is used in the engineering literature.

2.3.1.2. Engineering classification of clay shale

The basic purpose of an engineering classification is to provide terms that aid the user in distinguishing materials which have similar engineering properties. The more recent classification schemes for argillaceous materials have attempted to account for their potential engineering behaviour. However, classification of argillaceous material for engineering purposes has been particularly difficult. The difficulties arise from the transitional nature of some of these materials. This transitional nature creates confusion among many geotechnical engineers who are accustomed to viewing a material as either a rock or a soil, but not as a material that can have properties of both. An early engineering classification was proposed by Terzaghi (1936) that divided clays based on stiffness and the presence or absence of fissures into three major terms; soft clays free from fissures, stiff clay free from fissures, and stiff fissured clay. Bjerrum (1967) adopted a different approach, he proposed an overlapping three-fold classification, based on bond strength and extending up to shale materials. In his classification, these descriptive terms were followed: (a) overconsolidated clays with weak or no bonds, (b) clay shales i.e. overconsolidated clays with developed diagenetic bonds, and (c) shale i.e. overconsolidated clays with strongly defined diagenetic bonds. The two classifications have significant, but poorly distinguished overlap between them creating some confusion...
of terms. Further confusion has developed from the use of the British Standard Institute classification, which uses similar terms based on consistency or strength (Table 2.2).

Table 2.2  British Standard Institute classification (1957)

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Field indication</th>
<th>Strength $(q_u)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very stiff</td>
<td>Brittle or very tough</td>
<td>$&gt;150 \text{kN/m}^2$</td>
</tr>
<tr>
<td>Stiff</td>
<td>Cannot be molded in fingers</td>
<td>75–150</td>
</tr>
<tr>
<td>Firm</td>
<td>Molded in fingers by firm pressure</td>
<td>40–75</td>
</tr>
<tr>
<td>Soft</td>
<td>Easily molded in fingers</td>
<td>20–40</td>
</tr>
<tr>
<td>Very soft</td>
<td>Extrudes between fingers</td>
<td>&lt;20 kN/m$^2$</td>
</tr>
</tbody>
</table>

These classifications caused some ambiguities particularly when using terms such as \(\text{\textit{over-consolidated}}\) (Johnson, 1969; Fleming et al, 1970), and \(\text{\textit{stiff, fissured clay}}\) (Chandler, 1970) to indicate weakly bonded shale. This inconsistency in terminology has been most pronounced for the argillaceous materials that are transitional between normally consolidated clays and intact shales. Attempts were made by some investigators (Mead, 1936; Philbrick, 1950) to account for the potential changes in material behaviour with time. The influence of durability was considered and the term \(\text{\textit{slaking}}\) is introduced in their classification schemes. Based on correlations of material properties, such as moisture content, liquid limit, dry density, etc., Gamble (1971) carried out extensive investigation on the durability of varieties of shale, he strongly recommended that these materials could best be classified on the basis of the relationship between a two cycle slake durability index and their plastic index. Gamble suggested that more work was needed in order to correlate laboratory results with field behaviour, but no attempts were made to connect between his classification scheme and the pre-established terminology.

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Based on the realization of the importance of shale deterioration, another classification was proposed by Deo (1972) that classified argillaceous materials according to their susceptibility to deterioration rather than the initial state of the material. Three tests, all of which measure shale durability (i.e. slaking, slake durability, and sulfate soundness), were performed on various shales from Paleozoic deposits in Indiana. Using indices derived from these three tests, Deo categorized shale deposits into soil-like shale, two types of intermediate shale, and rock-like shale. A combination of earlier classification schemes based on initial properties and classification schemes based on durability was first attempted by Morgenstern and Eigenbrod (1974) who presented two classification schemes (Fig. 2.5), one based entirely on the slaking characteristics (i.e. the rate of slaking versus the amount of slaking), and a more significant scheme that included undrained shear strength, strength loss after softening, changes of water content after softening, and the time of softening. Although it was required that the scheme emphasize the influence of softening on strength and water content, the scheme first stipulated three potentially conflicting properties:

a) undrained shear strength,
b) the degree of strength loss after softening, and
c) the degree of changes in water content after softening.

These properties are given conditional values prior to dividing the argillaceous material into either soil or rock, and the classification is based only on these conditional values.

Only after this division, are slaking characteristics used to determine if any of the soil-like materials are clay shales. According to this classification, a shale that could be classified as rock-like according to its initial strength characteristics, could also be classified as soil-like based on its response to softening. According to this scheme, Italian clay shale, although rock-like in initial strength, slakes completely to a soft mud with only one cycle of the slake durability test (Belviso et al., 1977). Other engineering materials are classified according to their engineering properties that they presently exhibit. Yet, a "clay shale" is unique not in its present properties, but rather in its potential for significant deterioration.
of these properties as a result of interactions with water. None of the classification schemes to date have succeeded in recognising that. For instance, stiff clay, such as the

London clay; a clayey shale, such as the Pierre shale; or a well bonded shale such as Ashfield shale, are terms that define these materials according to their present engineering properties such as plasticity, slaking, and softening. However, based on the method of Morgenstern and Eigenbrod all of them regardless of the rate of deterioration can be further classified as nClay shaleò.

Figure 2.5 Two part classification scheme based on minimum 50% clay sized particles (after Morgenstern and Eigenbrod. 1974).
2.4  GENERAL CHARACTERISTICS OF CLAY SHALE

2.4.1  Index properties

Index properties such as water content, density, and porosity (void ratio), and specific gravity provide information that help our understanding of the behaviour of a material. These properties when applied to shale are influenced by factors such as degree of weathering, mineral components, texture and structure, and type of cementation if any.

2.4.1.1  Density and porosity

Density of shale is affected by the depth of burial and any defects and / or infillings involved. Density or unit weight of a rock is defined as its weight per volume. It may increase for the same rock type as the depth to the rock from the ground level increases. This is expected because of the increasing overburden. The density is also influenced by the mineral composition, porosity, joints and other open spaces present, even for the same rock type. In addition, an increase in density could be due to a decrease in open joints or cracks as a result of the high pressure caused by the overlying rocks. Organic content may also affect the density of shale if it involves part of its cementing agent. Tests on different shales have revealed that the bulk density of shales fall within the range 2.0 to 2.73 $t/m^3$ (Deen, 1981).

Knowledge about porosity is essential in the fields of geomechanics, geophysics, and petroleum engineering. Porosity is a rock property that affects the density, strength, and elasticity and it varies during the geological history of shale. Quantitative determination of this parameter is necessary to characterise models for compaction and deformation of shale (Olgaard et al., 1997).
The porosity-depth relationship is an important factor in studying compaction and burial depth. A porosity of 9 to 10% for shale under an overburden of 1800 m was reported by Hedberg (1935). These values were compared to the average porosity of the near surface silt and clay that ranged between 30 to 80%. This significant reduction in porosity was a result of the substantial compaction and burial depth. A similar study was undertaken on Canadian shale by Katsube and Williamson (1994b) who reported a reduction in the porosity of the shale from 30% at a depth of 1000m to 5% as the depth of burial increased to 2500m. Other factors such as erosion, stress, weathering, and lithification can play an important role in changing porosity.

Many researchers have reported the effect of porosity on the mechanical properties of rocks, but few have investigated the effect of stress on porosity. Prasad (2003) studied the behaviour of clay rich sediments from deposition to burial and lithification. She suggested that the time at which cementation occurs could significantly influence porosity. With early cementation, inter-particle voids are locked and further porosity decreases is retarded. Conversely, high stresses or compaction result in low porosity and hence no cement or late infilling.

Porosity and permeability of clay shale is very much influenced by lithification which is a result of reduction of voids, reorientation of particles and cementation. Reduction of permeability in a ductile shale was also investigated by Evans et al., (1990) who considered the decrease in permeability as a result of compactive deformation. Shea and Kronenberg (1993) reported the influence of compaction of clay shale in weakening the basal planes of the phyllosilicates. This influence may cause preferred orientations of the phyllosilicates and eventually to the creation of micro-defects that may contribute to an increase in the porosity of the material. These defects were investigated by Olgaard (1997) who described them as micro-cracks that have been formed as a result of bending the weakened phyllosilicates around detrital grains. This structural change is primarily due to the stress acting on the clay shale.
2.4.1.2 Water content

It has been well established that the moisture content of a shale can have significant effects on its physical and mechanical properties. Water contents of shale have been reported which vary from less than 5% to as high as 35% (Banks, 1971). This variation in water content has a marked influence on the mechanical behaviour, affecting both strength and deformation properties (Lashkaripour, 1999). This concept was investigated by Van Eeckhout (1976) who also studied the effect of water content on the strength of argillaceous rocks with different degrees of saturation. A significant reduction in the strength of clay shale due to an increase in water content from dry condition to saturated condition was found. Van Eeckhout’s findings were further investigated by Hsu and Nelson (1993) who reported a strong correlation between compressive strength and water content for clay shales of North America. Based on their analyses of considerable data for various clay shales, a correlation between the unconfined compressive strength and moisture content was demonstrated. They also reported a natural water content of 20% as a critical value above which the influence of moisture content becomes less significant.

The influence of the water content on the strength of shale was studied earlier by Salustowicz (1965) who found that the presence of moisture may decrease the strength by as much as 60%. Colback and Wiid (1965) attributed the reduction of strength to the degree of saturation of the shale. This was observed when they carried out uniaxial compression tests on shale specimens under various relative humidities and observed a decrease in compressive strength with an increase in the moisture content. Colback and Wiid found that the compressive strength of the shale under saturated conditions was about 50% of that under dry conditions (Figure 2.6). The figure shows a small change in water content of about 1% from saturated to dry condition. Studies carried out by A.G.I. (1971) on an Italian clay shale and on clay shale from Iran (Lashkaripour & Ajallosian, 2000) showed that a reduction of about 90% in the unconfined compressive strength can result from an increase in the natural moisture content from 0.05% to 4%. However, smaller values of reduction in strength due to increase in moisture content were reported by other researchers (Steiger & Leung, 1990; Ghafoori, 1995).
In order to show the influence of moisture content on strength, data from different clay shales were compiled (Table 2.3) and are shown in Figure 2.7. A correlation between natural water content and the strength of the shale was investigated further by Ghafoori (1995) who carried out a series of tests on fully saturated samples of Ashfield shale with geological descriptions ranging from intact to highly weathered. For these samples the moisture contents were in the range from 1.12% to 8.7% with a mean value of 3.85 and a standard deviation of 1.8%. The data indicates an inverse relationship between the UCS and the moisture content (Fig.2.8). This relationship is best described by the following equation:

\[
UCS = 600p_y \exp^{-0.415m_w}
\]  

(2.1)
Table 2.3 Influence of water content on UCS of clay shale from different regional locations (after Lashkari Pour et al., 2000; A.G.I, 1971; Ghafoori, 1995)

<table>
<thead>
<tr>
<th>No.</th>
<th>Australian clay shale</th>
<th>Italian clay shale</th>
<th>Iranian clay shale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W (%)</td>
<td>UCS (MPa)</td>
<td>W (%)</td>
</tr>
<tr>
<td>1</td>
<td>1.05</td>
<td>33.50</td>
<td>5.80</td>
</tr>
<tr>
<td>2</td>
<td>2.58</td>
<td>18.52</td>
<td>6.32</td>
</tr>
<tr>
<td>3</td>
<td>3.25</td>
<td>13.71</td>
<td>7.05</td>
</tr>
<tr>
<td>4</td>
<td>4.30</td>
<td>9.05</td>
<td>8.20</td>
</tr>
<tr>
<td>5</td>
<td>5.05</td>
<td>6.53</td>
<td>10.05</td>
</tr>
<tr>
<td>6</td>
<td>6.51</td>
<td>3.27</td>
<td>12.50</td>
</tr>
<tr>
<td>7</td>
<td>7.53</td>
<td>2.61</td>
<td>15.75</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>16.30</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.7 Visual presentation of data from table 2.3
in which

\[ m_c = \text{moisture content (\%)} \], and

\[ p_a = \text{atmospheric pressure (0.1} \text{MPa}) \]

It is evident from Figs 2.7 and 2.8 that \( UCS \) decreases significantly as moisture content increases and an exponential equation of the form of equation 2.1 would fit the data of many shales. However, there are several factors influencing the constants that include: mineralogy, microstructure, porosity, degree of saturation, and degree of cementation. The extent to which these factors influence relationship between \( UCS \) and moisture content is poorly understood.

2.4.1.3 Hydraulic conductivity

The intrinsic permeability \( (K) \) depends only on the porous media characteristics. In general it relates to the rate at which fluid moves through soil. However, since most
theories developed for water seepage through porous media are based on Darcy’s law, it is more common to use hydraulic conductivity \( (k) \) as the parameter describing the rate of water movement. Hydraulic conductivity depends on the properties of the fluid, degree of saturation, and the porous medium. Knowledge of this property is important in providing useful solutions to a large number of engineering problems in soils and rocks (Suzuki, 1982). For soils, it depends on soil texture, soil structure, the presence of compacted or dense soil horizons, and also to the size and distribution of voids in the soil (Charman and Murphy, 2000).

For rocks, it may also vary over several orders of magnitude based on mineral composition, grain size distribution, and the pore space distribution (Russel et al., 1996). For clay shale, various authors have investigated the relations between permeability, void ratio, effective stress, and the extent and frequency of any laminations. Since shale is anisotropic, it is not unreasonable to expect the permeability to be anisotropic, with a larger value parallel to the laminations. Katsube et al. (1991) reported that shale seems to approach a maximum state of compaction at a depth ranging from 2.4 to 3.2 km from the ground surface and that this is known as a critical depth of burial \((CDB)\). In their report, they claimed that the \( CDB \) is a transitional zone dividing the burial process into mechanical (above) and diagenesis (below). Katsube and Williamson (1994b) carried out tests on unconsolidated shale to investigate the influence of stress on permeability. Their test results revealed that the rate of hydraulic conductivity \( (k) \) decreases with depth and approaches a minimum value of \( 10^{120} m/s \) at an effective stress of about 50 MPa, which is equivalent to an overburden stress of about 3 km.

The low conductivity of shale has been used to explain the time delay before failure that often is experienced during the drilling of shale (Horsrud et al., 1998). Based on the initial state as well as the stress path of shale, drilling in low permeability shale may result in an immediate drop in pore pressure, it may take from hours to weeks before the pore pressure again reaches its initial value. The drop in pore pressure causes an increase in the effective stress and hence makes the formation more stable. Over time, the gradual
increase in pore pressure to reach equilibrium may result in reduction in the effective strength and eventually cause failure of the shale.

Shale conductivity values decrease, as is the case for most argillaceous materials, with applied pressure in the laboratory. Best (1995) reported that this variation is enhanced by the opening of microcracks, developed as a result of stress-release during sample removal, and their closure due to the confining stresses developed from compaction and depth of burial. In general, there is no agreement on whether the permeability-pressure relationship can be presented by a single mathematical expression. However, it is believed that it can be represented approximately by an exponential curve taking the form of:

\[ k = K_0 \exp(-\alpha P_e) \]  

(2.2)

where

- \( K_0 \) is the permeability at atmospheric pressure,
- \( P_e \) is the effective pressure, and
- \( \alpha \) is a constant.

The above equation was tested by Kwon et al. (2001a) who investigated the permeability of illite-rich shale from the Wilcox formation under different stresses. Their test results have shown that the equation would reasonably fit the data within a range of effective stresses from 3 MPa to 12 MPa.

In their studies on Wilcox shale, Kwon et al. (2001a) reported anisotropy in the conductivity at low effective stress, with values measured parallel to bedding 102 times greater than those measured normal to bedding. Chesnut (1983) suggested that permeability of intact shale in the Sydney Basin is very low, and that significant water flow is most likely only in the plane of laminations. Few laboratory data are available for Sydney basin shales, however Golder Associates (1979) measured \( k \) values of the Bringelly shale in the field. Following 48 hours for bore saturation, the measured values
ranged from $1 \times 10^{-7}$ to $6 \times 10^{-7}$ m/s. The relatively high conductivity was attributed to the presence of more permeable rock units within Bringelly shale and/or to the structural disintegration due to post-saturation swelling. Itakura (1999) managed to reduce the rate of the structural disintegration during saturation of Bringelly shale. He performed tests on three types of specimens during his studies on the advective transport of contaminants at the Castlereagh site in NSW. Itakura has reported a decreased conductivity of Bringelly shale to be of the order of $10^{-11}$ m/s to $10^{-12}$. His tests for conductivity also included clay specimens from the same site. Itakura used abbreviations to characterize different specimen types. His test results on clayey shale specimens (CBS) have shown that a rapid drop in the conductivity from $4.1 \times 10^{-7}$ to $8.5 \times 10^{-10}$ m/s and was also associated with a decrease in the void ratio from 0.16 to 0.11 respectively.

The trend of this relationship was also observed when specimens from sand shale (SBS), clayey shale (CBS), and reconstituted intact clay (RLC) were tested for conductivity measurements (Fig. 2.9). For example, the figure shows that the $k$ value for RLC1 is decreased from $1.4 \times 10^{-10}$ to $1.3 \times 10^{-11}$ m/s with a decrease in void ratio from 0.66 to 0.46. A drop in void ratio was also observed as the $k$ values of the sandy shale (SBS1) decreased from $4.1 \times 10^{-9}$ m/s to $8.4 \times 10^{-10}$ m/s. The test may confirm that a significant decrease in permeability can occur when specimen saturation is performed under high confining stress (>400 kPa), while dramatic increase in permeability could be a result of saturating the material under < 100 kPa. This agrees with (Urciuoli, 1994) who suggested that the influence of internal structure during unloading and/or the insufficient confining stresses to maintain the integrity of the material structure can affect the accuracy of measuring the laboratory permeability of a material.

2.4.2 Durability

The resistance of rock to short-term weathering is often estimated through a durability process called slaking. The term "slaking" describes an important process in engineering because it can cause rapid changes in strength and durability. The slaking process often results in dissolution of particles, creation of cracks, and flaking of surface layers (Santi
and Koncagul, 1996). Because of the physical interdependence between durability and slaking, durability of shale is often measured with slaking tests.

2.4.2.1 Assessment of durability

For shale, durability is considered as an index of its deterioration over time, particularly if used in an embankment or as fill (Okland and Lovell, 1982). Shale can also degrade to a certain extent with time. Problems in using shale as a construction material have occurred in numerous applications, including slopes, spillways, and unstable subgrades. In order to assess the durability of shales, Gamble (1971) used the slake durability index to establish an engineering geological classification scheme. This step was followed by Deo (1973) who also used slake durability test results in conjunction with other index test results to identify shale that was suitable for use in embankments. Moriwaki (1974) noted four modes of slaking:

![Figure 2.9 Relation between void ratio and hydraulic conductivity for the shale samples (after Itakura, 1999)]
(a) swelling, described as an increase in bulk volume without visible cracking or significant loss of material,

(b) body slaking, which appears to originate from internal processes and which rapidly traverses large portions of mass with no apparent deterioration between cracks,

(c) surface slaking, characterized by loss of mass due to "sloughing" of tiny flakes of grains from the entire surface with no apparent cracks in the underlying material, and

(d) dispersion, characterized by loss of mass resulting from the separation of clay-sized grains which go into suspension, rather than settling.

Clay shales are characteristically highly susceptible to slaking. It is possible that the process of slaking is closely related to swelling and softening of clay shale. However, the degree of softening and swelling is very much influenced by not only the clay contents, but more importantly by the type of clay species and their reactivity. In the absence of mineralogical changes, water content can be increased by (a) dilation during shear, (b) simple swelling related to elastic rebound following unloading, and (c) swelling and slaking related to the breaking of inter-particle bonds in response to wetting, or wetting and drying cycles in the absence of external load changes.

Shale can also degrade to a certain extent with time when it is removed or drying from its natural condition, this degradation results from water absorption upon unloading. This type of softening may lead to deterioration of the shale rock (Terzaghi, 1963). Depending on its physical and mineralogical properties, shale can be reduced from a rock like state to a soil-like material. Slaking and / or softening, are interrelated and believed to be important processes in the field of civil engineering due to their influence in causing rapid changes in strength and durability. These changes can lead to problems with erosion, slope stability, settlement, bearing capacity and drainage.
However, in order to account for such changes and variations in the engineering behavior of clay shale, it is important to understand the mechanisms by which the material properties of clay shale are altered. While many researchers have speculated on the causes of softening in clay shale, few have carried out extensive investigations on the fundamental mechanism of slaking.

Many studies have presented evidence (Nakano, 1970) that supports the hypothesis that some materials will not slake as long as the water content remains above a certain threshold, but if the water content is lowered below this threshold, slaking will occur during either drying or rewetting. Moriwaki (1974) disagreed with this hypothesis and concluded from his investigations on reconstituted shale that the dominant slaking mechanism is controlled by the clay mineralogy. Moriwaki further concluded that the susceptibility of any material to slaking would depend not only on the mineralogy, but also on the "physico-chemical characteristics", such as bonding, and the chemistry of the slaking fluid. However, it is worth mentioning that properties such as permeability and the presence of microcracks play an important role in determining the durability of shale.

Harper et al. (1979) and Richardson (1984) have supported Moriwaki’s approach and performed further tests to evaluate the rate of slaking in shale rocks. Test results showed that based on mineralogy, structure, water content, and porosity, durability could vary from very low to extremely high. Variations in durability could also be experienced as a result of temperature changes, and changes in humidity environment and degree of saturation (Grice 1968). However, Venter (1980) has claimed that the slake durability index of non-expandable shale is not effected by differences in temperature.

Some of these observations were further investigated for Wianamatta Group shales by Ghafoori (1995) who demonstrated the influence of durability on strength and natural moisture content of Ashfield shale. The test results revealed an inverse relationship between the durability of the rock and its natural water content, with durability and strength increasing with decreasing moisture content (Fig 2.10) and concluded that the natural moisture content was a good predictor of the durability of Ashfield shale.
of weathering was also studied by Ghafoori who claimed that it is to increase the clay and moisture contents and thereby reduce the strength and durabilities.

Figure 2.10  relationship between two cycles slake durability

And moisture content (after Ghafoori, 1995)

The slake durability test has played an important role in the development of various durability classifications (Gamble, 1971; Dick et al., 1994) and in the assessment of rock durability (Taylor and Spears, 1970; Olivier, 1980; Dick and Shakoor, 1992). Durability tests for index and design parameters for weak rocks are widely available. The most important and most commonly used durability test for index and design parameters is the slake durability index test. The main aim of this test is to evaluate the weathering resistance of shales. The test was first reported by Gamble (1971) and then developed by Franklin and Chandra (1972). They performed a wide range of the slake durability tests and devised a simple, but very useful classification for evaluating the weathering resistance of shales. The test procedures were recommended by the International Society for Rock Mechanics (ISRM, 1981) and standardized by the American Society for Testing and Materials (ASTM, 1990).

In the slake durability test developed by Franklin and Chandra (1972), ten representative lumps of shale each weighing 50 to 60 grams are oven dried and placed in a drum. After
10 minutes rotating in a partly immersed drum constructed of 2 mm mesh, the retained material in the drum is oven dried at 105°C for at least 6 hours and weighed. The cycle is repeated and the slake durability index is the dry weight percentage of material retained after the second cycle. Taylor (1988) performed slake durability tests on a wide range of clay-bearing rocks. He suggested that the two-cycle slake durability testing does not offer an acceptable indication of the durability of these rocks. Several studies have looked at increasing the number of slaking cycles (e.g. Moon and Beattie, 1995; Gokceoglu, 2000; Dhakal, 2002). Gokceoglu (2000) reported a significant increase in the amount of clay minerals passing from the drum after the third cycle, noting that repeated wetting and drying contributed to an increase in the amount of disaggregated clay minerals from the original shale sample. This was confirmed by Dhakal (2002) who performed a slaking test on Akita mudstone that was run for six cycles to ensure the passage of clay minerals from the drum. The slaking factor of Morgenstern and Eigenbrod (1974) is unique in that it is based on the one-dimensional free swell of a laterally-confined sample. In these tests, the change of height, and therefore the change of water content were measured as a function of wetting and drying cycles. Increased swelling is assumed to indicate progressive slaking within the specimen.

The slaking and compression softening tests were used by Morgenstern and Eigenbrod (1974) for identifying shale types. They performed the slaking tests on various clay shale specimens and other stiff clays. They also found a linear relationship between the maximum water content, obtained during the slaking test, and the liquid limit of the natural shale.

There is a lack of consistency among researchers on acceptable test procedures, and attempts to improve the slake test. Thus, although the slake durability test gives an indication of the susceptibility of a clay shale to slake and disintegrate, it does not provide a reliable quantitative measure.
A possible mechanism of slaking

Gokceoglu et al. (2000) suggested that hydration and chemical alteration of clay shale are closely related processes. He also claimed that responses to rock-slaking in shale with swelling clay minerals will vary based on the amount type of the constituent clay minerals. Increase of hydration and double layer repulsion force and negative pore pressure are the main slaking mechanisms in shale with significant amount of smectite. Internal microcracks in shale allow the entry of water carrying dissolved ions and lead into great expansion and destruction of the crystal lattice (Botts, 1998). Bjerrum (1967) suggested that the mechanism of slaking is a result of disruption of diagenetic bonds and the release of stored strain energy. This was further interpreted by Terzaghi and Peck (1967) who stated that the slaking mechanism could result from the compression of trapped air within the clay or shale mass, particularly in soils containing highly expansive clay minerals.

The degree of deterioration is believed to be based on the amount of clay minerals and their subsequent swelling effects, it can also vary according to the rock type (Varley, 1990). Considering the significant influence of durability on the unconfined compressive strength, there are very few publications that relate UCS to durability (e.g. Koncagul and Santi, 1999; Eigenbrod, 1972; Augenbaugh and Bruzewski, 1976). Huppert, (1988) reported that microstructure controls both strength and durability of shale and that high strength and durability are indicative of low porosity and a high degree of particle interlocking.

The influence of strength, weathering, clay content, and natural moisture content on durability of the Ashfield shale was examined by Ghafoori (1995) who demonstrated the effects of the moisture content on the durability and strength of the shale. It was evident from these examinations that increasing moisture content can reduce the strength and durability of the rock.
The methods that are commonly employed for investigating the process of slaking do not provide adequate information regarding the effects of slaking on the strength and stress-strain behaviour of clay shales in the field. Furthermore, it seems that the slake durability test does not necessarily measure the relative reduction of strength due to degradation of the material.

2.4.3 Strength

Determining the strength of soils for engineering applications is a highly complex issue. Different measures of soil strength are usually applicable in different applications. Knowledge of the strength is essential in the design and prediction of performance of a structure on or in the material encountered. The strength of clay shale can be influenced by density and bonding. However, determining the effects of these parameters on the strength of shale rocks appears to be difficult (e.g. Huang, 1994). For clay shale, because of the difficulty in sampling, storing, and their inherent anisotropy, e.g. laminations or micro and / or macro-cracks, strength tests are more complicated than for other common rocks. However, knowledge of the strength is essential for classification purposes and to assist with judgment about the suitability of these rocks for various construction purposes.

The aim of this section is to identify index tests that are useful for defining engineering properties, strength classification, and also for determining a suitable strength index for clay shale. The point load index test and unconfined compressive strength tests both provide procedures suitable for routine field use. The tests can be conducted quite rapidly and inexpensively on-site and / or in the laboratory and thus allow for quick on site monitoring of material strength.

2.4.3.1 The point load strength

The point load strength test provides a rapid and accurate strength index value that is useful for strength classification of shale rock. The test was first defined by Reichmuth
(1968) and a formula for the point load strength index was proposed by Broch and Franklin (1972). The test was then normalized by ISRM (1985), and later was updated by Brook (1993). The point load test is performed by loading a sample between two conical points having 60-degree conical points with a 5-mm point radius. Thus, a sufficient point load can be provided to fail hard rock samples using portable test apparatus. Results of point load tests are usually expressed in terms of the point load strength index $I_s$ given by the equation:

$$I_s = \frac{P}{D^2}$$

(2.3)

where

$I_s =$ the point load strength index  
$P =$ the applied load  
$D =$ the distance between the loading points

The point load strength test when first introduced, was mainly used to predict UCS (Broch and Franklin, 1972). Because $I_s$ is size dependent (Bieniawski, 1975), it should be correlated to a standard size i.e. 50-mm diameter core. Broch and Franklin (1972) introduced a size correction factor, $F$ as a function of core diameter for all rocks. From the experimental data, they proposed a size correction chart that can be used as a standard reference. This chart is reproduced in Figure 2.11 and can be used to determine $I_{50}$ in diametral point load index tests using the following expression:

$$I_{50} = F \times I_s$$

(2.4)

Where:

$F =$ a size correction factor

Chapter 2 – Previous Study on Shale Rocks
$I_s = \text{uncorrected point load strength}$

$$F = \left( \frac{D}{50} \right)^{0.45} \quad (2.5)$$

Germinger (1982) found that the size and shape effects in point load testing are independent of the degree of anisotropy and loading direction and that the point load strengths are a function of $D$ across the specimen and $D_e$ which is the equivalent core diameter. This was further investigated by Brook (1985) who used the term $D_e$ to compute $I_s$ for irregular specimens. The diameter is given by the following equation:

$$D_e^2 = \frac{4A}{\pi} \quad (2.6)$$

Where:

*Chapter 2 – Previous Study on Shale Rocks*
A = minimum cross-sectional area

He suggested that $D_c$ should be as close as possible to the site-size core diameter, especially where diametrical point load tests are also conducted. He also recommended that the test should be performed using a width-to-length ratio between 0.3 and 1.0 and that a minimum of ten specimens should be tested.

Point load tests may be performed on specimens with different sample geometries. Tests on core may be performed across the diameter of core samples (Bieniawski, 1975), or the core may be loaded axially. Alternatively tests may be performed on irregular lumps where no core is available. Smith (1997) has modified the test for rock that is nonuniform, has inclusions of weaker material, or is excessively brittle. In such cases, local crushing failure can occur without failing the entire sample. In order to ensure more reasonable results, Smith replaced the point loads by flat platens configured to pivot on the point load platens.

On average, unconfined compressive strength ($UCS$) is 20 to 25 times the point load strength ($I_{s(50)}$). However, it can vary over a much wider range (ISRM, 1985). Many correlations have been published in the literature from which strength parameters such as $UCS$ can be predicted. Published information on the point load test mainly relates to hard rock (Franklin, 1981; Richardson, 1985; Wiesner & Gillate, 1997), although some researchers have used the point load test on dredge material. Correlations with $UCS$ for weaker, saturated shales are not available (Smith, 1990).

Most rocks are to some extent anisotropic in their mechanical properties, even if they appear to contain no visible planes of weakness. Based on the direction of loading relative to that of weakness planes, strength of intact rock specimens can vary by a factor of ten or more (Broch and Franklin, 1972). The point load strength test was found to be a practical technique that is capable of measuring a strength anisotropy index, which is the point load strength ratio in two different directions. These directions are normally taken parallel and normal to any laminations.
Pells (1975) suggested that for certain rock materials the $UCS$ value that is predicted using diametral point load test and a conversion factors suggested by Broch and Franklin and / or Bieniawski is accurate enough for many engineering design purposes, particularly classification. However, Pells recommended that whenever point load test results are used to predict material strength under uniaxial or triaxial stress, at least some conventional uniaxial compression tests should be performed. Tsidzi (1991) stressed on the importance of the visual identification of the rock materials prior to the use of the conversion factor. He believed that the error associated with the prediction of the $UCS$ from the point load test can be reduced by more than 20%.

DGGT (1979) and Hassani et al. (1980) affirmed Pells approach and suggested that in practice, the use of an average conversion factor can be sufficient for determining $UCS$ values from the point load test results with no differentiation of rock type. However, this approach failed to adopt a common factor for determining the strength value, for example, DGGT used a conversion factor of 24 while Hassani used a factor of 29 between uniaxial compressive strength and diametral point load index. Based on limited point load test data gathered from Wianamatta shale and reported by Hadfield (1981), Won (1985) suggested correlation factors of 14 to 35 between $UCS$ and the axial point load strength and 22 to 35 between $UCS$ and the diametral point load strength. These correlations have been based on limited data and subject to the degree of substance defects and orientation of lamination and / or bedding. More data would be required to find reliable correlation factors between $UCS$ and axial and diametral point load strengths.

These are investigated in the present research. Ghafoori (1995) conducted an exclusive study on Ashfield shale and gave correlation factors of 24.1 and 38.2 for axial and diametral load strength respectively. The lower correlation values reported by Won may reflect the inclusion of Bringelly data with its different mineralogy and engineering properties within the undifferentiated results for Wianamatta shale. For the upper units of Bringelly shale, new data is required to find reliable correlation factors between $UCS$ and axial and diametral point load strengths.
axial and diametral point load strength. The difference between these correlation factors may reflect the different degree of anisotropy among clay shales and also reflects the difficulty of performing UCS tests as there is a wide spread practice of relying on point load index tests and relating these to UCS value on the basis of an empirical correlation. In the present research program, the correlation factor will be investigated.

2.4.3.2 Uniaxial strength

The standard uniaxial compressive strength $\sigma_c$ is one of the most important and commonly used properties of rocks. The uniaxial compressive strength has been widely used as a basis for classifications of rock substances versus rock mass for engineering purposes. For example, the primary intact rock property of interest for foundation design is unconfined compressive strength. The main purposes of the tests are to estimate the strength characteristics as well as determine the elastic parameters of rocks. Although it is known that strength of jointed rocks is generally less than unfractured portions of the rock mass, the unconfined compressive strength provides an upper limit of the rock mass strength and an index value for rock classification.

In shale, inherent weaknesses in the rock structure make collection and preparation of samples a difficult task. Among these weaknesses are bedding structures, microcracks, and swelling clay minerals. As a result of these, uniaxial compressive strength values determined from intact rock samples, which are usually the stronger and more easily prepared ones, are unlikely to be representative of a large rock mass.

The uniaxial compressive test is used to determine the compressive strength as well as deformation characteristics of a rock sample under a one dimensional stress state in the laboratory. The compressive strength $\sigma_c$ is the quotient of the uniaxial test failure load $F$ and the area of the sample $A$:

$$\sigma_c = \frac{F}{A}$$  \hspace{1cm} (2.7)
Researchers in the field of rock mechanics have found that uniaxial compressive strength decreases as length to diameter of a cylindrical specimen increases. The ISRM (1972) recommended a length to diameter ratio of 2.5 to 3 as a standard for UCS laboratory tests. However, a cylindrical specimen with a length to diameter of 2 to 2.5 is recommended by ASTM D2938 (1971). These ratios are applicable to a core size of NX (54 mm). To avoid end effects on the strength and deformation results, a cylindrical shape was chosen for the experiments reported in the present study. Because of the coring problems in Bringelly shale, an L/D ratio of 2 was used for uniaxial compression tests performed in this study.

Previous UCS data for shale in the Sydney metropolitan area were reported by many researchers (Burgess, 1977; Chesnut, 1983; Won, 1985, Ghafoori, 1995). In his investigation into the properties of Bringelly shale, Won (1985) reported UCS tests on core samples of Bringelly shale with uniaxial compressive strengths ranging from 5 to 80 MPa with a mean strength of 31 MPa. The data (Figure 2.12a) were based on a sample size of 65 tests and agrees with previously unpublished data provided by other agencies (Geological Survey, NSW, 1990; and RTA, NSW, 1990; and Won, 1985).

![Histogram of UCS data from Bringelly shale](after Won, 1985)
Uniaxial compressive strengths for Ashfield shale ranging from 5 to 75 MPa with a mean strength of 25 MPa were also reported by Won (1985). His data were based on 167 samples and were very close to test results reported by Ghafoori (1994) who carried out unconfined compressive tests on 150 samples of Ashfield shale. The range of strength reported for these samples was 1.8 MPa to 60.2 MPa with a mean strength of 20.1 MPa (Figure 2.12b). The wide range of strengths was related to the variations in porosity. As specimens were saturated good correlations with moisture content were observed. The range of porosity was related to weathering with increased weathering giving higher porosity and lower strength.

Figure 2.12b  Histogram of $UCS$ data from Ashfield shale  
(after Ghafoori,1995)

2.4.4 Mineralogy

The mineral composition of argillaceous rocks is one of their most important characteristics. Shales contain a wide range of minerals that are usually a mixture of clay-sized particles that are mainly clay minerals, and silt sized particles that are mainly quartz. In shale, clay minerals generally make up about 40-60% of the minerals in the rock. An understanding of clay minerals is important from an engineering point of view,
as some minerals expand significantly when exposed to water and this can have a significant influence on the mechanical behaviour of the material.

The type of clay mineral is a function of the source rocks, climate and diagenetic history. The main clay minerals found in shales are illite, kaolinite, smectite, and chlorite. Smectite and kaolinite for instance are more common in non-marine shales, while illite and chlorite are more common in marine shales (Brown et al., 1977). Different clay minerals have different effects on the engineering behaviour of soil and rocks. However, shales containing the smectite minerals are commonly more troublesome than others.

The clay mineral particles usually are less than 2 microns in diameter. Shales having higher clay mineral contents have smaller average grain sizes, while those with lower clay mineral contents generally contain coarser grained, silty particles. Attwell and Farmer (1976) reported that decreases in the ratio of clay mineral to quartz content result in reduced liquid and plastic limits, and may also result in increasing uniaxial compressive strength and uniaxial tensile strength of shale (Dusseault et al., 1986).

2.4.5 Microstructure

The microstructure and fabric of a material can influence its engineering behaviour. However, hence testing can quantify effects but can not be used to identify origin, it was suggested to examine material fabric by microscopy. For clay shale, mechanisms of deformation have generally been inferred from macroscopic and microscopic modes (e.g. Jordan et al., 1989). The mineralogical composition and fabric are the basic factors determining the properties of rock (Yumei et al., 1993). For instance, internal structural features such as micro-cracks in conjunction with water within the inter-layers of clays may control deformations and reduce the strength of shale.

The influence of micro-cracking was investigated by Ibanez et al. (1993) who performed triaxial compression experiment on illite rich shale at varying confining pressures. They reported that optical microscopy and transmission electron microscopy of deformed illite
rich shales showed that brittle micro-cracking and dilatant mechanisms were responsible for deformation. The deformation was accompanied by the development of fine scale bending in individual illite and chlorite platelets, and micro-crack bands were common within the shear zones regardless of the degree of orientation relative to the laminations. Previous studies of shale deformation by Bell et al. (1986); Christoffersen et al. (1990); and Mares et al. (1990) have also documented dilatant mechanisms of micro-cracking and fracture, while shear on clay platelets has been presumed to occur by frictional sliding on hydrated clay surfaces.

The effect of these deformation mechanisms is to cause a significant departure from strictly elastic behaviour. Yielding and an extended non-linear response can be observed when shale is loaded to failure. Such inelastic effects are known to result either from localised plastic yielding or from micro-cracking. The latter was examined by Vernik (1994) who attributed the origin of such micro-cracks in shale to five factors: (1) a differential elastic rebound of constituent minerals caused by overburden stress relief, (2) cooling induced thermal cracking, (3) overburden relief induced microhydraulic fracturing of overpressured fluid inclusion, (4) concentrated bottomhole stressing, and (5) microcracks in-situ. Even though, the first four factors could result from drilling and core recovery, Vernick suggested that all the factors could be present in situ. Furthermore, he described the mechanism of their formation as due to the relative weakness of the rock matrix. Rearrangement of clay platelets due to compaction (Fig. 2.13) may result in internal structural deformation causing a development of micro-cracks.

When pore pressure exceeds the normal total stress acting on the rock, microcracks are likely to develop. This was described by Meissner (1978) as a microhydraulic fracturing process and was suggested to be common in shale. It is developed during erosion when the formation pore pressure exceeds the total normal stress by a few megapascals.

These microcracks could determine the mode of failure in some shales. Yoshinaka et al. (1997) performed a series of compression tests on Kobe shale and reported that the increase of stress level can result in pore pressure build up due to the closure of pre-
existing microcracks inside the specimen. As new microcracks are initiated, such build up slows down until pore pressure reaches a maximum value. Thereafter, it decreases progressively to a negative value as crack propagation becomes predominant. This has

![Figure 2.13 Re-arrangement of clay platelets during the process of deposition (Bennet and Hulbert, 1986)](image)

resulted in a steep reduction in pore pressure at the end of unloading where specimens failed by bulging. These microstructural features are believed to be more common in clay shales where clay mineral content percentages exceed those of quartz content (e.g. Lee et al., 1993).

It is well known that microstructure of a soil is often controlled by geological processes. One of the most influential of these geological processes is cementation. The process of cementation often follows compaction and involves precipitation of mineral material in the pore spaces or pre-existing microcracks (Bell, 1993). This in turn leads to the formation of bonds between particles that subsequently resist the development of cracking. The cementation material may be derived from partial intrastratal solution of grains, or may be introduced into the pore spaces from an extraneous source by circulating water. Conversely, cement may be removed from a sedimentary rock by leaching. In a study carried out on Pierre shale, Olgaard et al. (1997) investigated the nature of cement in the shale. They reported that clay minerals appear to be cemented to
detrital grains such as quartz and calcite. They also explained the cementing mechanism as a wrapping process between the phyllosilicate framework and detrital grains. It is also believed that cementation processes in shale occur during deposition (Bennet & Hubert, 1986) where ions such as $K^+$, $Na^+$, $Ca^{++}$, may react directly with the clay in or at the bottom of depositing basin lakes and swamps. These ions are capable of occupying the open spaces that may have existed between the clay mineral platelets.

The process of growth of the new material is likely to take place at the points of contacts between the existing particles. This in turn creates a framework of interlocking crystals that may minimize or even eliminate pore space in which liquid may have been present as a result of the compaction process at a particular depositional environment. The cementation in natural geological materials can vary significantly because of the way it is formed. The cementation process may alter the mechanical properties such as density and porosity, in addition to their direct effects on the behaviour of the soil and rocks. For instance, deposition and then degradation of clay mineral cement in shale often contributes to damage of the shale microstructure.

Johnston et al. (1993) claimed that there are similarities in the mechanical responses between different types of soft rocks and the effect of cementation in soft rocks such as tuff, mudstone, and chalk is similar to its effect on clays, residual soils, and calcarenite. This suggests that the timing and nature of the cement are not critical. However, for shales where micro-cracking is common the observations of Johnston (1993) may not be applicable.

### 2.4.6 Aspects of swelling in clay shale

Shale, has long been known for its tendency to swell upon adsorption of water and on occasion lose its structural integrity due to swelling. Water sensitive shale formations cause costly and time consuming problems particularly in construction (Hatheway, 1991), mining (Steiger & Leung, 1988; Yoshida et al., 1997), and the petroleum industry (O'Brien and Chenevert 1973). In these industries, swelling of shale and its residual soil
has been focused on extensively. In the petroleum industry for example, about 75% of drilling operations' problems are related to shales (Dzialowski et al., 1993). It is believed that shale instability problems including swelling continue to dominate the drilling industry due to the lack of understanding of shale and drilling fluid interaction (Oart et al., 1994).

In the USA, swelling of residual soils is responsible for an annual economic cost that was estimated at around US$8 billion (Burland, 1995). In the petroleum drilling industry, the issue of wellbore instability has cost the industry substantial yearly expenditure (Osisanya, 1995). In China nearly fifty million US dollars per year is spent on maintenance in coal mines resulting from deformation due the swelling of shale rocks on wetting and stress relief (Yumei, et al., 1993). In Saudi Arabia, swelling of expansive shale has resulted in severe and widespread damage to residential buildings, sidewalks, and pavements (Al Mhaidib, 1999). Further examples of damage as a consequence of the swelling behaviour of the expansive shale from the middle region of Saudi Arabia can be seen in Figures 2.14a and 2.14b. Damage varies from minor cracking of pavements to structural cracking of buildings and displacement of footings. In the next section, the cause and influence of swelling on shale will be reviewed. In Australia, in the west of Melbourne, it was reported that 58% of the houses built on expansive shale and active clay have exhibited structural cracking problems (RAIA, 2002), and in the western

Figure 2.14a  Cracks across a main road pavement at town of Al-Ghatt (after Al-Mhaidib, 1998)
suburbs of Sydney, damage inflicted on residential construction by the influence of expansive clay shale (Bringelly shale), particularly when heavy rains following a drought season. This issue has not been recognized by the local authorities such as councils and local governments.

One consequence of swelling in clay shale is that standard water flush drilling techniques give very poor core recovery. This can be demonstrated by the difference in core recovery from the two shales of the Wianamatta group in Australia. Ashfield shale (Fig. 2.15a), that does not contain swelling clay minerals shows a good core recovery when conventional diamond drilling has been used. When using the same drilling technique, Bringelly shale (Fig. 2.15b) has given a very poor core recovery (Coffey Geosciences, 2004).

2.4.6.1 Previous investigation and preliminary findings on swelling

Extensive investigations have been made to study the causes and mechanism of swelling of some argillaceous rocks, particularly clay shale and marl. The results of these investigations were reviewed and assessed by researchers such as Balasubramanian (1972); Mitchell (1973, 1976); Lindner (1976); Huang et al. (1986) and others. In the
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following section, some of the important swelling mechanisms will be summarized so that their relevance to the swelling behaviour of clay shale can be assessed.
Shale swelling studies have been conducted using different approaches, equipment, and procedures (Mesri et al., 1994; Chenvert and Osisanya 1992; Mese 1995). Most of these tests have been aimed at evaluating the responses of intact shale when it is subjected to hydration and subsequent swelling. Common methods used to investigate this response are immersion tests and swelling pressure tests.

In conventional free swell tests, an intact specimen is immersed (under atmospheric pressure) in a certain solution so that reactivity between the rock and fluid can be evaluated. In addition to the volume change the tests can be assessed by means of a sample weight loss, rock surface hardness index changes, and sometimes by visual and tactile inspection. However, because no confining pressure is applied and there is no control of water content, the relevance of these tests to the rock in-situ is uncertain.

Swelling tests in which the rock is subject to confining and pore pressures, simulating the in-situ rock condition, have been reported by many authors. Murayama et al. (1966) were among those who attempted to measure the swelling pressure of clay shale under axially confined and laterally unconfined condition. They suggested that the swelling pressure is proportional to the swelling strain and that the main cause of failure in clay shale is non-uniform swelling caused by unequal distribution of water adsorption to clay platelets. Prior to the swelling tests, the above authors measured the suction of the clay shale samples. The results showed an inverse relationship between suction, measured in pF, and water content. Unfortunately, no mention of the degree of saturation of the samples was reported, and no relation between suction and the responses of clay shale to swelling were reported.

In an attempt to investigate the relationship between volumetric increase caused by immersing clay shale in water and their engineering properties, Sarman et al. (1990) conducted free swell tests on natural cubical samples. They related the increase in axial and volumetric strain to the swelling potential of the intact clay shale. A similar attempt was made by Lee et al. (1993) who investigated the effect of the time lag between sampling and testing on the swelling behaviour of intact shale. They performed free
swelling tests, using water, on samples up to 1,700 days after sampling. They claimed that the swelling strains measured in both horizontal and vertical directions (Fig. 2.16) were similar to those measured for intact samples tested immediately after coring. They suggested that the swelling potential is due to initial stresses due to sampling and not due to the changes in moisture content.

![Figure 2.16 Typical free swell test results of the Queenston shale sample (after Lee et al., 1993)](image)

The causes of the increase in volume have been debated by previous authors and it has been pointed out that many factors may contribute to the swell magnitude of clay shale. Factors such as type of clay minerals, stress history, properties of pore fluid, permeability, weathering, slaking, and exposure to free water were considered as the key issues that might cause the variations in the swell magnitude of different clay shales.

Bearing in mind that clay shale can deteriorate easily under various environmental changes, it is important to investigate these factors prior to testing (Lee & Klym, 1978; Madsen et al., 1985; Seedsman, 1986; Einstein & Bischoff, 1975; Lo, et al., 1990; and Yoshida et al., 1997).
Steiger (1993) investigated the hydration behaviour of Pierre shale in the triaxial apparatus. He measured the swelling pressures and strains using different test fluids. Sodium, potassium, calcium, and barium chloride solutions were used. Potassium chloride solution had much greater effects in minimizing the swelling potential of the rock. Kronenberg et al (1996) used the same approach in an effort to investigate the influence of fluid chemistry on the swelling potential of Wilcox shale. They suggested that using fluids such as \( \text{NaCl} \), \( \text{KCl} \), and \( \text{CaCl}_2 \) at different concentration could control the swelling potential of the shale. These studies suggest that the swelling and associated swelling pressure are a result of a redevelopment of double-layer water (Mesri et al., 1994). Although these studies showed why potassium is a major ingredient of inhibitive drilling fluid, but they did not pay much attention to the influence of such chemical solutions on the volumetric strain in the clay shale. In the oil industry, researchers have been interested in identifying an environmentally acceptable water based mud that could act as an alternative to oil-based muds (Simpson et al, 1995; Oart et al., 1996). Their results showed that oil based muds contain an emulsified water phase that can increase hydration, pore pressure, and hence weaken the rock. They suggested that a water soluble organic monomer can provide a mud that is capable of minimizing the swelling potential of the shale without weakening the rock.

Studies of the swelling potential in clay shale have found that absorption and adsorption can be used to explain the swell potential of shale (e.g. Sarman et al., 1990; Harrington et al., 1997). This conclusion was based on testing the material in water and no attention was given to the influence of the chemistry of the pore fluid. However, when different ambient fluids have been used, it has been suggested that that the mechanism of swelling is osmotic swelling due to ion-concentration differences between the double layer water and the pore fluid. These studies have shown on one hand the importance of the engineering properties in predicting the swelling potential, and on the other hand the necessity of examining the mineral composition and fabric to understand the mechanism of swelling in clay shale. The complexity of fluid / rock interactions in shale has led to a degree of confusion about the relationship between the phenomena of chemico-osmotic flow, swelling and hydration. For instance, from the swelling behaviour observed in

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Labiche shale (Wong, 1997), neither the osmotic swelling concept, nor water adsorption theory could explain why shale specimens swell when exposed to 1% and 3% NaCl solution when the pore fluid concentration is about 0.15%. However, Carminati et al. (1997) relate the cause of the above effect to suction pressure that could have been generated by insufficient saturation, unloading, and also due to the development of micro-cracks in the sample. However, Carminati et al.'s postulation has ignored the fact that swelling of Labiche shale is dependent on NaCl concentration (Wong, 1997).

§2.4.6.2 Possible mechanisms of swelling

In water saturated systems, local volume changes are a result of the transient movement of water due to effective stress changes. If the volume increases, the process is known as swelling. In partially or unsaturated systems, water interacts with cations attracted to negatively charged clay surfaces. Hydration of the cations on the silicate surfaces reduces the free energy of water and thus provides a driving force to cause spontaneous water adsorption and swelling until the activity or chemical potential of the associated water becomes equal to that of the free water (Low, 1992). This free water was defined by Hueckel (1992) as the water that fills the intramatrix pores and is able to move due to hydraulic gradients at room temperature.

In general, water is believed to be driven in or out of shale by different driving forces: hydraulic pressure, electrical and chemical potential, and temperature. However, researchers have adopted different concepts in an attempt to explain the mechanism that lead to swelling of shale. For example, the osmotic pressure concept was adopted by Oart et al. (1996); Santos et al. (1996); and Steiger (1993) who agreed that swelling occurs when there is a physico-chemical change in pore fluid chemistry. Osmotic movement of water from the intramatrix pores to the interlamellar pores occurs according to an ionic gradient. Because of its simplicity, the osmotic pressure concept is widely used, and can be used to quantify the swelling and consolidation (Barbour et al., 1989; Madsen et al., 1985). However, the osmotic pressure method appears to cater only for undisturbed
samples and samples that resist disintegration when in contact with water (Mody et al., 2002).

The water adsorption concept was first proposed by Chenevert (1970) who postulated that swelling of clay shale is a result of water adsorption that is capable of generating internal stresses. These stresses may lead to a reduction in compressive strength and consequently ultimate shale failure. Water adsorption was interpreted as a result of hydration and chemical forces. These forces are attributed to the ions and water movement between mineral layer surfaces (Pashely et al., 1983). The water adsorption mechanism can account for the molecular interactions between, pore fluid, clay particles, and the ions in the pore fluid. This approach was further explained by Low (1987) who suggested that the interaction of water with clay mineral surfaces reduces the chemical potential of the water, therefore creating a gradient in the chemical potential that enables additional water to flow into the system. The same concept was adopted by Yew et al. (1989) and Heidug et al. (1996) who succeeded in developing a swell model that is based on the continuity-governing equation by introduce a chemical potential parameter that was related to the driving force causing water migration and swelling.

Horsud et al. (1998) have suggested that suction is the main driving mechanism for shale swelling development. They stated that osmosis does not play a role, and that suction, which varies with the water content of the material, is the main cause of the swelling in clay shale. The limitation of this approach is that it cannot explain the influence of pore fluid chemistry.

2.4.7 Deformation characteristics

Shale can degrade to a certain extent with time when it is removed from its natural condition and is subjected to stress relief or when its environment changes or when it is subject to weathering processes. Thus the engineering properties can change with time. This time dependent deformation behaviour can be important in the design and construction of ground and underground structures built in swelling rocks. In order to
estimate the likely deformation of the ground and the displacements of structures reliably, accurate information on the deformation properties of shale is needed. For instance, elastic parameters such as modulus of elasticity and Poisson’s ratio are usually required as input for characterization of intact rock and/or rock mass deformability assessments.

These are required for a wide range of strain, from less than about 0.001% to that at peak strength. The distinction between elastic and plastic deformation properties is also important. It is difficult, however, to evaluate accurately these quantities by conventional triaxial compression test (Clayton et al., 1994; Tatsuoka & Kohata, 1995). In particular, data are required at strains of less than about 0.1%, which typically cover the strain levels experienced in the ground at working load levels (Burland, 1989; Atkinson & Sallfors, 1991; and Kim et al., 1994).

In their investigations on the deformation of the ground and displacement of structures, Kim et al. (1994) used local deformation transducers to measure small strains of less than 0.001%. They suggested that the initial portion of stress-strain relation at small strains is virtually linear and recoverable, allowing Young’s modulus to be defined confidently. They also attributed the S-shaped stress-strain curve to the closing of microcracks during axial compression. At large strains, the local stress-strain relation is noticeably non-linear. However, the degree of non-linearity was found to be smaller than is typical for poorly cemented material (Shibuya et al., 1991). Plastic strain prior to gross yielding is a common phenomenon in soft rocks. In principle, the yield point at the end of the straight portion of the stress-strain curve signifies the transition of elastic to plastic deformation. Although used widely, yield cannot always be exactly located in this way. Yoshinaka et al. (1996) performed triaxial compression tests on siltstone and mudstone samples from Japan. The results indicated that by plotting the variation of deformation moduli with respect to axial plastic strain, the yield point, and maximum elastic deformation modulus could easily be detected.

The correlation between the modulus of elasticity and confining stress has been investigated by Kulhawy (1975) who suggested a power law expression in the form:

\[
E = E_0 \left( \frac{P}{P_0} \right)^n
\]
\[ E_i = K' p_a \left( \frac{\sigma_3}{p_a} \right)^n \] (2.8)

where

\[ E_i = \text{initial tangent modulus} \]
\[ K' = \text{modulus number} \]
\[ n' = \text{modulus exponent} \]
\[ \sigma_3 = \text{confining stress} \]
\[ p_a = \text{atmospheric pressure in the same units as } \sigma_3 \text{ and } E_i \text{ to ensure that } K' \text{ and } n' \text{ are dimensionless numbers.} \]

Santarelli (1987) modified this expression using a power of \((1+\sigma_3)\). Ghafoori (1995) examined the modified equation and showed that the stress-strain response of the Ashfield shale was consistent with a cross anisotropic material with a ratio between modulus in the plane of the laminations and in the plane perpendicular to the laminations of 3. *Young's modulus* for specimens tested in uniaxial compression varied from 3 GPa to 7 GPa, these values increased with increasing confining stress according to a power law of the form:

\[ \frac{E}{p_a} = A \left( 1 + \frac{\sigma_3}{p_a} \right)^n \] (2.9)

where

\[ A = \text{modulus number, and} \]
\[ n = \text{modulus exponent, 0.035 from available data.} \]
It is believed that constant, \( A \), depends on moisture content in a similar way to UCS but insufficient data are available to provide a useful relation (Section 4.3.1). The effects of lamination orientation on the stiffness and shear strength parameters of clay shale have also been studied by many authors. Chenevert and Gatlin (1965) stated that no significant variation in Young’s modulus occurs within the bedding plane, but there is considerable variation between the modulus in the plane of anisotropy and in the plane perpendicular to this plane. Similar results were reported by Ibanez et al (1993) for Wilcox shale. Ghafoori (1995) performed uniaxial and triaxial compression tests on specimens with the laminations at orientation angles (\( \beta \)) vary from 0 to 90 degrees. He used the theory for a cross-anisotropic elastic material to analyse the deformation behaviour of Ashfield shale. The elastic response could be described by five parameters: \( E_1, E_2, \nu_1, \nu_2, G_2 \) where:

\[
E_1, E_2 \text{ are Young’s modulus, parallel and perpendicular to the lamination respectively}
\]

\[
\nu_1, \nu_2 \text{ are Poisson’s ratio describing the behaviour in parallel and perpendicular direction respectively}
\]

\[
G_2 \text{ is the independent shear modulus in planes perpendicular to laminations}
\]

Ghafoori (1995) demonstrated that the deformation moduli are dependent upon whether the load was applied parallel or perpendicular to the laminations (Fig 2.17). It was a maximum when the load was applied parallel to the lamination and a minimum when the load was applied perpendicular to laminations. Ghafoori showed also that the moduli were dependent on stress level and on moisture content (porosity) of the shale.

Because clay shales are not greatly cemented, their behaviour can be expected to be intermediate between rock and soil. For this reason concentration solely on stiffness at small strains may not be appropriate. One approach has been to use a pseudo-elastic secant stiffness. This approach is consistent with simple rock mechanics analysis (Bieniawski et al., 1970; Owen et al., 1980). Alternatively the problem can be approached from the perspective of cemented or structured soils. Several attempts have been made to establish the effect of structure on the mechanical characteristics of soil. For example,
Leroueil and Vaughan (1990) observed that during 1-D compression the behaviour will depend on the structural strength of the material. For intact (cemented) specimens, the material can reach states to the right of the de-structured normal compression line, and then after yield at Y approach the destructured response (Fig 2.18).

### 2.4.8 Strength of geomaterials

In all branches of geomechanics, the capacity of a soil or rock to withstand stress is generally described by reference to one of many strength criteria. Some of these criteria provide interesting insight into theoretical behaviour but fail to agree with experimental results. Others are highly empirical in nature but often oversimplify the complex behaviour observed in practice. In short, none of the available criteria is capable of simply describing the strength of geomaterials in general.

The shear strength of geomaterials is a primary concern in all stability analysis (foundations, retaining structures, slopes, boreholes, etc.). Based on the principle of effective stresses, the combination of $\tau$ and $\sigma$ which cause failure on the plane can be...
Figure 2.18 General framework of structure in soils

(Leroueil and Vaughan, 1990)

The Mohr-Coulomb effective stress failure envelope given by:

\[
\tau = c' + \sigma' \tan \phi
\]  

(2.10)

This criterion is widely used for soils and rocks, however $c'$ and $\phi'$ are not constants and alternative empirical failure criteria have been developed for rocks.

In an effort to explore the influence of confining stresses and volumetric swelling strains on clay shale, Wong (1997) performed a series of triaxial tests on LaBiche shale and suggested that strength of shale could be separated into two components, one dependent only upon void ratio or volumetric strain and the other dependent only upon stress. He
used Coulomb type expression (Eq.2.17) to rewrite the Hvorslev shear strength equation as follow:

\[ \tau = c_0 + \mu \sigma \] (2.11)

in which

- \( c_0 \) is the cohesive shear strength and \( \mu \) is the coefficient of internal friction.

\[ \tau = \tau (p_f, \varepsilon_{vf}) = c(\varepsilon_{vf}) + \sigma' \tan(\phi) \] (2.12)

in which

- \( \varepsilon_{vf} \) and \( p_f \) are the volumetric strain and mean confining stress at failure respectively.

It is believed that the above equation can be used if the clay shale specimens are expected to be subject to swelling during drained triaxial testing. As a consequence of this, normalization of the stress-strain relation with the confining stress variable alone may not be expected.

The Mohr-Coulomb failure criterion can also be expressed in terms of the major and minor principal effective stresses \( \sigma'_1 \) and \( \sigma'_3 \)

\[ \dot{\sigma}'_1 = \frac{1 + \sin \phi'}{1 - \sin \phi'} \dot{\sigma}'_3 + q'_{u} \] (2.13)

where \( q'_{u} \) is the uniaxial compressive strength of the material.

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It is more convenient to use an empirical failure criterion that is capable of modelling the highly non-linear relationship between major and minor principal stresses for isotropic and anisotropic rocks. This criterion was proposed by Hoek and Brown (1980). This criterion is presented in terms of principal stresses and uniaxial compressive strength and stated as:

\[
\sigma_1' = \sigma_3' + \sigma_{ci} \left( m \frac{\sigma_3'}{\sigma_{ci}} + s \right)^{0.5}
\]  

(2.14)

where

- \( \sigma_1' \) and \( \sigma_3' \) are the major and minor effective principal stresses at failure
- \( \sigma_{ci} \) is the uniaxial compressive strength of the intact rock material, and
- \( m \) and \( s \) are the material constants where \( s = 1 \) for intact rock and 0 for completely broken mass.

They suggested the value of \( m = 10 \) for argillaceous rocks e.g. shale and mudstone. The Hoek-Brown criteria does not explicitly allow for anisotropic behaviour even though it does include the effect of jointing.

Bieniawski (1974) found that a normalized form of an equation originally proposed by Murrel (1965) gave good predictions of rock strength. The equation has been widely used in practice in recent years and can be written in the following form:

\[
\frac{\sigma_1'}{\sigma_c} = 1 + B \left( \frac{\sigma_3'}{\sigma_c} \right)^a
\]  

(2.15)
where $\alpha$ is a constant equal to 0.75 for all rock types and $B$ is equal to 3 for siltstone and mudstones, $\sigma_1$ major principal stress, $\sigma_3$ minor principal stress, and $\sigma_c$ uniaxial compressive strength, where $\sigma_c = \sigma_{ci}$.

Shale shares many characteristics with both overconsolidated clays and harder rocks. Investigations into Melbourne mudstone have revealed a strong similarity between the mudstone strength behaviour and the strength characteristics of clay (Novello, 1988). Studies of the strength envelope for shale have revealed that the failure of highly weathered shale is reasonably represented by a linear Mohr Coulomb criterion, whereas slightly weathered shale shows a highly curved strength envelope and other characteristics more typical of harder rocks (Johnston, 1985). However, these variations in strength characteristics are a gradual progression from one extreme of weathering to the other. Limited data on the strength behaviour of fresh Ashfield shale in Sydney was reported by Won (1985). The strength parameters were given for intact shale, tested perpendicular to the lamination were $c = 3 \text{ MPa}$ and $\phi = 52^\circ$. However, this friction angle was reduced when specimens from the same area with lamination direction of $45^\circ$ were tested by Ghafoori (1995). Ghafoori has also measured strength parameters over a range of normal stress from 5 to 30 MPa and managed to obtain a friction angle of $\phi = 33^\circ$ and a cohesion strength of $c = 3.5 \text{ MPa}$ for fresh specimens tested normal to the lamination. Effective stress paths of Melbourne mudstone was investigated by Johnston and Chieu (1981, 1984). A highlight of the investigations into the soft rock has been the strong similarity between the soft rock behaviour and the characteristics of the clay. Novello (1988) described Melbourne mudstone as a saturated weathered weak Sillurian siltstone. His investigation to the engineering properties of Melbourne mudstone under triaxial and uniaxial stresses has confirmed the similarity between the weak rock and the overconsolidated clay and also confirmed that a range of friction angles between $20^\circ$ to $40^\circ$ has been achieved as the effective stress increased from 0 to 25 MPa. Novello contributed the change in the friction angle to the change in the natural water content of the material. Similarly, D'Elia (1980) investigated the influence of humidity on Italian clay shale and reported a reduction in strength parameters where $c$ and $\phi$ reduced from
35 kPa and 28° for fresh material to 12 kPa and 18° for partially humid material respectively. In a similar approach, in his investigation of Pierre shale, Botts (1998) illustrated the drastic reduction in the cohesion that resulted from the softening of the Pierre shale. The major reduction of strength was observed from the drop in the internal friction angle of the material from 54° to 48°. This reduction was reported within 0 to 200 kPa effective stress. Botts contributed the reduction in the strength to the decrease in the water content of the shale. This was also indicative from the reduction in the cohesion from 650 kPa to 0 kPa.

Based on the strength criterion proposed by Hoek and Brown (1980), Johnston (1985) proposed a criterion which assumes that the basic parameters are not only dependent on material type but also on the uniaxial compressive strength of the rock. The advantage of this criterion is that it is applicable to a wide range of intact materials ranging from soft clays through to very hard rocks for both compressive and tensile stresses regions.

\[
\sigma_{1n} = \left[ \frac{M}{B} \sigma_3 + 1 \right]^\frac{1}{n} \tag{2.16}
\]

where \( \sigma_{1n} = \frac{\sigma_{1n}'}{\sigma_c} \), \( \sigma_3n = \frac{\sigma_3n'}{\sigma_c} \), \( B \) and \( M \) are constants. \( M \) is a function of \( \sigma_c \) and material type whereas \( B \) is a function of \( \sigma_c \) only. Johnston (1985) suggested the following expressions for \( B \) and \( M \)

\[
B = 1 - 0.0172 (\log \sigma_c)^2 \tag{2.17}
\]

\[
M = 2.065 + k (\log \sigma_c)^2 \tag{2.18}
\]
where $k$ is equal to 0.231 for argillaceous rocks and $\sigma_c$ is expressed in MPa.

2.4.8.1 Critical state concept and its applications to soft rocks

The isotropic compression response of soft rock is qualitatively similar to the response of soils. Researchers have demonstrated that the compression characteristics for shale rocks display the general form of the loading path for overconsolidated soil with apparent preconsolidation pressures (Johnston and Chiu, 1981; Yoshinaka and Yamabe, 1981; Mesri et al, 1986, and Wong, 1997). For instance, the behaviour of some overconsolidated soils (e.g. London clay) at low confining pressures demonstrate a typical overconsolidated behaviour (Bishop et al., 1965) with relatively brittle response, well defined failure planes, and a degree of dilatancy. Moreover, Novello (1988) reported that drained shear response of Melbourne shale is similar to the behaviour of overconsolidated clay/soil (at low stress) and normally consolidated soil (at high stress) during shearing.

The critical state concept was developed in the 1960s to explain the behaviour of saturated reconstituted cohesive soils (Roscoe et al 1958; Roscoe and Burland, 1968), and since then has become firmly established in geotechnical engineering (Schofield and Wroth, 1968). The critical state concepts and the Modified Cam-Clay model have been shown to be applicable to a range of saturated, natural fine-grained soils (Wood, 1990).

In critical state soil mechanics, the stress-strain response of a soil specimen or element can be adequately described in a $q : p' : \nu$ three dimensional space. The projections of this three dimensional space in two dimensional $\nu : \ln p'$ space and $q : p'$, or the normalized format $q/p_c' : p'/p_c'$ where $p_c'$ is the initial isotropic stress, are more often used.

The normal consolidation line ($NCL$) is mathematically given by:

$$\nu = N - \lambda \ln p'$$

(2.19)
where $v$ is the specific volume, $\lambda$ the slope of the $NCL$ and $N$ the specific volume at $p' = 1$ kPa on the $NCL$.

The critical state line ($CSL$) is given by:

\[ v = \Gamma - \lambda \ln p' \quad (2.20) \]

and

\[ q = M p' \quad (2.21) \]

where $\Gamma$ is the specific volume at $p' = 1$ kPa in the $CSL$, and $M$ is a constant related to the frictional properties of the soil. The critical state line and normal consolidation can be plotted graphically in $v : \ln p'$ space (Fig. 2.19a).

Figure 2.19a State boundary surface in normalised space (Atkinson and Bransby)
Critical state soil mechanics states that all samples when sheared, whether normally consolidated or overconsolidated, would head towards and ultimately approach the critical state line \( (CSL) \) for which (plastic) shear distortion would occur without further change in volume or in effective stress. An isotropically normally consolidated specimen loaded to failure would traverse a unique normalized path (2.19b), i.e., the Roscoe surface, towards the critical state (Atkinson & Bransby, 1978). Therefore the Roscoe surface links the \( NCL \) to the \( CSL \). This framework has been justified by triaxial tests on various uncemented soils.

The Hvorslev surface is the yield locus defined as in Figure 2.19b. It is limited at one end by the \( CSL \) and the other by no tension condition. It serves for both normally consolidated and overconsolidated specimens. An overconsolidated specimen loaded to failure will be expected to reach the Hvorslev surface and then travel towards the \( CSL \). However, this does not always occur in practice because a shear plane or shear band often occurs in heavily overconsolidated specimens. The Roscoe surface, the Hvorslev surface and the no tension condition make up a state boundary surface \( (SBS) \). However, the \( SBS \) pattern may vary according to the material structure.

![Figure 2.19b Normal consolidation and critical states in \( v:\ln p' \) space (Atkinson and Bransby)](image-url)
The influence of cement on the mechanical behaviour of some material was investigated by different authors (e.g. Huang, 1994; Adachi et al., 1981; and Novello, 1988) who claimed that the critical state concepts are applicable for all types of materials and are not exclusive only for uncemented materials. Adachi et al., (1981) reported that the behaviour of the cemented soft rocks (e.g. Ohya tuff) can be generally described within the critical state framework. This was investigated by Chiu and Johnston (1984) who carried out studies on the behaviour of intact shales. They observed similarities between normalized stress paths of intact shale (Fig. 4.20a) and normalized stress paths of reconstituted shale (Fig. 4.20b) tested by Novello (1988). The patterns of the SBS of the two materials show that the Hvorslev and Roscoe surfaces do not meet as would have been expected for uncemented materials and that the stress paths in the two figures do not end in a critical state as expected. This pattern of the SBS has also been observed for other shale material (e.g. Dunbavan, 1988). Novello (1988) attributed the behaviour to the microcracks present in the intact shale, and suggested that these cracks grow in volume at high stress leading the shale to deviate from a critical state. However, this kind of behaviour may also result from other causes since it was also observed for another soft

![Figure 2.20a Normalised stress paths of intact shale rock (Chiu and Johnston, 1984)](image-url)
rock tested at high stress (e.g. Loe et al., 1992), and high stresses are normally expected to reduce cracking.

Picarelli et al. (1998, 2003) investigated the influence of fabric on the mechanical behaviour of highly plastic intensely fissured clay shales from Italy. They showed that the normalized failure surface of their intact shale lies below the surface for the reconstituted material tested at higher density and lower pre-consolidation stresses. When normalized by equivalent isotropic pressure $p_0$ (Fig. 5.21), the peak strength of natural material was significantly lower than that of the reconstituted material. Picarelli et al. interpreted the difference in behaviour as a clear effect of fabric, which makes negligible the role of both stress history and bonding. The difference in behaviour between different forms of specimens preparation are investigated in the present research program.
In this chapter previous studies into the classification, general characteristics, and strength of shale have been briefly reviewed to provide an overview of the behaviour of argillaceous rock in general and clay shale in particular.

Grain size, fissility, clay fraction, and clay minerals are among the important properties to consider in any geological classification of argillaceous rocks. According to the geological classifications, Bringelly shale can be defined as clay shale that classified as overconsolidated plastic clay with poorly developed diagenetic bonds. Although geological classification schemes can provide some useful information for engineers, they are generally inadequate for assessing potential engineering behavior of argillaceous materials.
The objective of an engineering classification scheme is to categorize geological materials according to their potential engineering behaviour. In this regard, an engineering classification is often oriented toward specific applications. This tends to cause some confusion among investigators when schemes employed for one application are considered valid for all applications. Classification of argillaceous materials for engineering purposes has been particularly difficult. Many of these difficulties have resulted because of the transitional nature between soil and rock. In addition, few engineering classification schemes account for the potential changes in material behavior which can occur in a relatively short time in many of these deposits.

Extensive efforts have been conducted by many researchers in an attempt to characterize the engineering properties of swelling shale. The rate of swelling has been found to be controlled by (i) hydraulic diffusion, i.e. flow of water from outside to the intermatrix pores according to the hydraulic gradient, and (ii) chemical diffusion-swelling. The chemical diffusion-swelling process includes ion diffusion, osmotic movement of water into double layers and water adsorption to clay minerals.

It was found that the shale deterioration and loss of its natural induration as a result of weathering upon exposure to wetting and drying cycles is by itself a major engineering problem. More important, however, is the fact that none of these durability tests measure the relative reduction of shear strength that occurs as these materials undergo deterioration in the field. Although wet and dry tests do measure the important tendency of a material to degrade, they do not necessarily measure the relative reduction of strength due to degradation.

A wide range of index properties have been reported for different types of shale. In general unconfined compressive strength and point load test results are based on test condition and the size of specimen. The point load test has been shown to be useful for weak, unsaturated, and brittle rocks that are typical of shale. For correlations of $I_s$ to UCS, a material-specific correlation factor should be used. However, the merit of these
findings is limited by the fact that the effect of mineralogy and cementation have not been independently investigated.

The major difficulties in assessing and predicting the engineering behaviour of clay shales can be attributed to several factors among which the transitional behaviour of shale between rock and soil, changes in the strength over a short period of time, and the role of their geological history are important. The factors that control the magnitude and time frame of these changes are not well understood. Evidence suggests that the presence of expandable clay minerals and microcracks, and the fabric play a very important part in the rapid loss of strength in clay shales. However, the geotechnical literature is surprisingly devoid of systematic studies concerned with theoretical or experimental aspects of swelling and fabric in the deterioration of the structural integrity of clay shales.

It appears that clay shale tends to deviate slightly from the critical state behaviour. This has been attributed to structural and diagenetic changes. The effects of these factors on the mechanical behaviour of clay shale are not fully understood. Further experimental work is required including consideration of other parameters such as the influence of degree of alignment of clay particles on reducing the strength and stiffness of clay shale and the role of saturation in influencing the strength and/or stiffness of these materials.

The review of data from Wianamatta shale is based on limited theoretical and experimental results. It is often assumed that the two shale members of the Wianamatta group are similar in their physical and mechanical behaviour. However, differences in mineralogy and core recovery between Bringelly and Ashfield shales have been noted, as well as a paucity of engineering data for the Bringelly shale. Thus, the current state of research on the engineering performance of Bringelly shale is still far from complete, and there are still many uncertainties that need to be addressed by more reliable experimental data.
Chapter 2 – Previous Study on Shale Rocks

END OF CHAPTER 2
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