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Mechanisms of failure of jointed rock masses and the behaviour of steep slopes

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Declaration

This thesis is the result of my own work and contains nothing which is the outcome of work done in collaboration. None of the material has previously been submitted for a degree at this or any other university.

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It does not exceed 80,000 words in length.

Owen G. Kimber
Durham
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Abstract

The geomorphological behaviour of steep jointed rock slopes has been studied using distinct element method computer models. In order to model steep slopes effectively, methodologies need to be combined from the studies of environmental modellers, geomorphologists and engineers. The distinct element method is ideal for the study of the development of jointed rock masses as the discontinuum approach can model the progressive failure of rock blocks along discontinuities.

Initial, theoretical modelling identified the limiting boundary conditions between the multiple block failure mechanisms of toppling, sliding and toppling-and-sliding, based upon the discontinuity geometry for a theoretically modelled limestone rock mass. It is demonstrated that joint dip, friction angle and spacing exert the greatest control upon rock mass failure mechanisms.

Two field locations, the Colorado Plateau and the Isle of Purbeck, have been chosen to provide a link between theoretical modelling and classic rock mass landforms which are controlled by variation in discontinuity geometry. In the Portland Limestone of the Isle of Purbeck, it is the joint geometry variation which influences development. Bedding steepens and average block size decreases in the coastal rock cliffs from east to west. Comparison between the model outputs highlighted that there is an increase in the rate of simulated cliff retreat from Winspit in the east to Durdle Door in the west. The Colorado Plateau rock cliffs form large, embayed plan-form escarpments and detached monoliths. It is the variation of joint set spacing in the cap-rock of cuesta-form composite scarps that controls development. Model results suggest there is a continuum of rock mass landforms, with buttes becoming detached at plan-form necks in the escarpment as determined by the joint geometry. The results show excellent similarity with the landforms observed in the field. This thesis introduces a research tool that can provide an understanding of slope behaviour.

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My family gave me the initial encouragement to take on this project. They have made it possible for me to be ambitious in my goals and have always been available to help me.

Finally, I wish to thank Hazel McGalliard. While I have gained the benefit of the many great experiences of this work, it seems at times that she has only had to bear my more difficult moments. I would like to thank her for always being a companion to me throughout my studies.

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The figures in this thesis include output from the UDEC computer simulation software. Most of the UDEC output is in the form of two-dimensional model block plots. The units of the horizontal and vertical axes are metres. The labelled notation (*10^1) indicates that the axis values need to be multiplied by 10. The legend includes an indication of the type of output plot, model cycle count and the model time, which is unrelated to real time. Where block velocity vectors are plotted, the scale is presented with units of m s^{-1}. The notation 1E 1 on the scale means that the scale is 1*10 m s^{-1} long. Again, the values of velocity do not relate to real velocity. Where UDEC plots of the history of total unbalanced forces for a model run are presented, the x-axis is model time (s), and the y-axis is force (kg m s^{-2}).

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List of Appendices

All appendices are to be found on the disk which is attached to the back cover of Volume 2 of this thesis. The file name corresponds with the appendix number. For the UDEC input files, where the letter 'v' is used, values are varied for different model runs.

Notations

Symbols are defined where they are introduced. The notation $B(u)$ means that B is a function of u. Dimensions of mechanically important quantities are given in brackets, with M = mass, L = length, and T = time. (0) means dimensionless. The corresponding SI units are the kilogram (kg), the metre (m) and the second (s). Some of the more commonly used symbols are the following:

b	length of base of a block (L)
c	cohesion (M L^{-1} T^{-2})
C_0	uniaxial or ultimate compressive strength (M L^{-1} T^{-2})
E	Young's modulus (M L^{-1} T^{-2})
E_{tp}	evapotranspiration
F	force (M L T^{-2})
Fs	factor of safety
g	acceleration of gravity (L T^{-2})
G	shear modulus (M L^{-1} T^{-2})
h	block height (L)
I	moment of inertia (M L^{2} T^{-1})
k	joint stiffness (M L^{-1} T^{-2})
K	bulk modulus (M L^{-1} T^{-2})
m	mass (M)
M	moment (M L^{2})
N	number of UDEC blocks
P	mean annual precipitation
R	Schmidt hammer rebound
s	used for purposes defined locally as both shear strength (M L^{-1} T^{-2}) and joint spacing (L)
t	time (T)
Δt	UDEC timestep
u, v	displacements parallel to x, y (L)
x, y, z	right-handed Cartesian coordinates
Z	depth below ground surface (L)

α	base plane angle (0)
δ	rotation about a centroid
δ_{ij}	Kronecker delta function
ε	normal strain (0)
θ	angular velocity (L T $^{-1}$)
λ, μ	Lamé constants (M L $^{-1}$ T $^{-2}$)
ν	Poisson's ratio (0)
ρ	mass density (M L $^{-3}$)
σ	normal stress (M L $^{-1}$ T $^{-2}$)
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (M L $^{-1}$ T $^{-2}$)
τ	shear stress (M L $^{-1}$ T $^{-2}$)
ϕ	friction angle; both internal and surficial (0)

Chapter 1: Introduction

Chapter 1: Introduction

1.1 Research objectives

The need for this study arises from the quest towards a comprehensive understanding of the development of jointed rock slope landforms. Innovative computer simulation techniques that allow the study of slope processes, which occur on a timescale that is difficult to monitor, are now available. The principal aim is to examine the behaviour and mechanisms of failure of jointed rock slopes by the computer modelling of real-world rock mass data. To achieve this, particular attention is given to the study of the Portland Limestone coastal cliffs of the Isle of Purbeck, central southern England, and the Chinle Formation, Wingate Sandstone and Kayenta Formation composite scarps of the Colorado Plateau, southwestern USA.

The research uses the Universal Distinct Element Code (UDEC) computer software in an embracing multidisciplinary study based on geological, geomorphological and rock geotechnical properties. The data are combined to consider slope evolution. The advantage of the approach is that a rock mass can be treated as an assemblage of blocks with failure at a cliff face being modelled by the movement of blocks along discrete joint sets. Field and laboratory data can be synthesised with model input including joint geometrical information, rock strength properties and cliff morphometric data. It is the first time that the approach has been used in the geomorphological study of jointed rock mass landforms.

To address the aim of the study, a number of primary research objectives were identified.

1. Introduce the UDEC computer simulation software as a geomorphological technique in the study of jointed rock cliff landforms.

2. Identify the conditions under which different failure mechanisms occur in jointed rock masses and assess the control exerted by relevant parameters in the processes of cliff development.

3. Study the controls on rock cliff and steep slope evolution using computer simulation based on data from real-world sites along the coast of the Isle of Purbeck, Dorset and the Colorado Plateau.

4. Compare the behaviour of rock slopes and cliffs for two contrasting environments through the application of a computer modelling approach.

1.2 Background to the present study

Geomorphological research on rock slopes has concentrated largely on descriptions of failure magnitude and frequency (Brunsden et al., 1984), slope retreat rates (Jones and Williams, 1991) and the study of failure products (Whalley, 1984). Jointed rock masses have received less attention than soft argillaceous sediments. Rock slope research is limited by the timescale over which behaviour needs to be monitored and the data-limited nature of the problem. There has been an upsurge in modelling research in rock mechanics (Starfield and Cundall, 1988) and the potential now exists for the use of advanced computer simulation codes in geomorphology.

Landform study is increasingly making use of the advantages afforded by following a multidisciplinary approach (Allison, 1997). Rock slope research has advanced by the consideration of material properties in a process-form framework (Allison et al., 1993; Cooks, 1983). Rock masses contain structural discontinuities which act to significantly reduce the shear strength of the mass below that of intact material. This study considers jointed rock masses where failures are composed of rock blocks along discontinuities. Others have studied the deformation of rock material by falls or avalanches (Azzoni et al., 1995; Maharaj, 1994). Geomorphologists have accounted for the control of discontinuities by using classification systems to assess the stability of landforms (Selby, 1980). However, the development of rock mass computer models allows for the study of slopes by a more quantitative method without a subjective weighting of data. The UDEC rock mass computer simulation uses physically based calculation systems and can allow for the synthesis of information in a rigorous scientific manner. Although the UDEC software has been extensively verified and a number of rock mechanics studies have successfully made use of the technique, the code has not been applied to the longer-term study of the geomorphic behaviour of jointed rock slopes.

The incorporation of the UDEC rock mass computer simulation code into a geomorphological framework provides the opportunity to re-examine rock slope landforms in two contrasting, but classic study locations. The Portland Limestone rock

cliffs in the Isle of Purbeck, Dorset, are formed in a coastal environment and act as a resistant rampart to the softer materials behind (Brunsden and Goudie, 1981). The Chinle Formation, Wingate Sandstone and Kayenta Formation rock cliffs on the Colorado Plateau occur in a low-precipitation, arid environment and form large, embayed escarpments and detached monoliths (Young, 1985). Both situations have cliffs which are controlled in development by a variation in discontinuity geometry. The Colorado Plateau is an ideal location for the study of rock mass landforms and much work has acknowledged the role of material strength variation on the stability of scarps (Koons, 1955; Schmidt, 1991; Schumm and Chorley, 1966). It has been suggested that the rock fabric of scarp caprock, in terms of joint orientation and spacing, is the dominant control of slope form (Nicholas and Dixon, 1986). The computer simulation approach provides the opportunity to assess the relative importance of joint geometry, and other factors, on the behaviour of Colorado Plateau rock slopes. Previous work on the Portland Limestone outcrop of the Isle of Purbeck has identified the structural control of the Purbeck Monocline on the stability of the cliffs (Allison, 1986; 1989). The opportunity exists to examine the control of a variation of bedding dip in the cliffs along the Isle of Purbeck coastline in some detail.

1.3 UDEC rock mass computer modelling

The most comprehensive, powerful and versatile discontinuum theory available is the distinct element method (Brown, 1987). The Universal Distinct Element Code (UDEC) is a two-dimensional numerical program based on the distinct element method for discontinuum modelling, originally developed by Cundall (1971). UDEC simulates the response of a jointed rock mass, represented as an assemblage of discrete blocks, under loading from either gravity or external forces. UDEC runs by simulating the motion of the blocks along the discontinuities as governed by linear or non-linear force-displacement relations for movement in the normal and shear directions solved by Lagrangian calculations. The program uses explicit time-marching to solve the equations of motion. UDEC has several built-in material behaviour models, for both the intact blocks and the discontinuities. Blocks can be made deformable by sub-dividing into a mesh of finite difference elements. The code is able to simulate the flow of fluid through the discontinuities and voids in the model, the transient flux of heat in

materials, linear inelastic behaviour of joints, plastic behaviour and fracture of blocks (Lemos *et al.*, 1985). The user can generate plots of the model and any problem variable and histories of change of a variable as a function of calculation steps can be recorded. Sequences of model output can be stored and replayed as a 'movie'. It is therefore possible to monitor the failure of a rock mass (Itasca, 1993).

UDEC has been extensively verified (Cundall and Strack, 1979; Lemos, 1990) and a number of studies have successfully applied the software to rock mechanics situations (Barton *et al.*, 1990; Pritchard and Savigny, 1991). It is best used as a means of modelling the progressive failure of rock slopes where block size is a key control on the problem. The code is ideally suited to study potential modes of failure directly related to the presence of discontinuous features. In introducing the distinct element method, Cundall (1971) suggested that the advantages of the model were that there is no limit to the amount of displacement or rotation of blocks meaning that progressive failure can be monitored. The program allows the individual study of the effects of joint geometry, joint parameters, loading conditions and excavation procedure. Other methods assume that the intact properties of the rock and the stiffness of the joints play a negligible part in the processes of failure of rock masses. UDEC recognises new block-block contacts automatically as the calculation progresses, which allows for the modelling of large numbers of blocks whose interactions are not known in advance (Konietzky *et al.*, 1994). Many other modelling approaches, concerned with short-term stability problems, are restricted to small displacements and do not address the changes in force distribution that accompany displacements of blocks. UDEC is ideally suited to the long-term geomorphic study of the behaviour of large, jointed rock slopes.

1.4 Approach and organisation of thesis

The text, tables and references of this thesis are to be found in Volume 1. Volume 2 comprises the figures and plates. The appendices are to be found on a computer disk which is appended inside the back cover of Volume 2. Two further papers have been published that contain some of the work presented in this thesis (Allison and Kimber, 1998; Kimber *et al.*, 1998).

The contents of this thesis may be summarised as follows. Chapter 2 reviews the study of jointed rock slopes and the geotechnical properties which need to be considered

for this research. Consideration is also given to the modelling approaches used in geomorphology. Chapter 3 assesses the various rock mass models used in rock mechanics and describes the operation of the UDEC software. Chapter 4 examines the limiting boundary conditions between the multiple block failure mechanisms toppling, sliding and toppling and sliding, based upon the discontinuity geometry for a theoretical limestone rock mass modelled using UDEC. The identification of the boundary conditions is presented as a precursor to considering the results of a parameter sensitivity test of UDEC input parameters which are relevant for the study of jointed rock slopes. The consideration of important rock mass behavioural controls is used as a base for much of the remainder of this study.

Chapter 5 provides a background to the field site locations of the Isle of Purbeck, Dorset and the Colorado Plateau, USA. Geological influences, previous work and geomorphological sites descriptions are reviewed. Chapter 6 presents the results of investigations into the mechanisms of failure and the behaviour of the Portland Limestone coastal cliffs of the Isle of Purbeck, Dorset, including the computer modelling of rock slopes from key field sites. Chapter 7 presents the results of investigations into the mechanisms of failure and the behaviour of cliffs on the Colorado Plateau, including the computer modelling of rock slopes from key field sites.

Finally, in Chapter 8, the original contributions made by this thesis are reviewed and the respects in which previous work have been extended is discussed. In conclusion, recommendations are made for further study.

Chapter 2: Background: The characteristics and study of jointed rock masses

Chapter 2: Background: The characteristics and study of jointed rock masses

2.1 Introduction

This chapter aims to present the background information necessary for this study. There are four main topics of discussion. The methodology used as part of this study accounts for general issues which concern the modelling of environmental issues and how geomorphologists deal with the study of slope development. A background of the issues associated with the measurement of both joint and intact block properties in rock slopes form parts two and three of this discussion. In order to study jointed rock masses, consideration needs to be made of both the strength of the rock blocks and the movement of blocks along discontinuities. Thus, the final section gives a background to the study of the stability of rock masses by combining information upon intact rock block strength and discontinuities between blocks.

2.2 Environmental modelling

Modelling is a broad concept which has been central to the study of geomorphology since the time of Gilbert (1880). Recently there has been an upsurge in modelling research in rock mechanics. This has been attributed to three reasons (Starfield and Cundall, 1988): more computer packages (ISRM, 1988; Spink, 1998), greater ability of packages to include geological detail and an acknowledgement of the success of modelling in other disciplines such as mechanical engineering.

There are several important considerations when modelling landforms. A balance needs to be made between using irrelevant details in a model and cutting out essential features of the real-world. The design of a model must take into account the accuracy of the data on which it will be run. Alonso (1968) explained that measurement errors can cause error in a model by up to 70% through algebraic operations. The more complex the model, the more the measurement errors accumulate as data are processed through the system. The gains in correctness of specification in a more complex model may be offset by the compounding of measurement errors. When accurate data are available, complex models are possible. When data are poor, simple models are advisable. Such considerations have been known for a long time. William of Occam, a

fourteenth-century English philosopher, stated *Non sunt multiplicanda entia praeter necessitatem* (things should not be multiplied without good reason).

In real-world environmental systems, chaotic behaviour has often been observed. Chaos is irregular and complicated behaviour which is regulated by some deterministic rule (Gleick, 1987; Thompson and Stewart, 1988). If a chaotic state is possible, then a very slight difference in initial parameter conditions can lead to a great difference in final outcome (Lorenz, 1976). Numerical slope simulations have shown that regolith evolution may reach a steady state, undergo simple or complex response cycles or behave chaotically depending upon the rate of bedrock weathering and erosional debris removal (Phillips, 1993). In rock mechanics, there seem to be at least two sources of the seemingly erratic behaviour, and both can be simulated by UDEC. First, certain geometrical patterns of discontinuities in a rock mass force the system to choose, apparently at random, between two alternative outcomes. For example, if apexes of triangular blocks touch, a choice will depend upon microscopic irregularities in geometry properties or kinetic energy (Cundall, 1990). Second, a positive feedback process of 'softening' arises when one or more stress components in an element are able to decrease with increasing strain. A region that has more strain softens more, and thereby attracts more strain (Cundall, 1990). In this study, behaviour from several runs of a model was monitored to overcome problems of chaos.

Simplification is a crucial part of rock mechanics modelling and there is a balance between geological detail and engineering understanding. In modelling rock masses a data-limited system is considered (Starfield *et al.*, 1990). Field data, such as *in situ* stresses, material properties and geological features, will never be completely known. It is futile to expect a model to provide design data, such as expected displacements, when there is uncertainty in the input data. However, a numerical model is still useful in providing a picture of the mechanisms that may occur in a particular physical system (Starfield and Cundall, 1988). Computer routines create the potential to save time by balancing a level of precision in research which is relevant to the problem.

As a contrast, it has been argued that geomorphological models have to become more detailed to incorporate aspects of the real system and reduce the range of generalisations (Anderson and Sambles, 1988). Natural environmental systems exhibit a number of characteristics which render description, explanation and prediction very

intricate. As processes are often linked, it is difficult to define meaningful system boundaries which enable the subject of investigation to be isolated (Howes and Anderson, 1988). Generally, there have been two ways in which geomorphologists design a model: by assembling small, known and unknown, elements of a system or by starting with a coarse representation of the system, and quantitatively describing component interactions (Carson and Kirkby, 1972; Ahnert, 1996). Models, such as UDEC, which make use of deductive, physical relationships have greater explanatory value. However, the environmental modeller has to beware of the use of randomness to explain natural systems. Modelled slope profiles generated with randomly varied processes can result in a general similarity to natural landforms (Ahnert, 1994; Chorley, 1964).

The laws of operation of environmental systems are scale-dependent and the type of landform change has implications for the modelling methodology. For instance, the larger a landform is, the longer it lasts and the higher the number of influences (Ahnert, 1988). A general modelling aim in geomorphology is long-term environmental simulation, but potential model inaccuracy is likely to increase (Anderson and Sambles, 1988). UDEC is designed for use as a tool for assessing the short-term stability of rock masses, but this study is concerned with simulation of slope evolution. The division between long- and short-term hillslope study is often considered to be around 100 years (Anderson and Richards, 1987).

Some attention has been given to the problem of the past in models which are based on present environmental conditions (Douglas, 1988). Hillslope development may be a matter of an extreme event followed by adjustment due to modifying processes. The age and persistence of the landform adds to the complexity of the modelled hillslope situation (Brunsden, 1993). Rock slopes will contain stresses which are a result of previous failures and loading. UDEC accounts for the landform history to some extent by allowing the model to reach initial equilibrium before movement is allowed. The process allows stresses between blocks to reduce and balance. However, unless a history of failures is known for a real-world rock slope, the modelling methodology has to acknowledge some limitation in real-world representation.

The purpose of a model needs to be clearly defined to avoid ambiguity with the explanation offered by model results. The relevant aspects of the real world and the

level of detail have to be identified. Modelling has been defined as a purposeful representation of whatever is being modelled (Starfield *et al.*, 1990). The modelling methodology used throughout this thesis differentiates between designing a model of a landform and designing a model which replicates features of a landform. No model can completely represent a real-world landform, but it is possible to gain an understanding of the development of a landform by using simple models which highlight important aspects of a system. Particular attention has been given to the level at which the UDEC joint geometry will match the real-world jointing pattern. The UDEC model geometry must represent the physical problem to a sufficient extent to capture the dominant mechanisms related to the geologic structure in the region of interest. A further theme which is central to this thesis is that a simple, theoretical approach to modelling, by isolating parameters, has advantages for geomorphic understanding. Time has been spent running a range of possible scenarios of investigation and designing simple test models to gain an appreciation of a response of a system. Sometimes, even large uncertainties in conditions do not alter the conclusions that can be drawn from the predicted response of a model.

Given the background information concerning rock mechanics modelling and the long-term simulation of geomorphic landforms, this thesis uses a well constrained modelling methodology. Importance is attached to a parameter sensitivity study in order to ascertain which geotechnical and slope morphology variables have an important control on the behaviour of steep slopes. Models have been designed to simulate the important characteristics of real-world landforms, and accurate data have been collected for the controlling parameters. UDEC has the modelling methodological advantage of rigorous physical principles. Empirical models do not have such explanatory power.

2.2.1 Background to slope study in geomorphology

A concept which has great influence on current geomorphic methodology is dynamic equilibrium (Gilbert, 1880; Hack, 1960). In contrast to Gilbert, who emphasised adjustment between present forms and processes, Davis (1892; 1899; 1930) suggested the cycle of erosion based on the systematic progression of landforms through time, initiated by uplift of the landsurface. An associated idea was that of slope decline through time as the rate of downcutting by streams decreases. Penck (1925) argued that

the form of landscapes depends on whether the rate of uplift is increasing, decreasing or constant through time. The Penck framework allows the possibility of parallel retreat of vertical slopes (Tuan, 1958). King (1957) showed that a number of escarpments in South Africa, with low-angled pediment bases, have experienced parallel retreat. However, a more complex understanding of slope retreat is evident in present-day geomorphology, with the mode of slope evolution depending upon the environment, morphometry and structure, as well as process (Chorley, 1964).

Much discussion has occurred within geomorphology with regard to the link with other scientific disciplines. Close synergy exists between many aspects of slope study undertaken by geomorphologists and engineering geologists. There is a desire in both to understand how the earth works at and close to the ground surface (Allison, 1997). It is believed that a more holistic explanation of earth surface systems is possible by integrating geomorphology and engineering geology. It is important that functional relations are maintained with materials and landforms. Continuous links can be identified between geomorphology and cognate subjects (Allison, 1997) (Figure 2.1). At the same time, it has been suggested that there is too much study of processes and that more landform development geomorphology is required (Ahnert, 1996). An obvious distinction in studies of slope stability is that engineers study systems over short timescales. The true distinction lies in the objectives of analysis. Engineers are interested in gaining a statement of stability for a specific slope, whereas geomorphologists are interested in an appraisal of the role of failure processes in slope evolution.

Accounting for the mechanical properties of landform materials is often part of geomorphological studies. It has been suggested that if a landform has ensystemic change, future predictions can be made after good observation of forms, materials and processes (Ahnert, 1988). This is known as functional geomorphology and does not involve changes which occur outside the system, such as fluctuations in climate. Much more can be achieved by examining geotechnical data and considering the principles used in other disciplines. It is possible to define geomorphological research within a continuum with extremes focusing on form, processes and materials (Allison, 1996a) (Figure 2.2). Individual studies can be plotted relative to the importance of the form, processes and materials variables. Attention is given in this

study to the development of rock slope form by considering processes affecting a rock mass and the strength of the rock mass material. A good understanding of the material properties composing jointed rock mass landforms, particularly joint parameters, is necessary when studying steep slopes (Allison, 1993).

More geomorphic attention has focused upon slopes in softer sediments, partly because more engineering stability problems occur in softer materials (Statham, 1977) and because the material is easier to analyse. In the study of jointed rock masses, the properties of the intact material have to be included as well as the deformation of the mass along discontinuities. The underlying process in hillslope studies is often the weathering of the surface of a consolidated rock mass into loose debris. The development of a hillside is thus controlled or limited by the pattern and rate of weathering upon it (Carson and Kirkby, 1972). In contrast to the study of jointed rock masses, much hillslope development study regards a slope as a smooth continuous profile shape. For instance, Kirkby (1971) used a mathematical approach relating the rate of change in slope height and rate of change in sediment transport. It is shown that a family of gradient- and distance-dependent transport processes lead to a family of slope forms of differing convexity and concavity.

Some useful concepts relating to slope development have been discussed in soft slope geomorphology. There are two important general problems which have been identified and are common to all studies of slope development. First, substantial landform change is often not observable in a human lifetime. Second, erosion is dominant and past forms may not be present in the landscape. Soft-slopes, such as badlands, are suited to rapidly changing slope form models (Davis, 1882; Gilbert, 1880; Howard, 1997; Schumm, 1956), or may be linked to a space / time substitution approach to study. For instance, at the Laugharne spit in South Wales slopes have been identified which have been progressively isolated from the sea (Savigear, 1952). The younger slopes have a steeper profile and inferences were made about changes through time. Sequences of modelled sites from the two field locations in this study used a space / time substitution concept to explain the development of rock mass landforms. The most effective slope studies are of processes at a point over short time periods and of slope forms over an area (Carson and Kirkby, 1972).

Early geomorphological slope models were the product of field investigation and were based around classification. Models are now more composite in character and predict long-term slope development which cannot be validated in the field (Anderson and Sambles, 1988; Howes and Anderson, 1988). An important concern of any simulation research is that of how realistic the modelling should be. Chorley (1978) identified the realist approach in geomorphology and Richards (1990; 1994), Rhoads (1994) and Bassett (1994) have continued the appraisal. Rhoads (1994) stated that the profitable aspects of realism were emphases on a reductionist search for the underlying causal mechanisms, the potential for emergence in complex open systems, and explanatory power rather than predictive accuracy using empirical data as the basis for theory acceptance. Computer simulation provides an efficient and rapid means to examine the validity of a range of possible relationships and it requires clarity of thought and precise specification and commitment (Howes and Anderson, 1988).

Slope development modelling has primarily analysed slopes with softer sediments. For instance, the SLOP3D model has been used to model landform changes as a response to changes in the gross rate of fluvial downcutting (Ahnert, 1988). SLOP3D uses the concept of dynamic equilibrium in slope evolution, relating the rate of creep to slope angle (Howard, 1988). The program has been used to model gully and valley development, inselberg development and karst landforms (Ahnert, 1994; 1996). One area of relative omission of slope modellers is that of rock slope processes in regard to rock slope controls (Anderson and Sambles, 1988). However, the difficulty of modelling jointed rock slopes has been noted (Selby *et al.*, 1988). It is suggested that a full understanding of rock slopes can be achieved only by exploiting all physical, numerical, geotechnical and schematic descriptive models. Such an understanding is now possible as sophisticated computer simulation packages such as UDEC are available.

2.3 Intact rock strength characteristics

The concept of stress-strain behaviour is integral to a comprehensive understanding of how materials at the surface of the earth respond to the processes acting upon them and the resultant landforms (Allison, 1996). It is important that the strength of rock blocks is known when studying jointed rock mass landforms as it acts as a control upon the failure of slopes. The intact rock strength characteristics are required by the UDEC modelling code. Intact rock is a polycrystalline solid consisting of a natural aggregate of materials, the properties of which depend upon the physical properties of the constituents and the type of bonding of the constituents to one another. Rock resistance to deformation involves toughness, resilience, strength and elasticity (Deere, 1966). A number of different laboratory and field tests are possible to understand links between stress, strain and shear and the resulting deformation of a material may be volumetric and / or distortion-based (Brown, 1981; Goodman, 1980; Hoek and Bray, 1981). Types of tests include basic field estimates based upon Schmidt hammer testing, static laboratory stress / strain testing, dynamic sonic wave propagation testing and correlations with petrographic properties.

2.3.1 Schmidt hammer testing

A portable N-type Schmidt hammer was used at the Colorado Plateau field sites as part of this study to measure the intact strength of the rock. The Schmidt hammer measures the distance of rebound of a controlled impact on a rock surface. The elastic recovery of a rock surface depends upon the hardness of the surface. As hardness is related to mechanical strength, the distance of rebound R gives a relative measure of surface hardness or strength (Day and Goudie, 1977). There are two types of Schmidt hammer, L and N-type. Reliable correlations have been observed between the two types (Ayday and Goktan, 1992). The advantages of correctly using the Schmidt hammer are that it provides some useful indicators of rock quality, it is cheap and it is fast (Campbell, 1991a). Laboratory methods of testing rocks are time-consuming, and the strength test is carried out on a specimen which has been severed from the mass of rocks, a condition contrary to reality in the solid rock (Hucka, 1965).

The technique has been employed extensively by geomorphologists to make rapid estimates of the strength of rocks. The Schmidt hammer has successfully been

used for the study of weathering in association with late-lying snow patches (Ballantyne *et al.*, 1993; Hall, 1993). Strong correlations have been gained between Schmidt hammer values and weathering on the Swedish coast (Sjöberg and Broadbent, 1991) and at Writing-On-Stone Provincial Park, Alberta (Campbell, 1991b). Where it is possible to take many measurements from a number of different sites, then relatively small differences in the mean R values can be statistically significant (McCarroll, 1991), although data have been recorded with a skewed frequency distribution (Ballantyne *et al.*, 1989). Allison (1991) suggested that the Schmidt hammer is basically an unconfined compression test despite being crude in nature. Readings are a function of rock surface hardness and roughness, but studies have used the hammer to estimate the joint compressive strength (Barton and Choubey, 1977).

There is no theoretical way of relating Schmidt hardness measurements to other strength properties, but empirical relations can be made (Deere, 1966). In comparison with other, similar rock strength tests, such as the point load test, the Los Angeles abrasion test and the slake durability tests, the Schmidt hammer provides the best correlation at a strength of less than 150 MPa, but is not so good at higher strengths (Cargill and Shakoor, 1990). However, the abrasion test and slake durability test can only be undertaken in the laboratory (Christaras, 1996). Aggistalis *et al.* (1996) attempted to correlate the uniaxial compressive strength and Young's modulus of 63 gabbros and 30 basalts with the Schmidt hammer rebound value. Deere (1966) related Schmidt hammer rebound values to the ultimate compressive strength C_0 and Young's modulus E. Twenty-four Schmidt hammer rebound values were recorded and the top 50% of values were used in correlation.

Allison (1990) stated that the accuracy of Schmidt hammer tests has been questioned and that a correlation of only 0.33 resulted with Hoek cell data from the Devonian limestone of the Napier Range, Australia. However, only thirteen data points were used in the correlation analysis. Campbell (1991a) suggested that Schmidt hammer rebound data from Allison (1990) were not all that different in accuracy from other more sophisticated tests. Improvements to correlations by combining rebound value with ultrasonic pulse velocity or dry density have been recommended (Arioglu and Tokgoz, 1991; Kolaiti and Papadopoulos, 1993). Stronger correlations have been

recorded between Schmidt hammer rebound values multiplied by density and the uniaxial compressive strength (Augustinus, 1991; Xu *et al.*, 1990).

Correlations between Schmidt hammer R values and strength properties give a wide range of results. Empirical relationships have been published for the conversion of Schmidt hammer R values and other intact rock properties (Table 2.1). Graphs of the relationships are shown in Figures 2.3 and 2.4. Although there is some variation of relationships, there are four similar relationships for Schmidt hammer R values between 30 and 60; the range expected for the testing of hard rock (Figures 2.5 and 2.6). It is interesting to note that three of the four relationships were derived from tests on sandstone or sandstone derivatives. Thus, for the purposes of this study, the valid conversion of a data set of R values by one of the four relationships was undertaken.

Reference	Equations	Based Upon	Result
E.L.E. (Manufacturer)	$\log C_o = 2.324 \log R - 1.92$	Concrete	$C_o = 63.56$ MPa
Hucka (1965)	$C_o = 3.38R - 71.9$	Sandstone	$C_o = 63.3$ MPa
Deere (1966)	$\log C_o = 0.0088 dnR + 1.01$	Colorado Plateau Sandstone	$C_o = 66.0$ MPa
	$E_t = 0.1867 dn^2 R - 7.865$		$E_t = 31.64$ GPa
Aufmuth (1974)	$\log(C_o/6.9) = 1.348 \log(dnR) - 1.325$		$C_o = 145.0$ MPa
	$\log(144.93E_t) = 1.061 \log(dnR) + 1.861$		$E_t = 60.73$ GPa
Yaalon and Singer (1974)	$\log(C_o/9.81 \times 10^4) = 0.0387R + 0.826$	Calcrete	$C_o = 23.2$ MPa
Irfan and Dearman (1978)	$C_o = 7.752R - 213.35$		$C_o = 96.7$ MPa
Beverly et al. (1979)	$C_o = 12.74 \times e^{(0.0185 dnR)}$		$C_o = 69.9$ MPa
	$1000E_t = 192(Rdn^2) - 12710$		$E_t = 42.7$ Gpa
Kidybinski (1980)	$C_o = 0.447 \times e^{(0.045(R + 3.5) + dn)}$		$C_o = 31.6$ MPa
Haramy and DeMarco (1985)	$C_o = 0.2869R^{1.3252}$	Coal	$C_o = 38.1$ MPa

Table 2.1:   Equations published for the conversion of Schmidt hammer rebound values, R, into intact rock geotechnical data (uniaxial compressive strength, C_o, and Young's modulus, E_t).

Results are calculated using Schmidt hammer value $R = 40$, and density $d = 2.3$ Mg m^{-3}.

2.3.2 Standard testing methods

Much geomorphological application of material properties has made use of simple stress / strain relationships to gain parameters associated with sample failure or shear deformation (Selby, 1987). Parameters required as part of this study for the modelling of jointed rock masses include porosity, density, Young's modulus, Poisson's ratio, the

friction angle and cohesion. Some account also needs to be made of the *in situ* stress situation. The required parameters can be gained from laboratory tests that replicate the shear failure of rocks. Uniaxial and triaxial shear tests have been run to derive values for the rock strength parameters used in this study. For this study, it was possible to make use of previous test results for both the Portland Limestone and the Colorado Plateau sandstones.

Cut samples prepared for failure testing are weighed, dried and weighed again to calculate porosity and bulk density. The usual definition of the strength of a material is the stress necessary to produce a permanent failure (Middleton and Wilcock, 1994). Strength is influenced by the porosity, porewater, amount and type of cement and grain composition of a sample. The unconfined compressive strength is requested nine times more often by engineers than the second most sought after property, triaxial strength (Bell, 1983). The compressive strength is the ratio of the maximum load at failure to the cross sectional area of the specimen before the test (Deere, 1966). A triaxial test varies a stress applied in the vertical direction (σ_1) upon a sample at a chosen confining pressure applied horizontally (σ_3). The deformation of the sample is measured by the strain, the proportional change in length of the specimen, and is generally plotted against the differential, or deviatoric, stress $\sigma_1 - \sigma_3$.

The shear failure of a rock specimen is normally analysed by the Mohr-Coulomb theory to derive the angle of internal friction, ϕ, which is due to the normal forces and cohesion. Coulomb (1776) related shear strength

$$s = c + \sigma \tan \phi$$

$$(2.3.2 - 1)$$

where c = cohesion and

σ = normal strength.

By running a number of failure tests, the Coulomb failure criterion can be estimated by the Mohr graphical method. Cohesion is plotted against strength, with circles drawn as defined by σ_1 and σ_3. A line drawn tangent to all the different Mohr circles representing failure defines an envelope of failure. The gradient of the line is the apparent friction angle of the samples and the cohesion axis intercept of the envelope represents the apparent cohesion of the sample. However, there is often much variation in the results of

a number of tests and the failure envelopes for most rocks lie between a parabola and a straight line (Goodman, 1980). Tests were conducted with a triaxial Hoek cell at horizontal confining pressures of 15 MPa, 30 MPa and 60 MPa for the failure of the Portland Limestone (Allison, 1986; 1989). Strength values of Colorado Plateau Sandstones, which were also required as part of this study, were gained from uniaxial tests at three values of normal stress and analysed by the angle-envelope method (Haverland, 1976).

The friction angle of a rock can be determined from a grit covered tilting table laboratory experiment (Bruce et al., 1989). With weathering there is a decrease in cohesion and an increase in friction angle (Carson and Kirkby, 1972). Guidelines concerning the effect of moisture saturation on the mechanical properties of rocks have been published (Ojo and Brook, 1990). The friction angle of discontinuities can be related to parameters which statistically summarise rock surface roughness (Reeves, 1985). However, assumptions are made about the probability distribution functions of rock surface features.

Deformability means the capacity of a material to strain under applied loads. Stress and strain may be theoretically related by using the Hooke theory of linear elasticity. Static elastic properties can be gained from the gradient of the stress / strain curve at the loading to failure of a rock sample (Deere, 1966). In the vertical direction, the stress / strain relationship of a sample can be defined by

$$\sigma = E\varepsilon$$

$$(2.3.2 - 2)$$

where σ = normal stress,

E = Young's modulus and

ε = normal strain.

Young's modulus is a good indicator of rock deformation under load, which has geomorphological importance when studying rock slopes (Allison, 1988; Augustinus, 1991). In the horizontal direction, Poisson's ratio v is the ratio of the lateral unit deformation to linear unit deformation within the elastic limit and theoretically varies between -1 and 0.5. Elastic properties are meaningless unless the conditions under which the properties are obtained are specified. However, the gradient of a stress / strain

curve is not linear when testing rock samples because of the presence of pores and cracks. Brady *et al.* (1985) tested a closely jointed rock specimen and showed that hysteresis is expected under loading. Elastic properties were attained in the unloading phase. Ofoegbu and Curran (1991) suggested that structural defects such as microcracks cause the load bearing capacity and elastic stiffness of rock to decrease.

All material models for deformable blocks in UDEC assume an isotropic material behaviour in the elastic range described by two constants, bulk modulus K and shear modulus G. The elastic constants, K and G, are used in UDEC rather than Young's modulus E and Poisson's ratio v because it is believed that bulk and shear moduli correspond to more fundamental aspects of material behaviour. Bulk modulus K and shear modulus G are related to the more commonly attained elastic properties, Young's modulus E and Poisson's ratio v, by:

$$K = \frac{E}{3(1-2\upsilon)}$$

(2.3.2 - 3)

$$G = \frac{E}{2(1+\upsilon)}$$

(2.3.2 - 4)

$$E = \frac{9KG}{3K+G}$$

(2.3.2 - 5)

$$v = \frac{3K-2G}{2(3K+G)}$$

(2.3.2 - 6)

Natural stresses in rock masses comprise gravitational stresses, tectonic stresses, residual stresses and thermal stresses and are determined by engineers using overcoring, flatjack or hydraulic fracturing methods (Herget, 1988). Studies analysing the stress field in jointed rock masses have demonstrated that the vertical stress is always a principal stress and equal to the weight of the overlying material. Horizontal stresses are strongly correlated to the rock mass fabric and are different for conditions of no lateral strain and no lateral displacement (Amadei and Pan, 1992). If a material is elastic and

no horizontal movement is possible, then the horizontal component of gravitational stress

$$\sigma_H = \frac{\upsilon}{1 - \upsilon} \sigma_V$$

<div align="right">(2.3.2 - 7)</div>

where σ_V = the vertical stress and

ν = Poisson's ratio (Mohajerani, 1990).

However, Pan *et al.* (1995) illustrated that non-zero horizontal compressive stresses exceeding the vertical stress develop at ridge crests, and that horizontal tensile stresses develop under isolated valleys, although the addition of horizontal stress has little effect on the magnitude of the vertical stress. For the purposes of this study, horizontal initial stresses were set, as recommended, to half the vertical stress value (Herget, 1988; Itasca, 1993). This value is suggested as many naturally recorded and calculated horizontal stresses are approximately half the vertical stress value.

2.3.3 Sonic wave propagation

There are two alternative methods for the determination of the elastic properties of a rock sample, either the gradient of the first part of the unloading curve as has been discussed, or dynamic wave propagation procedures. When a vibration is propagated in a solid, four different kinds of waves are generated; compressional (P) waves, shear (S) waves, Rayleigh and Love waves. The ability of a body to resist forces which tend to induce compressive, tensional, shear or volumetric deformations is determined by its elasticity (Cooks, 1981; Selby, 1987). In practice, a network of micro-cracks in the rock block being tested also has an influence, although the effect can be negated if porosity is considered (Goodman, 1980). Intact rock elasticity is useful in solving geomechanics problems and it can be measured *in situ* (Davis and Salvudurai, 1996).

Allison (1988) introduced the Grindosonic apparatus for measuring the speed of propagation of waves through a rock sample, and the calculation of dynamic Young's modulus. The apparatus has been used in the field, with tests being conducted on the Napier Range, Australia. The apparatus has also been used to determine dynamic Young's modulus and Poisson's ratio on Jurassic Portland Limestone as part of this study (Allison, 1989). However, the Grindosonic test apparatus is bulkier and more

expensive than a Schmidt hammer for non-destructive testing (Allison, 1991). Parameters such as mineralogy, texture, density, porosity, anisotropy, water content and temperature affect the propagation of waves (Augustinus, 1991). The dilation wave velocity increases with the state of stress, rock density and the closing of microcracks (Deere, 1966). The propagation of sonic waves through a rock can also be used to determine the pore aspect ratio within a sample (Burns *et al.*, 1990). Seismic wave velocity as a measure of rock quality was also used from five different rock types in South Africa and the USA (Cooks, 1981; Cooks, 1983). There is a strong relationship between rock quality and landform development of drainage basins, as described by morphometric properties.

2.3.4 Representation and accuracy of test results

The material properties entered into the UDEC model should correspond as closely as possible to the actual strength of the real-world intact rock. Laboratory experiments only partially reproduce conditions under which rocks deform *in situ*. The dimensions of samples are quite small compared with those in the field and drilling disturbs the rock (Cristescu, 1989). The presence of discontinuities in the model will account for a good portion of the scaling effect on properties, although some adjustment of properties will still probably be required to represent the influence of heterogeneities and micro-fractures on the rock mass response.

International Society for Rock Mechanics guidelines need to be followed when undertaking laboratory strength tests in order to maintain relevance (Brown, 1981). The loading rate in a triaxial test may influence the compression results and work hardening of rocks occurs. Loading conditions are a more accurate reproduction of field conditions for a triaxial test as opposed to a uniaxial test (Cristescu, 1989). However, great differences between 'measured' and 'real' values of rock properties may occur (Litwiniszyn, 1989). Values of parameters are influenced by the type of test, the apparatus used, and possible disturbance. It has been suggested that aspects of the standard testing methods need to be reviewed as rocks vary in response (Dobereiner *et al.*, 1990). In the consideration of volumetric strain relations, weaker rocks differ markedly from stronger rocks.

Intact rock blocks are said to be anisotropic as they have physical, dynamic, thermal, mechanical and hydraulic properties that vary with the direction of the principal stresses. Intact rock behaviour is not only non-linear, but anisotropy is important when measuring rock parameters (Amadei, 1996). Chosen testing points and volumes for the measurement of rock mass parameters affect the results of laboratory and field tests (Cuhna, 1990). Anisotropy causes rocks to be 70% weaker parallel to joints (Augustinus, 1991). The only way to evaluate strength anisotropy is by the systematic laboratory testing of a number of specimens drilled in different directions from an oriented block sample (Goodman, 1980). It is also important to recognise that joint strength properties measured in the laboratory typically are not representative of those for real joints in the field (Itasca, 1993). Often, the only way to guide the choice of appropriate parameters is by comparison with similar joint properties derived from field tests (Kulhawy, 1975).

Some of the problems of sample representation, laboratory testing and anisotropy are overcome by using non-destructive field apparatus. Stress / strain methods of measuring elastic properties assume that the rock is linearly elastic. Values of moduli obtained from the velocity of propagation of sonic waves are more accurate, but results are close to those from the initial third of an unloading curve. Thus, there is now an increasing tendency to use the dynamic procedure for the determination of elastic moduli (Cristescu, 1989) (Section 2.3.3). Relationships can be derived theoretically between the static deformation characteristics and the propagation of elastic waves through intact rock. However, statically determined values are always lower because of microcracks and the non-linear response of rock to static stress. The most divergent results between dynamic and static elastic properties occur for rocks with low moduli. If the rock is truly elastic, the elastic properties should approach constant values as the rock cracks close up under increasing stress (Deere, 1966).

2.4 Discontinuity characteristics

2.4.1 Genesis

Rock masses typically contain many structural discontinuities, which greatly reduce the shear strength of the mass below that of the intact material (Hencher, 1987). It is important that the influence of discontinuities is examined in studies of jointed rock slopes. Discontinuities are termed cracks, fractures, joints, bedding planes, schistosity or foliation and are classified into the four groups of tension, shear, sedimentation and metamorphic, according to the mechanical or environmental processes which formed them (Aydan and Kawamoto, 1990). The initiation of joints will depend upon the nature of the material and also pre-existing weaknesses such as cleavages and grain boundaries (Whalley et al., 1982) and can be deduced by analysing the joint aperture and spacing (Narr and Suppe, 1991; Pascal et al., 1997). The overburden pressures causing rock fragmentation have been calculated by relating the average joint spacing to the thickness of rock bedding (Angelier et al., 1989). In south-eastern France and the Gulf of Suez, the average joint spacing decreases when the degree of rock consolidation increases (Qin Huang and Angelier, 1989). Deformation patterns on the Colorado Plateau have been recorded at Arches National Park, where jointing is related to the formation of salt upwelling (Cruikshank et al., 1991; Zhao and Johnson, 1991).

2.4.2 Measurement and analysis

The engineering behaviour of a rock mass is controlled by the presence of discontinuities of a scale that can be physically measured (Attewell and Farmer, 1976). In the field, two geometries of the discontinuity need to be measured using a compass-clinometer. The dip is the maximum inclination of a structural discontinuity plane to the horizontal, measured from $0°$ to $90°$ with a clinometer (Hoek and Bray, 1981). Often, when measured on a rock mass as a whole, the line where the discontinuity plane intersects the rock face is measured, giving a result which is usually less than the true dip. The strike is the trace of the intersection of an obliquely inclined plane with a horizontal reference plane and it is perpendicular to the dip and dip direction of the oblique plane (Brown, 1981) and is measured from $0°$ to $180°$ with the compass. The two simply measured parameters, together with an indication of the direction in which the plane is dipping, are all that are required to record a three-dimensional plane.

At each field location selected for study, 100 strike and dip measures were collected (Goodman, 1980). Whether more are necessary can be ascertained from analysis, which indicates if the sample is large enough to represent the apparent discontinuity trends. The representative volume of a rock mass which should be surveyed is defined as the minimum volume beyond which any sub-mass behaves like a whole mass (Oda, 1988). Joints should be recorded along a scan-line, with the amount of bias decreasing with length (Sen and Kazai, 1984). It is suggested that the length of a scan-line must be at least three times larger than the typical joint length in order to survey a representative volume of a rock mass (Oda, 1988). In this study, tapes were laid parallel and perpendicular to the bedding and each discontinuity encountered was recorded. The results would obviously be more accurate and representative if the entire height of the outcrop was incorporated and the discontinuities were not measured as they were systematically encountered, but access is often not possible.

Hemispherical projection is a graphical method whereby three-dimensional planar data can be presented and analysed in two dimensions. It is often referred to as stereographic projection (Phillips, 1971). A stereoplot method is very useful in representing the individual planes as a great circle and analysing their relationships and intersections. To focus on the trends in the discontinuity data, the technique is not very clear (Swan and Sandihands, 1995). By representing the planes as points on the equal area net, statistical and vector analysis is possible (Mauldon and Goodman, 1996). The pole of each great circle is the point which is centre of the great circle on the sphere. By plotting the poles for each measured discontinuity on a stereoplot, statistical contouring methods can be used to identify pole concentrations (Priest, 1985). Thus, mean joint set characteristics can be deduced.

The process described above can be undertaken by entering the joint strike and dip data into the computer program ROCKLORD (Bromhead, 1987). The computer processing greatly decreases the time spent analysing joint data and has been used in this study. However, it is important to understand the method. Stereographic projection is not well adapted when the discontinuities are well dispersed and other computer programs have been written for the analysis of three-dimensional block structures (Baroudi et al., 1990; Matheson, 1988; Priest, 1993; van Everdingen et al., 1992; Vollmer, 1995; Zhang Xing, 1989).

Other measured discontinuity parameters include the spacing between parallel pairs of joints and the persistence of individual joint segments (Gabrielsen, 1990). It has been suggested that a log-normal or an exponential statistical model is useful for the fit of joint spacing data (Mohajerani, 1989). Knowledge of spacing and size of discontinuities in a rock mass is of considerable importance for the prediction of rock behaviour (Priest and Hudson, 1981) and the control of these parameters will be analysed as part of this study. A second survey was undertaken to measure joint spacing at the sites used in this study with tapes laid perpendicular to the strike of each joint set. The spacing between joint segments of the relevant joint set was noted as encountered upon traversing the section. Experiments were made whereby spacings were measured along the two perpendicular transects used for the measurement of joint orientation. The spacing was noted, and the actual mean spacing between each joint set was calculated trigonometrically. Because the joint sets at the two field locations used as part of this study were repeatable and had low variability, the former method for measuring spacing was thought the most suitable. Some attempts were also made at measuring joint segment lengths to gain an indication of joint persistence. However, joints at both field locations were highly continuous.

Various difficulties have been noted when measuring joint geometry characteristics of a rock mass. Rock mass studies usually take the conservative approach of assuming full persistence and biases are frequently introduced in sampling for geometrical properties (Kulatilake and Wu, 1984). If the discontinuity spacing is known, estimates of block size can be made (Wang *et al.*, 1990). However, a major problem of measuring persistence is that direct mapping internally in a rock mass is not possible (Einstein, 1983). A possible solution uses the SLOPSIM program which relates the spatial variability of joints to a random persistence variable used to give a probability of rock slope stability (Einstein *et al.*, 1983). Other studies have used a probabilistic analysis based upon joint shape, distribution of centroids and the probability function of joint distribution to predict the three-dimensional joint orientation in an outcrop (Kuroda *et al.*, 1991).

2.4.3 Strength properties

Joint properties for UDEC input are derived from laboratory triaxial and shear tests of joints and a knowledge of joint stiffnesses is required under both normal and shear loads (Swan, 1981). Values for normal and shear stiffnesses for rock joints can typically range from 100 MPa m^{-1}, for joints with soft clay in-filling, to over 100 GPa m^{-1} for tight joints in granite and basalt, while the Poisson's ratio range is from 0.02 to 0.73 (Kulhawy, 1975). Approximate stiffnesses can be calculated from information on the deformability and joint structure in the rock mass and the deformability of the intact rock (Itasca, 1993). Joint normal stiffness

$$k_n = \frac{E_m E_r}{s(E_r - E_m)}$$

(2.4.3 - 1)

where E_m = rock mass Young's modulus,

E_r = intact rock Young's modulus and

s = joint spacing.

Joint shear stiffness

$$k_s = \frac{G_m G_r}{s(G_r - G_m)}$$

(2.4.3 - 2)

where G_m = rock mass shear modulus and

G_r = intact rock shear modulus.

The nature of the material which fills rock joints is important. Although the shear strength of infilled rock joints has been long studied a complete understanding of the parameters controlling the process has never been reached (DeToledo and DeFreitas, 1993). Closure, shear displacement and dilation are the joint strength components which dictate the performance of rock masses (Bandis, 1993). It is argued that the global behaviour of a rough rock joint depends on the microfeatures of the contact planes on the joint (Dong and Pan, 1996). Secondary sources have been used as recommended for this study to obtain values of joint normal and shear stiffness (Itasca, 1993). Joint stiffness values are a reflection of joint wall roughness, the strength of asperities and the

infilling material. Data for the stiffness properties of rock joints can be found in Piteau (1973), Kulhawy (1975), Rosso (1976) and Bandis *et al.* (1983).

The mechanical response of rock joints can be characterised by using a constitutive modelling approach called the disturbed state concept (DSC) (Desai and Ma, 1992). Consideration is made of friction, cohesion, roughness, asperities, deformation and degradation of joints. It is possible to estimate the joint friction angle parameter and shear strength of joints based upon Schmidt hammer rebound readings and a residual tilt test (Barton and Choubey, 1977). Other estimation models for the strength of joints have included the joint roughness coefficient (JRC) (Bandis *et al.*, 1983). The JRC can be related to the joint compressive strength (JCS) as derived from Schmidt hammer readings (Barton and Bandis, 1990). It has been noted that many models have been formulated to predict the behaviour of rough rock joints, but they are questionable, because they are too simplistic, rely too heavily on empiricism or require complex input parameters that are typically way beyond current capabilities of normal site investigation practice or laboratory procedures (Haberfield and Johnston, 1994). Other problems with the measurement of the material properties of rock joints occur because of the anisotropy in strength (Jing *et al.*, 1992). Due to the difficulty of gaining laboratory joint strength data and the problems with such techniques, it was decided to use published data when ascribing joint properties in this study.

2.5 Rock Mass Stability

2.5.1 The effect of joint properties on rock mass strength

The geometry and strength characteristics of rock joints decrease the strength of jointed rock masses compared with the strength of intact rock. When investigating the slope stability of opencast mines and quarries it is important to include the strength of geological structural features and take into account the choice of mechanical parameters for the joints (Cojean, 1995). The mechanical behaviour of a rock mass will depend on the mechanical behaviour of the rock element and of discontinuities and their orientation with respect to the applied load and constraint conditions (Aydan and Kawamoto, 1990; Naugle, 1988). It has been demonstrated that a relationship exists between jointing and topography (Fleischmann, 1991). The reduction in rock shear moduli because of the

presence of joints tends to be greater than corresponding reductions in direct moduli and Poisson's ratio (Gerrard, 1982).

The classification of jointed rock masses, based upon the incorporation of weakening of rock material by discontinuities, is a commonly used engineering method to gain an indication of comparative rock mass strength (Goodman, 1980). The rock quality designation system (RQD) was developed by Deere (1969) and classifies rock as excellent, good, fair, poor or very poor, giving an indication of slope stability. The RQD can be related to sonic velocity measurements (Sen and Kazai, 1984) and discontinuity spacing in a rock mass (Priest and Hudson, 1981). However, the RQD ignores factors of rock strength, joint character and environmental properties (Goodman, 1980).

Three rock mass rating schemes have been widely used and developed for engineering practice. Barton $et\ al.$ (1974) developed the NGI tunnelling quality index Q related to the RQD:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

(2.5.1 - 1)

where J_n = number of joints,

J_r = joint roughness number,

J_a = joint alteration number,

J_w = water reduction factor and

SRF = stress reduction factor.

The Q value varies from 0.001 for poor rock to 1000 for good quality rock. The application of the Q system was demonstrated by relating with 38 categories to an excavation support ratio (ESR), which varies with the use of excavation and the extent to which some degree of instability is acceptable (Barton $et\ al.$, 1974). Bieniawski (1978) developed an empirical relation for a modulus of rock mass deformation E_m from numerous field test results and a rock mass rating (RMR):

$$E_m = 2(RMR) - 100$$

(2.5.1 - 2)

The RMR is an index from 0 to 100 derived from the strength of the rock, drill quality, ground water, joint spacing and characteristics (Bieniawski, 1973). Schultz (1996)

analysed Mohr failure envelopes obtained from the RMR classification system and showed that rock mass cohesive strength, tensile strength and unconfined compressive strength can be reduced by as much as a divisor of ten relative to the values for the unfractured material. The most commonly accepted approach to estimate rock mass strength is that proposed by Hoek and Brown (1980). The major principal stress at peak strength

$$\sigma_{1s} = \sigma_3 + (m \, \sigma_c \, \sigma_s + s \, \sigma^2_c)^{1/2}$$

(2.5.1 - 3)

where σ_3 = minor principal stress,

m and s = constants that depend on the properties of rock and

σ_c = uniaxial compressive strength of intact rock material.

It is also possible to estimate friction angle and cohesion to meet the demands of software written in terms of the Mohr-Coulomb failure criterion (Hoek and Brown, 1997).

Rock mass classification techniques are useful to gain an appreciation of the relative strength of a rock mass, and have been used in many studies to complement rock mass modelling (Cameron-Clarke and Budavari, 1981; Dershowitz and Einstein, 1988; Hoek 1983; Price, 1993; Ramamurthy and Arora, 1994). Engineering classifications have been modified for field use by geomorphologists. The rock mass strength (RMS) classification was first introduced by Selby (1980) for the analysis of jointed rock masses. Simplifications were designed so that a geomorphologist can make a rapid, unsupported estimate of RMS in remote field locations (Selby, 1987; 1993). The strength of intact rock as measured by the Schmidt hammer, weathering of rock, groundwater, spacing, orientation, continuity and width of discontinuities are all measured and rated in weighting for a classification of up to 100. The RMS classification values for rock masses correlated very well with slope angles for slopes in Antarctica and northern New Zealand (Moon, 1984; Selby, 1980). Thus, the concept of a strength equilibrium slope was developed as stronger slopes are more likely to be inclined at steeper angles (Abrahams and Parsons, 1987).

Easily quantified feedback between rock strength and slope development is possible using the Selby RMS classification because any slope can be assigned a

numerical rating (Selby, 1993). The landscape on the Clarens Sandstone Formation in the Golden Gate Highlands National Park, South Africa is characterised by two slope forms (Moon and Munro-Perry, 1988). The steeper cliffs are in strength equilibrium and the RMS has been used to demonstrate different stages in slope development. Selby (1982) observed a widespread occurrence of strength equilibrium slopes in the Namib desert. Parallel retreat of slopes on inselbergs only occurs if the rock strength is uniform throughout the mass, but a small change in joint spacing may alter the strength. On the Napier Range in the Kimberley region of Western Australia, RMS correlated not just with slope angle, but also a form parameter (Allison and Goudie, 1990a; 1990b). The more convex the slope form, the stronger the RMS value.

The RMS and other classification techniques are limited because of the simplifications in measurement and the lumping of properties together to make up one parameter. All successful ratings systems have been developed from the experience of many users, although all suffer from the weighting methodology (Price, 1993). The altering of weighting systems can markedly influence results. In the real-world, different properties may have different influences depending upon the other properties in the rock mass and the stage of slope development. For instance, the angle of dip of bedding planes in a rock mass may have a large influence if the slope is steep but if the slope is gentle, then the behavioural control of the bedding may be negligible. Advances in the understanding of slope behaviour have been made using the RMS technique but the UDEC modelling of slopes based on a physical, deductive approach is a more rigorous scientific method. The advantage of using modelling approaches over empirical classification methods is that insight can be gained for rock mass failure mechanisms and how the change in slope form through time can lead to different behaviour.

2.5.1.1 Scale effects upon rock mass properties

One of the principal obstacles in rock mechanics is the fact that small-scale discontinuities differ from those on a large scale (Leal Gomes and Cuhna, 1995). Microcracks influence rock quality and strength but are difficult to quantify (Gerrard, 1988). The factors affecting the representative sample of a rock mass may provide the solution. The concept of representative elementary volume for rock mass measurement is often used to account for the scale effects which are caused by the varied effect of

rock material properties and joint geometries (Cuhna, 1990; Odling, 1997). Mohammad *et al.* (1997) reviewed forty papers reporting rock mass geotechnical characteristics and noted the reduction factors, because of scale effects, which have been used for the modelling of jointed rock masses. The average reduction, attributed to micro-fractures, of Young's modulus from laboratory values is by a factor of 0.47, although many engineers do not use reduction factors (Mohammed *et al.*, 1997).

The use of rock mass classification techniques is not just confined to the study of comparative mass strength. An important application of engineering classification techniques is at smaller scales. Results from laboratory tests of intact rock samples are affected by the volume involved. The variation of test results with the specimen size is called the scale effect (Cuhna, 1990) and microscopic discontinua have a control. If the strength properties of a laboratory sample have application in the study of rock block deformation, reduction factors need to be used (Barton, 1990). Thus, for input of deformation parameters into UDEC, rock mass classification techniques must be used to convert laboratory-measured elastic moduli to the scale of a whole block being modelled.

2.5.2 The stability of rock masses

By accounting for the strength of rock masses which is affected by discontinuities and intact rock properties, assessment can be made of the stability of slopes. Early studies of slope stability emphasised the value of joint surveys in factor of safety studies. Terzaghi (1962) included in stability analysis the angle of shearing resistance of the jointed rock, the effective cohesion and the water pressure in joints. The factor of safety for rock slope failures is greatly reduced when pore pressures are accounted for in stability analysis (West, 1996) and the value of mechanical parameters for the joints and rock material is thought to be an influence on the mode of failure of a rock slope (Cojean, 1995). The probability of failure for slopes shows little change for low and high slope angles, but varies rapidly in the 40° to 60° range (Carter and Lajtai, 1992). Also, the stability of rock failure is shown to be affected by the discontinuous sliding because of asperities on joints (Estrin and Brecht, 1996).

Commonly occurring in nature are rock slopes overlying softer bases. Much understanding has yet to be gained for such situations (Poisel, 1990). Failure

mechanisms have several stages of movement (Pausto and Soldati, 1996). The rock mass initially sinks in its centre, leading to bulging of sediments at the slope edge. This may cause topples and slides in the rock mass, which subsequently induce rotational movements in the softer material. Steger and Unterberger (1990) modelled three types of failure: sliding of blocks upon the softer base, direct toppling of rock blocks, and the slumping of rock blocks leading to bulging of the softer material. Slope bulging has also been observed at the Isle of Portland, Dorset (Brunsden et al., 1996). The study of composite rock slopes needs to account for the properties of a discontinuous rock mass and a continuous material (Steger and Unterberger, 1990).

Geomorphological studies of slope development in jointed rock masses account for both the properties of the intact rock, and the properties of the discontinuities. It is difficult to combine the various characteristics of jointed rock into a coherent model (Gerrard, 1988). Although cliff retreat of the Jurassic Portland Limestone coastal cliffs of the Isle of Purbeck, Dorset showed no regular spatial or temporal pattern, trends can be identified by accounting for the discontinuities in conjunction with the rock strength measurements (Allison, 1989). Where the discontinuities indicated more stable cliffs, the intact rock strength was weakest, and where the discontinuity pattern indicated more unstable cliffs, the intact rock strength was greatest. Computer simulation of the cliffs has identified a change in mechanism of cliff failure along the Isle of Purbeck coastline (Allison and Kimber, 1998; Kimber et al., 1998).

Some studies have attempted to examine slope behaviour without incorporating the effect of joints. The material strength of cliffs along the West Wales coastline was measured and found to have no relation to the recession rates (Jones and Williams, 1991). It was concluded that the volume of beach material was the dominant variable. However, on the West Wales coast, failure mechanisms and cliff stability were related to cliff geometry and jointing patterns (Williams et al., 1993; Williams et al., 1994). At a regional scale, different parameters are important in explaining slope behaviour. The main factor controlling scarp form is areal variation in process rates, such as erodibility, fluvial incision and groundwater sapping (Howard, 1995). It was suggested that controls on slope form near Picton, New South Wales, Australia are the sandstone caprock, slope processes and the channel at the base of the slope (Pain, 1986). It is postulated that younger slopes undergo parallel retreat and that older slopes decline. Long-term controls

over erosion rates are the rate of base level lowering and rock dip (Howard and Selby, 1994).

Studies have also attempted to explain slope development without incorporating the strength of the intact blocks. Fracture patterns have a critical influence upon the landform development in granite (Twidale, 1993). Ehlen (1991; 1992) related three types of tor on Dartmoor, Devon to the joint spacing distributions for the rock masses. Joints have also provided explanations of the form of roches moutonnées in SW Finland (Rastas and Seppälä, 1991), fjord systems in Norway (Nesje and Whillans, 1994) and major escarpment knickpoints (Weissel and Seidl, 1997).

2.5.2.1 Failure mechanisms in jointed rock masses

The link between the failure mechanism of rock blocks from slopes and the geomorphological behaviour of rock mass landforms is a major theme in this thesis. There are three basic mechanisms of rock block failure under gravitational forces: plane sliding, wedge sliding and toppling (DeFreitas and Watters, 1973; Goodman, 1980; Hoek and Bray, 1981) (Figure 2.7). The orientation of discontinuities determines the conventional stability of rock blocks (Leung and Quek, 1995). Plane sliding can occur when the plane of potential movement is within 20° of the slope angle (Cruden, 1985; Goodman, 1980; Sharma et al., 1995). The failure plane must intersect the slope free face and dip more steeply than the angle of internal friction, ϕ, of the discontinuity. Wedge failures occur along the intersection of two or more discontinuities (Hoek and Bray, 1981). The dip of the line of intersection must be greater than the angle of internal friction across the surface, but less than the dip of the slope face. Toppling is a more complex mechanism and has only been recognised for the last 30 years (Goodman and Bray, 1976). Toppling involves the rotation of blocks when the centre of mass for a block overlies a pivot point (Figure 2.8). Failure may be instantaneous and catastrophic or merely involve the cambering of a slope.

Toppling has been identified as an important mode of failure which involves a large volume of rock in steep slopes in layered, well-jointed or foliated rock. Failures have been categorised into flexural-toppling, block toppling and block-flexural-toppling (Goodman and Bray, 1976). In slopes typically composed of slates or schists with one discontinuity system, columns break in flexure as they bend forward. Harder rocks, such

as limestone and sandstone, can exhibit block toppling, due to the pressure exerted by surrounding blocks, when individual columns are divided by widely spaced planes of bedding. Block-flexure-toppling occurs when harder rock columns are interbedded with chert and shale and toppling may also occur as a secondary motion to sliding (Evans, 1981). Block topples and block-flexure-topples have been identified on the sandstone and shale slopes of the Highwood Pass, Alberta, Canada (Cruden and Hu, 1994). The toppling mode is controlled by ratios of joint spacing to bedding thicknesses, rock strength and topography. The finite element code, which includes the bending of stresses in its formulation, can be used to model the flexural toppling of slopes in foliated rock masses with no cross-joints (Adhikary et al., 1996). In a geomorphic study of toppling failures from the dolerite alpine cliffs on Ben Lomond, Tasmania, two modes of toppling have been observed depending upon the location of the contact with the clay cliff base (Caine, 1982). Toppling only occurs where the cliffs have been steepened due to glaciation, which occurred more than 100,000 years ago. Thus rates of cliff retreat have been estimated at 0.2 mm yr^{-1}. Large-scale toppling failures have been observed on the slopes of Ben Attow, Scotland (Holmes and Jarvis, 1985). The term 'sackung' was used to describe failure where there is no continuous failure plane.

The stabilisation of slopes which fail by a toppling mechanism has been widely discussed by engineers (Aydin and Kawamoto, 1992; Cruden et al., 1993; Scavia et al., 1990). At the Glennies Creek Dam, Australia, a new rock cutting exposed toppling slopes (Woodward, 1988). Stabilisation was attained by reducing the angle of the slope face. Also, a granite slope in British Columbia, Canada and a sandstone slope of the Rocky Mountains were monitored and engineering stability was ensured by using rock bolts to increase the effective friction angle (Wyllie, 1980). Slope stabilisation can be achieved by such measures as reducing slope height, flattening the slope face, bolting slabs together or supporting the toe (Wyllie and Wood, 1983). The distinct element method has been used in an analysis of the stability of toppling slopes at the Delabole Slate Quarry and replicates a complex failure mechanism (Coggan and Pine, 1996) and a range of failure mechanisms are apparent along the excavations of the Aqaba Ras El Naqab highway, Jordan (AlHomoud and Tal, 1997).

Much study of natural rock slope failure mechanisms has focused on field sites in Canada. Observations made in Kananaskis Country, in the Front Ranges of the

Rockies, suggest that rock-slides occur on overdip slopes along penetrative discontinuities and topples may be catastrophic if the bedding dips steeply into the slope (Cruden, 1988). The most hazardous Canadian slopes are determined by the relationship between the orientation of the slope and the penetrative discontinuities (Cruden and Hu, 1996). At Hell's Gate Bluffs, British Columbia, zones of stresses with a toppling slope were successfully modelled using a finite element analysis (Kakani and Piteau, 1976). Slides and topples acting in conjunction have been monitored based upon ground-motion vectors using an EDM in the Southern Coast Mountains, British Columbia (Bovis and Evans, 1995) and the results are consistent with feasible gravitational movements (Bovis and Evans, 1996). Pritchard and Savigny (1990) demonstrated that it was possible to model block toppling using UDEC and quantitatively assess slope stability. An example of a toppling failure at the Heather Hill landslide, British Columbia, was simulated with UDEC (Pritchard and Savigny, 1991). The variation in joint spacing and the intercalated change in lithologies account for the curvilinear failure surface.

Theoretical analysis of weak rock masses with prominent discontinuity sets has been carried out using the distinct element method (Hsu and Nelson, 1995). Data from testing on Eagle Ford Shale, Texas, USA were used to derive stability relationships between slope height, discontinuity dip and discontinuity spacing. Slopes failed by sliding and toppling failure mechanisms. It was observed that when using deformable blocks, the range of instability was far greater and that slopes fail by a flexural toppling failure mechanism. The distribution, density, inclination, arrangement and friction angle of discontinuities have also been identified as governing toppling instability in slopes (Jiang et al., 1995). Using a theoretical UDEC simulation of hard rock masses containing rigid blocks, joint geometrical characteristics define the failure mechanisms of steep slopes (Kimber et al., 1998). The possibility of a toppling failure from horizontally bedded masses was identified and linked with field observations from the Isle of Purbeck, Dorset (Allison and Kimber, 1998). It is thought that toppling can occur for a greater range of conditions than originally considered (Cruden, 1989).

2.6 Conclusion

This chapter describes the methodology for the determination of relevant rock slope parameters for modelling the behaviour of jointed rock masses. To model the development of steep slopes, methodologies and techniques need to be combined from environmental modelling, geomorphology, geology, rock mechanics and computing. The approach used for this thesis treats models as representations of reality with the aim of increasing understanding of steep slope landforms by an appraisal of the sensitivity and influence of rock mass parameters. Geomorphologists have made advances by using material property information in landform studies, particularly when studying slopes composed of soft materials. Hard rock slope research accounts for the geometry of the discontinuities which cut a mass as well as the strength of intact blocks. Failures of jointed rock masses occur by the sliding or toppling, or sliding-and-toppling, of rock blocks along discontinuities.

The measurement and analysis of discontinuities is well established in engineering geology, although there are difficulties with the inclusion of joint strength properties in rock mass models. Use can be made of the various rock mass classification techniques to examine rock masses, but this study will analyse the influence of joint strength on rock slope development, and consider the accuracy of joint strength information required. There is also much discussion about the most suitable method for determining the strength properties of intact rock. The most useful parameters for understanding the response of rock blocks under stress from surrounding material in a geomorphological situation are the elastic properties. Where possible use will be made of data derived from sonic wave propagation of rock blocks, although analysis will account for the need for accurate data, and whether cheap and quick techniques, such as the use of the Schmidt hammer and secondary data sources, will suffice.

Previous studies of rock mass stability and rock slope form have contributed much understanding by considering relevant rock mass parameters or by weighting the controls in classification techniques. It is difficult to consider the characteristics of jointed rock masses together, but the use of the UDEC code offers the advantage that data can be synthesised in a deductive, scientific manner.

Chapter 3: Modelling approaches

3.1 Development of rock mass models used in engineering

The difficulty of predictions of the engineering response of jointed rock masses derives largely from their discontinuous and variable nature. A decision needs to be made based on the scale of the problem between a discontinuous and a continuous modelling approach (Brown, 1987). Purrer (1997) has demonstrated that some state-of-the-art computer models are not capable of determining the relevant failure mechanisms as numerical methods are fed with variations of rock parameters. It is suggested that the first step should be to analyse a rock mass based upon qualitative scenario investigations. The most suitable approach for numerical models is not as a complete representation of reality, but to provide an insight into the physical phenomena of a rock mass and suggest areas where more information is needed (Lemos, 1990). The ISRM (1988) and Spink (1998) have described computer programs developed by universities, research institutions and companies for modelling rock mechanics problems. The three most common numerical methods in rock mechanics are the finite element, boundary element and discrete element methods (Pande *et al.*, 1990). The boundary element method applies to sub-surface rock mass problems and is not relevant here.

Early development of rock mass models included the design of physical, scale representations. Two approaches have been used to build physical representations of jointed rock masses. The base friction modelling approach uses a set of physical blocks with gravity being simulated by the drag of a belt moving along the underside of the model (Bray and Goodman, 1981). The tilt table method has been used to simulate the toppling of blocks contributing to a quantitative understanding (Pritchard and Savigny, 1990) and to determine the friction angles of various rock types (Bruce *et al.*, 1989). Lemos (1990) built a horizontal section through an arch dam abutment with 1,300 plaster blocks, exerted a load by a hydraulic jack and measured displacements. However, it has been suggested that physical models lack the ease, flexibility and quantitative basis of other rock mechanics models (Pritchard and Savigny, 1990).

Limit equilibrium methods can model multiple intersecting discontinuities and determine a factor of safety (*Fs*) for a particular set of conditions. Vector analysis is used to establish whether it is kinematically possible for any block in a blocky system to

move and become detached from the system without reference to the forces which cause blocks to move. Once the mode of failure is known, it is then possible to evaluate the probability of failure, although only minimal reference is made to the strength parameters of the block. The limit equilibrium method can be applied by approximating a slope as a series of columns (Cruden and Eaton, 1987; Goodman and Bray, 1976). The failure mode for each block is determined by solving two statics problems, one assuming block sliding and one rotation, with the higher resultant force indicating the mode of failure (Pritchard and Savigny, 1990). Thus, it is not possible to consider blocks failing by a simultaneous toppling and sliding failure mechanism. The resultant is then applied to the next block all the way to the toe. Due to the calculation procedure, the method is effective for rock mechanics problems with very simple block geometries, although it has been applied successfully to some scenarios (West, 1996).

The limit equilibrium method has been verified by analysing a physical laboratory model of a rock mass with two discontinuity sets (Aydan et al., 1989). Under simple conditions, the limit equilibrium approach was concluded to be valid and effective. Geological data sets collected from an open pit uranium mine have been converted in a block generator software, RESOBLOCK, before being input into a block stability analysis using the limit equilibrium method (Baroudi et al., 1990). Two-dimensional analysis of rock blocks resting on a stepped failure surface in an opencast operation in limestone was carried out using limit equilibrium analysis of rock blocks (Scavia et al., 1990). The method has also been applied to the multiple block toppling failures in order to stabilise three slopes (Wyllie, 1980) and to analyse the stability of Ravedi's Dam, Italy (Lunardi et al., 1995).

Limit equilibrium methods have been applied to the geomorphological study of soil slope stability. For such uses, individual results are of limited value, but, when combined with experience of application in similar conditions and extensive field survey, the results are a useful input to the decision making process (Nash, 1987). Carter and Lajtai (1992) compared two limit equilibrium method approaches for analysing the stability of rock masses. The deterministic approach evaluates the stability of a single rock wedge through three-dimensional vector algebra, whereas the probabilistic approach evaluates the probability of failure for a whole rock slope by examining the distribution of factors of safety from all potential wedges. However, it has been

suggested that the limit equilibrium method is restricted to the analysis of small scale toppling where the process is limited by a planar failure surface and failure is facilitated by joint shear and separation (Pritchard and Savigny, 1990).

The kinematic basis of block theory for the determination of 'key blocks' in a slope was introduced by Goodman and Shi (1985). It was developed to provide a theoretical basis for decision-making and requires knowledge of the three-dimensional geological structure and only the friction angle of rock joints. However, many types of rock mass are not amenable to analysis by block theory or discontinuous deformation analysis (Goodman, 1990; 1995). It is now possible for computer programs to find key-blocks directly from joint maps of engineering sites. The distribution of keyblock volumes and the number of keyblocks per square metre along a tunnel excavation has been estimated using a kinematic program (Shapiro and Delport, 1991).

A computational procedure to analyse the stability of a single three-dimensional block based on vector analysis of the block was introduced by Warburton (1981). The block could be any arbitrary polyhedron, although possible movements are limited to translation only. Basic kinematic analysis of rock slopes are often computational versions of the conventional stereographic projection method, but can efficiently analyse joint sets to identify the failure mechanisms of plane sliding, wedge sliding and toppling (Leung and Kheok, 1987). The benefit of kinematic analysis is that there is much less computational effort and that a good understanding can be gained by using a small number of blocks (Lin and Fairhurst, 1988).

There are two basic classes of computational methods for the modelling of discontinuous rock masses (Brown, 1987). Differential continuum methods, such as the finite difference and element methods, require approximations to be made throughout the problem domain and discontinuum methods require approximations to be made only on a boundary. The continuum, implicit approach uses a single, effective medium defined to deform like an assemblage of blocks and joints whereas the discrete, explicit approach models motions across joints and between blocks separately (Senseny and Simons, 1994). Continuum models have also been successfully applied to the modelling of water movement in a jointed rock medium (Peters and Klavetter, 1988).

Many computer programs based upon a continuum mechanics formulation can simulate the variability in material types and nonlinear constitutive behaviour typically

associated with a rock mass. A finite element problem domain is divided into discrete elements which provide a physical approximation to the continuity of displacements and stresses and the governing equations are solved for nodes (Brown, 1987). Finite modelling methods require that physical or mathematical approximations be made throughout a bounded region. Many limitations of the limit equilibrium method are overcome by using the constitutive relations of the intact rock mass and the joints being more realistically modelled (Pritchard and Savigny, 1990). Also, pore pressures can be introduced and any geometry, geology and loading history can be used. Finite difference methods are also available but are more widely used in solving dynamics problems and approximate numerical solutions are obtained to the governing equations at an array of points within the problem domain (Brown, 1987).

In an early application of the finite element method, it was demonstrated that for a high groundwater condition, an otherwise stable slope at Hell's Gate Bluffs, British Columbia, failed by a toppling mechanism (Kakani and Piteau, 1976). The method has been verified by comparison with physical model tests (Zhu and Wang, 1993). Duncan (1996) used a finite element analysis to assess the stability of soil slopes of dams and embankments. The principal requirement for accurate results is the suitable representation of the stress-strain behaviour of the solid involved. The finite element method has been used to model the toppling of rock slopes due to the tensile breaking of rock columns (Adhikary et al., 1996) and the study of a symmetrical wedge of rock in a rectangular set (St John, 1971).

Slopes sometimes show aspects of a continuum as well as aspects of a discontinuum and finite methods are limited in dualism situations where a rock mass overlies a softer base (Poisel, 1990). However, finite element methods have been combined with distinct element methods to analyse slopes where part of the failure occurs along discontinuities and part of the failure within blocks (Pan and Reed, 1991). Non-linear joint equations have been introduced to a finite element analysis in order to model the Malpasset dam failure (Steger and Unterberger, 1990). This approach overcomes some problems where a competent rock mass lies on a incompetent base. A full understanding of rock slopes can be achieved only by exploiting all the model properties (Selby et al., 1988). Problems with finite element or difference modelling approaches are due to the boundary definition and the inclusion of field stresses which

leads to increased data preparation and computing (Brown, 1987). Also the finite element approach is limited when a rock slope failure occurs as the whole continuum matrix needs reforming (Pritchard and Savigny, 1990). A modelling approach which allows for the change in form and associated change in behaviour of rock slopes through time is more useful for the geomorphological study of rock mass landforms.

3.2 The Universal Distinct Element Code (UDEC)

The most comprehensive, powerful and versatile discontinuum theory available is the distinct element method (Brown, 1987). The distinct element method was developed by Cundall (1971) and it simulates the response of a jointed rock mass, represented as an assemblage of discrete, rigid or deformable, blocks, under loading. UDEC, the commercially available distinct element method code, was introduced in 1985 (Lemos *et al.*, 1985). The method is best used to model the progressive failure of rock slopes where block size is a key scale on the problem. UDEC is ideally suited to study potential modes of failure directly related to the presence of discontinuous features. The advantages of the distinct element approach are that there is no limit to the amount of displacement or rotation of blocks; progressive failure occurs; and the program allows the individual study of the effects of joint geometry, joint parameters, loading conditions and excavation procedure (Cundall, 1971). Other methods assume that the intact properties of the rock and the joint stiffness of the joints play a negligible part in the processes of failure of rock masses.

The distinct element method is a class of discrete element program (Cundall, 1990). The name 'discrete element method' applies if a program allows finite displacements and rotations of discrete bodies and recognises new contacts automatically as the calculation progresses (Konietzky *et al.*, 1994). The rotation of blocks within and from a rock mass is an important jointed rock mass mechanism, and the second attribute allows the modelling of large numbers of blocks whose interactions are not known in advance. UDEC was originally developed to perform stability analysis of jointed rock slopes. The discontinuum formulation for rigid blocks and the explicit time-marching solution of the full equations of motion facilitate the analysis of progressive, large-scale movements of slopes in blocky rock. UDEC has also been used widely in studies related to mining engineering, underground construction and fluid

flow through rock beneath gravity dams. It can capture the anisotropic, scale-dependent behaviour of jointed rock and it has been demonstrated that insight is given on the estimation of the representative elementary volumes of rock masses (Kulatilake and Swoboda, 1994).

Many finite element, boundary element and Lagrangian finite difference programs have interface elements that enable them to model a discontinuous material. However, their formulation is usually restricted to small displacements; there are problems when many intersecting joints are used and there is no way of recognising new contacts. The limit equilibrium method computes the static force equilibrium of the bodies and does not address the changes in force distribution that accompany displacements of the bodies. It has been demonstrated that UDEC can accurately represent an assemblage of physical blocks, deriving the same conclusions more rapidly (Lemos, 1990).

Senseny and Simons (1994) compared results between two distinct element and three finite element codes for a problem involving stress-wave loading of a lined circular tunnel in a jointed medium. The best results came from the distinct element method model where the mechanics for intact rock and joints were both considered. Pritchard and Savigny (1990) suggested that the main disadvantage of UDEC was the small time step leading to a long run time. However, advances in computing capabilities mean that the code can now be run on a conventional personal computer.

3.2.1 The UDEC calculation procedure

UDEC simulates the response of a jointed rock mass under loading. The motion of the blocks along the discontinuities is governed by linear or non-linear force-displacement relations for movement in the normal and shear directions solved by a Lagrangian calculation scheme. The program uses explicit time-marching to solve the equations of motion directly. UDEC has several built-in material behaviour models, for both the intact blocks and the discontinuities, and blocks can be made deformable by sub-dividing into a mesh of finite difference elements. The code assumes a two-dimensional plane strain rate. This condition is associated with structures with constant cross-section acted on by loads in the plane of the cross-section. Discontinuities are considered as planar features oriented normal to the plane of analysis. UDEC is able to simulate the

flow of fluid through the discontinuities and voids in the model, the transient flux of heat in materials, linear inelastic behaviour of joints and plastic behaviour and fracture of blocks (Lemos *et al.*, 1985). The user can generate plots of the model and any problem variable and histories of change of a variable as a function of calculation step can be recorded. Sequences of model output can be stored and replayed as a 'movie'. Thus, it is possible to monitor the failure of a rock mass.

In the distinct element method, a rock mass is represented as an assemblage of discrete blocks and joints as interfaces between distinct bodies (Pritchard and Savigny, 1990). UDEC uses dynamic relaxation techniques to solve Newton's laws of motion in order to determine the forces between, and displacements of, blocks during the progressive, large-scale deformation of jointed rock masses (Brown, 1987). The forces and displacements at the joints in a stressed rock mass are related to the movements of blocks. Movements result from the propagation through the rock mass of disturbances caused by applied loads. The speed of propagation depends upon the physical properties of the discrete system. The dynamic behaviour is represented numerically by a time-stepping algorithm in which the size of the timestep is limited by the assumption that velocities and accelerations are constant within the timestep. The timestep is unrelated to explicit time (Iofis *et al.*, 1990). For rigid blocks, the block mass and the joint stiffness between blocks define the time-step limitation (Itasca, 1993).

The task of the solution scheme is to determine a set of displacements that will bring all elements to equilibrium or indicate the failure mode (Cundall, 1987). The dynamic relaxation calculations performed in UDEC alternate between the application of a force-displacement law at all joint contacts and Newton's second law at all blocks (Senseny and Simons, 1994). The force-displacement law is used to find contact forces from known displacements and Newton's second law gives the motion of the blocks resulting from the known forces acting on them (Cundall, 1971) (Figure 3.1). Each timestep produces new block positions which generate new contact forces. Resultant forces are then used to calculate linear and angular accelerations of each block. Block displacements are then determined by integration over increments in time. Dynamic relaxation is physically more realistic than other relaxation schemes, but it requires more computational effort (Cundall, 1987). With a mass being acted on by a varying

force, $F = F(t)$, Newton's second law of motion can be written as

$$\frac{dv}{dt} = \frac{F}{m}$$

where v = velocity,

t = time and

m = mass.

The distinct element method uses a central difference scheme with calculations ordered. The force / displacement calculation is done at a time instant, with velocities stored at the half-timestep point. Displacement, u, at the half-timestep point:

$$u^{(t + \Delta t)} = u^{(t)} + v^{(t + \Delta t/2)} \Delta t$$

For blocks which are acted upon by several forces, including gravity, the velocity becomes

$$v_i^{(t + \Delta t/2)} = v_i^{(t-\Delta t/2)} + (\frac{\sum F(t)}{m} + g_i) \Delta t$$

$$\theta^{(t + \Delta t/2)} = \theta^{(t - \Delta t/2)} + (\frac{\sum M(t)}{I}) \Delta t$$

where θ = angular velocity of block about centroid,

I = moment of inertia of block,

M = moment acting on block,

v_i = velocity components of block centroid and

g_i = components of gravitational acceleration (body forces).

The indices i denote components in a Cartesian coordinate frame.

The new velocities are used to determine the new block location according to

$$x_i^{(t + \Delta t)} = x_i^{(t)} + v_i^{(t + \Delta t / 2)} \Delta t$$

(3.2.1 - 4)

$$\delta^{(t + \Delta t)} = \delta^{(t)} + \theta^{(t + \Delta t / 2)} \Delta t$$

where δ = rotation of block about centroid and

x_i = coordinates of block centroid.

The equations used in UDEC are based on the spring-loaded physical interaction of bodies (Figure 3.1). The laws of conservation of momentum and energy are satisfied exactly.

Different representations of joint material behaviour are available. The basic model, which was used for this study, is the Coulomb slip criterion, which assigns elastic stiffness, friction, cohesive and tensile strengths and dilation characteristics to a joint. A rock joint is represented numerically as a contact surface between two block edges (Cundall and Hart, 1992). With rigid blocks, contacts are created at block corners, with deformable blocks, contacts are created at all gridpoints located along the block edge (Lemos, 1994). An unrealistic response can occur when block interaction occurs close or at two opposing block corners as a result of the assumption that block corners are sharp or have infinite strength. In reality, the crushing of a block corner would occur due to stress concentration. However, a realistic numerical representation can be achieved by rounding the block corners. Contact points are updated automatically during UDEC operation as block motion occurs. Algorithm efficiency is maintained by using domains of contacts which allows for a large number of blocks to be modelled. Problems therefore occur when blocks become detached from the rock mass as the domain structure is ill-defined. It is suggested that blocks which become disconnected from the main rock mass during the modelling process should be deleted (Itasca, 1993).

For the basic joint behaviour model used in UDEC captures several features which are representative of the physical response of joints (Itasca, 1993). In the normal direction, the stress-displacement relation is assumed to be linear and governed by the

joint normal stiffness k_n such that

$$\Delta\sigma_n = k_n \, \Delta U_n$$

where $\Delta\sigma_n$ = effective normal stress increment and

ΔU_n = normal displacement increment.

There is a limiting tensile strength for the joint. If the effective normal stress exceeds the limiting tensile strength, then there is no displacement. In shear, the joint response is governed by a constant shear stiffness, k_s. The shear stress, τ_s, is limited by a combination of cohesive (c) and frictional strength (ϕ). Thus, if

$$|\tau_s| \le c + \sigma_n \tan \phi = \tau_{max},$$

then

$$\Delta\tau_s = k_s \, \Delta U_s^e,$$

where ΔU_s^e = the elastic component of the incremental shear displacement.

In addition, joint dilation may occur at the onset of slip and is limited by a high normal stress level, or by large accumulated shear displacement. This corresponds to the crushing of asperities which prevents a joint from dilating.

Blocks may be defined as rigid or deformable in the distinct element method. The formulation represents rigid blocks as a set of distinct blocks which do not change their geometry as a result of loading. Rigid blocks are most applicable where the behaviour of a rock mass is dominated by discontinuities and for which material elastic properties may be ignored. Such conditions arise in low stress environments and / or where the material possesses high strength and low deformability (Senseny and Simons, 1994). For the purposes of this thesis blocks were defined as rigid. However, experiments were undertaken to consider the effect of blocks that are allowed to deform in shape. The UDEC code has seven built-in material models for deformable blocks. Fully deformable blocks are discretised into finite difference triangular elements. The iterative cycle is modified slightly in that the displacements of grid points within the blocks are linked to the displacements of grid points forming the block boundaries

(Pritchard and Savigny, 1990). Thus, the complexity of the deformation of blocks depends upon the number of elements into which the block is divided. Plane-strain conditions are assumed and the equations of motion for each gridpoint are

$$\ddot{u}_i = \frac{\sum \sigma_{ij} n_j ds + F_i}{m} + g_i$$

(3.2.1 - 8)

where s = the surface enclosing the mass m lumped at the gridpoint,

n_j = the unit normal to s,

F_i = resultant of all external forces applied to the gridpoint and

g_i = gravitational acceleration.

During each timestep, strains and rotations are related to nodal displacements in the same fashion as for the rock blocks. The constitutive relations are used in incremental form:

$$\Delta \sigma^e_{ij} = \lambda \, \Delta \varepsilon_v \, \delta_{ij} + 2 \, \mu \, \Delta \varepsilon_{ij}$$

(3.2.1 - 9)

where λ, μ = the Lamé constants,

$\Delta \sigma^e_{ij}$ = elastic increments of the stress tensor,

$\Delta \varepsilon_{ij}$ = incremental strains,

$\Delta \varepsilon_v$ = increment of volumetric strain and

δ_{ij} = Kronecker delta function, δ_{ij} = 1 iff $i = j$.

Of the seven failure models for the deformable blocks, the basic model is the Mohr-Coulomb failure criterion. Other non-linear plasticity models available in UDEC include an elastic / plastic Drucker-Prager failure model. UDEC is primarily intended for the failure along joints within a rock mass. However, in many problems, the failure and collapse of intact material must be incorporated in the model. As different intact block failure models can be used within one model mesh, it is possible to simulate masses composed of both hard and soft rock. It is important to recognise that failure of the intact material may overestimate collapse load due to mesh locking. A method used in codes designed to model softer material is mixed discretisation. In UDEC, an alternative zoning generator creates diagonally-opposed triangular elements in blocks.

Damping is used in the distinct element code to solve mechanical problems (Cundall, 1971). Since an elastic system would continue to oscillate forever, damping must be provided so that the steady state is approached (Cundall, 1987). The equations of motion are automatically damped to reach a force equilibrium state under the applied conditions. The magnitude of the damping force is proportional to the velocity of the blocks. The viscosity of the damping is a constant proportion to the rate of change of kinetic energy in the system, with the adjustment to the viscosity being made by keeping the following ratio, R, equal to a given ratio (Cundall and Strack, 1979):

$$R = \frac{\sum P}{\sum E_k}$$

(3.2.1 - 10)

where P = the damping power for a node, and

E_k = the rate of change of nodal kinetic energy.

The velocity-proportional form of damping has the advantage that damping-induced body forces, which may erroneously influence the mode of failure, are reduced because the damping power tends to zero as the system approaches steady state.

The solution scheme for the distinct element method is only stable at the end of a timestep. A limiting timestep is determined that satisfies both the stability criterion for calculation of internal block deformation and that for inter-block relative displacement. The timestep required for the stability of block deformation computations is

$$\Delta t_n = 2 \min (m_i / k_i)^{1/2}$$

(3.2.1 - 11)

where m_i = mass associated with the block node i and

k_i = a measure of stiffness of the elements surrounding the node, affected by the material elastic properties.

For calculations of inter-block relative displacement, the limiting timestep is calculated as

$$\Delta t_b = (frac)\ 2\ (M_{min}\ /\ K_{max})^{1/2}$$

(3.2.1 - 12)

where M_{min} = mass of the smallest block in the system and

K_{max} = maximum contact stiffness.

The term 'frac' accounts for the fact that a single block is in contact with several blocks. The controlling timestep is therefore

$$\Delta t = \min\ (\Delta t_n,\ \Delta t_b)$$

(3.2.1 - 13)

3.2.2 The operation of UDEC

The UDEC software is operated by a series of ordered and structured commands. The input command procedure commences with the creation of a rock mass form and joint geometry. The second stage defines the material properties and behaviour and specifies the boundary and initial conditions. The model is then run to a mathematical equilibrium state, which effectively simulates the process of block consolidation within a fixed mesh. Finally, fixed model boundaries are released to allow rock mass failures to occur.

The creation of a mesh begins with a single block which spans the physical region being modelled. The initial block is cut by joints and regions can be deleted to create the model mesh form. The corners of blocks are rounded in order to prevent large stress concentrations occurring at block corners. A rounding distance of between 0.1% and 1% of the block length was generally used in this study. An automatic joint set generator within UDEC is invoked in order to define a two-dimensional joint mesh across the model block. By inputting mean joint set characteristics, the joint generator creates a pattern for the whole mesh. Joint parameters required are dip angle on the mesh from horizontal, joint continuity, gap length between discontinuous joint segments and joint spacing normal to joint tracks. For each of the parameters a maximum random

deviation from the mean can be assigned and, as with every UDEC command, if a parameter is not defined, a default value of zero is given.

Individual blocks and defined regions of blocks may be deleted from the mesh in order to assimilate the morphological conditions of the modelled profile. Typically, a region on the left-hand-side of a model may be deleted for jointed rock slope problems in order to outline the slope free face and the edge and toe of the slope. It is recommended that very small blocks, with size less than 0.01% of the whole mass, which are created in the joint mesh are deleted in order to increase calculational efficiency. For N blocks, or deformable gridpoints, defined in the UDEC modelling region, the computation time is proportional to $N^{3/2}$. Very small blocks do not play any role in the failure of a rock slope. Furthermore, small blocks with tight acute angles which may form in the centre of a rock mass would be crushed in the real-world due to the stress build-up in the block corners. Numerically the blocks with such rounded corners may overlap, so it is best if they are deleted.

A further issue which is encountered when using UDEC for rock slope problems is the mapping of a three-dimensional joint set of planes on to the two-dimensional mesh (Jing and Stephansson, 1994). The dip of a joint set as represented by a concentration of poles on a stereographic projection is a value which is fixed by the dip direction in three dimensions. Unless the strike of the UDEC model profile is perpendicular to the strike of the joint set, then the dip value on the UDEC mesh is different. For instance, a real joint set may be dipping at an average angle of 45° to the south. If it is desired to model a cliff which has a north-south profile, then the dip of the joint set on the mesh would be at 45° above the horizontal. However, if the cliff has an east-west profile, then the represented dip on the mesh would be horizontal. A computer program was written to transform joint sets defined by a stereographic strike and dip and a defined UDEC mesh. The program is based on a vector approach and the angle of intersection between two planes (Appendix 3.1). In most rock slope problems encountered in this study, the bearing of the UDEC profile is usually taken to be perpendicular to the free face strike of the cliff being modelled, although where a headland or embayment plan feature is modelled, the UDEC profile bisects the centre of the feature.

Once the UDEC mesh has been defined, material behaviour models and properties need to be assigned for all blocks and discontinuities. Deformable blocks can be created by defining the length of one side of a mesh of finite difference triangular zones for each block. For most slope failure models in hard rock, a rigid block assumption may be applied. There are seven material models available to UDEC users which can model different types of deformable block. Different parts of the UDEC mesh may be assigned different types of material model. For most rock slope problems, where failure is of hard rock blocks along discontinuities, the default elastic model is appropriate. The required properties for this model are density, bulk modulus K and shear modulus G. The derivative of bulk modulus and shear modulus from Young's modulus and Poisson's ratio is given in Section 2.3.2. In addition to the block material models, a material model and properties must be assigned for all discontinuities in the model. There are four built-in constitutive models which can be used to represent different types of discontinuity. The most appropriate model for rock slope problems is the default elastic-perfectly plastic joint area contact Coulomb slip model which is used in this study. The properties required for this model are normal stiffness, shear stiffness, friction angle, cohesion, dilation angle and tensile strength (Section 2.3.2). There may be problems of block interpenetration if the specified joint normal stiffness is very low. However, solution convergence will be very slow if high stiffness is specified. The joint normal stiffness, k_n, and the joint shear stiffness, k_s, should be set to at most ten times the equivalent stiffness of the stiffest neighbouring zone:

$$k_n \text{ and } k_s \leq 10.0 \text{ x max } (\frac{K + 4/3G}{\Delta Z_{min}})$$

(3.2.2 - 5)

where ΔZ_{min} = the smallest width of an adjoining zone in the normal direction.

Once the block cutting is complete and the material models and properties have been assigned, boundary and initial stress conditions must be applied. For rock slope problems, it is common to apply zero velocity boundary conditions to the sides and lower part of the model. This has the effect of spatially fixing the boundary of the model. The model boundary must be far enough away from the region of study so that model response is not influenced adversely. Once the blocks have consolidated in the mesh and the model has reached equilibrium, it is then possible to free one of the two

side boundaries to allow failures to develop. Initial stresses must be specified for all blocks in the model as there is an *in situ* state of stress acting in any real world rock mass. For a rock slope, the vertical component of a stress force on a particular block is calculated from the weight of the material above that block and is $g\rho z$, where g is the gravitational acceleration, ρ is the mass density of the material, and z is the depth below the surface. UDEC automatically creates a stress gradient through the model depending upon a block's relative position within the model and relating a gravitational force to the density of blocks. Care is required when different layers in a rock mass have different densities. The *in situ* horizontal stresses are more difficult to estimate and can be considered in terms of gravity being applied to an elastic mass which cannot move laterally. However, the condition does not often apply due to the history of the landform which may have been affected by tectonic stresses, failure and removal of material and locked-in stresses. It is typical to compromise. A set of horizontal initial stresses is generally taken to be the recommended half of the vertical stress (Herget, 1988; Itasca, 1993). By the time that the model is run to equilibrium and the system changed for the study of a landform, stresses will be reasonable for the situation.

The UDEC model must be at an initial force-equilibrium state before fixed boundaries can be freed to allow rock mass failures to develop. In order to reach equilibrium, the model is run to a user-specified number of calculation steps. The model is at equilibrium when the net nodal force vector at each centroid of rigid block is zero. The maximum nodal force vector, called the unbalanced force, can be monitored in order to assist the user with the decision. One way to think of the process is to use the analogy of a quantity of building blocks being instantaneously placed into a box. The box acts as a fixed boundary, the blocks have a varied amount of stress upon them due to the weight of surrounding blocks and will take a little time to settle. Once the blocks have settled, one side of the box could be cut away to allow blocks to fall out of the box if unstable conditions occur. Once the boundary alteration is made, the model can be run for a specified number of time steps.

When entering data into UDEC, it is important to follow a consistent magnitude and unit convention for parameter values. In all cases, SI units are used. Data are converted into metres for a length property, kg m^{-3} for density, Newtons for force, Pascals for stress, m s^{-2} for gravity and Pa m^{-1} for stiffness. Commands are entered into

an edited data file which can be called into the UDEC program. It is possible to record all commands and code messages from a UDEC work session into a log file for future reference. There are several forms of output which can be used to assess the state of the numerical model. Save files can be made during the process which completely record the model's status at any point and can enable status to be restored. At any point the user can plot the blocks on the screen or the boundary of the model with other variables, such as block displacement, superimposed. History files and plots can be made for the quantitative response of any variable through the modelling process.

Commonly examined variables include the value of unbalanced forces acting on a block which is displayed on the screen as the model is run. The total unbalanced forces can also be saved as a history and displayed as a graph. The magnitude of unbalanced forces is important for analysis. A stable rock mass will still have reactionary unbalanced forces acting at gridpoints and block contacts, but the forces will be generally seven or eight orders of magnitude below the forces for an unstable rock mass. The displacement of individual blocks and the gridpoints of deformable blocks may be assessed by plotting the whole field of velocities onto a plot of blocks, or by selecting certain points in the model and tracking their velocities with histories (Starfield and Cundall, 1988). Steady state conditions are occurring if the history plot is horizontal. If the history plot is close to zero, then equilibrium has occurred for the model. Again, the magnitude of the velocity vectors needs to be considered. A block field of randomly oriented velocity vectors with low magnitude is an infallible indicator of stability. If the vectors in the velocity field are coherent and their magnitude is quite large, then blocks are failing, or plastic flow is occurring within blocks, or the system is still adjusting elastically. Failing blocks are easily identified by associated displacement. Movie files can be created to capture any graphics images which appear on the screen during the modelling process. Play back of movie images may enable the user to view the succession of model block plots which show the development of a failure mechanism.

The operation and capability of UDEC can be demonstrated by following a simple example of a slope in which an engineering cut is made. A model slope was created at angle of 45°, with the two-dimensional modelled profile having a depth of 80 metres and a height of 50 metres. Two joint sets were automatically generated; one

dipping at 70° into the slope face and the other dipping at 40° out of the slope face. The model was then run for 10,000 steps in order to reach equilibrium. The block displacement plot shows randomly oriented vectors of a very low magnitude (Figure 3.2) and the history of total unbalanced forces (Figure 3.3) indicates that the modelled mesh has consolidated and is at equilibrium. At this point an engineering cut of 10 m is made in the lower part of the slope and the model is allowed to run again. By 105,000 steps (Figure 3.4) it is clear from the block displacement vectors that the blocks are failing by a sliding mechanism. Blocks are sliding on the 40° joint set into the engineering cut. By 955,000 steps (Figure 3.5), it is clear that the modelled mesh has reached equilibrium again and motion is prevented by a key stop block. At this point, the modelling exercise is completed, unless further changes are made to the cliff form or joint geometry.

3.2.3 Verification and applications

Code verification is a necessary part of the development of a rock mass modelling approach but is difficult (Brown, 1987) and there have been relatively few comparisons of numerical results of UDEC (Lemos, 1990). Often the only method of verification is by controlled physical tests, as field tests are not suitable as material properties, boundary conditions and loading conditions cannot be accurately defined, controlled or reproduced. Lemos (1990) compared the numerical and physical results of the deformation and failure of a block system and found that good agreement of the displacement field could be achieved, provided that low values of joint stiffness were used in order to account for the lack of fit of the blocks in the physical test. It has also been demonstrated that the distinct element method modelled the complex behaviour of an assembly of discs accurately (Cundall and Strack, 1979). The force vector diagrams closely resembled those obtained photoelastically. Video tracking for UDEC validation has also been used for laboratory experiments (Hryciw et al., 1997).

Brady et al. (1990) confirmed that UDEC can represent discontinuous deformation of jointed rock and continuous deformation of individual blocks. In a simple problem involving the loading of a block transected by an inclined joint, the UDEC analysis produced results for the stiffnesses which were virtually identical with the independent, closed-form solution. Pritchard and Savigny (1990) used UDEC to

model three rock mechanics verification problems. The limiting surface of failure of a small, toppling base friction model was closely approximated by the distinct element code. The limit equilibrium analysis performed by Goodman and Bray (1976) was repeated to demonstrate that UDEC could accurately reproduce the known solution for limiting stability. Also, the failure of an opencast mine working was reproduced. Despite the difficulty of comparing UDEC results with field observations, known characteristics of the rock mass retreat correlated well. Excellent agreement between UDEC predictions and laboratory measurements for joint shear displacement for the loading of a 2 m cube of Precambrian gneiss has also been described (Chryssanthakis *et al.*, 1991).

Much advance in the understanding of rock slope failure using UDEC has been made using theoretical slope models and an experimental, parameter sensitivity approach is used initially as part of this study. Hsu and Nelson (1995) modelled weak rock masses with a single discontinuity set and related cliff stability to cliff height, cliff angle, discontinuity dip and discontinuity spacing. Theoretical models have also suggested that toppling slope failure mechanisms are influenced by discontinuity density and friction angle (Jiang *et al.*, 1995). The influence of joint-geometry parameters such as joint density, ratio of joint size to block size and joint orientation on the deformability of jointed rock is also great (Kulatilake *et al.*, 1994). The importance of the choice of joint constitutive model on the stability of jointed rock slopes has also been demonstrated (Souley and Homand, 1996).

An important application of the UDEC software in simulating real-world slopes involves the understanding of failure mechanisms. Models using deformable blocks, despite the low stress environments, have proved to be effective in illustrating the type and mechanism of block displacement of undermined Loire Valley chalk cliffs near Saumur (Homond Etienne *et al.*, 1990). The toppling of the Heather Hill Landslide, British Columbia, Canada has been simulated using the distinct element method (Pritchard *et al.*, 1990; Pritchard and Savigny, 1991). It is demonstrated that the UDEC approach can be used to provide a good understanding, despite generalisations of the discontinuity pattern. Further landslide hazards in south-western British Columbia have also been investigated using this technique (Savigny and Rinne, 1991). By allowing for the rotation of blocks, UDEC can accurately simulate toppling failures. It has been used

to model toppling structures in welded tuff near to Mount Ontake, Central Japan (Ishida, 1990; Ishida *et al.*, 1987) and the failure mechanisms of a slate quarry at Delabole, Cornwall (Coggan and Pine, 1996). Here, distinct planar failure surfaces were identified and modelling confirmed the detrimental influence of a raised water-table on the stability of the slope.

The influence of discontinuous joints upon rock mass stability has been noted (Kulatilake *et al.*, 1992). However, in previous versions, UDEC did not directly model discontinuous joints, although several parameters could be altered to account for the change in rock mass strength. UDEC has also been applied to the effect of fault slip on fluid flow in rock masses (Chen and Lorig, 1997), to the analysis of a dry rock avalanche triggered by earthquakes (Uchida and Hakuno, 1990) and the ability to model frictional sliding behaviour has been assessed (Lorig and Hobbs, 1990).

Much application of the UDEC software has been to problems in underground rock mechanics. Although different modes of failure are involved, and the timescale differs from geomorphological study, such work provides a useful background and leads to an appreciation of the successful use of the code. Sensitivity analysis has been performed using UDEC in order to provide insights into important deformation mechanisms in a large cavern in the Himalayas (Bhasin and Hoeg, 1998). It was concluded that the size of blocks and joint friction angle were the most important control on cavern failure. The study indicated the importance of completing a parameter sensitivity analysis to develop understanding of a data-limited situation. UDEC models have been created for the proposed Sellafield radioactive waste repository (Barton *et al.*, 1992), the 62 metre span Norwegian Olympic Ice Hockey Cavern at Gjovik (Barton *et al.*, 1994) and the twin, three lane Fjellinjen road tunnels under Oslo (Makurat *et al.*, 1990). They have been used for performance monitoring at the site of an underground powerhouse cavern in the Himalayan region of India (Bhasin *et al.*, 1996). The three-dimensional distinct element code has been used to simulate a sub-level stoping experiment at the Kiruna mine, Sweden (Jing and Stephannson, 1991). UDEC models of mining-induced subsidence in Australia reproduced many important aspects of roof strata behaviour, although it was noted that results depended upon the rock removal path taken (Choi and Coulthard, 1990). Such examples highlight the importance of running a number of possible scenarios for the situation being modelled. Ravi and Dasgupta

(1995) highlighted the usefulness of a UDEC analysis over a continuum analysis in the examination of an underground mine which is based upon the stability of a jointed cap rock. Landforms on the Colorado Plateau examined as part of this thesis are controlled by differences in cap-rock strength. It has also been possible to verify the UDEC software in underground situations because of the rock mass response timescale involved. UDEC simulations of an overstressed borehole and a large cavern have been successfully compared with instrumentation results (Barton *et al.*, 1993) and measured stress distributions (McNeary and Abel, 1993).

UDEC is able to simulate the flow of fluid through discontinuities in a jointed rock mass and it is possible to model a ground-water table within a mass. The quantity of water within a slope is often considered to be an important geomorphological control on landform development (Ahnert, 1966; Gerrard, 1988; Selby, 1993). Several studies have successfully applied the UDEC software to the modelling of fluid flow. Fluid is often injected into geological formations primarily for resource extraction or storage. Hydromechanical behaviour of jointed rock masses involves complex interactions between joint deformations and effective stress, causing changes in aperture and thus hydraulic conductivity (Lemos and Lorig, 1990). The distinct element code was chosen for fluid flow study because it is able to represent explicitly the fractures and to couple mechanical deformation to the fluid flow (Last and Harper, 1990). UDEC modelling has been used to simulate possible conditions in a sandstone reservoir at 3 km depth (Harper and Last, 1989). Results showed that the higher the rate of injection, the greater the number of fluid pathways (Harper and Last, 1990a; 1990b).

An advantage offered by the scientific power and flexibility of the UDEC simulation software is that a number of alternative rock mechanics problems can be analysed. Natural rock columns often assume strange geometries which appear unstable (Hall, 1996). However, UDEC analysis of six mushroom rocks from the Chiricahua Mountains, Arizona demonstrated that none of the columns were close to compressional or tensile failure. Cundall (1990) used UDEC in modelling discontinuity development. It was suggested that very little work has been undertaken before and that by analysing locked-in stresses in rock masses that understanding can be gained of joint spacing, angles and continuity. Barton and Bandis (1990) developed the Barton-Bandis (BB) joint model which has been incorporated into special versions of the UDEC code. The

advantage of the BB joint model is that it can be related to joint roughness coefficient (JRC) measurements, as opposed to joint stiffness parameters (Bandis *et al.*, 1983) (Section 2.4.3). The joint roughness coefficient is easier to measure in the field. Other additions which have been developed for the UDEC code include a reinforcing element model (Choi, 1992). It is possible to model non-linear fault behaviour without defining joint stiffness parameters (Beer and Poulsen, 1994). Also, Chryssanthakis *et al.* (1991) used a UDEC-BB code in order to model a rock mass of 4 km by 4 km loaded by an ice sheet 3 km high and simulated deglaciation. The simulated geometry was similar to that from the Lansjarv region of northern Sweden.

3.3 Conclusion

Data on the strength of intact rocks and discontinuities have been combined in the consideration of the stability of engineered slopes. Computer models now account for much rock mass stability analysis, and a variety of codes are commercially available. The UDEC code has been selected for the study of the development of steep slopes, as the discontinuum approach can model the progressive failure of rock masses due to the geometrical distribution of discontinuities. The time-marching solution scheme and interactive output allow the development of a rock slope to be monitored. The main advantages offered by the software for geomorphological steep slope research are that there is no limit to the amount of displacement or rotation of blocks, progressive failure can be related to changes in slope form and the program allows the individual study of the effects of joint geometry, joint parameters, material properties, block deformability and fluid flow in joints. The UDEC code has a strong mathematical and physical basis and it has been accurately verified and applied to many rock mechanics problems. Previous applications of UDEC provide a useful background to the first geomorphological application.

Chapter 4: Simulations of failure mechanisms in jointed rock masses

Chapter 4: Simulations of failure mechanisms in jointed rock masses

4.1 Background

4.1.1 DeFreitas and Watters' study of failure mechanisms

DeFreitas and Watters (1973) described the conditions for the sliding, toppling and toppling-and-sliding failure of a single, rectangular block standing upon an inclined base. The kinematic failure of the rock block was defined by the block geometrical parameters of the ratio between the base length, b, and height length, h, of the block, the angle of the base plane surface, α, and the angle of friction between the block and base plane, ϕ (Figure 4.1). The paper is regarded as the earliest description of natural slope movements by toppling (Cruden, 1989). The conditions are defined in Table 4.1 and form a continuum between stability and instability.

Block Failure Mechanism	Conditions
Stable block	$\alpha < \phi$ and $b / h > \tan \phi$
Sliding	$\alpha > \phi$ and $b / h > \tan \phi$
Toppling	$\alpha < \phi$ and $b / h < \tan \phi$
Toppling-and-sliding	$\alpha > \phi$ and $b / h < \tan \phi$

Table 4.1:   Forms of single rock block failure mechanism (from DeFreitas and Watters, 1973).

The basic limiting conditions for a rock block failure can be represented as four regions on a graph of base plane angle α against b/h ratio (DeFreitas and Watters, 1973) (Figure 4.2). Blocks which are relatively tall and thin, having a low b/h ratio, are more likely to topple. Blocks which are shorter and wider are more likely to be stable. At the same time, blocks resting on a base plane inclined at a low angle are more likely to be stable. If the base plane angle α is greater than the friction angle ϕ then the block will also slide. Blocks with a low b/h ratio are more likely to topple and slide simultaneously. Three field examples in a variety of rock types at a variety of scales were used to illustrate the toppling mechanism (DeFreitas and Watters, 1973). At

Gowlish Cliff, North Devon, topples formed in interbedded sandstones and shales and at Glen Pean in the Scottish Highlands, topples formed in the metamorphosed Moine Series. However, at Nant Gareg-Lurig, Glamorgan, slopes which are composed of a horizontally bedded sandstone with vertical joints fail by toppling. As toppling should not kinematically occur for such slopes, the explanation is based upon the fact that weaker coal and shale layers exist below the sandstone.

It was possible to use the UDEC computer simulation software to model the failure of a single block standing on an inclined block. By varying the dimensions of the block and angle of the base block, and comparing the result with the DeFreitas and Watters' failure conditions, the use of UDEC can be verified for modelling the failure of blocks. Four models were run to represent each of the failure types defined by DeFreitas and Waters (1973) and block stability and each had a friction angle of 40°. The input files for each of the four models are listed in Appendix 4.1. One model had a block with a *b/h* ratio of 0.8 and was resting on a plane angled at 39°. The UDEC output clearly shows the block toppling (Figure 4.3a). This conforms to the failure mechanism expected for a block with such dimensions. Also resting on a base plane of 39°, a block was modelled with a *b/h* ratio of 1.0. The block, as would be suggested by DeFreitas and Watters, was stable (Figure 4.3b). A block also with a *b/h* ratio of 1.0, but resting on a base plane angled at 41°, fails by sliding (Figure 4.3c), demonstrating the boundary between conditions due to a defined friction angle of 40°. If the *b/h* ratio is reduced to 0.8, but the block still rests on a 41° plane, the block fails by sliding-and-toppling (Figure 4.3d). The four models chosen contained individual blocks with dimensions close to the limiting boundary conditions of the failure of a block as defined by DeFreitas and Watters (1973).

4.1.2 Methodology: the boundary conditions for modelled hard jointed rock mass failure

Natural rock slope behaviour depends upon the dynamic interaction of numerous blocks. The displacement of a block depends upon the weight of overburden and surrounding stresses. Goodman and Bray (1976) suggest that the occurrence of rock mass toppling is not related to the kinematic failure conditions of a single block, but the height of the slope, the thickness of the layers, the strength of the rock and the regularity and spacing

of the joints. It is now possible to reconsider the kinematic limiting conditions for failure of a single block proposed by DeFreitas and Watters (1973) for the failure of a whole rock mass. It would not be possible to study the failure conditions of a rock mass by a kinematic approach as behaviour is controlled by the weight of interacting blocks. Advanced computer simulation techniques, such as the distinct element method, can deal with the dynamics of a modelled jointed rock mass and the resulting failure mechanism can be monitored as output. UDEC is ideally suited to study the potential modes of failure directly related to the presence of discontinuous features. It has been demonstrated that the distinct element method is suited to the analysis of toppling failure as models are consistent with field conditions on Mount Ontake, Japan (Ishida, 1990; Ishida *et al.*, 1987). The ability to model frictional sliding behaviour has also been extensively verified (Lorig and Hobbs, 1990). There is a need to understand the nature of jointed rock mass failure as geomorphologists deal with landforms composed of numerous blocks and have had to assume the conditions of a single block failure in analysis (e.g. Caine, 1982; Dikau *et al.*, 1996; Leung and Kheok, 1987). At the same time there is greater use of the material properties in geomorphic study. A dynamic approach to the study of rock mass failure mechanisms considers the geotechnical properties of the jointed rock mass.

The UDEC computer simulation software was used to examine how the geometrical pattern of discontinuities in a rock mass, rather than the limiting conditions on an individual block, affect the failure mechanism. To compare with DeFreitas and Watters' (1973) calculations for a single block, the same parameters were used in the analysis and a number of models were run for various points on the graph. The main parameters, base plane angle α and the base-to-height (b/h) ratio, were assumed to be constant throughout each modelled rock mass (Figure 4.4). The base plane angle α is represented by the dip of bedding in a rock slope. The b/h ratio was defined by the geometry of the discontinuities in the rock mass. The models were designed with blocks fixed at a height of four metres by fixing the spacing between the bedding planes at four metres. The b/h ratio was altered by varying the spacing between the second discontinuity set used in the model. All other UDEC input parameters were held constant throughout the duration of the exercise so that comparisons could be made.

The command input file for the UDEC computer simulation exercise is listed in Appendix 4.2. In all, 112 meshes were defined and run to fix the boundaries. The model was set up with a 45 metre high cliff inclined at 85° and a profile width of 80 metres. Two joint sets were determined, one which had a dip fixed at 70° and the bedding which varied with the base plane angle α. Rigid blocks were used for the initial part of the exercise, and material properties similar in strength to the Portland Limestone of Dorset (Allison, 1986) were ascribed (Table 4.2). The blocks have a density of 3,000 kg m^{-3} and a joint friction angle, ϕ, of 40° was defined. The default linearly-elastic, isotropic block model and joint area contact with Coulomb slip failure were used. Initially the model was run for 4,000 steps with fixed boundaries to allow the redistribution of forces and block settlement until equilibrium was reached. A side boundary was then freed to allow displacement of blocks. The model was run for another 200,000 steps and the UDEC output was monitored to discern the mechanism of rock mass failure.

Once the conditions of failure in a rock slope had been established, the exercise was extended by deriving further, comparative b/h ratio / α graphs from models with different input parameters. By varying individual UDEC input commands, the graph acts as a template for parameter sensitivity analysis and leads to a further understanding of the controls on rock mass landforms. The theoretical model approach has several advantages: it can isolate the effects of individual parts of the system; it permits the extrapolation of observed processes over longer time-spans; and it can serve as a means to test hypotheses (Ahnert, 1988). Gerrard (1988) considered the characteristics of greatest importance to rock stability to be joint dip, spacing, nature of joint surfaces, thickness, infill material and joint persistence. Much attention has been given to the large control of slope height and angle on rock slope stability (Dikau et al., 1996; Ross-Brown, 1980). The effect from varying the height of the slope (Hsu and Nelson, 1995) and the angle of the slope face (Jiang et al., 1995) for rock masses modelled using UDEC has been considered elsewhere and will not be repeated here. A rock mass with a high slope face and angle is more unstable for given discontinuity dip and spacing values.

Parameters		1	2	3	4	5	6	7	8	9	10	11	12	13	14
Height	m	45	45	45	45	45	45	45	45	45	45	45	45	45	45
Angle	x°	85	85	85	85	85	85	85	85	85	85	85	85	85	85
Joint A: angle (α)	x°	v	v	v	v	v	v	v	v	v	v+/-10%	v	v	v	v
Joint A: space (h)	m	4	4	4	4	4	4	4	4	4+/-2	4	4	4	4	4
Joint B: angle	x°	-70	-70	α - 90	-70	-70	-70	-70	-70	-70	-70+/-10%	-70	-70	-70	-70
Joint B: space (b)	m	v	4	v	v	v	v	v	v	v+/-2	v	v	v	v	v
Bulk density	kg m^{-3}	3000	3000	3000	3000	2320	2650	2320	2650	3000	3000	3000	3000	3000	3000
Joint stiffness	GPa m^{-1}	1	1	1	1	1	1	1	1	1	1	1	1	5	1
Friction angle (ϕ)	x°	40	40	40	60 / 20	29	46	29	46	40	40	40	40	40	40
Bulk modulus	GPa	27.6	27.6	27.6	27.6	9	12	9	12	27.6	27.6	27.6	27.6	27.6	27.6
Shear modulus	GPa	15.8	15.8	15.8	15.8	8	9	8	9	15.8	15.8	15.8	15.8	15.8	15.8
Zones	m	-	-	-	-	-	-	3	2	-	-	-	-	-	-
Persistence		full	full	full	full	full	full	full	full	full	full	v	full	full	full
Water table	m	-	-	-	-	-	-	-	-	-	-	-	35 / 50	-	-
Platform	m	0	0	0	0	0	0	0	0	0	0	0	0	0	50

Table 4.2: UDEC input parameters used for models to define the boundary conditions between failure mechanisms for jointed rock masses and also for parameter sensitivity testing.

A 'v' indicates that the parameter was varied between different models. Tests undertaken correspond with the number in the first row.

Key for Table 4.2:

1.   Models run to define the original boundary conditions for modelled limestone jointed rock mass failure mechanisms (Section 4.2).

2.   The effect of varying block height (Section 4.2.2).

3.   The effect of using rectangular blocks (Section 4.2.3).

4.   Models run to define the original boundary conditions for modelled limestone jointed rock mass failure mechanisms with joint friction angles of 20° and 60° (Section 4.3.1).

5.   Models run to define boundary conditions for modelled sandstone jointed rock mass failure mechanisms (Section 4.3.2).

6.   Models run to define boundary conditions for modelled granite jointed rock mass failure mechanisms (Section 4.3.2).

7.   Models run to define boundary conditions for modelled sandstone jointed rock mass failure mechanisms with deformable blocks (Section 4.3.3).

8.   Models run to define boundary conditions for modelled granite jointed rock mass failure mechanisms with deformable blocks (Section 4.3.3).

9.   The effect of varying joint spacing (Section 4.4.1).

10.   The effect of varying joint dip (Section 4.4.2).

11.   The effect of varying joint persistence (Section 4.4.3).

12.   Models run to define boundary conditions for modelled limestone jointed rock mass failure mechanisms with a ground water table (Section 4.5.1).

13.   The effect of varying joint stiffness (Section 4.5.2).

14.   Models run to define boundary conditions for modelled limestone jointed rock mass failure mechanisms with a wave-cut platform (Section 4.5.3).

New b/h against α graphs were created having altered the following parameters from the original input command file in turn: friction angle, block properties, random, varied joint angle, random, varied joint spacing, joint persistence, joint stiffness, block deformability, rock mass water pressure and the model boundary condition. The input geometrical properties for each test are listed in Table 4.2 and the actual input command files are listed in appendices. Pritchard and Savigny (1990) suggest that the influence of the strength of rock material or rock mass forming toppling columns has never been

quantitatively assessed. An input command file was set up identical to the original, but with a joint friction angle of 20° (Appendix 4.3) and 60° (Appendix 4.4). Thus two further comparative graphs for the conditions of failure of rock masses were created in order to understand the control exerted by the joint friction angle. In order to vary the intact block properties, it was decided to use real-world data for sandstone (Appendix 4.5) and granite rock (Appendix 4.6). The behaviour of rock masses composed of limestone, sandstone and granite could then be compared. Using the input data files for the granite and sandstone blocks, commands were issued to ascribe deformability to intact rock blocks (Appendices 4.7 and 4.8).

The control of discontinuities on the failure of modelled rock masses was further analysed by altering the input commands which generate the UDEC mesh. A graph of b/h against α was created having run a series of models with a variable dip of the joint set (Appendix 4.9) (Table 4.2). A permitted random variance of up to 10% from the specified joint dip was allocated using the UDEC joint mesh generator. Likewise, a command file was created which was identical to the original, but with a permitted random variance of up to two metres from the specified joint spacing (Appendix 4.10). The original data file included fully persistent joints across the rock mass. Full persistence does not always occur in real rock masses, and although persistence is difficult to measure in the field, its effect on rock slope failure mechanisms can be elucidated using this technique (Appendix 4.11). The UDEC joint stiffness parameter is also difficult to measure, and reflects properties such as joint dilation, infill, roughness and width. The input command file used to gain the comparative graph for joint normal and shear stiffnesses of 5 GPa is given in Appendix 4.12. Two further input parameters were tested for control on rock mass failure. It is possible to model a rock mass which includes a fluid pressure within the discontinuities. Two situations were modelled: one with a water table with a height of 35 metres (Appendix 4.13) at the rear of the rock mass, and one with a water table at a height of 50 metres (Appendix 4.14). In both situations, the height of the water table was set to the base of the cliff face at the left-hand edge of the model. Also tested was the influence of a platform at the base of the cliff which reflects the geomorphic situation of many jointed rock masses. An input command file was set up which included a 50 metre wide platform (Appendix 4.15).

4.2 Results: The sliding and / or toppling of modelled rock masses

The resulting failure mechanisms from 112 models were recorded so that the boundary defining each failure could be drawn on a graph of base-to-height ratio (b/h ratio) against base plane angle (α) (Figure 4.5). Failure type and values for b/h and α for each of the models are listed in Appendix 4.16. It can be seen that the shapes of boundary conditions between rock mass failure criteria are quite different from those for an individual rock block (Figure 4.6). The implication is that the discontinuity pattern is an important control on stability and that the dynamic response of the rock mass material is quite different from the kinematic failure of a single block. One of the most notable differences is that at base plane angles which are greater than the joint friction angle, ϕ, there is only a small region in which toppling-and-sliding occur together as the movement mechanism. In addition, toppling can occur where the bedding plane angle is horizontal or even where it dips into the rock face, providing that there is a very low b/h ratio, confirming the suspicions of Cruden (1989). This is a significant finding concerning toppling failure mechanisms and has implications for rock mass stability studies and slope development. However, it is interesting to note that the shape of the boundary curve between a toppling failure and stable conditions is similar for both a single block and a limestone rock mass. At lower angles of base plane, it is more likely that stable conditions exist. The curve for the modelled limestone rock masses has been displaced by about 40° on the base plane axis and there is a far greater range of conditions for which toppling can occur for a rock mass. Such an observation may be expected as the pressure of surrounding blocks in a rock mass would exert a greater force promoting failure than for a single block.

Four examples of model output representing each failure mechanism and a stable slope can be used to explain and understand the location of the boundary conditions on Figure 4.5. A sliding modelled rock mass model is characterised by a joint geometry which has a steep bedding plane angle and a relatively high b/h ratio (Figure 4.7a). Blocks slide on the bedding planes with the velocity vectors plotting parallel to the bedding plane which indicates that no element of toppling is occurring. There is a sharp boundary at the joint friction angle ϕ between sliding and a region of no movement (Figure 4.5). The models which have a discontinuity geometrical pattern giving a high b/h ratio and low bedding angle fell into the region of no movement. The model which

is shown in Figure 4.7b has a *b/h* ratio of 2.0 and a base plane angle, α, of 25°. Stable models were easy to discern as there was a random pattern of very low value velocity vectors.

A modelled rock mass which fails by toppling has a bedding angle α below the 40° joint friction angle and a low *b/h* ratio. The example illustrated in Figure 4.7c has a base plane angle of 35° and *b/h* ratio of 0.3. The rotation in the failing blocks is clear, and further evidence is given if the displacement vector arrows are plotted; vectors plot at an acute angle to the basal sliding surface, out of the slope as opposed to parallel to the dip of the bedding. The region of toppling on the graph is much greater for the rock mass than the single block failure and it has been demonstrated that it is possible for a toppling failure mechanism to occur if the bedding planes are horizontal or dip into the slope face. The dynamic stress from surrounding blocks makes a block topple when, under the same geometric conditions, it would be kinematically stable when standing on its own. Further observations made during the modelling exercise noted that tension cracks often occurred in the top slope between blocks which had failed, and blocks which remained stable (Figure 4.7c). Tension cracks have been observed in the field (Bovis and Evans, 1996; Caine, 1982; Ishida *et al.*, 1987), which suggests that blocks within the rock mass coalesce to form one toppling column which decreases the *b/h* ratio. Goodman and Bray (1976) suggest that deep tension cracks and the resulting formation of obsequent scarps are distinctive geomorphic features which make it possible to identify toppling as the applicable mechanism. In some instances, for example with a base plane angle of 30° and a *b/h* ratio of 1.4, two tension cracks develop, each representing the daylighting of a toppling failure plane. The results of this study suggest that toppling is a much more widespread than has been thought and an understanding has been gained of the situations under which toppling may occur.

Under some discontinuity geometric situations in rock masses, toppling-and-sliding failure occurs. An example is a modelled rock mass with a *b/h* ratio of 0.3 and a base plane angle, α, of 48° (Figure 4.7d). It can be seen that there is a plane of bedding within the model upon which a mass of blocks have slipped. The failed blocks can be seen to be rotating at the same time. The displacement vectors are oriented at a variety of angles rather than parallel to the sliding plane. It is interesting to note that the zone of

sliding-and-toppling is much smaller than for an individual block (Figure 4.6). Perhaps the sliding motion of surrounding blocks prevents rotation of a block which standing alone could topple as well as slide. However, it should be noted that rock mass models plotted close to the boundary line, but included in the sliding only region, did comprise a few individual blocks standing on the slope face which toppled. But the model failure, as influenced by the forces within the mass, was entirely a sliding mechanism. The fact just illustrates that the kinematic failure of an individual block with such a geometry is different from the rock mass failure.

A further point to be made about the graph of boundary conditions between failure mechanisms in a rock mass (Figure 4.5) is that the line was very difficult to define for models with a bedding angle close to the joint friction angle ϕ. Thus, a gap has been left in the line in Figure 4.5 which relates to the gap in understanding. There was a sharp change at 40° between sliding models and non-sliding models, but it was difficult to define whether toppling was occurring. It is possible that the two lines are exponential at angles close to 40° and that in reality they never join. It was also noted that if the models are left to run over a long period of time, it is observed that those models which fail by sliding retreat in a parallel fashion, whereas those which fail by toppling undergo a gradual decline in slope. The sliding models result in a successive loss of rock layers due to movement across the base bedding planes. The rock masses therefore exhibit parallel retreat across the steeply dipping bedding planes. Models which display a toppling failure mechanism lose a mass of blocks above the gently angled bedding planes, before the slope stabilises at a lower angle. Holmes and Jarvis (1985) note that scarps formed in toppling slopes strike parallel to joint sets. However, a scarp section often remains at the top of the slope profile as a remnant of the tension cracks which once developed during toppling failure.

4.2.1 The activity of rock slope failure mechanisms

As a UDEC model is being processed, it is possible to keep a record of the forces acting on each block at the end of each time-step. From plotting the time record of total unbalanced forces for the whole model as it is run, it is evident that activity in the rock mass occurs in distinct phases or pulses. For the sliding model which is illustrated in Figure 4.7a, there are rapid movements separated by short periods of less intense

activity (Figure 4.8a). The activity reaches a maximum just before the model stabilises. For the toppling model (Figure 4.8b) there is less of a surge of activity, with more consistent peaks and troughs in the unbalanced forces plot. It appears that the forces in the rock mass increase significantly to peaks, before decreasing to equilibrium. Total unbalanced force activity for the toppling-and-sliding model (Figure 4.8c) demonstrates characteristics of behaviour from both of the single failure types. The model (illustrated in Figure 4.7d) appears to fail initially by predominantly sliding, before toppling exerts greater control at a lower level of activity. There appears to be a distinct break between the two levels of activity. Bovis and Evans (1996) suggest that at most sites where deformations have been monitored in British Columbia, Canada, that sliding is the initial mode of movement, followed by or accompanied by toppling and toppling-induced sliding movements.

Pulsed events which affect landforms have been widely discussed in geomorphology and this behaviour appears to have been highlighted during the exercise (Brunsden, 1990; Kennedy, 1992; Renwick, 1992). Explanation for pulsed activity in jointed rock slopes could be related to the presence of key blocks in the mass which due to a stable position prevent movement of surrounding blocks. Forces acting on the key block from surrounding blocks gradually increase to a point where the stresses promoting movement exceed those holding the block which is promoting stability. Once an event commences, the activity in the slope rapidly increases before stabilising again. Key blocks in the model runs can be identified by the orientation displacement vectors opposing the general rock mass movement.

Data from the record of total unbalanced forces can be extracted from the UDEC program for analysis after a model run. A value for the total unbalanced force is collected after every ten UDEC cycles. If the data from the sliding model (Figure 4.8a) are taken, nearly 50,000 data points can be extracted (Figure 4.9). By plotting the points on a logarithmic scale for the total unbalanced forces, a better spread of points is evident, suggesting that the increase in activity is close to exponential. By focusing upon an individual peak of activity in the plot (Figure 4.10), it can be seen that the early rise in activity is exponential followed by a period of consistent high forces. The decrease in activity occurs sharply to a low level again. By focusing still further (Figure 4.11), it can be seen that there are four peaks of high activity, which are separated by

time intervals of roughly equal periodicity. The exercise thus demonstrates that modelled rock slope activity does not proceed as a continuous process but as a pulsed event sequence between stability, movement and the establishment of an equilibrium slope form.

4.2.2 The effect of varying rock block height

The models which were run to define boundary conditions between different mechanisms of failure for a limestone rock mass depending upon joint geometry used blocks with a fixed height of 4 m. The width of the base of the blocks was altered by varying the spacing between the second joint set, inclined at 70°, in order to control the b/h ratio. It was therefore decided to repeat the experiment under the same settings, again varying the b/h ratio and the bedding angle, α, for a limestone rock mass. However, the b/h ratio was altered by varying the height of the blocks, not the width, thus resulting in a different rock mass geometry (Appendix 4.17) (Table 4.2).

In all, 85 models were run in order to examine the behaviour of rock masses with block height varied (Appendix 4.18). It was not possible to define the boundary between the different types of failure mechanism by relating the b/h ratio to the base plane angle. Rock masses which had a base plane angle of 10° or less were stable. However, for rock masses which had a base plane angle of 20°, a b/h ratio of between 0 and 20 created a geometry of rock blocks which failed by toppling. If a b/h ratio more than 20 was defined, a very large number of blocks were generated and the UDEC code took a long time to process. The difference in joint geometrical conditions which result when block height is varied can be appreciated by examining a plot with a base plane angle of 25° and a b/h ratio of 2.0 (Figure 4.12). If the plot is compared with that from the original test (Figure 4.7b), which has the same conditions, it can be seen that a greater number of discontinuities occur in the cliff face. Thus, the model is not only more unstable, but equivalent tests require a larger number of blocks to be used in each model.

4.2.3 The effect of using rectangular blocks

The models which were run to define boundary conditions between different mechanisms of failure for a limestone rock mass depending upon joint geometry defined blocks using two joint sets. One of the joint sets varied in dip to represent the changing

conditions of the base plane and the second was fixed at 70°. It was decided to repeat the experiment using rectangular blocks, whereby both joint sets are varied. The second joint set was altered in line with the base plane angle in order that perpendicular internal angles were maintained for blocks (Table 4.2). The use of rectangular blocks is of interest not only because DeFreitas and Watters used rectangular blocks, but also because such blocks are often apparent in nature. Again, the b/h ratio and the bedding angle, α, were varied for a limestone rock mass (Appendix 4.19).

The boundary between the different types of failure mechanism is drawn on a graph of b/h ratio versus α for a rock mass with rectangular blocks (Figure 4.13). In all, 58 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.20. The shape of the boundary between the different types of failure differs from the original graph for the limestone rock mass with a fixed 70° joint set (Figure 4.14). Using rectangular blocks, it is not evident that a toppling failure occurs for bedding planes inclined at less than horizontal. However, for modelled rock masses which contained bedding inclined at greater than 30° there are more conditions for which a toppling failure mechanism can occur. The graph plot is still very different from the conditions of failure of a single block. For a rock mass with a base plane angle of 50° and a b/h ratio of 3.2, it is clear that the blocks are failing by sliding-and-toppling (Figure 4.15a). It can be seen that there is a rotation of blocks as the angle of the 40° joint set has changed, and that blocks at the base of the cliff are sliding. A rock mass containing blocks with the same b/h ratio and α using a fixed 70° joint set would not have failed by this mechanism. Also with a 50° base plane angle, but a b/h ratio of 3.6, a rock mass fails by a purely sliding mechanism (Figure 4.15b). The mechanism can be deduced from the fact that the velocity vectors are oriented parallel to the base plane angle, and that there has been no rotation of the 40° joint set.

4.2.4 Summary

The numerical simulation results presented here using suites of theoretical 'data' illustrate that there is a great difference between the previously defined boundary conditions for different failure mechanisms for the kinematic failure of a single block and the boundaries for the dynamic behaviour of numerous blocks in a rock mass. Different rock masses may behave in different fashions. Thus, the new boundary

conditions, as described for a modelled limestone rock mass with two continuous discontinuity sets, can now act as a template for further investigations analysing the sensitivity of input parameters. A greater understanding of the response of a rock slope landform has been gained, which has been aided by observations made during the exercise. Although the observations from the simulated limestone rock mass cannot be used to propose definitive rules of rock mass behaviour, they can be used to add weight to debates occurring within geomorphology. Some evidence can be given for the pulsed nature of failure processes operating in rock mass landforms, and whether the slopes develop through parallel retreat or through a gradual decline in angle appears to be influenced by the geometry of discontinuities and the nature of the failure mechanism. The debate about the nature of slope behaviour begun with Penck (1925) and Davis (1930), developed by the likes of Howard and Selby (1994) and Parsons (1988) still continues.

4.3 Material property variation

4.3.1 Joint friction angle

The joint friction angle for rock masses is an important influence upon slope stability (Terzaghi, 1962). The models of limestone rock masses which were used to fix boundary conditions between failure mechanisms had a joint friction angle, ϕ, of 40°. It was decided to repeat the modelling, again varying the b/h ratio and the bedding angle α, for rock masses with joint friction angles of 20° and 60° (Table 4.2). All other parameters were input as before. The command files are listed in Appendices 4.3 and 4.4. The exercise aimed to create two new curves of boundary conditions, graphically comparing the results with the original b/h versus α curve. Thus, it would be possible to isolate the control of the joint friction angle parameter upon modelled rock slope failure and landform development.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a rock mass with a joint friction angle of 20° (Figure 4.16). In all, 188 models were run in order to fix the line. The corresponding mechanisms of failure are listed in Appendix 4.21. A large number of model runs were required because it became difficult to identify the mode of failure for a number of situations. To overcome the

identification problem, it was decided to define another failure mechanism type and an extra zone has been designated on the graph (Figure 4.16).

Models which demonstrated the initial stages of a rotational movement of blocks but then stabilised were observed during the experiment. The term creep toppling and stabilising was given for this type of failure, and the region is denoted on the b/h ratio /α graph (Figure 4.16). A creep toppling and stabilising failure mechanism occurred in modelled rock masses which had a base plane angle dipping into the slope face at 20° or more (Figure 4.17). After the initial phase of block settlement, the blocks in the UDEC models then failed by a combination of slipping on the bedding planes and rotating to various degrees, before stabilising. Stability was confirmed by observing that the magnitude of the displacement vectors was low, and that vectors were randomly oriented throughout. All models demonstrating a creep toppling and stabilising behaviour were run for a large number of steps to ensure that failure did not recommence. The boundary between creep toppling and stabilising and totally stable rock masses was easy to delimit, although the line is coincident with a large b/h ratio (Figure 4.16).

The shape of the boundary between the different types of failure is similar to the original graph for a rock mass with a joint friction angle of 40° (Figure 4.18). The central peak of the graph, consistent with the joint friction angle, has been displaced by 20°, demonstrating the control of this parameter. At bedding plane angles α greater than the friction angle ϕ there is again a small zone of toppling-and-sliding failure (Figure 4.16). The curve has a steep, negative gradient from a maximum b/h value at 20°. A rock mass which has a base plane angle of 40° will fail by sliding, unless the geometry of the discontinuity sets leads to the occurrence of blocks which have an extremely low b/h ratio. At base plane angles below the joint friction angle of 20° to a bedding base plane angle which dips at 60° into the slope face, toppling will occur if there is a low b/h ratio. Again the curve between toppling and no movement has a similar shape to the original, although the peak is not quite as high. However, there are more inflections in stable/toppling boundary curve for modelled rock masses with a friction angle of 20°. The experiments were repeated several times to fix the curve accurately, and observations made during the modelling process yielded no clues explaining the inflections.

The boundary between the different types of failure mechanism is drawn on a graph of *b/h* ratio versus α for a rock mass with a joint friction angle of 60° (Figure 4.19). In all, 121 models were run to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.22. The three types of failure, toppling, sliding, toppling-and-sliding, and a region of no movement, have been designated on the graph.

Once again, the shape of the boundary between the different types of failure is similar to the original graph for a rock mass with a joint friction angle of 40° (Figure 4.20). A toppling failure was observed for a wide range of modelled discontinuity geometries, with base plane angles varying from 59° to horizontal, providing that the *b/h* ratio is low. Sliding occurred for models which had bedding dipping at greater than the joint friction angle of 60° and a small zone of sliding-and-toppling failure is delimited on the graph. However, there are three notable differences between the boundary line of failure mechanisms for the rock mass with a friction angle of 60° and a rock mass with a friction angle of 40° (Figure 4.20). First, are several inflections on the curve delimiting the boundary between toppling failure and a region of no movement. Again, the experiments were repeated several times to fix the curve accurately, and observations made during the modelling process yielded no clues explaining the inflections. Second, the peak of the curve at values of bedding angle close to the joint friction angle 60° does not correspond to such high values of *b/h* ratio as does the peak of the boundary conditions for the joint friction angle of 40° curve. Finally, it was difficult to delimit the failure mechanism when the angle of bedding was greater than 80°. The discontinuity geometry leads to inherently stable rock masses in such circumstances and due to the uncertainty of definition, the curve has not been designated at a base plane angle greater than 80°.

If the three boundaries between the different types of failure for rock masses with joint friction angles of 20°, 40° and 60° are compared, (Figure 4.21) it can be seen that the curves plot approximately parallel. The joint friction angle clearly controls the location of the peak of each curve and was identified as an important rock strength control upon toppling failures in Tasmania (Caine, 1982). However, several facets of behaviour were observed whilst investigating rock masses with joint friction angles of 20° and 60° that did not occur in modelling with a joint friction angle of 40°. The modelled creep toppling and stabilising failure mechanism which was observed has not

been recorded before, and is difficult to interpret. Buckles have been observed deep in rock masses where rock layers fold under pressure from overlying material (Hu and Cruden, 1993). Once the creep toppling movement has commenced with limited rotation of blocks, the altered joint geometry would suggest that continued failure was more likely than stability. Also, the boundary between toppling failure and no movement for models with a joint friction angle of 20° and 60° was inflected in several places.

Explanation of the two spurious observations may be related to data consistency. Although the exercise demonstrates the control of the joint friction angle input parameter, such rock masses would not occur in the field. In general, a hard jointed rock mass would have a friction angle of between 25° and 50° depending upon rock type (Deere, 1966). Also, the models used Portland Limestone rock strength data, and it would be unrealistic for limestone to have a friction angle of either 20° or 60°. Generally, it would be appropriate for weak sediments or granular materials to have a joint friction angle of 20°. Perhaps the explanation for the differences between the boundary failure conditions for the 20°, 40° and 60° models is that the input data for the 20° and 60° model runs are incompatible. Thus, it was decided to run the experiments with a complete data set for different rock masses.

4.3.2 Intact rock properties: limestone, granite and sandstone

The models used to fix the original boundary conditions between different types of failure mechanism were based upon Portland Limestone geotechnical data (Allison, 1986) and had a joint friction angle ϕ of 40°. Given the problems identified in isolating joint friction angle as a control, it was decided to repeat the experiment under the same settings, again varying the b/h ratio and the bedding angle α, for rock masses composed of granite and sandstone (Table 4.2). Representative data for the geotechnical properties of different rock types were analysed from literature sources (Daly et al., 1966; Deere, 1966; Doberiener et al., 1990; Kulhawy, 1975; Shakoor and Bonelli, 1991). It was decided to set up models using sandstone and granite intact rock properties as they occurred at the opposite ends of the hard rock strength spectrum. Data typical for an average plutonic granite were prepared for UDEC entry. The modelled rock blocks had a density of 2,650 kg m^{-3} and a joint friction angle of 46° (Table 4.2). The sandstone model blocks had a density of 2,320 kg m^{-3} and a joint friction angle of 29° (Table 4.2).

All other parameters were input as before, and the command files are listed in Appendices 4.5 and 4.6. Again, the exercise was aimed to create two new curves of boundary conditions between the different types of failure, graphically comparing the results with the original b/h versus α curve.

The boundary between the different types of failure mechanism is drawn on a graph of b/h ratio versus α for a sandstone rock mass with a joint friction angle of 29° (Figure 4.22). In all, 41 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.23. Fewer models were used to define the conditions between failures because of time constraints of this study and the experience gained in identifying boundaries. The shape of the boundary between the different types of failure is similar to the original graph for a limestone rock mass (Figure 4.23). At bedding plane angles α greater than the friction angle ϕ there is a slightly larger zone of toppling-and-sliding failure for the sandstone rock masses. An example of a toppling-and-sliding rock mass, with a bedding plane angle of 50° and a b/h ratio of 0.2, is illustrated in Figure 4.24a. A modelled rock mass with b/h ratio of 1.2 and a base plane angle of 40° clearly slides as indicated by the displacement vectors (Figure 4.24b). Toppling occurs for modelled joint geometries which have a low b/h ratio, and bedding which dips between 29° out of the slope face and 50° into the slope face. The example illustrated clearly shows that the centre of mass of the individual blocks does not overhang the pivot point of rotation with the base planes dipping at 20° into the rock (Figure 4.24c).

The shape of the curve of boundary conditions between different failure mechanisms is similar for the two rock masses and it can be concluded that the joint friction angle controls the location of the peak of the curve. The other rock strength parameters which were changed for the sandstone rock modelling exercise appear to have less importance. The resulting curve is similar to what might be expected if only the joint friction angle were changed to 29° from the original limestone data set. But, there are fewer inflections in the curve, and the creep toppling and stabilising failure mechanism was not observed, even though toppling occurred for bedding dipping into the slope face at angles greater than the 29° friction angle. Thus it is important to use a complete and consistent set of intact rock strength parameters to avoid the situations

encountered in the experiments on joint friction angle (see Section 4.3.1) (Figure 4.21), even though the friction parameter exerts the greatest control.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a granite rock mass with a joint friction angle of 46° (Figure 4.25). In all, 35 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.24. Again, the shape of the boundary between the different types of failure is similar to the original graph for a limestone rock mass (Figure 4.26). At bedding plane angles α greater than the friction angle ϕ there is a similarly small zone of toppling-and-sliding failure for the granite rock masses. The boundary line with the sliding failure for the granite rock masses plots nearly parallel with the boundary for the limestone rock mass. The boundary between the toppling failure mechanism and the stable granite rock masses has a small inflection. However, the inflection is observed at the same point in the curve for limiting boundary conditions in the sandstone rock masses (Figure 4.27), and it is the curve for the limestone rock masses that demonstrates unique conditions (Figure 4.28).

Toppling occurs for modelled joint geometries which have a low b/h ratio and bedding which dips between 46° out of the slope face and 20° into the slope face. A rock mass with a b/h ratio of 1.8 and bedding planes dipping at an angle of 30° out of the rock face clearly topples (Figure 4.29a). The history plot of total unbalanced forces for this toppling granite model demonstrates that activity increases in a close to exponential fashion to peak values (Figure 4.29b). However, there appears to be an irregular cyclicity between periods of movement. A modelled granite rock mass which has a b/h ratio of 0.8 and a base plane angle of 60° fails by sliding (Figure 4.29c). There are no block displacement vectors plotting out of the slope face, suggesting that no rotation of blocks is occurring. A rock mass also with a bedding angle of 60°, but a b/h ratio of 0.4, fails by toppling-and-sliding (Figure 4.29d). The history plot of unbalanced forces for the toppling-and-sliding model again appears to demonstrate two distinct phases of activity (Figure 4.29e). After the free face has been released after the model consolidation, seven separate periods of low activity occur. Then there is a change to two long period phases of activity, with levels building to a final, high activity spell. The two phase activity was also observed for the limestone toppling-and-sliding models (Figure 4.8c).

It is interesting to compare the conditions defining the failure mechanisms for modelled sandstone, limestone and granite rock masses (Figure 4.28). It can be seen that it is possible to consider a similar shape of the boundary curve for hard jointed rock masses, but the actual position of the curve varies, depending upon the value of the friction angle. Thus, for the same joint geometrical conditions in each rock mass, different states of stability occur. For instance, at a base plane angle of 30° and a *b/h* ratio of 2.0, a modelled sandstone rock mass fails by sliding-and-toppling, a modelled limestone rock mass fails by toppling and a modelled granite rock mass is stable (Table 4.3).

		Sandstone $\phi = 29°$	Limestone $\phi = 40°$	Granite $\phi = 46°$
$\alpha = -30°$,	$b/h = 0.4$	stable	stable	stable
$\alpha = -20°$,	$b/h = 0.4$	topples	stable	stable
$\alpha = -10°$,	$b/h = 0.4$	topples	topples	stable
$\alpha = 0°$,	$b/h = 0.4$	topples	topples	topples
$\alpha = 30°$,	$b/h = 2.0$	slides-and-topples	topples	stable
$\alpha = 41°$,	$b/h = 2.0$	slides	slides-and-topples	topples
$\alpha = 50°$,	$b/h = 2.0$	slides	slides	slides-and-topples
$\alpha = 60°$,	$b/h = 2.0$	slides	slides	slides

Table 4.3: A comparison of stability conditions for modelled sandstone, limestone and granite rock masses at given rock mass joint geometries.

4.3.3 Deformable blocks

In simulating jointed rock masses using the UDEC modelling software, the possibility exists to use deformable intact blocks. Rigid blocks do not change in geometry under loading, and the failure of the rock mass occurs entirely along discontinuities. The use of deformable blocks is generally restricted to the modelling of high pressure environments, such as may be encountered during a deep mine excavation. Cliffs composed of a hard jointed rock generally fail by block displacement at lower stresses than is required for intact blocks to deform. However, there are geomorphic scenarios

when a control may be exerted by the deformability of intact rock in cliffs (Cooks, 1983). The development of cliffs composed of a massive rock containing few discontinuities, such as occur on the Colorado Plateau, may be influenced by blocks deforming before failure occurs (Oberlander, 1977). In weaker rocks, including sandstone, the elastic range is limited to very low stress levels (Doberiener et al., 1990). It has been demonstrated that UDEC models of slopes, composed of soft jointed granodiorite, using fully deformable blocks can be used to study flexural topples (Pritchard and Savigny, 1990). The failure surface in the broken material was curvilinear. Hsu and Nelson (1995) suggested that the range of instability for deformable blocks is far larger than for rigid blocks when modelling a weak shale sediment. Again, it is confirmed that a flexural toppling failure mechanism can be replicated when modelling softer jointed sediments with deformable blocks and that combinations of sliding, toppling and shearing are observed.

It was decided to run the experiments using the sandstone and granite rock masses, analysed in Section 4.3.2, with deformable blocks. It was thought that any effects would be more pronounced in rock masses at the opposite end of the strength spectrum. The input of data was the same, but a command was issued to set block deformability (Table 4.2). UDEC automatically discretises any block into triangular, constant strain zones. Motion is calculated for the zones as opposed to the whole block with all gridpoints being located on the block edge. It was decided to set zones with a side length of 2 m for the sandstone blocks and 3 m for the granite blocks. Within UDEC, calculations are set up using the equation of motion of deformable gridpoints with strains and rotations being related to nodal displacements acted upon by the elastic increment of the stress tensor. The Poisson's ratio effect is represented for modelled rock masses. For low values of $E / s\, k_n$ (E is Young's modulus, s is joint spacing and k_n is the joint normal stiffness), the Poisson effect of the rock mass is dominated by the elastic properties of the intact rock. For high values of $E / S\, k_n$, the 'Poisson effect' is dominated by jointing.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a sandstone rock mass using deformable blocks (Figure 4.30). The input command file for the models is listed in Appendix 4.7. In all, 48 models were run to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.25.

The shape of the boundary between the different types of failure is similar to the original graph for a limestone rock mass (Figure 4.31), although the curve does not form such a sharp peak. Thus, there is a greater region of toppling failure below the joint friction angle of 29°, although the peak is lower for the boundary conditions of the deformable sandstone mass. It is only just possible for a rock mass with a *b/h* ratio of 2.0 to fail by toppling, whereas it is possible for the limestone rock mass containing blocks which have a *b/h* ratio of 3.0 to fail by toppling.

When the boundary conditions for failure of a sandstone rock mass containing deformable blocks are compared with the boundary conditions of a sandstone rock mass containing rigid blocks (Figure 4.32), it can be seen that the deformable blocks exert little control. The greatest effect appears to be for models with base plane angles close to the joint friction angle, as there is a difference in the height of the peaks. Generally, the rock masses containing deformable blocks can fail by toppling at slightly higher values of the *b/h* ratio. Perhaps the most interesting comparison is that the two lines for the boundary between the toppling and stable regions plot almost parallel. The slight inflections occur in the same place for both graphs, which suggests that there is an explanation for the occurrence of inflections as opposed to being a consequence of modelling or observation errors.

For both a sandstone model using rigid blocks and a sandstone model with deformable blocks, toppling will occur when the discontinuity geometry gives a *b/h* ratio of 0.6 and a base plane angle of 30° into the rock mass. The output for the model using deformable blocks plots the displacement vectors from the centre of each deformable zone (Figure 4.33a). The deformable zones for this model are plotted in Figure 4.33b. However, the history plot for the toppling model demonstrates a decrease in activity from an early maximum, before settling at a consistent level (Figure 4.33c). Another toppling model using deformable sandstone blocks illustrates a different form of failure (Figure 4.33d). The model has a *b/h* ratio of 1.8 and a base plane angle of 10° and plots close to the boundary of no movement (Figure 4.30). The centre of mass of each block clearly does not overhang the block rotation point. It is interesting to note that there are no tension cracks in the top of the profile and that failure is only occurring for a certain number of blocks lower in the profile. The fact that the upper blocks remain attached to the main part of the cliff suggests that the plot is a slightly spurious output.

For comparison, the output of a stable model, which has a *b/h* ratio of 2.6 and a base plane angle of 20°, plots displacement vectors of low value pointing randomly (Figure 4.33e). The history plot confirms that no activity is occurring (Figure 4.33f).

When the line representing the boundary conditions between a sliding failure mechanism and a sliding-and-toppling failure mechanism is compared for a model using rigid sandstone blocks with a model using deformable sandstone blocks (Figure 4.32), it can be seen that there is a large difference at base plane angles of 30°. A model which has a bedding angle of 30° and a *b/h* ratio of 2.4 and is composed of deformable sandstone blocks clearly fails by sliding (Figure 4.33g). There is no suggestion of rotation as indicated by the displacement vectors. Another example of a sliding model plots close to the boundary line, and demonstrates that it is possible for individual blocks to fail by toppling over (Figure 4.33h). However, the rock mass, which has a base plane angle of 40° and a *b/h* ratio of 1.0, as a whole fails by a sliding mechanism. Most movement occurs along bedding planes which occur low in the rock mass. The upper blocks seem to fall back into the voids created by the movement rather than topple away from the face. The history of unbalanced forces plot indicates that activity for this model increases in magnitude and decreases in frequency as the model is run (Figure 4.33i). Activity appears to increase exponentially towards peaks, and highlights a magnitude / frequency relationship which is commonly discussed in geomorphology (Ohmori and Hirano, 1988; Richards, 1990). A model using deformable sandstone blocks, which also has a base plane angle of 40°, but a slightly lower *b/h* ratio of 0.8, fails by sliding-and-toppling (Figure 4.33j). The buckling of the top side of the model indicates that sliding is occurring, although toppling appears to account for the most movement. The history of unbalanced forces for the sliding-and-toppling failure mechanisms demonstrates irregularly spaced periods of activity of different magnitude (Figure 4.33k). The plot is influenced by the combined effects of both toppling-and-sliding activity.

The boundary between the different types of failure is drawn on a graph of *b/h* ratio versus α for a granite rock mass using deformable blocks (Figure 4.34). In all, 29 models were run to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.26. The shape of the boundary between the different types of failure mechanism is similar to the original graph for a limestone rock mass (Figure 4.35),

particularly for base plane angles greater than the joint friction angle. However, there is a smaller region of toppling failure for the curve for the granite model with deformable blocks as the peak is lower. Again, it is only just possible for a rock mass with a *b/h* ratio of 2.0 to fail by toppling, whereas it is possible for the limestone rock mass containing blocks which have a *b/h* ratio of 3.0 to fail by toppling.

When the boundary conditions for failure of a granite rock mass containing deformable blocks are compared with the boundary conditions of a granite rock mass containing rigid blocks (Figure 4.36), it can be seen that the deformable blocks exert little control as the two graphs are very similar. The main difference is that the graph for the boundary conditions between a toppling failure mechanism and a stable rock mass modelled with deformable granite blocks contains a large inflection. This hints at some control being exerted by the use of deformable blocks. However, for both sets of input parameters, toppling will occur for models with a bedding angle between 46° dipping out of the slope and 20° dipping into the slope. Also, the boundaries between rigid granite and deformable granite models which fail by sliding and toppling-and-sliding occurs in a similar position on the graph.

It is possible for models composed of deformable granite blocks, which have a horizontal base plane, to fail by toppling. A model with horizontal bedding and a *b/h* ratio of 0.4 plots close to boundary of the toppling region (Figure 4.37a). It can be seen that there is a zone of major toppling activity and also that a second tension crack occurs further to the rear of the model top. The triangular deformable zones for the model are plotted in Figure 4.37b. A model composed of deformable granite blocks with a *b/h* ratio of 0.8 and a base plane dipping at 60° out of the slope face fails by toppling-and-sliding (Figure 4.37c). The bulk of failure was by toppling and the remnants of toppled blocks are seen towards the base of the slope. However, the blocks at the top of the profile are creeping down-slope with the joint friction causing an imbalance.

Observations made during the modelling experiments using deformable blocks support a number of points to be made about the further use of deformable blocks in the simulation of rock mass landforms using UDEC. There are few geomorphic situations where the deformable nature of hard rock blocks may exert a control on the rock slope form. However, problems encountered elsewhere in the modelling of jointed rock masses using UDEC arose when the joint geometry defined very small, triangular

blocks surrounded by larger stones. In reality, such blocks would be crushed as the stress build-up at the sharp point of the block would be great. But, UDEC deals with large stress build-up by numerically overlapping the blocks into larger blocks. It was thought that by creating deformable blocks such problems would be overcome, although the deletion of blocks would also be a practical solution as the problem blocks are so small that they would have insignificant control.

It can be seen in the comparative graphs (Figures 4.32 and 4.36) that there is little difference in the boundary conditions for different types of failure mechanism due to the use of deformable blocks. In certain places, the accuracy of the position of the boundary curves for models composed of deformable blocks is questionable because of spurious modelling results (Figure 4.31 for example). Also, due to the strength and elastic properties defined for hard rock mass blocks, situated in comparatively low stress environments which occur with surface rock slope landforms, there was no evidence during the exercise of any block deforming to the extent that the block dimensions altered. However, a conclusive argument can be made for not using deformable blocks in modelling, for logistical reasons. The time taken by the computer to model 100,000 cycles for a rock mass composed of deformable rock mass was compared with a model containing rigid blocks under the equivalent discontinuity geometric conditions. The results are presented in Table 4.4.

Model	deformable granite	granite	deformable sandstone	sandstone
b/h=0.8, α=0°	3 hrs 29 mins	57 mins		
b/h=0.8, α=60°	12 hrs 38 mins	62 mins		
b/h=0.6, α=-20°			6 hrs 20 mins	1 hr 18 mins
b/h=1.0, α=-10°			5 hrs 57 mins	
b/h=2.4, α=30°			5 hrs 49 mins	23 mins
b/h=0.4, α=50°				2 hrs 11 mins
Average	8 hrs 3 mins	60 mins	6 hrs 2 mins	1 hr 17 mins

Table 4.4: The time taken to complete 100,000 cycles for models composed of deformable and rigid blocks.

Given the extra time required for modelling using deformable blocks which exert little control upon the rock slope development, it is argued that the use of deformable blocks during UDEC modelling of rock slopes is inappropriate.

4.4 Discontinuity variation

4.4.1 Spacing variability

The model of the limestone rock mass which was used to fix the original boundary conditions between different types of failure had a regular, fixed joint spacing. If a set of real joint spacings are analysed, a statistical spread is evident (Mohajerani, 1989; Priest and Hudson, 1981; Sen and Kazai, 1984). It was decided to repeat the experiment under the same settings, again varying the b/h ratio and the bedding angle α, for rock masses with joint sets which could vary randomly by up to 2 m from the assigned discontinuity spacing value (Table 4.2). All other parameters were input as before, and the command files are listed in Appendix 4.10. The exercise was aimed to create a new curve of boundary conditions between the different types of failure mechanism, graphically comparing the results with the original b/h versus α curve. Thus, it would be possible to isolate the control of statistical variation in joint spacing parameter upon modelled rock slope failure and landform development.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a rock mass with varied joint set spacings (Figure 4.38). In all, 34 models were run to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.27. The shape of the boundary between the different types of failure mechanism is very similar to the original graph for the limestone rock mass with regular joint spacings (Figure 4.39). It can be argued that a variation in joint spacing has very little influence upon the development of jointed rock masses. If data upon the statistical variability of discontinuity spacing measurements are not available for a rock mass it will have little effect upon conclusions made about rock mass development. A balance needs to be sought between the increased effort of collecting a reliable data set and gaining an indication of the mean discontinuity spacing (Priest and Hudson, 1981). The only slight difference which exists between the two sets of boundary conditions is that there is an accentuated inflection on the boundary between toppling and no movement

for the rock masses with variable joint spacing. An explanation might invoke a random variation of spacing. The average b/h ratio for a rock mass with variable spacing may not be equivalent to the fixed b/h ratio of the original limestone rock mass.

The similarity between the boundary conditions is surprising considering the appearance of the discontinuity geometry. A model with a horizontal bedding angle and a b/h ratio of 0.8 fails by toppling (Figure 4.40a). It may have been supposed that the failure mechanism would be controlled by the narrow blocks that have a centre of mass close to the pivot point, but this appears not to be the case. The toppling model illustrated plots close to the boundary line on the b/h ratio versus α graph, as a model with a b/h ratio of 1.0 is stable (Figure 4.40b). The boundary between sliding failure and toppling-and-sliding failure for a rock mass with variable joint spacing is clearly defined. For example, a rock mass with a base plane angle of 60° dipping out of the slope and a b/h ratio of 0.4 fails by sliding (Figure 4.40c) and a rock mass with bedding inclined at 50° and a b/h ratio of 0.6 fails by a combination of toppling-and-sliding (Figure 4.40d). It is interesting to note that even though the two rock masses are geometrically similar as plotted on the graph, there is a difference in the appearance of the models due to the random variation in joint spacing.

4.4.2 Dip variability

The model of the limestone rock mass which was used to fix the original boundary conditions between different types of failure mechanism had a regular, fixed discontinuity pattern. Few rock masses will have such a precise geometry of joints. It is often apparent when analysing discontinuity data by stereographic projection that plotted data for individual discontinuity poles form a cluster around the mean joint dip value (Kulatilake and Wu, 1984). The variation within the cluster could be converted for UDEC model entry. It was decided to repeat the experiment varying the b/h ratio and the bedding angle α, for rock masses with joint sets which could vary randomly by up to 10% from the assigned discontinuity dip value (Table 4.2). Thus, a joint set which has an average dip of 50° on the UDEC mesh may have individual joint segments which vary in dip between 41° and 59°. All other parameters were input as before, and the command files are listed in Appendix 4.9. The exercise was aimed to create a new curve of boundary conditions between the different types of failure, graphically comparing the

results with the original b/h versus α curve. Thus, it would be possible to isolate the control of the statistical variation in the joint dip parameter upon modelled rock slope failure mechanism and landform development.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a rock mass with varied joint set dips (Figure 4.41). In all, 46 models were run to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.28. The shape of the boundary between the different types of failure mechanism is similar to the original graph for the limestone rock mass with regular joint spacings (Figure 4.42). The main difference is that for the boundary between toppling and stable rock masses, the curve for the model using a variable dip setting includes three small peaks in the curve. It is interesting to note that the boundary between toppling-and-sliding and sliding failure is smooth and plots very close to the boundary for the original rock mass. It can be argued that a variation in joint spacing has little influence upon the development of jointed rock masses, except that localised fluctuations from expected results may occur.

From looking at the discontinuity mesh, an explanation can be offered for the fluctuations in the boundary curve. Again, the average b/h ratio for a rock mass with variable joint dip may not be equivalent to the fixed b/h ratio of the original limestone rock mass. A model with a bedding angle of 20° out of the slope face and a b/h ratio of 1.6 fails by toppling (Figure 4.43a). It can be seen that by varying the joint dip by up to 10% for each joint set that the geometrical appearance of the rock mass changes, while there are large differences in block size and shape. It is therefore surprising that the curves for boundary conditions for the rock mass with a fixed joint pattern and with a variable joint dip are so similar (Figure 4.42). The history of total unbalanced forces plot for the model (Figure 4.43b) is similar to the history plots for other toppling models (Figure 4.8b for example). Activity occurs over different time periods and increases nearly exponentially to peaks which are of different magnitude. It appears that a sliding failure mechanism for a rock mass with a variable joint dip is controlled by the average sliding plane dip, as sliding did not occur for average dip values below the joint friction angle ϕ of 40°. A model with a base plane angle of 45° and a b/h ratio of 1.4 fails by a combination of sliding-and-toppling (Figure 4.43c). In the first part of the model run, blocks failed by a toppling failure, but by 400,000 cycles, most of the blocks are sliding.

The combination is reflected in the history of total unbalanced forces in the model (Figure 4.43d).

4.4.3 The effect of joint persistence

The model of the limestone rock mass which was used to fix the original boundary conditions between different types of failure had a regular discontinuity pattern with fully persistent joints. There are few real rock masses for which individual joints cut through the entire rock mass, and although difficult to measure in the field (Brown, 1981), it is thought that persistence has a strong control over rock mass form (Dershowitz and Einstein, 1988; Einstein *et al.*, 1983). Priest and Hudson (1981) suggest that persistence is commonly overestimated and that samples are biased.

A model mesh was set up whereby the 70° joint set contained individual discontinuities which were 40 m long, with a gap of 1 m before the next joint section, and the second joint set contained individual discontinuities which were 10 m long, with a gap of 2 m between the joint segments (Table 4.2). The mesh was constructed using the automatic joint generator facility within UDEC. It was decided to repeat the experiment varying the b/h ratio and the bedding angle α, for rock masses with joint persistency as set out above. All other parameters were input as before, and the command files are listed in Appendix 4.11. The exercise was aimed to create a new curve of boundary conditions between the different types of failure, graphically comparing the results with the original b/h versus α curve. Thus, it would be possible to isolate the control which a different joint persistence has upon modelled rock slope failure mechanism and landform development.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a rock mass containing impersistent joints (Figure 4.44). In all, 41 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.29. The shape of the boundary between the different types of failure mechanism is very different from the original graph for the limestone rock mass containing individual joints with full persistence (Figure 4.45). The main difference is that the regions for toppling-and-sliding and toppling only are much smaller. The plotted line for the boundary conditions between different failure mechanisms still reaches a peak at base plane angles α close to the joint friction angle ϕ of 40°. Toppling

will only occur for modelled rock masses under impersistent joint conditions between bedding which dips at 40° out of the slope face and bedding which dips at 10° into the slope face. It can be concluded that joint persistence exerts a large control over rock mass development. Rock masses with decreased persistence are more stable, and toppling can only occur for masses which have a *b/h* ratio of 0.5 or less. However, if observations made during the modelling exercise are accounted for then the extent of the control which a lower persistence exerts can be questioned.

The UDEC automatic joint mesh generator creates zones within the rock mass where there are no joints, and regions where there are persistent joint sets. Some of the individual block shapes seem incongruous and would certainly promote stability. The representation of the connectivity of joints is a major part of the persistence debate (Einstein, 1993). Kulatilake *et al.* (1992) suggested that fictitious joints need to be created to model rock masses with impersistent joint sets accurately. Toppling occurs in a region where joints are persistent. Given the forces which generate discontinuities in rock masses (Bergerat *et al.*, 1992), such mesh patterns do not resemble real patterns (Angelier, 1989). Therefore, although a decrease in joint persistence does increase rock mass stability, the difficulty of analysing joint persistence accurately using the UDEC simulation software confirms that it is difficult to measure and use (Einstein *et al.*, 1983). Its use in rock slope development problems should be cautious and it is recommended that slopes which contain persistent and repeatable joints are studied.

4.5 Other relevant properties

4.5.1 Groundwater level and water flow

The model which was used to fix the original boundary conditions between different types of failure mechanism was based upon Portland Limestone geotechnical data (Allison, 1986) and was assumed to be under totally dry conditions. Kakani and Piteau (1976) demonstrate that only when a high groundwater condition is included in the model of topples does the tensile stress region reflect real conditions. West (1996) suggests that the factor of safety for sliding-and-toppling is greatly reduced when pore pressures are present. In the Delabole Slate Quarry, Cornwall, modelling has confirmed the detrimental influence of a raised water table on the slope stability and that the observed seasonal opening and closing of tension cracks may be attributed to seasonal

rise and fall of the water table (Coggan and Pine, 1996). Other rock mechanics modelling codes have been used for the analysis of groundwater flow simulation (Bulut et al., 1996; Dewers and Hajash, 1995; Garga and Baolin Wang, 1993; Peters and Klavetter, 1988; Stibitz, 1996). A quantitative characterisation of the joint geometry in a rock mass is often used to determine the fluid flow in a slope (Antonellini and Aydin, 1995; Wei et al., 1995).

The possibility exists within UDEC to run a basic water flow algorithm within the modelling process, which has been extensively verified (Lemos and Lorig, 1990). The distinct element method has been applied to analysis of fluid injection into rock masses for the purposes of oil extraction (Harper and Last, 1989): the higher the rate of injection, the greater the number of available flow pathways (Harper and Last, 1990). Flow is idealised as laminar viscous flow between parallel plates. A coupled mechanical-hydraulic analysis is used where fracture conductivity depends on mechanical deformation and, conversely, joint water pressures affect the mechanical properties. Hydromechanical behaviour involves complex interactions between joint deformations and effective stress, causing changes in aperture and thus hydraulic conductivity (Bandis, 1993; Lemos and Lorig, 1990). At each timestep in the mechanical calculation, the computations determine the updated geometry of the system, thus yielding the new values of apertures for all contacts and volumes of all domains. Flow rates are then calculated from hydraulic algorithms. Given new domain pressures, the forces exerted by the fluid on the edges of the surrounding blocks can be obtained.

In virtually all of the environments of the world, jointed rock mass landforms will contain water at a certain level flowing between the blocks. Therefore, it was decided to repeat the experiment under the same settings, again varying the b/h ratio and the bedding angle α, for limestone rock masses containing a water table. Two experiments were attempted, one with a water table set at approximately half way up the back of the jointed part of the model at a height of 35 m, and one with a water table set to the top (50 m) at the back of the rock mass (Table 4.2). In both cases the water table was set to the base of the slope face at the front of the model. A steady-state flow algorithm was selected, fluid density set to 1 kg m^{-3}, fluid pressure set to 10 kPa and the pressure gradient set to -0.01 Pa m^{-1}. All other parameters were input as before, and the

command files are listed in Appendices 4.13 and 4.14. Again, the exercise was aimed to create two new curves of boundary conditions between the different types of failure, graphically comparing the results with the original b/h versus α curve.

The boundary between the different types of failure mechanism is drawn on a graph of b/h ratio versus α for a rock mass containing a water table which is set to 35 m at the back of the rock mass and set to the base of the slope face at the front of the rock mass (Figure 4.46). In all, 33 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.30. The shape of the boundary between the different types of failure is very similar to the original graph for a dry limestone rock mass (Figure 4.47) and there are no noteworthy differences. It may have been expected that a rock mass with fluid water flow in the discontinuities would decrease in stability, and be affected in joint friction angle, but the water table appears to exert very little influence.

A modelled rock mass which has a base plane angle of 20° dipping out of the slope face and a b/h ratio of 1.6 fails by toppling (Figure 4.48a). If the flow rates are plotted for the model (Figure 4.48b), it can be seen that most of the water flow occurs between the toppled columns and at the toe of the slope face. If the pore water pressure is plotted for the model (Figure 4.48c), the location of the water aquifer at the base and to the rear of the rock mass is illustrated. It is impossible to ascertain whether the high fluid pressure at the base of the toppling columns has any influence upon the failure mechanism. For a stable modelled rock mass, which again has a base plane angle of 20°, but a b/h ratio of 2.0, virtually all of the water flow is concentrated at the toe of the slope (Figure 4.48d). Also, the plot of pore water pressure indicates that virtually all of the water is located in the aquifer at the rear of the model (Figure 4.48e). The question then arises: is the water surrounding the toppling columns of the unstable rock mass (Figure 4.48c) causing the toppling, or is there a pore pressure in that part of the model because of the toppling. Goodman and Bray (1976) suggest that water will not be found in the toppling zone because of the openness of the joint system. The curves of boundary conditions for failures are similar between wet and dry rock masses, suggesting that the latter is the correct answer, although further work is required here.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a rock mass containing a water table which is set to 50 m at the back

of the rock mass and set to the base of the slope face at the front (Figure 4.49). In all, 37 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.31. The shape of the boundary between the different types of failure is similar to the original graph for a dry limestone rock mass (Figure 4.50), but there are some differences from the curve for the rock mass with a water table set to 35 m (Figure 4.51). There appears to be a good similarity for the boundary conditions between toppling-and-sliding and sliding failure, but the boundary between toppling failures and stable rock slopes is inflected for the rock mass containing a water table set at 50 m (Figure 4.49). Thus, the occurrence of a high water table does appear to have some effect on rock mass failure mechanisms. At base plane angles of between 20° and 40° there is a greater level of rock mass instability, as indicated by the position of the limiting boundary, but it may have been expected that a rock mass with such a fluid flow in the discontinuities would show a far greater difference in stability.

A modelled rock mass which has a base plane angle of 41° dipping out of the slope face and a *b/h* ratio of 4.4 is starting to fail by toppling-and-sliding, as indicated by the displacement vectors (Figure 4.52a). A plot of the flow rates for the model (Figure 4.52b) shows that most of the water flow occurs between the toppled columns and at the toe of the slope face. Flow is also along the line of the tension crack in the rock mass between the toppling columns and stable columns. If the pore water pressure is plotted for the model (Figure 4.52c), the location of the water aquifer at the rear of the rock mass is illustrated, but there is some pressure along the discontinuity where failure is about to take place. Again, it is impossible to ascertain whether further water pressure will build up in this region as a prelude to further movement, or the pressure builds up as a consequence of the movement. The history plot of total unbalanced forces for the model (Figure 4.52d) suggests that a large failure event has yet to occur. The creep of the blocks close to the slope face is reflected in small peaks of activity.

From the exercise of parameter sensitivity using the rock mass with a water table set to 50 m, it can be concluded that water pressure does have a very small influence upon rock mass failure when there is a high water table. The result has implications for studies of climate change, as rock mass landforms which occurred under wetter conditions may have undergone slightly greater rates of retreat as the jointed slopes were less stable. However, it is extremely difficult to judge the water pressure and level

of the water table at depth in the rock mass, and the quantity of water in the rock mass fluctuates with short- and long-term variations in climate. Pritchard and Savigny (1990) suggest that pore water, within a reasonable range of pressure, does not appear to affect the morphology of slope failure. Thus, it is recommended that the water pressure parameter is not used in the modelling of rock slope landforms.

4.5.2 Joint stiffness

A water table in a rock mass has the effect of exerting a pressure in the joints which affects the strength of the joint contact. Yoshinaka and Yamabe (1986) demonstrated that the whole behaviour of a jointed rock mass is affected by joint stiffness. The original strength contact value is entered into UDEC by assigning values to the parameters of joint normal and shear stiffness. The Coulomb slip model within UDEC runs a stress-displacement relation in the normal direction governed by the joint normal stiffness (Swan, 1983). In shear, the response is similarly governed by shear stiffness, but is limited by a combination of cohesion and frictional (ϕ) strength. As the fluid pressure in joints had little effect on the failure mechanism of a rock mass, it was decided to see whether an increase in the joint stiffness properties would have any influence. Therefore, it was decided to repeat the experiment under the same settings, again varying the b/h ratio and the bedding angle α, for limestone rock masses which had higher joint stiffness settings (Table 4.2).

The original limestone rock mass had joint normal and shear stiffness set to 1 GPa (Bandis *et al.*, 1983; Barton, 1976; Jaeger and Cook, 1979; Kulhawy, 1975). The model used in this experiment had joint normal and shear stiffness of 5 GPa. The joint normal and shear stiffness should be set to approximately ten times the equivalent stiffness of the stiffest neighbouring zone according to Itasca (1993):

$$k \approx \max\left[\frac{k + 4/3g}{\Delta Z_{min}}\right]$$

(4.5.2 - 1)

where k is the value of stiffness,

g is bulk modulus, and

ΔZ_{min} is the smallest width of an adjoining zone in the normal direction.

Rock joints exhibit a wide spectrum of shear strength under the low effective normal stress levels operating in most rock engineering problems (Bandis, 1993; Barton, 1976). Bandis *et al.* (1983) suggested that the normal and shear stiffness of joints will be influenced by the initial contact area, the joint wall roughness, the strength and deformability of the asperities and the thickness and type of infilling material. The experimental study, using five different rock types, generates quantitative characteristics of rock joint deformation. The behaviour of a joint also depends upon the micro-features of the contact planes on the joint (DeToledo and DeFreitas, 1993; Dong and Pan, 1996). Joint stiffnesses vary from 10 to 100 MPa for joints with a clay infilling to 100 GPa for tight joints in granite and basalt (Itasca, 1993). A constitutive law for rock joints has been developed based upon two laboratory tests, but is based upon various assumptions, particularly that the joints are free of material (Leichnitz, 1985). Other attempts have made use of a disturbed state modelling concept (Desai and Ma, 1992). Empirical methods have been used to estimate the joint roughness coefficient and the joint compressive strength (Barton and Choubey, 1977) and a sub routine has been written to incorporate the measures into the UDEC code (Barton and Bandis, 1990). It has been suggested that the Barton roughness coefficient is too simplistic, and further equations have been developed (Reeves, 1985). Many joint models are too simplistic, rely too heavily on empiricism or require complex input parameters which are beyond the capabilities of a site investigation (Haberfield and Johnson, 1994). It is very difficult to measure joint stiffness (Aydin and Kawamoto, 1990) and it is recommended that published values are used (Itasca, 1993).

In all, 37 models were run in order to fix the boundary line (Appendix 4.12) and the corresponding mechanisms of failure are listed in Appendix 4.32. The boundary between the different types of failure is drawn on a graph of *b/h* ratio versus α for a rock mass with joint normal and shear stiffnesses set to 5 GPa (Figure 4.53). As with the water table parameters, the shape of the boundary between the different types of failure is very similar to the original graph for the limestone rock mass (Figure 4.54) and there are no noteworthy differences. It may have been expected that a rock mass which had joint stiffnesses five times greater would increase in stability, but, if there is any difference, it is that the original rock mass which had a joint normal and shear stiffness

of 1 GPa is very slightly more stable. There were also no interesting or unusual observations made during the exercise. However, it can be concluded that the exact value assigned for joint stiffness in rock mass modelling does not have a large effect upon the failure of a rock mass. Other parameter sensitivity studies using UDEC to simulate rock mass failure mechanisms have suggested that joint strength parameters have little control (Bhasin and Hoeg, 1998). As the parameter is very difficult to measure, either in the laboratory, or from correlation with joint material factors, it is appropriate to use text-book values for modelling exercises.

4.5.3 Geometry of the model boundary conditions (length of the wave-cut platform)

The model which was used to fix the original boundary conditions between different types of failure mechanism was formed from a simple, rectangular block. However, in virtually all geomorphic scenarios, the toe of the slope is bounded by either a wave-cut platform for coastal cliffs, or a pediment section. Therefore, it was decided to repeat the experiment under the same settings, again varying the b/h ratio and the bedding angle α, for limestone rock masses bounded by a wave-cut platform at the base of the slope. A horizontal rock platform was set up to extend 50 m from the base of the slope (Table 4.2). All other parameters were input as before, and the command file is listed in Appendix 4.15. Again, the exercise aimed to create new curves of boundary conditions between the different types of failure, graphically comparing the results with the original b/h versus α curve.

The boundary between the different types of failure is drawn on a graph of b/h ratio versus α for a rock mass bounded by a 50 m wide wave-cut platform (Figure 4.55). In all, 27 models were run in order to fix the line and the corresponding mechanisms of failure are listed in Appendix 4.33. The shape of the boundary between the different types of failure mechanism for a rock mass bounded by a wave-cut platform is virtually identical to the original graph for a limestone rock mass (Figure 4.56). It can be concluded that the mode of rock mass failure is unaffected by the nature of the slope or platform occurring at the toe of the slope. A model which has a base plane angle of 10° dipping into the slope and a b/h ratio of 0.4 fails by toppling (Figure 4.57a). It may have been supposed that a sliding form of failure mechanism could be affected by the

position of a platform preventing the movement of lower layers of the rock mass. But a rock mass which has a bedding angle of 60° and a *b/h* ratio of 0.4 fails by sliding (Figure 4.57b) and a rock mass which has the same base plane angle, but a *b/h* ratio of 0.2, fails by toppling-and-sliding (Figure 4.57c).

4.5.4 Failure mechanisms of rock masses above a softer base

There are geomorphological situations where a hard jointed rock mass occurs above a softer base which has a strong influence on the slope development (Cancelli and Pellegrini, 1987; Pasuto and Soldati, 1996). Situations which have been discussed include the Isle of Portland, where the Portland Limestone overlies a Kimmeridge Clay base (Brunsden *et al.*, 1996), and the Austrian Alps, where three modes of failure are observed (Steger and Unterberger, 1990). Blocks from the hard, jointed rock mass fail by toppling, by sinking or by riding on the softer material.

The possibility exists within UDEC to model a basic mass composed of softer sediment, and it was thought to be interesting to study the failure of a jointed Portland limestone rock mass resting above an inclined clay base. The clay part of the model contained deformable zones and used typical textbook geotechnical properties (Bell, 1983; Stepkowska, 1990). The clay density was set to 2,200 kg m^{-3}. A cohesion of 4 MPa, a block dilation angle of 20°, block friction angle of 26° and a tensile strength of 20 MPa were also given. The Drucker-Prager plasticity model was used for the clay part of the model which provides a simple representation for a material yielding in shear. The UDEC plasticity model comprises two basic functions: a yield criterion and a plasticity flow rule. It was decided to repeat the experiment under the same settings, again varying the *b/h* ratio and the bedding angle *α*, for a limestone rock mass resting above a clay base. The clay base was inclined at 30° to the toe of the limestone scarp face, and extended for 50 m from the scarp. The model boundary, including the deformable zones for the clay, is illustrated in Figure 4.58. The command file for the UDEC input is listed in Appendix 4.34 and all the parameters for the limestone part of the model were entered as for the original experiment.

The exercise was aimed to create two new curves of boundary conditions between the different types of failure, graphically comparing the results with the original *b/h* versus *α* curve. Due to the plastic movement of the clay base, all models

failed, whatever the dimensions of the blocks in the rock mass cap-rock. Generally failure occurred by slumping of limestone into the clay which caused movement of the clay base. However, it was possible to identify models where toppling, sliding and toppling-and-sliding failure mechanisms occurred together with the slumping. As the failure within the rock mass depended upon discontinuity geometry, it was possible to plot the boundary between mechanisms (Figure 4.59) (Appendix 4.35). When compared with the boundary conditions between failure for a limestone rock mass without a clay base (Figure 4.60), it can be seen that a soft base exerts a great control upon rock mass failure. There are a much greater number of conditions for which toppling and also a sliding-and-toppling occurs. It could be that the reduction of support from a base below the columns of rock affects the degree to which columns can rotate.

A model which has a limestone mass with a b/h ratio of 1.6 and a base plane angle of 20° dipping into the face fails by toppling (Figure 4.61a). The toppled columns from the limestone part of the mass collapse into the clay, which then extrudes and bulges. The clay has extruded by more than 20 m at the base of the model when compared with the starting conditions (Figure 4.58). The observations made here support arguments put forward by Brunsden et al. (1996), who suggested that the Portland Limestone causes the basal Kimmeridge Clay to bulge and extrude on the Isle of Portland, Dorset. Not only does the model demonstrate clay extrusion, but early stages of the toppling failure into the clay lead to the development of caves within the limestone rock mass (Figure 4.61a). Caves have been reported at similar positions in the rock masses on the Isle of Portland (Brunsden et al., 1996). The history plot of total unbalanced forces for the model indicates a large peak of activity during the early part of the model run, but lower, frequent levels of activity during the remaining time (Figure 4.61b).

A limestone model with a discontinuity geometry defined by a bedding angle of 40° dipping into the slope and a b/h ratio of 0.6 would be stable if standing alone. However, with a clay base below, the limestone blocks appear to slump (Figure 4.61c). There is no evidence of any toppling mechanism, although there is evidence of extrusion and bulging of the clay. A limestone model with a discontinuity geometry defined by horizontal base planes and a b/h ratio of 1.0 would also be stable if standing alone. But when supported by a clay base the model definitely fails by toppling, and causes more

than 50 m of clay extrusion (Figure 4.61d). By the monitoring of velocity vectors in the plot, clay consolidation can be seen a shear surface develops within the clay material. Perhaps the greatest extent of extrusion of clay occurred when models slid into the base. A rock mass with bedding dipping at 50° into the slope face and a *b/h* ratio of 2.0 fails by sliding and causes not only a large bulge in the forefront of the failed blocks, but also a large quantity of clay extrusion (Figure 4.61e). The clay below the sinking blocks will be consolidating and compacting. The observations from the exercise increase understanding of the processes that occur when a rock mass overtops a softer base. Clay below a rock mass exerts a very large control on the behaviour of the rock mass. Not only does block slumping occur whatever the conditions, but toppling occurs for a greater range of joint geometric conditions in the limestone rock mass. Hypotheses of slow sediment extrusion that is related to the nature of the failure of the overlying rock mass are confirmed. The exercise provides the impetus for further investigation into such landform development, although the amount of sinking of the rock blocks needs to be examined.

4.6 Conclusion

The limiting boundary conditions between the failure mechanisms of toppling, sliding and toppling-and-sliding in conjunction have been defined, based upon the discontinuity geometry, for a modelled limestone rock mass using UDEC. The boundary conditions for a rock mass, which responds to the dynamic forces of the interaction of multiple blocks, are very different from the boundary conditions for the failure of a single block which can be defined kinematically. It is possible for rock masses which have horizontal bedding to fail by toppling and there are only a small number of conditions for which toppling-and-sliding may occur. While defining the limiting conditions, it was noted that rock mass activity occurs in distinct pulses. The pattern of activity reflects the mode of failure, and the magnitude/frequency relationship exhibited in the pattern has been widely discussed for other landforms.

The boundary conditions for the failure of the limestone rock mass can be used as a template for testing the parameter sensitivity of the UDEC rock mass models. The large control of slope height and angle has been analysed elsewhere (Hsu and Nelson, 1995; Jiang *et al.*, 1995). The methodology used highlights the understanding which can

be gained from using well-defined models. A number of experiments were run to examine the effect of different rock mass properties on slope stability (Table 4.5). Of the input material property parameters the joint friction angle has the most important control on the stability and the failure mechanisms of rock masses, as it influences the location of the peak in the boundary conditions curve. The joint friction angle has been identified elsewhere as being important, together with the discontinuity arrangement (Caine, 1982; Jiang *et al.*, 1995). However, the joint friction angle needs to be input consistently with other intact rock strength parameters, which have less influence. The use of deformable blocks makes little difference to the failure of rock slopes, but there is uncertainty with some of the results and the models using deformable blocks take a long time to run. The factors which affect the joint stiffness values input into UDEC, including the presence of water in the joints, also have little influence upon rock mass stability. The discontinuity geometry input parameters appear to have a greater control upon the failure of rock slopes. A decrease in the joint persistence of individual discontinuities increases the stability of rock masses, but joint persistence is difficult to measure in the field and due to modelling difficulties it is difficult to ascertain the control of this property. Statistical variation in both spacing and dip leads to a surprisingly small difference in the graph plot for limiting boundary conditions given the appearance of the joint mesh after the parameters had been altered (Table 4.5). However, it is the value of dip of the discontinuities and joint spacing which exerts the greatest control upon rock slope failure mechanisms as it is these parameters that control the discontinuity geometry that in turn governs the limiting boundary conditions.

Experiment	Section	Indication
Varying block height	4.2.2	Rock masses are more unstable, but large number of blocks required.
Rectangular blocks	4.2.3	At low angles of base plane, more stable; at high angles, less stable.
Friction angle 20°	4.3.1	Creep toppling and stabilising evident. Property influences location of the graph peak.
Friction angle 60°	4.3.1	Again, the property influences the location of the graph peak.
Sandstone rock mass	4.3.2	More realistic than friction angle experiments. Graph peak occurs at 29°.
Granite rock mass	4.3.2	Graph peak occurs at 46°.
Deformable blocks	4.2.3	Little effect. Strong, well-jointed rock masses fail along discontinuities. Long model run-time.
Varying joint spacing	4.4.1	Little effect.
Varying joint dip	4.4.2	Little effect.
Varying joint persistence	4.4.3	Large effect, but difficult to represent this property.
Water table	4.5.1	Little effect, even with a high water table.
Varying joint stiffness	4.5.2	Little effect.
Wave-cut platform	4.5.3	No effect.
Clay base	4.5.4	Large effect. Rock mass above the clay is much less stable.

Table 4.5: Summary of experiments undertaken to examine the effect of different rock mass properties on slope stability.

Chapter 5: The field study areas: Isle of Purbeck, Dorset, UK and Colorado Plateau, USA

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5.1 Introduction to The Isle of Purbeck, Dorset field area

The Isle of Purbeck, Dorset, central southern England, is a distinctive geological region separated from surrounding districts by a chalk ridge that runs between Bat's Head (SY 795 803) in the west and Ballard Point (SZ 048 813) in the east (Figure 5.1) (Jones *et al.*, 1983). The coast of the Isle of Purbeck is best known for its variety of rocks and famous fossil localities, but visitors are struck by the equally impressive coastal landforms (Goudie and Brunsden, 1997). It is one of the best locations in Britain to demonstrate the links between rocks and relief. The coast includes classic features such as Durdle Door (SY 806 802), Lulworth Cove (SY 827 797), Worbarrow Bay (SY 868 795) and St. Alban's Head (SY 961 753). The plan form and profile of the coastline is a direct consequence of sedimentary rocks that have differing resistance to weathering and erosion. The harder rocks of the Portland, Purbeck and Chalk form resistant cliff barriers to erosion, while the softer Wealden Beds, Upper Greensand and Gault are eroded into embayments.

The Isle of Purbeck is one of the warmest regions of the United Kingdom. It has a temperate climate with few days of frost and less than five days of snow each year (Manley, 1952). A principal event leading to slope movements and landscape development in its current form was the last rise in sea level (Brunsden and Goudie, 1981). Since the end of the Holocene, the sea level in the English Channel has been rising at 1 mm a^{-1}, although the effects are increased by simultaneous tectonic subsidence of the land mass (Shennan, 1989). Bays such as Worbarrow and Lulworth have been developed by a combination of slope instability and marine processes. During the last glacial, streams running on permanently frozen ground would have cut through the current coastal cliffs in places, running to the central English Channel. Much of the Isle of Purbeck comprises grass downland hills of greater than 150 m and associated coastal cliffs that reach 100 m or more in height at Bat's Head, Gad Cliff and St Alban's Head. However, it is the coastal landscapes that have attracted greatest interest from scientific researchers (House, 1993).

5.1.1 Geology

The visible succession at the Isle of Purbeck begins with the Upper Jurassic, represented by the Kimmeridge Clay, and continues in sedimentation up to the Lower Tertiary (House, 1958) (Figure 5.2). The rock outcrops run approximately parallel to the coastline and topographic changes conform closely to both lithological and structural differences (Figure 5.3) (Jones *et al.*, 1983). There is regional thinning of marine sediments related to distance from the shore, with the greatest thickness of sediment to the south-east of Dorset. Resistant Chalk forms the Purbeck Hills, a ridge that runs for 20 km at the northern boundary of the region with a height of 199 m at Godlingston Hill. To the south of the Isle of Purbeck, a second ridge of Portland Limestone reaches 120 m at Gad Cliff. Between the two ridges the centre of the district is underlain by the Wealden Beds, which broaden in outcrop from 50 m at Durdle Door to 4.5 km at Swanage. The Greensand and Gault outcrop between the Wealden and Chalk, and the Purbeck Beds between the Wealden and Portland Limestone are susceptible to failure.

There is great stratigraphic diversity both within and between individual rock units that has a considerable effect on the engineering performance of the rocks (Allison, 1986). Evidence from cores suggests that the sediments of eastern Dorset are underlain by Triassic mudstones in troughs, and Lower Jurassic Lias (Figure 5.2) (House, 1993). Above, the black shales and overconsolidated clays of the Late Jurassic Kimmeridge Clay are exposed in a 10 km length of the coastal cliffs between Gad Cliff and St. Alban's Head (Figure 5.3). The Kimmeridge Clay was deposited in a deep-marine environment, but the conformable succession above indicates a progressive shallowing of depositing environments with several short-term sea-level cycles (Coe, 1996). It comprises 500 m of clays and black shales that are often bituminous with occasional bands of lime mudstone, silty mudstone, limestone and dolomite (Cope, 1978). The limestone bands act as controls on process and form. Where limestone outcrops at sea level, for instance at the Kimmeridge Ledges, bands can extend offshore, reducing marine energy at the cliff foot. The upper surface acts as a potential failure surface, with the seepage of water occurring below clays.

Above the Kimmeridge Clay in the succession is the Portland Group (Figure 5.2). The Group is composed of Portland Sand and Portland Limestone and seven formations (Figure 5.4) (Melville and Freshney, 1982). The Portland Sand and

Limestone units seldom both outcrop at the same place. In a few places along the Isle of Purbeck coastline, such as Chapman's Pool and Dungy Head, Portland Sand is exposed. The Portland Sand is a blue-grey, silty, sandy dolomitic mudstone and has more affinity with the Kimmeridge Clay below than the Portland Stone above (Arkell, 1947). It is composed of four members (Townson, 1975), all of which are predominantly plastic materials and very susceptible to weathering, erosion and the initiation of mass movement (Allison, 1986).

The most important part of the Upper Jurassic in Dorset is the resistant Portland Limestone Formation that forms the entirety of the sea cliffs in many places in the Isle of Purbeck (Arkell, 1933; Strahan, 1898). The Jurassic Portland Limestone is a bioturbated, hard, brittle, shelly, crystalline sediment with masses of chert and fossils (Arkell, 1947; Cox, 1929). The extent of the outcrop along the Isle of Purbeck coastline is indicated in Figure 5.5. The outcrop thins from east to west. In the east, the Portland outcrop is first exposed in the cliffs at Durlston Head and runs for 10 km to St Alban's Head. Between Chapman's Pool and Gad Cliff the unit forms an inland ridge, with Kimmeridge Clay outcropping at the coast. To the west of Gad Cliff, the Limestone forms the sea cliffs for a major part of the coastline, breached by embayments at Pondfield, Worbarrow, Lulworth and St Oswald's. Air photographs show a submerged reef of Portland Limestone across these bays marking the position of the original cliffline (Allison, 1986). The Portland Limestone Formation forms a resistant rampart to erosion in many locations, and the development of the coastal cliffs cut into the outcrop forms the basis of this study.

Portland is a well known building stone, becoming famous due to use by Sir Christopher Wren after the Great Fire of London (Edmunds and Schaffer, 1932; Fraaye and Collins, 1996). Its durability, low anisotropy and lack of tectonic disturbance are properties that make it ideal for quarrying (Hounsell, 1952), although it has relatively low strength compared with more widely used natural materials (Clark, 1988). Discontinuities and planes of bedding are well defined throughout the Portland Limestone outcrop and relate to the regional structure. Rocks become detached from Portland Limestone cliffs by joint control mechanisms. There is a variation in intact rock strength between the members depending on the proportion of ooliths (Clark, 1988), with the Roach beds of the Winspit Member having the greatest unconfined

compressive strength. The intact geotechnical properties are typical for clastic sedimentary rocks, with greater strength than arenaceous or argillaceous sedimentary rocks, but much weaker than igneous rocks. The hardness of the rock, along with its anisotropy and regular discontinuity pattern, makes the Portland Limestone outcrop ideal for the study of jointed rock landforms.

The Isle of Purbeck Portland Limestone outcrop varies in thickness from 26 m at Dungy Head to a maximum of about 42 m at St Alban's Head (Townson, 1975). The lithostratigraphy of the Portland Limestone Formation was first described by Strahan (1898). Arkell (1947) divided the Formation into the three zones of Basal Shell Bed (1.8 m), Cherty Series (15.2 - 18.3 m) and the upper Portland Freestone (7.6 m) (Figure 5.4). Townson (1975) described a basal Dungy Head Member (4.5 - 14 m), a Dancing Ledge Member (6 - 15 m) and an upper Winspit Member (6 - 19 m). The biostratigraphy was described by Wimbledon and Cope (1978) who split the Portland Limestone Formation into two based upon the occurrence of the fossils *Galbanites kerberus* and *Titanites anguiformis*.

The lower beds of the Dungy Head Member are pale, bioturbated limestones rich in calcified *Rhaxella* spicules and with masses of chert (Melville and Freshney, 1982). Arkell (1947) described the upper bed of the Dungy Head Member (lower Cherty Series) as a shelly, greyish brown limestone with some chert. The two contrasting beds are named the Portland Clay and the Portland Shell Bed (Brunsden *et al.*, 1996). The Dancing Ledge Member is defined by the proportion of *Rhaxella* spicules and is a fine-grained shell-sand limestone (Townson, 1975). The Winspit Member is a fine-grained shell-sand limestone with oolitic layers increasing towards the west of the Formation and forms a sharp junction with the Dancing Ledge Member (Melville and Freshney, 1982). The Winspit Member forms most of the Portland cliffline between Durdle Door and Worbarrow Tout and its general competence allows the formation of vertical cliffs (Allison, 1986). Townson (1975) proposed a three-fold division of the Winspit Member into the Base Beds and the Base Bed Roach, a soft white limestone, the middle beds including the Curf and Flinty Bed and the upper bed including the Roach and Whit Beds, which were deposited in a shallow, high energy environment (Fraaye and Collins, 1996). It is the beds of the Winspit Member, particularly the Roach beds, which are quarried, from the Isle of Portland, the cliffs between St Alban's Head and Durlston

Head and inland at Worth Matravers. Discontinuity measurements were taken in the Winspit Member for this study.

Above the Portland Group, the Purbeck Group overlies without a sharp break (Figure 5.2). The Purbeck Beds are composed of several formations of lacustrine and lagoonal limestones and marls, shales with gypsum and calcareous mudstone, representing a marine to lagoonal transition. It is well exposed at the entrance to Lulworth Cove and outcrops extensively throughout the Isle of Purbeck (Anderson and Bazley, 1971; Gorman and Williams, 1996; Westhead and Mather, 1996). The Purbeck Beds are divided into a lower Lulworth Formation and an upper Durlston Formation and thin rapidly to the west (Melville and Freshney, 1982; Townson, 1975). Dirt bed seams and the marls and shales are weathered easily by mechanical and chemical disintegration and are removed by marine action. This has the effect of destabilising screens of limestone resulting in extensive collapse of overlying beds. At both Lulworth Cove and Durdle Door, folds occur within the Purbeck strata due to the movement of the incompetent Wealden sediments at the contact with the more competent Purbeck Formation (West, 1992). This leads to complex failure of the material (Allison, 1986).

The Jurassic / Cretaceous boundary in Dorset is drawn at the junction between the Purbeck Group and Wealden Group at about 144 million years BP. The less resistant Wealden Group includes terrestrial alluvial fan and fluvial deposits (Anderson, 1967) and outcrop throughout the area forming the subdued relief of central Purbeck. The sandstones, marls, clays and grits of the Wealden Group are well exposed in Lulworth Cove where mudslides and marine removal have resulted in the large embayment (Burton, 1937). The coastal cliffs are blanketed in landslides characteristic of soft, plastic, incompetent materials, while at many inland locations slopes in the Wealden are mantled with degraded landslides (Allison, 1986).

During the mid-Cretaceous a series of marine transgressions resulted in the deposition of Lower Greensand, Gault and Upper Greensand (Figure 5.2). These outcrops are all small and do not contribute significantly to the geomorphology of the area (Allison, 1986). The Lower Greensand is a shallow, marine, variegated sand and clay deposit (Casey, 1961), the Gault is a marine clay, while the Upper Greensand is a glauconitic, marine sandstone. A major unconformity, due to Albian tectonic activity, cuts Gault and Upper Greensand onto the Lower Greensand. The Chalk of the Upper

Cretaceous provides the next resistant rock to erosion. It is widespread in its extent, and appears in the coastal cliffs of the Isle of Purbeck at Bats Head, Bindon Hill at the back of Lulworth Cove, Whiteway Hill at the back of Worbarrow Bay and in the east at Ballard Point. It outcrops continuously in an east-west ridge from Ballard Down to Bat's Head (Kennedy and Garrison, 1975). The Chalk is thought to represent an open-water shelf deposit, with rock horizons representing shallower water periods (House, 1958). It comprises a series of bioturbated, white and light grey limestones with subordinate marls (Kennedy and Garrison, 1975). In a few locations about Lulworth, Tertiary fluvial sands and gravels occur above the Chalk and are affected by folding and faulting which was occurring at the same time.

The rocks of the Dorset coast have played a significant part in the establishment of a biostratigraphic scale for the Jurassic, Cretaceous and Lower Tertiary rock succession (House, 1993). Correlation has been made with the stratigraphy of northern France (O'Schmarnn, 1988). Biostratigraphically, the Portland Limestone Formation and Portland Sand Formation make up the Portlandian Stage, which are divided into smaller correlation zones on the basis of particular fossils. Marine ammonite fossils that exist in the Portland Stage divide five zones, three in the Portland Sand Formation and two in the Portland Limestone Formation (Wimbledon and Cope, 1978). Non-marine ostracods divide the Purbeck Group.

5.1.2 Structural link with the present-day coastline

There have been two major tectonic periods that have affected the sediments of the Isle of Purbeck. The Albian period is indicated by a major discontinuity between the Lower and Upper Cretaceous sediments and the resulting tensional structures affected later tectonic activity. The second period occurred during the early to mid-Tertiary when faulting and folding caused prominent compressional structures in southern England (Bevan, 1985). The Isle of Purbeck is principally influenced by the asymmetric Purbeck Monocline that runs east to west and slightly oblique to the coast and with a gentle easterly plunge (Figure 5.5). The Purbeck fold is associated with a mid-Cretaceous normal fault that was reactivated by reversed movement (House, 1993). South of the coastline, strata gently dip to the south. To the north of the Isle of Purbeck, the axis of the shallow Frome Syncline passes from Dorchester to Poole. To the south in the

English Channel is the Shambles Syncline, whose axis crosses the southern point of the Isle of Portland (Brunsden *et al.*, 1996), and further to the west is the Weymouth Anticline (Figure 5.6). Offshore, the Kimmeridge Clay occupies a considerable area of the seafloor and Portland Limestone forms prominent ridges (West, 1998).

The surface geology evident along the present-day coastline of the Isle of Purbeck is a consequence of a juxtaposition of layers of sedimentary rock of varying resistance to erosion and the relative position of locations on the Purbeck Monocline. The resulting suite of landforms are commonly regarded as classic features (Brunsden and Goudie, 1981). Different parts of the structure are exposed along the cliffs, as is illustrated in Figure 5.7. In the east, at St Alban's Head (Figure 5.7, section A-B), the central section of the Purbeck Monocline is evident at the coast. Therefore, the stratigraphy at the coast is horizontally bedded. The coastal cliffs between St Alban's Head and Durlston Head are entirely composed of Portland Limestone that acts as a resistant rampart to the structural plateau of the Purbeck Beds behind. From St Alban's Head and Chapman's Pool, the structural control upon the landscape form is clear. The Portland Limestone outcrop caps a toe slope Kimmeridge Clay section in the coastal cliffs. Rockfalls from the Limestone are evident upon the toe slope surface, and there are deep-seated sides and mudslides within the Clay. Between Chapman's Pool and Gad Cliff the present-day coastline exists to the south of the Portland Limestone outcrop, and sea cliffs are composed of Kimmeridge Clay. At the western end of Gad Cliff (Figure 5.7, section C-D), the Portland Limestone outcrop occurs at sea level. Here, the northern limb of the Purbeck Monocline is exposed in the coastal cliffs, with bedding inclined at 25° north.

Across Worbarrow Bay, the Portland Limestone has been penetrated, and the softer sediments of the Purbeck and Wealden Beds have been washed away, exposing resistant Chalk cliffs at the back of the bay. An explanation of the behaviour of the Chalk cliffs of the Isle of Purbeck was attempted by May and Heaps (1985). Rates of retreat were estimated between 0.05 to 1 metre each year and rock fall debris was monitored. It was concluded that the main eroding agent is sub-aerial and that the marine role is two-fold: the sea develops notches and transports small calibre sediment from rock falls. The Portland Limestone outcrop continues at the western end of Worbarrow Bay, at Bacon Hole. The Wealden valley is much narrower at Worbarrow

Bay than at the eastern end of the Isle of Purbeck, near Swanage. A steep dip, a thinner Wealden succession and some strike faulting are the major reasons for the narrow outcrop in the west (West, 1998).

Lulworth Cove is a commonly visited geological locality as it provides a fine example of marine erosion in steeply dipping strata of very unequal resistance. It is almost circular, about 500 m in diameter, with the breach in the Portland Limestone being 120 m wide (Damon, 1884). It is situated closer to the axis of the Purbeck Monocline, with the dip of the bedding in the Portland Limestone here being 36° to the north (Figure 5.7, section E-F). The rocks of the Purbeck crop out at the top of the coastal cliffs here. Again the Portland Limestone has been penetrated, by fluvial and marine action, and the Chalk is exposed at the rear of the Cove. The understanding of the evolution of the coastline in the Lulworth Cove region is often based upon the principle of space-time substitution (Goudie and Brunsden, 1997). A sequence of forms commences with features such as the small embayment at Stair Hole. Here the sea has penetrated the barrier Limestone creating narrow arches. The sea is close to undercutting the Wealden Beds and landslides and mudslides have recently been activated (Allison, 1986). When the softer Wealden Beds are exposed, rapid erosion develops behind the Limestone entrance, leading to a symmetrical bay, such as Lulworth Cove (Plate 5.1). Mupe Bay, Worbarrow Bay and St Oswald's Bay represent a condition where bays have possibly coalesced, the Portland and Purbeck Beds are destroyed until only small stacks are visible from submerged reefs, and the rear of the bays have reached the Chalk (Plate 5.2). Each side of the Durdle Door promontory represents relict horns, similar to which are formed in the Portland Limestone at the entrance to Lulworth Cove. However, complexity is added to the space-time substitution model by three main factors. The beds thicken towards the east along the Isle of Purbeck coastline, and thus, the development of the coastline is greater at Worbarrow Bay than at St Oswald's Bay. Second, it has been suggested that Lulworth Cove and Worbarrow Bay were breached by high discharge streams during colder climates when the sea level was lower (Goudie and Brunsden, 1997). Thus, the two bays represent ria features, and differ from the partially developed Stair Hole. Also, the resistance of the Portland Limestone barrier changes along the coastline. The dip of the bedding steepens from about 25° at Worbarrow Bay, to nearly vertical at the Durdle Promontory.

The most westerly point where the Portland Limestone Formation is exposed in the coastal cliffs is at the Durdle Promontory (Figure 5.7, section G-H). The Monocline is vertical or overturned in the promontory, but to the south, the Portland Limestone dips at about 50° north. As a result of the thinning of the sediments and the Purbeck Monocline the distance between the Portland Limestone and Chalk reaches 4.5 km at Swanage, but is little more than 50 m at Durdle Door (Arkell, 1947). Traces of the Portland Limestone outcrop can be seen at low tide across St Oswald's Bay and to the west of Durdle Door. The axis of the Purbeck Monocline passes out to sea at Bat's Head, 2 km to the west of the Durdle Promontory. Thus, due to the Purbeck Monocline, there is a gradual change in dip of nearly 90° of sediments exposed in the Isle of Purbeck coastline. This has considerable implications for cliff behaviour.

The Portland Limestone cliffs of the Isle of Purbeck have generally been ignored by geomorphologists. Allison (1989) demonstrated that retreat of the Portland Limestone cliffs show no regular spatial or temporal pattern. Mapped changes in the location of the cliffline of the Isle of Purbeck were related to spatial trends of intact limestone strength, and cliff stability based on discontinuity survey. The strongest rock occurred in the cliffs at the western end of the Isle of Purbeck. However, the discontinuity pattern indicated that the most stable cliffs occurred in the east. The role of the Portland Limestone Formation as overburden on the Isle of Portland has been considered by Brunsden et al. (1996). The landslide pattern on the Isle of Portland was related to the island morphology, leading to a tentative model of landscape evolution in which the loading of the Portland Limestone causes clay extrusion below.

5.1.3 Field sites description

The Portland Limestone outcrop of the Isle of Purbeck is suitable for jointed rock mass study. The geology provides a fundamental control on the coastal landforms. Not only does the changing control of the Purbeck Monocline on the coastal cliff discontinuity pattern affect the behaviour of rock mass landforms, but the difference in the engineering properties of the units affects coastal evolution. Ten locations along the Isle of Purbeck coast were identified in the Portland Limestone cliffs for the collection of field and laboratory data (Figure 5.8). Much of the basic data for these sites were available from a previous study (Allison, 1986; 1989).

The first site at the western end of the Isle of Purbeck is the part of the Portland Limestone outcrop which occurs at the Durdle Promontory (Plate 5.3) (SY 806 802). The parabolic form of the arch at the western end of the Promontory reflects the stability generated by the overburden weight distribution through the supports and the shape of the arch is largely controlled by two sets of joints (West, 1998). On the eastern pincer of the Promontory, the Portland Stone dips at about 50° to the north, whereas on the western pincer that contains the arch, the beds dip at an angle close to vertical. From the landward side of the arch, the Portland Stone is hardly visible, and the surface observed is the lower Purbeck stromatolitic limestone (West, 1998). The Portland Limestone cliffs are 30 m high and bedding dips steeply to the north. The intact rock from the Durdle Promontory has the greatest strength of all Isle of Purbeck field sites and may be due to tectonic hardening that occurs in the Chalk (Allison, 1989). The location of the Durdle Promontory is very close to the angular foresyncline of the Purbeck Monocline (Figure 5.5). At the 'neck' of the promontory, the Purbeck Group rocks can be seen dipping at a greater angle to the north, with much deformation and some faulting. The more resistant promontory is almost entirely composed of Portland Limestone.

To the west of Lulworth Cove, the Portland Limestone coastal cliffs have been penetrated by the sea at Stair Hole (Plate 5.4) (SY 822 798). The breaching of the cliffs is at an early stage; there is a small entrance at the western end of the site, and caves occur within the Portland Limestone outcrop. However, the softer sediments behind the Portland Limestone ramparts have been removed more easily and there are landslips in the Wealden Group (Allison, 1986). In storms, the sea directly attacks the Purbeck Formation. The upper part of the cliffs culminate in the Purbeck Beds, with the cliffs reaching 55 m, and bedding in the Limestone dipping at 27° to the north. The contorted Purbeck Beds at Stair Hole are known as the Lulworth Crumple. At Lulworth Cove (SY 827 797), the opening of the Portland Limestone cliffs is much greater, and a large, rounded embayment has developed behind the two pincers at the entrance. The valley, with a stream entering the back of the embayment, gives an indication that the Portland Limestone Formation may have been ruptured by both marine and fluvial action (Jones et al., 1983). A good view of the dip of the bedding can be gained in the Portland Limestone each side of the entrance (Plate 5.1). The data collection point for the

Lulworth Cove field site was at the eastern rampart of the harbour entrance (Plate 5.5). Here the cliffs are 28 m high, and the bedding dips at 27° to the north.

Between Lulworth Cove and Worbarrow Bay, the Portland Limestone outcrop plunges directly into the sea and two field sites with similar characteristics occur along this stretch of coastline. Fossil Forest (Plate 5.6) (SY 834 796) has a stepped cliff profile, with the Portland Limestone Formation forming the lower part of the sea cliff and the rocks of the Purbeck Group occurring in the upper part. The Portland Stone here is a massive oolite forming the outer cliff that descends to the sea and contains thick-shelled marine molluscs (West, 1998). The cliff is 43 m high, and the bedding dips at 25° to the north. The site has joint characteristics that are similar to a site at Potters's Hole, half-way between Fossil Forest and Lulworth Cove. Fossil Forest is famous for the fossilised and silicified tree stumps which occur in the basal limestone layer of the Purbeck Group and have been dated at about 140 million years old. Woody material is not evident, but boles of trees exist, with hollows in the middle (House, 1993). Foliage and some timber can be found at several other levels within the Purbeck Formation (West, 1998). Further to the east along this stretch of coast, the resistant Portland Limestone ramparts at the entrance to Worbarrow Bay are evident at Bacon Hole (SY 839 796). At Bacon Hole, the Portland Limestone outcrop has been breached in several places (Plate 5.7). The sea cliffs here have similar characteristics as Fossil Forest, with cliff height being 37 m and the bedding in the Limestone dipping at 18° to the north.

The Portland Limestone outcrop is exposed again at the other side of Worbarrow Bay. Again it is suspected that the bay was breached fluvially and by marine action. The rampart at the eastern edge of the bay is called Worbarrow Tout (SY 868 795). On the western edge of the Tout, the cliffs are 58 m high, and topped by the rocks of the Purbeck Group (Plate 5.8). The bedding here is on the gently dipping part on the Purbeck Monocline and dips at 30° to the north. At the eastern side of the Tout there is Pondfield (SY 872 796). The field data were collected at the eastern edge of the bay at the point where the Portland Limestone is lifted above the Kimmeridge Clay and Portland Sand in Gad Cliff (Plate 5.8). The cliffs are therefore higher (99 m) and the bedding of the Limestone dips at 28° to the north.

The Portland Limestone outcrop again plunges directly into the sea to the east of St Alban's Head. The 7 km stretch of coast to Tillywhim on the flat part of the Purbeck Monocline yields three field sites with similar characteristics. Both Winspit and Seacombe sites are quarries situated at the entrance of two small dry valleys that reach the coast about 1 km apart. Winspit (Plate 5.9) (SY 977 760) has a cliff height of 43 m and horizontal planes of bedding. The discontinuity pattern here forms large, square blocks that are ideal for quarrying (Arkell, 1936). Seacombe (Plate 5.10) (SY 983 767) has a cliff height of 38 m and again has horizontal bedding. The cliffs here are thought to be stable (Allison, 1989). At the eastern-most point of the Portland Limestone outcrop on the Isle of Purbeck the cliffs have again been quarried, which gives ideal access for geological survey. The cliffs at Tillywhim (SZ 031 770) are 28 m high: again, large, square blocks are cut (Plate 5.11).

Data have been collected for a further three sites where the coastal cliffs are composed of Portland Limestone underlain by Portland Sands and Kimmeridge Clay. The cliffs provide an interesting comparison with surrounding sites on the Portland Limestone outcrop. St Alban's Head cliffs reach 108 m (SY 961 754) and the cliff face angle turns 90° as controlled by two joint sets. It is an excellent place to study the geological characteristics of the Portland Limestone, and failed rock blocks remain embedded below the face (Plate 5.12). Despite proximity to the Winspit site, the bedding here is dipping at 5° to the north. Emmet's Hill is 1.5 km to the north of St Alban's Head (SY 957 765). Again the cliffs have bedding which dips at 5° to the north. From Emmet's Hill, which overlooks Chapman's Pool, the Portland Limestone outcrop trends inland. Dungy Head (SY 816 799) is located between the Durdle Promontory and Stair Hole. Faulting has upthrust the Portland Limestone block, and Portland Sand and Kimmeridge Clay are also exposed at the coast (Melville and Freshney, 1982). Again this site is of interest because it is possible to view the failed Portland Limestone blocks that have become embedded into the Kimmeridge Clay. The bedding here dips at 43° to the north.

5.2 Introduction to the Colorado Plateau field area

The Colorado Plateau is one of the 15 United States geographical provinces covering 450,000 square kilometres in four states from Rifle, Colorado in the north-east to Flagstaff, Arizona in the south-west, and Cedar City, Utah in the west to nearly Albuquerque, New Mexico in the south-east (Lohman, 1981) (Figure 5.9). The Plateau ranges in height from 1500 to 3000 m with depths in the Grand Canyon as low as 600 m. This results in a range in precipitation with altitude and corresponding zonations in vegetation.

Geomorphologically, the Colorado Plateau has been principally influenced by the rapid incision of the Colorado River system about five million years ago. This has exposed the horizontally bedded sedimentary strata to the effects of differential erosion. Canyons, cuesta scarps, and plains stripped to bedrock dominate, with details including major discordances between stream courses and geologic structures; the presence of both ingrown and entrenched stream meanders; canyon sinuosity variations related to jointing and stratigraphic dips; evidence that groundwater sapping has influenced the growth of tributary canyons; rock arches, natural bridges, and exfoliation effects in massive sandstones; relict large-scale slope detachments; small areas of active sand dunes; and recently recognised aerodynamic forms (yardangs) resulting from aeolian erosion of consolidated rock (Oberlander, 1994b) (Plate 5.13). However, it is the work of running water which has dominated geomorphic research on the Colorado Plateau (Bishop, 1995; Graf *et al.*, 1987).

The Plateau is surrounded on its north-western and north-eastern sides by two branches of the southernmost part of the Rocky Mountain chain. The Precambrian igneous rocks of the Rockies have been uplifted to the extent that there are many summits of elevations greater than 4000 m. The boundary on the southern and western side is marked by volcanic plateaus. Virtually all of the geology exposed on the Colorado Plateau is sedimentary beds which have been uplifted (Eardley, 1962). There are igneous rocks at the depths of the Grand Canyon and also the Colorado National Monument. The fact that the sedimentary layers have remained largely horizontal, resisting the forces which crumpled the surrounding Rocky Mountains, can be attributed to the faulting, and makes the plateau remarkable geologically (DeCourten, 1993).

Added to this, the climate limits the development of soils and vegetation, thus exposing the rock, making the Plateau an ideal location for geological study.

All but the very highest parts of the Plateau have an arid or semi-arid climate with less than 250 mm a^{-1} of precipitation, although much of this is evaporated. Precipitation from the Pacific Ocean is intercepted by the Sierra Nevada Mountains, California for most of the year (Ely *et al.*, 1993). Some snowfall is carried by air from the north during the winter, and winds from the south in July and August give summer storms known locally as the 'monsoon'. Borehole temperature data indicates between 0.4 °C and 0.8 °C warming over the past 200 years (Harris and Chapman, 1995) but there have been climatic fluctuations throughout the Holocene, especially in precipitation (McFadden and McAuliffe, 1997). The Colorado Plateau has been classified as a desert by a UNESCO report which delimited aridity on the basis of the ratio of the mean annual precipitation P to the mean annual potential evapotranspiration E_{tp} (UNESCO, 1977). The central part of the Plateau was delimited as being arid ($0.03 < P / E_{tp} < 0.20$), with the edge being semi-arid ($0.20 < P / E_{tp} < 0.50$). However, the Colorado Plateau is commonly described as a cold desert because it has cool winters. The average annual temperature in Moab, Utah over the last 100 years is 13 °C, although summer maximum temperatures can reach 40 °C and winter minimum temperatures can be less than 0 °C (Harris and Chapman, 1995).

The Colorado Plateau contains numerous jointed rock mass scarps and other classic rock mass landforms. Mesas are rock masses which are wider than high; buttes are at least as high as wide; monuments and spires have a thin top (Lee Stokes, 1969). The distinctive environment is well known because of its appearance in many 'western' films. The geometrical distribution of joints in the rock masses leads to a variation in resistance of rock layers and zones, provoking explanations of rock mass development.

5.2.1 Geology and formation of the Plateau

The nature of the plateau landscape is associated with the Colorado River and its tributaries (Graf *et al.*, 1987; Harden, 1990; Kieffer, 1990). There has been much debate about the history of the Colorado River, particularly associated with the formation of the Grand Canyon. But the most widely accepted hypothesis is that stream capture to the west of the Kaibab Upwarp, on the western edge of the Colorado Plateau, during the late

Tertiary five million years ago, was followed by headward erosion of the drainage system (Huntoon, 1990; Lucchitta, 1990).

During the Laramide Orogeny towards the end of the Mesozoic, from 80 to 50 Ma, the collision of the North American Plate with the East Pacific Plate uplifted the Colorado Plateau and the Rocky Mountains (Spencer, 1996). The Laramide Orogeny has had the greatest influence on the Colorado Plateau. Uplift was broken by normal faults inherited from the Precambrian and was accompanied with erosion of rock layers. The Colorado Plateau behaved as a stable rigid block, while surrounding areas were intensively deformed during the orogeny (Bergerat et al., 1992; Thompson and Zoback, 1979). Young (1985) suggested that scarp retreat during this period was rapid (1500 to 3800 m / Ma) when base levels were stable or rising. Since the end of the orogeny, much slower rates (160 to 170 m / Ma) have occurred. As much as 600 m of Mesozoic and Palaeozoic sediments may have been stripped (Graf et al., 1987). Uplift appears to promote rapid, uniform scarp retreat, whereas when base levels are lowered individual scarp face tributaries erode headward and dissect scarps, slowing the rate of landscape evolution by an order of magnitude (Young, 1985). Further tectonic warping in the Basin and Range region in the Miocene, up to 5 Ma ago, was coincident with the incision of drainage systems across the Plateau. The establishment of a drainage divide with the lowering of base level led to the Mogollon Rim, to the south of the Grand Canyon, which marks the physiographic boundary of the Colorado Plateau (Mayer, 1979).

Much work relates the geometry of joint patterns over the Colorado Plateau to tectonic activity (Angelier, 1989). Because joints are formed when rocks are under shear or tension, different sets can be used to build a chronology. Nine joint sets from 7200 vertical discontinuity readings have been defined as characteristic (Bergerat et al., 1992), and can be compared with readings in this study. The succession of tectonic events reconstructed includes a pre-Laramide compressional event, three Laramide compressional events and three Neogene extensional events (Bergerat et al., 1991; Bouroz et al., 1989). Most analysis of discontinuity patterns on the Colorado Plateau has occurred at Arches National Park. Here joints can be related to collapsed salt structures as well as to tectonic faults (Cruikshank et al., 1991; Zhao and Johnson,

1991) to develop a model of fluid flow (Antonellini and Aydin, 1995). Joint widening then leads to the development of fins and arches (Blair *et al.*, 1975).

As the Plateau has developed, magma has intruded through faults, building into volcanoes and lava flows. The last eruption on the Plateau, Sunset Crater, occurred 900 years ago. Contemporary active seismicity is associated with boundaries of the Colorado Plateau which involve substantial crustal thinning. The Plateau is tectonically distinctive from surrounding regions: the crust of the interior of the Plateau is approximately 45 km thick as compared to 30 km elsewhere (Keller *et al.*, 1979). In the central part of the Plateau, nearly 1000 earthquakes with Richter magnitude between 0 and 3.3 were recorded in six years (Wong *et al.*, 1987). This is a low level of activity compared with the west US coast, but quite active when compared with the mid-continent. It is concluded that there is ongoing crustal deformation within the plateau, even though major deformation probably ceased at the end of the Laramide Orogeny 40 million years ago.

The extent of the Colorado Plateau makes description of the stratigraphy of the whole Plateau irrelevant, but descriptions of the stratigraphy of the field areas will be given in Sections 5.2.3 and 5.2.4. Rocks on the Plateau range from Precambrian (2 billion years old) through to the present. Generally, horizontal sedimentary layers broken by faults occur, with sediments being deposited in a variety of environments. Where deformation has occurred, rocks are jointed, but many massive sandstones are to be found. However, a graphical impression of the stratigraphy across the Plateau can be gained from fence diagrams (Figure 5.10) (Chronic, 1984).

5.2.2 Slope development on the Colorado Plateau

The first scientific expedition to explore the Plateau was led by John Wesley Powell in 1869. He documented a daring trip by boat down the Green and Colorado Rivers through the Grand Canyon, surviving hazards which at the time were unknown except to indigenous people (Powell, 1875). Following Powell, G.K. Gilbert explored and completed surveys on large parts of the higher plateau lands which inspired conclusions on structure and erosional processes (Baker, 1996; Gilbert, 1880; Weisheit, 1995). Studies of slope development on the Colorado Plateau were initiated by Gilbert (1880). The Henry Mountains were used to demonstrate that the sculpture of the land was due to

the law of structure and the law of divides. The Henry Mountains stand as eminences built of harder intrusions: they are steepest at their crests and concave outwards. It was noted that exceptionally the divides of badlands are convex at the crest. Davis (1892) suggested that convex divides are due to mass wasting. The central part of Davis' life work was that slopes declined as the landscape evolved (Davis, 1899), but he acknowledged that in arid regions, such as the Colorado Plateau, that the vertical cliff element of slopes may undergo parallel retreat broadly as suggested by W. Penck (Davis, 1930).

Three broad categories of steep slope occur on the Colorado Plateau. Cuesta-form composite scarps cut into several sedimentary layers with different characteristics. Commonly, composite scarp slopes are composed of a hard jointed cap-rock above a massive, vertical cliff-forming unit and a gently angled, soft basal layer. Classic Colorado Plateau rock landforms such as spires and buttes have often developed from composite scarps cut into mesas. Scarps of compound rock types are also cut into canyons, with some slope research being conducted within the Grand Canyon (Schmidt, 1987). A second category of steep slopes includes scarps which cut into massive sedimentary layers to develop distinctive forms. Also common are steep slopes in softer sediments. The badland slopes, with a high drainage density, are special slopes formed from softer sediments (Schumm, 1956). Figure 5.11 gives a summary of slope development studies on the Colorado Plateau and the type of slopes apparent. It can be seen that there are different types of slope on the Colorado Plateau which are studied at different scales, by various geomorphological approaches. There is some degree of overlap between the sections shown on Figure 5.11; clearly studies can encompass different approaches.

The manner of cliff retreat on the Colorado Plateau has been described as discontinuous, consisting of sudden rock falls separated by periods of stability as the talus debris is removed (Koons, 1955). The study also showed that slope angles varied little, depending upon the angle of repose and the angle of sliding friction of the materials involved. Ahnert (1960) suggested that most activity on scarp faces has occurred under a different climate from the present, so that the water table was higher and lower scarp slope shales were saturated. Scarps in different structural settings were examined. It was observed that scarps with the massive sandstone reaching the base of

the cliff form a rounded profile, as opposed to the vertical profile of compound scarps. Schumm and Chorley (1966) attempted an understanding of the rates and mechanisms of scarp retreat. Explanation is given in terms of the rocks comprising the scarp face and the structure of the scarp. Four variables are the relative resistance of the caprock, the joint spacing of the caprock, the direction of dip, and the proportion of weaker rock exposed in the scarp.

A further classic study of slope development on the Colorado Plateau does not analyse the abundant compound scarps with strata of varying resistance, but looks at scarp evolution in one massive rock unit. Oberlander (1977) demonstrated that partings in the massive Entrada Sandstone which separate slope sections leads to allometric slope retreat. The two components of the slope were termed the rounded slick rock slope and the vertical slab wall, with the opening and closure of an effective parting between the two varying the rate of retreat. The lower unit mimics the behaviour of a thin-bedded substrate, producing footslopes, while the cliffed upper unit plays the role of caprock, as in the Koons (1955) model of slope retreat. One question which is not addressed is how the buttes have become disconnected from the scarp faces that have developed. Examples occur at the Courthouse Towers, Arches National Park, with features such as Organ Rock becoming detached from the main cliff line. However, it is acknowledged that the parallel retreat of compound scarps is the dominant mode of slope development on the Colorado Plateau. Cuesta-form landscapes in arid regions are the epitome of weathering-limited systems, in which the rock surfaces are kept free of waste products by the disparity between weathering rates and erosional efficiency. Cuesta-form landscapes seem to exemplify the equilibrium concept of slope development, in which each rock type is associated with a particular slope angle that equates erosional stress to surface resistance, resulting in efficient removal of weathering products. This appears to produce parallel rectilinear slope retreat (Oberlander, 1977).

It has been suggested that contemporary slope development on the Colorado Plateau is by the parallel recession of scarps through the horizontal strata of varying resistance (Young, 1985). A characteristic feature developed in horizontal strata in an arid climate is the retreating escarpment. A series of beds 30 to 300 m thick may be stripped away for kilometres, yet the remaining portion may suffer little or no loss of height. This uneven erosion seems to be dominant on arid plateaus (National Park

Service, 1985). The multi-level nature of scarp retreat on the different layers of rock on the Colorado Plateau makes the process almost inexhaustible (Schmidt, 1989). After the Laramide Orogeny, scarp retreat was initiated radially from the centre of the plateau uplifts on the youngest rocks. Thus, for instance, at Monument Valley at the centre of the Monument Upwarp, older Permian sandstones are retreating, whilst further away, Tertiary and Cretaceous scarp faces are exposed (Baars, 1962; Baker and Reeside, 1929). Resistant scarps retreating across the Plateau are caught at the base by faster retreating softer scarps which combine with and are controlled by the caprock of the resistant scarp. Above the caprock, softer layers retreat more rapidly forming a plateau surface. Thus on the Colorado Plateau, scarp retreat can operate independently at different elevations in a kind of staircase with steps of different heights and widths and different velocities of recession (Schmidt, 1989).

Schmidt and Meitz (1996) identified four different types of cuesta-form scarp slope in different altitude bands to evaluate climate change in a space-time substitution study. During the last glacial, increasing moisture availability at lower elevations due to lower evapotranspiration rates led to an increase in vegetation on slopes. In the soft Mancos shale slopes to the north of Grand Junction, Colorado, the establishment of woodland led to stabilisation and decreased dissection. With the compound cuesta scarp slopes of Canyonlands National Park, the Wingate sandstone forms a vertical cliff unit. At higher elevations, the angle is relaxed. But the last glacial phase was not long enough to shift the cliff form at lower altitudes due to the rate of cliff retreat (Nishiizumi *et al.*, 1993).

At a smaller scale, Schmidt (1991) demonstrated that the scarp backslope is eroded by the backwearing of scarp recession. The length of the backslope is controlled by the resistance and thickness of the scarp caprock unit, as well as the structural dip and the strength of the overlying rocks. Further morphometric analysis (Schmidt, 1994a) suggested that Colorado Plateau scarps are less embayed in plan with decreasing resistance and increasing thickness of the caprock and increasing structural dip. Nicholas and Dixon (1986) suggested that the rock fabric of the caprock in terms of joint orientation and spacing is the dominant control of scarp form and that the rock strength plays a minimal role. Cliff retreat is greater in embayments, and headlands remain as resistant projections where joint spacing is greatest. Headlands may become

detached from cliff faces to form buttes and pinnacles which are common on the Colorado Plateau. The importance of scarp geometry and aspect, as well as the lack of hydraulic throughflow, has been illustrated in the failure of Wingate Sandstone cliffs at Canyonlands National Park (Butler and Nicholas, 1989). Howard (1995) studied escarpment planforms at the intermediate scale, suggesting that form is controlled by areal variations in the rates of processes acting upon scarps. Simulated scarps incorporating the processes of cliff erosion, fluvial incision, and groundwater sapping are compared with natural scarps.

One common process which affects the development of cliffs on the Colorado Plateau is groundwater sapping. Sapping is the process leading to the undermining and collapse of valley head and side walls by weakening or removal of basal support as a result of enhanced weathering and erosion by concentrated fluid flow at a site of seepage (Laity and Malin, 1985). Ahnert (1960) introduced the idea that slopes on the Colorado Plateau could have been influenced by sapping processes. It is suggested that sapping between the vertical cliff faces and less resistant shale base during a wetter climate has led to the distinctive morphometry of compound scarps on the Colorado Plateau. But Schmidt (1996) believed that a study of scarps in past climates is possible because there were similar amounts of precipitation as today. Certainly the scarps are retreating at the present time (Nishiizumi et al., 1993), and the role of sapping may not be as great as Ahnert (1960) suggested (Butler and Nicholas, 1989). However, there are certain situations where sapping is an important geomorphic process. The study by Laity and Malin (1985) highlighted the contact between Navajo Sandstone, which has a high porosity, and the Kayenta Formation in the Glen Canyon region, causing seeping leading to canyon growth. The Kayenta Formation prevents the aquifer above from reaching lower layers such as Wingate Sandstone (Zhu et al., 1998).

Slopes which are made up of more readily weathered and eroded rock units, generally shales or poorly cemented sandstones or alluvium, are commonly eroded into badlands where the layers are thick (Howard and Selby, 1994). Badlands occur on steep slopes, often where there is a break in the slope angle, and are typified by high drainage densities, a shallow regolith, and rapid erosion rates (Howard, 1994; 1997). The intriguing miniature landscapes created are a common sight on the Colorado Plateau with numerous exposures of shale and high runoff of rainfall during storms.

Geomorphic research has largely concentrated upon applying the concepts of badlands to more areally significant geomorphic systems (Bryan and Yair, 1982). On the Colorado Plateau most badlands occur in the south-eastern part and were most extensively developed during the late Holocene, a period of increased aridity (Wells and Gutierrez, 1982).

Two field areas have been identified on the Colorado Plateau for the study of the behaviour of jointed rock masses. Both are situated on mesas and are composed, in profile form, of a Chinle Formation basal unit, a vertical cliff-forming Wingate Sandstone and a jointed cap rock of the Kayenta Formation. In order to study the three-dimensional development of the Colorado Plateau rock cliffs, consideration needs to be made of the planform of the cliffs as well as the profile. In plan form, the scarps can be divided into headland and embayment situations, similar to coastal cliffs. Work was undertaken in the Canyonlands Region and the Colorado National Monument analysing the joint geometry of the ledge-forming Kayenta Formation in order to model the development of the cliffs. Lohman (1965) stated that the vertical cliffs and shafts of the Wingate Sandstone endure only where the top of the formation is capped by beds of the next younger rock unit, the Kayenta Formation. The Kayenta is much more resistant than the Wingate, so even a metre or so of the Kayenta protects the rock beneath. Both mesas are formed from horizontal layers of sandstone and made up of cliffs up to 400 m in height, which vary in their development, so that there are scarp plan convexities (headlands) separated by scarp plan concavities (embayments) with numerous isolated buttes.

Work completed on the Plateau in this study and discussed in the sections on the Colorado National Monument and Canyonlands National Park (Section 5.2.3 and 5.2.4) is aimed to create an understanding of the rates and mechanisms of change of cuesta-form composite scarps in plan and profile at the small scale. By computer modelling the development of cliffs the work can include information on the morphometry of the landforms, the material properties making up the landforms, and analyse how the processes of failure from the scarp face influence development.

5.2.3 Canyonlands National Park and Dead Horse Point State Park

The Canyonlands of the Colorado Plateau are a physiographic region at the centre of the Plateau based on the Colorado and Green Rivers (Figure 5.9). The region has low precipitation, little vegetation, salmon-coloured rocks stripped bare of cover, canyons and an abundance of classic rock mass landforms (Chronic, 1990). The sedimentary layers are flat-lying as the region lies just North of the Monument Upwarp, where the oldest deposits lie at the centre of the Plateau. The Canyonlands region is an excellent area to study the development of jointed rock mass compound scarps, as the rocks control slope behaviour. This study has focused upon cliffs in the Island in the Sky mesa that occurs in Canyonlands National Park and Dead Horse Point State Park (Figure 5.12), because the regularly jointed cap-rock has a strong influence upon slope development.

The Island in the Sky mesa rises gradually to the south of the Moab Fault near Arches National Park and reaches an altitude of 1800 m. The mesa is surrounded on three sides by the Colorado and Green Rivers with cuestaform compound scarps overlooking the river canyons (Figure 5.12). The confluence of the rivers is to the south (Figure 5.13). Both the Island in the Sky District of Canyonlands National Park and Dead Horse Point State Park are on top of the mesa and the dominant features of the parks are the almost vertical cliffs of the mesas and buttes, cut into horizontally bedded sandstone and up to 400 m in height. The top of the mesa is located in the desert shrub belt and the lower part of the pinyon-juniper woodland. However, large areas of the region are devoid of soil cover and vegetation (Plate 5.14). The average precipitation recorded on the mesa is 233 mm a^{-1}, although up to 37 mm have been recorded in 3 hour storms (Butler and Nicholas, 1989). In July, the maximum temperature exceeds 30 °C on an average of thirty days and in January, the minimum temperature is less than 0 °C on an average of thirty days, and the annual range may be as much as 55 °C. There are still few people familiar with these lands of rugged beauty as wilderness is partly maintained by the fact that water and services are only available at Moab, 90 km away.

This discussion will cover the Island in the Sky section of Canyonlands National Park and Dead Horse Point State Park together, as these sections are geomorphologically similar. Dead Horse Point State Park was established by the Utah Department of Natural Resources in 1956 covering 52 km^2 and has so far resisted calls

for merger with Canyonlands National Park. A study of the geological structure of the parks leads to an elucidation of the geomorphological features within the park (Figure 5.14) (Plate 5.15). From the bottoms of the canyons to the tops of the mesas, bands of horizontally bedded rock with varying resistance form a ledge-slope-ledge topography. In the Island in the Sky district there are two main ledges. One is formed with White Rim Sandstone and occurs above the canyons containing the Colorado and Green Rivers and the other occurs at the top of a second band of cliffs which confines the Island in the Sky mesa. Starting at the base of the column at the confluence of the Green and Colorado Rivers is the Permian Rico Formation followed by the Cedar Mesa Sandstone of the Cutler Formation. The extensive ledge-forming unit at the top of these cliffs is the Permian White Rim Sandstone, an erg and coastal dune-deposited sandstone with a thin, capping marine veneer (Kamola and Huntoon, 1994). The cliffs capped by White Rim Sandstone are largely controlled by the tributaries and erosion of the rivers in the canyon below.

The soft red siltstone of the Chinle Formation and Triassic Moenkopi Formation forms the gently angled base of the large upper cliffs (Figure 5.14). The Chinle Formation is composed of beds of heterogeneous thickness and lithology, mainly siltstones and fine sandstones (Schmidt and Meitz, 1996). These are deeply coloured in brick red due to a relatively high concentration of iron oxides, were formed as floodplain deposits, and hence are soft, intensely fractured rock masses (Dubiel, 1992). Steep-sided rills and gullies are common and steps are caused by resistant beds in the Chinle Formation. Very sparse blackbrush (*Coleogryne ramosissima*) communities vegetate the slopes at this altitude. The Wingate sandstone is the main cliff-forming unit in the Canyonlands region. This soft fine-grained sandstone was formed during the Late Triassic as an eolian deposit and is only moderately cemented with calcium carbonate (Schmidt and Meitz, 1996). The exposure of cliffs composed of Wingate Sandstone has been dated using cosmogenic nuclides (Nishiizumi *et al.*, 1993). It was estimated that rock failure occurred every 10,000 years. The Wingate Sandstone is much less resistant than the overlying Kayenta. Without its protecting cover, the cliffs become more susceptible to salt weathering and freeze-thaw, and there is a rapid recession of the cliff (Paradise, 1997; Schmidt, 1994b). Failure occurs because internal cohesive forces of the sandstone do not equal shear forces produced by the rock's own weight (Butler and

Nicholas, 1989). The cap rock of the cliffs and out-lying buttes is the resistant Kayenta formation cemented with silica. This formation is a fluvial deposit, with lenticular sandstone packages of reddish-brown arenite interbedded with minor reddish-brown siltstone / mudstone and carbonate conglomerate (Bromley, 1991; Luttrell, 1987), which has a high porosity of 22% (Burns *et al.*, 1990; Piwinskii, 1977).

Much of the geomorphology of the Island in the Sky Mesa is controlled by variations in the Kayenta Formation caprock. The unit is well-jointed with horizontal bedding layers approximately 2 m thick and a complex pattern of nearly vertical jointing, with an average spacing of about 5 m. The joints within the Kayenta Formation create the planes of weakness within the rock that controls the development of the whole 400 m high cliffs. Schmidt (1989) demonstrated that caprock thickness is not related to the rate of scarp retreat, but when this property is combined with rock resistance (which includes jointing in its determination) there is a strong correlation with retreat. Schmidt and Meitz (1996) identified this type of Chinle-Wingate-Kayenta scarp as a prominent morphological feature of the Colorado Plateau. Where the precipitation is less than 350 mm a^{-1} the Wingate Sandstone unit is vertical and retreats parallel fashion by toppling. The rate of retreat is estimated to be 1 m / 10^3 yr.

Eroded back from the top of the cliffs on the Island in the Sky mesa are small outcrops of the massive dune-deposited Jurassic Navajo Sandstone. A further point of geological interest on the Island in the Sky is a large crater 2 km in diameter known as Upheaval Dome. Opinion is divided on whether this was formed by a meteorite impact or the collapse of a salt dome. The crater resembles collapsed salt domes in the Gulf (Lohman, 1974) and there are large salt deposits in the region. But three concentric rings around the crater give weight to the meteorite impact idea. A reason that the Upheaval Dome has received so much attention from geomorphologists is that interpretation can be used by analogy in planetary geological studies. The Canyonlands National Park has been used for Martian studies because of its limited rainfall, diversity of landforms, and rock exposure (Graf *et al.*, 1987). Laity and Malin (1985) suggest that there are striking similarities in land form on Mars and that the gross geomorphic process may be similar.

When the plan form of the cliffs of the Island in the Sky Mesa is considered (Figure 5.13), as well as the profile (Figure 5.14), it can be seen that the trace of the crest of the cliffs carves headland (Plate 5.16) and embayment (Plate 5.17) features. It is

often said that to stand on the edge of a mesa in the Canyonlands Region is akin to standing on a coastal cliff. Often detached from headlands are buttes (Plate 5.18), which demonstrate various stages of development (King, 1957). Butler and Nicholas (1989) studied 512 landslide deposits in Canyonlands National Park, of which 89% originated from the Kayenta / Wingate cliffs. The morphometric characteristics of the deposits were best explained by the landslide position beneath a headland or embayment. Landslide deposits beneath embayments are significantly longer and larger in area, suggesting that the degree of jointing in the sandstone is important. The aim of this study is to demonstrate how jointed rock mass failure mechanisms and rates of change affect the development of composite scarps to control such landforms.

5.2.4 The Colorado National Monument

The Colorado National Monument is in the north-eastern part of the Colorado Plateau (Figure 5.9) on the northern tip of the Uncompahgre Plateau, a north-westward dipping anticline 200 km long by 50 km wide. The park covers 83 km^2 and was established as a National Monument in 1911 after the efforts of John Otto in trail breaking into the cliffs to the south of Grand Junction and the Colorado River (Plate 5.19). The Redlands Fault, which follows the north-eastern side of the park, has a throw of 250 m and causes the Colorado National Monument cliffs to be upthrown relative to the Bookcliffs on the opposite side of the Colorado River valley (Jamison and Stearns, 1982). Thus the dominant features of the park are the almost vertical cliffs of the mesas and buttes which are cut into horizontally bedded sandstone. The cliffs of the Colorado National Monument are up to 150 m high with trails and a rim rock road allowing good access to the top of the cliffs. Formations exposed here extend into adjacent states and appear in other parks of the Colorado Plateau (Chronic, 1984). Although similar in appearance to the cliffs in the Canyonlands Region, the deposits are thinner, leading to lower heights. Also, there is a slightly greater average annual precipitation of about 250 mm a^{-1} at the Colorado National Monument, leading to greater pinyon and juniper (*Pinus edulis, Juniperus osteosperma*) vegetation cover in the park.

The cliff profile is shown in Figure 5.15. At the bottom of the canyons in the park is the dark rock of the Precambrian complex made up of schist and gneiss, and granitic dykes (Lohman, 1965). It is topped by an erosion surface, which constitutes an

uncomformity in the column. Above the erosion surface, the geological cliff-forming succession resembles that in the Canyonlands Region. The soft red siltstone of the Chinle Formation forms the gently angled base of the large cliffs and the Wingate sandstone is the main cliff-forming unit in the Colorado National Monument (Chronic, 1980). Evidence of the weathering of the Wingate Sandstone, without the protective cover of the Kayenta Formation, is evident at Colorado National Monument (Plate 5.20). The cap rock of the cliffs and out-lying buttes is the resistant Kayenta Formation cemented with silica, and it forms a continuous boundary with the Wingate.

The Kayenta Formation forms a bench layer at the top of the cliffs of the Colorado National Monument. Above the Kayenta is the Jurassic Entrada Formation, which is a massive sandstone containing no joints. In other parts of the Colorado Plateau, the Navajo Formation would normally occur between the Kayenta and the Entrada, but it has been eroded here along with the lowest member of the Entrada, the Dewey Bridge Member. The Slick Rock Member of the Entrada is formed from coastal sand dunes and contains calcium carbonate, which leads to solutional rock architecture. The cliffs of the Entrada Formation occur some distance behind the main Colorado National Monument cliffs due to greater erosion. Above the Entrada in the park column occur softer sediments. The Late Jurassic Morrison Formation is a fluvial deposit of siltstone and mudstone which also contains numerous dinosaur bones. It is from the Morrison Formation that the world's largest dinosaur bones were once found at Riggs Hill, 2 km outside the National Monument. Above this is the Cretaceous Burro Canyon Formation, a shale which is to be found in the highest part of the park at altitudes of about 2200 m.

Much of the shape of the Colorado National Monument has come about since the Pliocene when the River Colorado captured the River Gunnison to flow down the Grand Valley, followed by uplift of the Uncompahgre Plateau. Canyon cutting has occurred during the Quaternary within 500 m of the river. Initially the canyons had a V shaped profile with ephemeral streams in the bottom carrying the disintegrated, fallen rock away as sand. When the harder Precambrian rock was reached at the bottom of the canyon the rate of downcutting slowed and the canyons developed with a U shaped profile.

Lohman (1965) suggested that the character of the canyon walls is governed by several factors: the climate; the character and hardness of the rocks; the presence or absence of joints; the relative positions of layers of hard and soft rocks; freezing and thawing; and the amount of sunshine the canyon walls receive. The climatic factors affect the angle of the cliff face and the other factors affect the cliff development. Cliffs facing the sun are vertical whereas north-facing cliffs are gentle enough to be climbed and allow talus development. The amount of sunshine on the cliff affects the amount of vegetation on the slope and thus the amount of weathering. At the Colorado National Monument it is clear that there are differences in the valley profile and the cross-valley vegetation cover due to the aspect of the cliff faces (Plate 5.21). Schmidt and Meitz (1996) suggested that a precipitation of less than 350 mm a^{-1} is necessary to maintain a vertical Wingate Sandstone cliff. At greater precipitations, or where the north facing slope is in shade, the lower Chinle slope decreases in angle to 36° and pinyon-juniper vegetation becomes denser. On the upper part of the cliff, segmentation of the Wingate Sandstone occurs and vegetation claims ledges.

As in the Canyonlands Region, much of the geomorphology of the Colorado National Monument is controlled by variations in the Kayenta Formation caprock. The unit is well-jointed with horizontal bedding layers approximately 1 m thick and a complex pattern of nearly-vertical jointing with an average spacing of about 2 m. Lohman (1981) suggested that there are no apparent regular joint systems or patterns. The cliff plan form at the Colorado National Monument (Figure 5.16) can again be described using features such as headlands (Plate 5.22) and embayments (Plate 5.21). It is thought that differences between the joint sets at different locations control the rate of cliff retreat at that location. For instance the joint geometry at the back of embayments of the cliffs in plan form means that the Kayenta Formation is weaker in these locations and so the cliffs have retreated further. The buttes which outlie the cliffs, such as Independence Monument, have a stronger rock mass cap-rock (Plate 5.23).

5.3 Methodology and data acquisition

On both the Colorado Plateau and the Isle of Purbeck, Dorset, it is possible to identify jointed rock mass landforms at different stages of development. By identifying links between landforms, and the reasons for the differences in slope behaviour, strong conclusions can be made about the formation of landforms in the past. Space-time substitution has already been used to understand landscape development along the Isle of Purbeck coastline (Allison, 1989; Goudie and Brunsden, 1997), and to link jointed rock masses with indicators for climate change (Schmidt, 1994b). On the Colorado Plateau, talus and pediment flatirons were formed during a wetter climate and have become detached from the main escarpment face during the subsequent dry phase (Schmidt, 1996). From morphometric measurement, rates of cliff retreat are estimated at 2 m / 10^3 yr. Also on the Colorado Plateau, jointed rock escarpments were identified within the same geological sequence at different altitudes (Schmidt and Meitz, 1996). The angle of slope was observed to decrease with altitude and it was questioned whether the duration of a colder spell was sufficient to change slope form.

At each of the field locations, the Isle of Purbeck and the Colorado Plateau, morphometric data, discontinuity geometry data and rock strength data were collected. For the Isle of Purbeck sites, morphometric data included the height of the cliff above sea level and the bearing of the coast at the cliff, measured from Ordnance Survey maps (Evans, 1986). The angle of the free face was measured by interpolation using a clinometer in the field. The depth of the cliff below sea level was measured by dropping a weight from the edge of the cliff and converted trignometrically if not vertical. The sea floor profile offshore of each of the field sites was gained by taking a series of echo depth readings from a boat, and calibrating with the status of the tide at the time from Admiralty tide tables. The distance of the boat from the shore was measured with a Global Positioning System, and the overall profiles were compared with reference points on Admiralty Charts. On the Colorado Plateau, sites were selected at the top of the cliffs at Dead Horse Point State Park, Canyonlands National Park and Colorado National Monument. The sample points reflect the varying extent of cliff development between the headlands and embayments along the cliff plan. At each place the height of the cliff, the mean slope angle of the free face and the angle of the basal unit, the

orientation of the free face, and other relevant morphometric characteristics were recorded from United States Geological Survey maps.

Discontinuity data were collected by laying tapes parallel and perpendicular to the strike of the bedding at each field site with recordings being made as each discontinuity intersected the base line (Plate 5.24; 5.25). The dip and strike of each discontinuity and plane of bedding were logged. Care was taken to collect a sufficient sample; generally 100 recordings were made (Kulatilake and Wu, 1984; Terzaghi, 1965). The number of readings taken for each joint set reflected the proportion it represented of the total number of joints. The guidelines proposed by Oda (1988) were followed to reduce error. Spacing between individual joints was recorded along transects oriented perpendicular to the strike of joint sets (Mohajerani, 1989; Qin Huang and Angelier, 1989). This proved complex, but an indication was gained of the spacing between obvious sets of discontinuities. For the Isle of Purbeck field sites, it was important that the joint surveys were undertaken at each site within the upper Winspit Member, a shell-sand limestone with oolitic layers (Cox, 1929). Not only does the upper part of the outcrop exert the greatest control over the failure mechanism of the rock mass, but the Winspit Member is the quarried stone that forms the most regular blocks and between-site consistency could be maintained. It is impossible to collect discontinuity data for the entirety of the rock mass (Starfield and Cundall, 1988) and statistical assumptions have to be made about the representation of the joint geometry within the rock mass. However, it appears that for both areas that discontinuity sets are consistent throughout the rock masses. For instance, where the Portland Limestone had been breached, the three-dimensional view of the rock mass demonstrates that discontinuity sets are consistent. On the Colorado Plateau, it was noted in the field that the joints were persistent, and where rock masses are exposed due to lack of soil cover, consistency is clear (Plate 5.14).

Rock strength data were collected by two means. Rock hardness values were measured in the field using the Schmidt hammer and rebound values were correlated with standard rock strength characteristics (Day and Goudie, 1977; Deere, 1966; McCarroll, 1987). At each site on the Colorado Plateau, twenty Schmidt hammer impact readings were recorded on five different rocks. By moving the hammer across the surface of the rock, problems of rock anisotropy were overcome. By measuring different

rock blocks at a site, differences within a geological formation are considered. However, Allison (1988) argued that a very poor correlation resulted for the Portland Limestone rocks. It was fortunately possible to make use of previous laboratory test results for both the Portland Limestone (Allison, 1986; 1989) and Colorado Plateau sandstones (Fisher, pers. comm.). In Dorset, *in situ* rock blocks were also removed from the cliff along intersecting discontinuities, their orientation noted and returned to the laboratory for material property analysis (Allison, 1989). A triaxial Hoek cell was used to determine the compressive stress, with tests being conducted at confining pressures (σ_3) of 15 MN m^{-2}, 30 MN m^{-2} and 60 MN m^{-2}. Standard test procedure was adopted (Brown, 1981), with 38 mm diameter cores being cut and weighed following removal of weathering rind. Further test cores were used for ultrasonic tests to determine Poisson's Ratio and Dynamic Young's Modulus (Allison, 1988). For the purpose of this study, the use of previous laboratory test results is a satisfactory resolution, as the approach is to construct simple, representative models of the field sites, not perfect representations of real-world conditions. Moreover, there is much discussion in the engineering literature on the accepted techniques for collecting rock strength data (Litwiniszyn, 1989). Large anisotropies may occur in rocks (Amadei, 1996) and on the Colorado Plateau the effects of zones of microscopic deformation bands on rock properties have been measured (Antonellini and Aydin, 1995). The high porosity due to micro-cracks and pore structure in the Kayenta Formation causes localised hardening (Burns *et al.*, 1991). In the cliff forming Wingate Sandstone, the primary sedimentary features (cross beds and cross-bed-set boundaries) impart anisotropy in strength which decreases in deformation zones (Jamison and Stearns, 1982). However, the analysis and modelling of rock slopes will always be a data-limited problem, but, strong conclusions can be made using simple approaches (Starfield and Cundall, 1988).

5.4 Conclusion

The introduction to the field study areas has highlighted that jointed rock cliffs occur in contrasting environments. The rock cliffs in the Isle of Purbeck are formed in a coastal environment where removal of waste material by the sea has maintained exposure. The Colorado Plateau rock cliffs are formed in a low precipitation, arid environment and form large, embayed escarpments and detached monoliths. However, there are common factors which make both locations ideal for this study. Both locations have been identified as being the best locations where the geological structure is directly related to landscape development. Jointed rock cliffs are exposed in both environments and are controlled in development by variation in discontinuity geometry. In the Portland Limestone of the Isle of Purbeck, joint geometry variation as caused by the relative position of the Purbeck Monocline along the outcrop controls development. On the Colorado Plateau, variation of joint set spacing in the cap-rock of cuestaform composite scarps controls development. Also, in both areas, there are classic, spectacular rock landforms of outstanding interest to the geomorphologist.

Chapter 6: The mechanisms of failure and behaviour of the Portland Limestone coastal cliffs of the Isle of Purbeck, Dorset

Chapter 6: The mechanisms of failure and behaviour of the Portland Limestone coastal cliffs of the Isle of Purbeck, Dorset

6.1 Data analysis of Portland Limestone rock slope properties

Work was undertaken along the coastline of the Isle of Purbeck, Dorset, analysing the joint geometry and properties of the Portland Limestone outcrop and the cliff morphometry in order to model the mechanisms of failure and behaviour of coastal cliffs. Ten sites have been selected for examination and are shown in Figure 5.9. The sample points reflect the varying extent of cliff development at each site due to the structural setting, are geographically spread and include different cliff forms.

6.1.1 Morphometric and joint data

If the discontinuity pattern at the ten field sites along the Isle of Purbeck is considered, an indication of cliff stability and some explanation of cliff changes begins to emerge. By the contouring of poles on equal area stereographic projections, mean joint set characteristics at all ten sites can be identified (Hoek and Bray, 1981; Priest, 1985) (Table 6.1). Stereoplots from the key sites of Durdle Door, Lulworth Cove, Fossil Forest and Winspit are presented (Figures 6.1 to 6.4). It can be seen that there is generally little variability of joint set orientation at each site as the pole clusters are concentrated.

A link throughout the Portland Limestone outcrop can be identified by comparing the representative data for the joint sets at each site (Table 6.1). Values can conveniently be sub-divided into six joint sets which occur in the Isle of Purbeck. At the eastern end of the Isle, the bedding is close to horizontal and joint sets 'A' and 'B' can be identified. Set 'A' strikes approximately north-east to south-west and is close to vertical. Set 'B' strikes approximately east to west and again is close to vertical. The interaction of sets 'A' and 'B' with horizontal bedding at the sites of Tillywhim, Seacombe and Winspit produces rectangular blocks. Joint data from Pondfield, Worbarrow Tout, Bacon Hole, Fossil Forest, Lulworth Cove and Stair Hole are characteristic of wedge failures (Hoek and Bray, 1981). The bedding for these sites strikes at a bearing between east and north-east, and dips at angles of 20° - 30°. Wedge failures are controlled by joint sets 'C' and 'D', which intersect to form a notch along which blocks slide. Joint set 'C' strikes at approximately south-east to north-west, and

dip varies between 35° and 65° for the six sites. Joint set 'D' strikes at approximately south-west to north-east, and dip varies between 70° and 90°. At Durdle Door, the bedding dips at 47° to the north, joint set 'E' strikes south / south-west to north / north-east dipping at an angle of 36° and joint set F strikes north-east to south-west dipping at 44° (Table 6.1). The interaction of the three joint sets with the morphometry of the Durdle Peninsula landform indicates a complex failure mechanism. The combination of joint intersections would suggest indicate that both toppling and sliding failures are possible.

	Bedding	Set A	Set B	Set C	Set D	Set E	Set F
Tillywhim	0	214/88					
Seacombe	0	204/85	273/90				
Winspit	0	28/82	277/90				
St. Aldhelms Head	0	46/84	293/85				
Emmett's Hill	351/4	36/86	275/83				
Pondfield	338/28			151/36	243/88		
Worbarrow Tout	333/30			160/41	248/84		
Bacon Hole	1/18			118/63	233/66		
Fossil Forest	352/21			177/56	225/70		
Potter's Hole	336/24			152/68	232/70		
Lulworth Cove	358/27			164/50	226/71		
Stair Hole	24/27			123/55		204/30	
Dungy Head	344/37			83/90		169/63	
Durdle Door	0/47					190/36	242/44

Table 6.1:   Representative values for the bedding and joint sets of the Portland Limestone outcrop for sites on the Isle of Purbeck.

Data are listed as pairs. The first value is the dip direction (degrees: 0° to 360°) and the second value is the dip (degrees: 0° to 90°). The ten field sites relevant to this study are labelled in normal text. Further sites for which data is available are included for comparison and are labelled in *italics*.

It has been suggested that the discontinuity pattern is regular throughout the Portland Limestone outcrop in the Isle of Purbeck, and that joint orientation remains constant as the bedding changes. The regularity of the joint structure with respect to the dip of the bedding can be considered by calculating the angle of intersection between joint sets 'A', 'B', 'C', 'D', 'E' and 'F' and the bedding planes. Such analysis has been used to gain an insight into discontinuity genesis (Angelier *et al.*, 1989; Bergerat *et al.*, 1991). A computer program was written in Basic which calculates the angle of intersection between two planes in three dimensions (Appendix 6.1). The results are presented in Table 6.2.

	A/Bed	B/Bed	C/Bed	D/Bed	E/Bed	F/Bed	A/B	C/D
Tillywhim	88°						49°	
Seacombe	85°	90°					69°	
Winspit	82°	90°					69°	
St. Aldhelms Head	84°	85°					68°	
Emmett's Hill	83°	82°					60°	
Pondfield			64°	89°				90°
Worbarrow Tout			71°	82°				84°
Bacon Hole			72°	78°				81°
Fossil Forest			77°	83°				45°
Potter's Hole			88°	77°				74°
Lulworth Cove			76°	68°				66°
Stair Hole			63°		57°			
Dungy Head			85°		80°			
Durdle Door					83°	75°		

Table 6.2: Angles between the joint sets of the Portland Limestone outcrop.

The data confirm that the joint sets have been correctly labelled and grouped. For instance, the angle of intersection between joint set 'A' and the bedding for the five listed sites varies between 82° and 88°. However, it is not possible to make statistical conclusions upon the regularity of the joint sets throughout the outcrop from the data. If there was a regular distribution, then the intersection angles listed between joint set 'C'

and the bedding would resemble the values for the intersections between joint set 'A' or 'B' and the bedding. In other words, the relative joint geometry and intersection angles would be the same for all locations along the Purbeck coastline, but some sites may have a rotated geometry due to the Purbeck Monocline. It has already been demonstrated in this study that the failure mechanism depends largely upon the geometrical pattern of the discontinuities which is largely controlled by joint spacing between the joint sets (Chapter 4). Tapes were laid perpendicular to identifiable joint sets in the field and spacings between individual discontinuities were recorded. Results of statistical analysis of joint spacing data from four sites are presented in Table 6.3.

	mean	s.d.	no.	skew	kurt	mode	min.	max.
Winspit Total	1.32	0.57	150	1.42	5.22	1.26	0.46	3.26
Winspit A	1.03	0.30	50	0.09	2.62	1.00	0.46	1.68
Winspit B	1.59	0.84	50	0.57	2.02	1.33	0.48	3.26
Winspit bedding	1.35	0.22	50	-0.82	4.07	1.35	0.75	1.70
Fossil Forest Total	0.93	0.55	110	0.31	1.40	0.79	0.24	2.64
Fossil Forest C	1.50	0.66	30	0.28	1.85	1.44	0.55	2.64
Fossil Forest D	0.79	0.33	30	1.28	4.61	0.70	0.32	1.74
Fossil Forest bedding	0.66	0.29	50	0.51	2.31	0.58	0.24	1.36
Lulworth Cove Total	0.56	0.18	60	0.70	3.65	0.53	0.20	1.11
Lulworth Cove C	0.65	0.24	20	-0.01	2.53	0.68	0.20	1.11
Lulworth Cove D	0.57	0.14	20	-0.37	2.32	0.61	0.29	0.79
Lulworth Cove bedding	0.47	0.08	20	0.97	3.94	0.46	0.36	0.69
Durdle Door Total	0.43	0.23	80	1.35	4.39	0.37	0.11	1.10
Durdle Door E	0.42	0.14	28	0.38	3.10	0.41	0.11	0.75
Durdle Door F	0.68	0.28	19	0.16	1.54	0.62	0.31	1.10
Durdle Door bedding	0.29	0.09	33	0.18	1.69	0.27	0.15	0.45

Table 6.3:   Joint spacing statistics from the four sites of Winspit, Fossil Forest, Lulworth Cove and Durdle Door.

Spacings are measured in metres between each of the joint sets listed in Table 6.1, denoted by the label, and totalled for each site. The listed statistics are the mean,

standard deviation, number of measurements, skewness, kurtosis, mode, minimum value and maximum value. Skewness and kurtosis describe the distribution of the data. A normal distribution has a skewness of 0 and a value of kurtosis of 3.

By considering the graphical shape of the data distribution of joint spacing values between sites, greater understanding can be gained of differences. It might be expected that a natural data set would spread close to a normal distribution, or where a large number of small values are recorded, close to a log-normal distribution (Mohajerani, 1989). The data spread for joint spacing measurements from Winspit is close to a normal plot with a positive skewed distribution, although there is some evidence of a bimodal distribution (Figure 6.5). The distribution of total joint spacing data from Fossil Forest has a strong positive skew and the shape appears to be similar to a log-normal fit of data as suggested by Mohajerani (1989) (Figure 6.6). Lulworth Cove (Figure 6.7) and Durdle Door (Figure 6.8) have distributions of joint spacing data with less skewness that fit closely to a normal plot for a set of data which has the same mean and standard deviation.

From the joint spacing measurements gained at the four sites it is clear that there is a decrease in spacings in a westerly direction along the Portland Limestone outcrop (Figures 6.5 to 6.8). Winspit has a mean joint spacing of 1.32 m, Fossil Forest has a mean of 0.93 m, Lulworth Cove has a mean of 0.56 m and Durdle Door has a mean of 0.43 m. Difference of mean Student t-tests demonstrate that there is a significant difference between all sites (Table 6.4). If the individual data sets are plotted against each other, it is graphically obvious that there are large differences between the sites. The quantile-quantile graph plots the ordered values of joint spacing for one site against the ordered values for another site. If the data sets are of the same size and distribution, then data would plot along the line $y = x$, which is shown. As the data for total joint spacings for Winspit against total joint spacings for Fossil Forest plot above the line of equality, it can be concluded that joint spacing data for Winspit are greater (Figure 6.9). For the plot of Fossil Forest data against Lulworth Cove data all but one of the points occur above the line (Figure 6.10) and for the plot of Lulworth Cove data against Durdle Door data the majority of points occur above the line (Figure 6.11). The importance of the control of joint spacing on rock mass stability and failure mechanisms was discussed in Chapter 4. The statistical analysis of the joint spacing data along the Portland

Limestone coastal cliffs presented here would suggest that rock mass stability increases towards the eastern part of the outcrop. By plotting the cumulative spacing values for each of the four sites on the same plot, the difference is clear (Figure 6.12). The fact that the curves are similar to an 'S' shape indicates that the plotted distributions are close to normal, although the spacing values are plotted on a log scale. However, a greater understanding of rock mass behaviour can be gained by treating the joint spacing statistics for individual sites together with other important rock mass controls in rigorous models.

	Winspit	Fossil Forest	Lulworth Cove
Winspit			
Fossil Forest	Win > Fos		
Lulworth Cove	Win > Lul	Fos > Lul	
Durdle Door	Win > Dur	Fos > Dur	Lul > Dur

Table 6.4: Results of Student t-test for difference of means of joint spacing measurements.

Six tests are recorded. The '>' symbol is used to denote a result where the mean joint set spacing for a site is statistically greater than another site. Where the mean of a data set is indicated to be greater, the result is statistically significant at the 5% level.

At places where the joint pattern is exposed through the Portland limestone rock outcrop in three-dimensions, for instance at the entrance to Lulworth Cove, it can be seen that there is considerable continuity back into the rock mass (Plate 5.1). The joint sets are repeatable and continuous, so there is no need for difficult assumptions about joint persistence. Of the main joint sets for each site, there appears to be very little variability in dip and spacing throughout the outcrop. One site identified with particular characteristics is the Durdle Promontory. There appears to be a sharp change in joint characteristics due to its position relative to the axis of the Purbeck Monocline. In the Portland Limestone outcrop on the seaward side of the Promontory (Figure 6.13), bedding at the eastern end of the Promontory dips more gently than at the western side. The joint characteristics were measured at the eastern end as only there can safe access

to the outcrop be gained. At the eastern end, bedding dips at about 50° to the north, whereas close to the Durdle Door sea-arch, bedding dips at about 85° to the north (Plate 6.1), although this has overturned slightly in the upper part of the cliff. The figures for the joint dip are confirmed by the appearance of the cliff slope in the Portland Limestone on the northern side of the Promontory, which is inclined at a similar angle to the bedding dip.

Other morphometric properties measured included offshore depth. Measured points were converted to mean sea-level and have been linked to cliff height and the sea depth at the base of the cliff. Profiles are plotted for Tillywhim (Figure 6.14), Winspit (Figure 6.15), Pondfield (Figure 6.16), Bacon Hole (Figure 6.17), Fossil Forest (Figure 6.18), Potter's Hole (Figure 6.19), Lulworth Cove (Figure 6.20), Stair Hole (Figure 6.21) and Durdle Door (Figure 6.22). There are groups of similar profiles. At the eastern end of the Isle of Purbeck, offshore profiles at Tillywhim and Winspit are close to horizontal close to the cliff base, before gently deepening at a relatively consistent gradient (Figure 6.23). For the sites in the central part of the Isle of Purbeck, profiles initially decrease in depth close to the cliff faces (Figure 6.24). The rise could be related to the possibility of mound structures of cliff debris which often occur in the immediate offshore (Allsop et al., 1996). At about 30 m from the cliff base, the offshore gradient changes and a concave offshore profile occurs which flattens at a distance of approximately 150 m from the cliffs. At Stair Hole and Durdle Door the offshore profile is initially horizontal before deepening at a steady gradient until 500 m from the cliff face (Figure 6.25). The offshore profile is one of several properties used to calculate the wave pressures acting upon cliffs (Allsop and Vicinanza, 1996), and the initial gradient for the first 10 m at the base of the cliff is included in this study.

6.1.2 Rock strength data

It was possible to make use of secondary data for the rock strength parameters required as part of this study (Allison, 1986; 1989). Rock samples were collected at the Isle of Purbeck field sites and tested with a Schmidt hammer, Hoek Triaxial Cell and Grindosonic apparatus. Intact rock strength data are presented in Table 6.5. Cut samples for the Hoek Cell were weighed, dried and weighed again in order to calculate porosity and bulk density. Density values range from 2260 Kg m^{-3} at Winspit to 3190 Kg m^{-3} at Stair Hole and porosity values range from 10.12% at Winspit to 1.80% at Stair Hole.

Property	Porosity	Bulk Density	Yield Stress (15 MPa)	Yield Stress (30 MPa)	Bulk Modulus	Shear Modulus	Dynamic Young's Modulus	Schmidt Hammer	Estimated Young's Modulus
Units	%	kg m^{-3}	MPa	MPa	GPa	GPa	GPa	'R'	GPa
Tillywhim	9.86	2350	205	267	39.56	22.61	56.97	34	28
Seacombe	7.69	2390	204	255	28.35	16.20	40.82	28	22
Winspit	10.12	2260	153	188	27.63	15.78	39.78	31	22
Pondfield	9.96	2350	204	255	30.36	17.35	43.72	32	25
Worbarrow Tout	2.31	2790	259	307	40.39	23.08	58.16	46	59
Bacon Hole	6.66	2420	226	262	32.38	18.50	46.63	31	26
Fossil Forest	7.19	2390	173	271	36.87	21.07	53.09	33	27
Lulworth Cove	2.51	2580	240	299	42.99	24.56	61.90	38	39
Stair Hole	1.80	3190	251	317	35.66	20.38	51.35	27	43
Durdle Door	3.59	2570	280	373	41.13	23.50	59.23	29	28

Table 6.5:   Results from tests on rock samples collected at sites to examine the mechanical properties of the Portland Limestone (Allison 1986).

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From six test specimens for each site, Hoek Cell tests were conducted at confining pressures (σ_3) of 15 MPa, 30 MPa and 60 MPa. Results are presented as values of stress at failure drawn as Mohr's circles (Figure 6.26). The Portland Limestone at Durdle Door displays the highest yield strength and the lowest occurs at Winspit. Broadly, strength increases along the Isle of Purbeck coastline from east to west (Allison, 1989).

Bulk modulus and shear modulus were calculated from dynamic Young's Modulus and dynamic Poisson's ratio for Portland Limestone rocks from the Isle of Purbeck field sites (Section 3.2.2). Ten sonic wave propagation readings were made for each specimen, although there is very little variability in the results. Readings were analysed and converted to the elastic moduli using the EMOD computer software (Lemmens Elektronika, 1988). Shear modulus varied between 15.78 GPa for intact rocks at Winspit and 24.56 GPa at Lulworth Cove and bulk modulus values from 39.78 GPa at Winspit to 61.90 GPa at Lulworth Cove (Table 6.5). The deformation modulii listed here can be directly entered as UDEC input. Schmidt hammer 'R' values have been converted into values of Young's modulus for each site using guidelines set out by Deere (1966) (Section 2.3.1). However, as the Schmidt hammer method provides only an estimation of Young's modulus, and there is a correlation of 0.58 with the dynamic modulus values, it was decided not to use the Schmidt values for UDEC input.

The joint friction angle parameter is an important control upon the failure of rock masses (Section 4.3.1). The joint friction angle is generally calculated by considering the angle of the line which connects the upper part of Mohr's circles on a plot of compressive strength at failure, τ, against confining pressure, σ. The τ axis intercept is the cohesion and a line for a number of circles can be produced by regression. However, the method assumes that the failure envelope for the Mohr's circles is linear which is not appropriate for the Isle of Purbeck field sites (Figure 6.26). Also, there are large differences in the compressive strength at failure for samples from the same site tested at the same confining pressure. The overall effect results in a failure envelope being summarised by a linear relationship which has an unsatisfactory correlation strength. The solution was to use a friction angle value of 36° for each site calculated from all of the strength test data for the Portland Limestone outcrop. Although the friction angle exerts a major control, not many failure planes are close to 36° for the Isle of Purbeck model meshes. The problems encountered in attaining a

result for the joint friction angle from the Portland Limestone add further evidence to the debate upon the representation and accuracy of laboratory test results (Section 2.3.4). Problems can be associated with the representation of samples removed from the field, laboratory testing and anisotropy, but results obtained by sonic propagation of waves appear to be very repeatable.

6.2 Characteristic field sites and UDEC input

After data analysis, similarities and differences can be identified between the ten Isle of Purbeck field sites. Based upon the discontinuity geometry data, sites can be grouped into four categories depending upon relative position along the Purbeck Monocline. At the eastern end of the Isle of Purbeck, the Portland limestone sea cliffs of Winspit, Seacombe and Tillywhim have horizontal bedding. At the western end of the Isle of Purbeck, the measured bedding at Durdle Door dips at 52° to the north. In between, the sites of Stair Hole and Lulworth Cove are characterised by bedding which dips at approximately 30° to the north, and the sites of Fossil Forest, Bacon Hole, Worbarrow Bay and Pondfield have bedding which dips at approximately 20° to the north (Table 6.1).

At the same time, similar groups of sites can be designated based upon the differences in the strength properties of the Portland Limestone intact blocks. If the results of the triaxial tests are examined by plotting Mohr Circles for the shear stress at failure τ_f against the confining stress σ, the different sites can be clustered into groups (Figure 6.27). The least competent intact Limestone occurs at Winspit. At the other end of the strength spectrum is material from Durdle Door where there may be a structural control of tectonic hardening. Two further groups can be defined: the sites of Fossil Forest, Bacon Hole, Pondfield, Seacombe and Tillywhim, and the remaining locations of Stair Hole, Lulworth Cove and Worbarrow Tout.

Based upon grouping by rock discontinuity and intact strength, four sites can be selected representing the changing rock mass properties along the Isle of Purbeck coast (Figure 6.28). The selection for modelling is aimed at providing an understanding for representative sites which exhibit a failure mechanism and rate of cliff retreat characteristic for different sections of the Isle of Purbeck coast. The Durdle Promontory has steep bedding, material of high yield strength and a cliff height of 30 m (Plate 5.3).

A cross-section of the Portland Limestone cliffs at each end of the promontory is modelled as well as a section across the classic sea arch of the promontory. Lulworth Cove has bedding which dips at 27° to the north and a cliff height of 28 m. The site is representative of Portland outcrops which are relatively steeply dipping and have an intermediate rock yield strength. Two cross-sections are modelled based on data from Lulworth Cove. A model is made for the whole rock mass height which is characteristic of locations on this part of the Portland Limestone outcrop (Plate 5.1) and a model is made across a pincer at the entrance to the Cove including a point where the rock geotechnical data were collected (Plate 5.5).

To the east of Lulworth Cove, Fossil Forest has a cliff with a height of 43 m and bedding which dips at 25° to the north (Plate 5.6). It is one of a cluster of sites which has gently dipping bedding planes and an intermediate rock strength. The modelled cross-section designed to simulate characteristics of this site has a stepped profile which is apparent in the field. Winspit is typical of coastal cliffs at the eastern end of the Isle of Purbeck. The site has horizontal bedding which cuts the rock into large blocks, the cliffs have a height of 43 m, and the rock has a low yield strength at failure (Plate 5.9). Two models are constructed in order to represent features of the rock mass landforms formed in the Portland Limestone outcrop at Winspit. One has a step in the profile upon which quarrying activity has occurred and the second is a continuous cliff which rises from the sea. Both profiles are to be found at the site.

The modelling methodology used to simulate geomorphological slope evolution at different parts of the Portland Limestone outcrop on the Isle of Purbeck used current cliff profiles as a starting point. The profiles are stable initially at the point when the mesh has consolidated. The models constructed are based upon the real cliff profiles and controlling rock mass characteristics from sites at Winspit, Fossil Forest, Lulworth Cove and Durdle Door in order for conclusions to be made on the processes of failure and relative rates of retreat between models. Relevant parameters for model input sub-divide into a number of groups. Information such as rock mass morphology, discontinuity characteristics and intact rock block properties has been collated from field and laboratory work. It is important to maintain a link between the modelling of real-world rock slopes and the understanding gained in the theoretical parameter sensitivity study of rock mass controls. Based upon the study of background

considerations, importance is attached to model accuracy in parameters such as cliff dimensions, discontinuity geometry and joint friction angle. However, it is emphasised that some level of simplicity has to be maintained in order to understand model response and that the meshes are not complete scale representations of real-world conditions at each site. Some factors, such as the removal of blocks due to sea action which is difficult to quantify, have been assumed to be constant between sites to permit spatial and temporal comparison. But models do replicate the important characteristics which differentiate parts of the Portland Limestone outcrop along the Isle of Purbeck coast. Parameters entered into the UDEC code files for each model run are listed in Table 6.6 and the actual input files are in Appendices 6.2 to 6.9.

Parameter	Units	Winspit	Fossil Forest	Lulworth Cove	Durdle Door	Durdle Prom.
Cliff height above m.s.l.	m	40	40	28/18	30	19
Cliff height below m.s.l.	m	4	6	7	7	7
Free face angle	x°	85	66	68	74	various
UDEC mesh bearing	x°	317	000	005	355	085
Joint set 1: angle	x°	90	-53	-48	35	-4.5
Joint set 1: spacing/s.d.	m	7.95/4.2	7.5/3.3	3.25/1.2	2.1/0.7	2.1/0.7
Joint set 2: angle	x°	-67	-74	-66	21	50
Joint set 2: spacing/s.d.	m	5.15/1.5	3.95/1.65	2.85/0.7	3.5/1.25	3.5/1.25
Bedding: angle	x°	0	21	27	-47/-72	-57
Bedding: spacing/s.d.	m	6.75/1.1	3.3/1.45	2.5/0.4	1.5/0.5	1.5/0.5
Bulk density	kg m^{-3}	2260	2390	2580	2570	2570
Bulk modulus	GPa	27.6	36.9	43.0	41.1	41.1
Shear modulus	GPa	15.8	21.1	24.6	23.5	23.5
Joint normal stiffness	GPa m^{-1}	20	20	20	20	20
Joint shear stiffness	GPa m^{-1}	4	4	4	4	4
Joint friction angle	x°	36	36	36	36	36

Table 6.6:   Model input parameters.

There are some similarities in input between each site (Table 6.6). In order to permit comparison between sites along the Isle of Purbeck coast, cliff profile meshes were taken perpendicular to the bearing of the coastline. However, the dips of each joint set on the two-dimensional UDEC mesh have to be converted from joint set data, which have a dip defined at the bearing of the joint strike. Thus, all of the mean joint set data were run through a program based upon the intersection of two planes which was specially written to convert the data into the appropriate dip on the UDEC mesh (Appendix 3.1). Standard deviations of the joint spacing data were input for each joint set as indicators of spacing variability. Although the spacing variability parameter has been shown to have very little control upon UDEC rock mass models (Section 4.4.1), this data were included as they are available. Accurate data were available in the literature for the joint stiffness parameter. Furthermore, the parameter sensitivity testing (Section 4.5.2) suggested that the control exerted on the UDEC models by the joint stiffness parameters is not great, and it is very difficult to accurately measure joint stiffness properties.

Individual model blocks were defined as rigid units, since all the Portland Limestone cliff outcrops have a relatively low altitude, small gravitational stress and relatively high material strength, with failure usually taking place along the discontinuities rather than through the material. Intact rock properties and joint friction angle for each model were taken from the laboratory test results and input directly. Gravitational acceleration was set at 9.81 m s^{-2} and initial vertical model stresses were set to act as a gradient through the model to simulate the overburden weight of surrounding blocks. Initial horizontal stresses were set to the recommended half of the value of the vertical stresses at a point (Herget 1988). However, by the time the model has been run to initial equilibrium, before the free face is released, stresses are mathematically balanced throughout the rock mass. At the beginning of each model run, the boundary of the mesh representing the cliff face was fixed to allow the blocks to consolidate. The purpose of the consolidation phase is to allow equilibrium to be reached before boundary conditions are freed to permit failure of the blocks at the cliff face. For each of the models run to simulate characteristics of the Isle of Purbeck coastal cliffs, equilibrium was achieved by 6,000 steps, with an exponential reduction of the unbalanced forces towards zero.

In order to simulate characteristics of the site at Durdle Door, three model meshes were constructed. Two of the model meshes cut through the Durdle Promontory perpendicular to the cliff free face at a bearing of 355° and the third mesh models the profile of Durdle Door at a bearing of 085°. The mesh set up to simulate characteristics of the sea-arch at Durdle Door (Figure 6.29) occurs in the part of the Durdle Promontory where the bedding dips at 85° to the north. Thus, after conversion the bedding dips on the mesh at 57° to the west, joint set E dips at 4.5° to the west and joint set F dips at 50° to the east. The arch dimensions were estimated from scaling photographs and included 7 m that occur below mean sea level. The right-hand side of the mesh, the left-hand side of the mesh and the boundary of the arch were fixed in order for the blocks to consolidate. At equilibrium, the boundary of the arch and the left hand side of the model were released to enable block failure to occur. Further model meshes were constructed in order to simulate the profile section across the Durdle Promontory. At the eastern end of the Promontory bedding dips at 52° to the north. On the mesh after conversion, the bedding dips at 47° to the north, joint set E dips at 35° to the south and joint set F dips at 27° to the south (Figure 6.30). The profile section on the southerly, right-hand side of the model mesh has a free face angle of 74°. On the northerly side of the mesh the cliff face dips at 47°, the same as the angle of bedding, representing conditions in the field. Both faces were fixed in order for the model to reach equilibrium before block failures were allowed to develop. At the western end of the Durdle Promontory, the bedding in the field dips at approximately 85° to the north (Figure 6.31). A simple model was constructed in order to compare differences with the eastern model which has gentler bedding, although accurate joint measurements were not available for the western profile. Therefore, the model was constructed in a similar fashion as the eastern mesh, but a converted bedding value of 72° was input, and the steepness of the northern cliff face was altered in order to represent the higher bedding dip.

In order to simulate characteristics of the Portland Limestone rock cliffs for the Lulworth Cove area, two model meshes were constructed. One of the models was constructed in order to examine the profile of the pincer at the entrance of Lulworth Cove embayment, and the second model was designed to replicate characteristics of the full height rock cliff which occurs at locations very close to the Cove entrance. Both meshes have a bearing of 005° which is perpendicular to the coastal cliffs at Lulworth

Cove and the same joint data were entered for the two. After conversion, the bedding on the UDEC mesh dips at 27° to the north, and two other joint sets dip at 66° and 48° to the south. The mesh set up for the pincer at Lulworth Cove models a rock mass which is 25 m high and has a sea face angle of 68° on the southern side (Figure 6.32). The slope on the northern side of the pincer occurs at the same angle as the dip of the bedding as measured in the field. The offshore profile slope increases in height to the south of the sea cliffs at Lulworth Cove and is represented in the model. Both sides of the model pincer were fixed for the model mesh to consolidate. The second model constructed for the whole cliff height here has a height of 35 m (Figure 6.33). The left hand boundary of the model was fixed throughout the model run, but the southern free-face was released when the model reached equilibrium to allow failures to develop.

Only one model mesh was set up to examine characteristics of the central part of the Portland Limestone outcrop for which Fossil Forest is a typical site. The bearing of the UDEC mesh profile of the Fossil Forest site cuts a north-to-south section. The mesh modelled has a height of 46 m, of which 6 m occur below mean sea level at Fossil Forest, and a step is cut into the profile based on morphometric data collected from the field site (Figure 6.34). After conversion, the bedding on the UDEC mesh dips at 25° to the north and the two joint sets dip at 74° and 53° to the south. The offshore profile immediately adjacent to the sea cliffs at Fossil Forest is horizontal and this information is included in the model. As before, the northern, left-hand edge of the mesh is fixed throughout the model run and the southern, sea cliff face is freed after mesh consolidation.

The two model meshes set up to simulate the rock mass landforms at Winspit have a height of 44 m, of which 4 m is below mean sea level (Figures 6.35 and 6.36). As the coastline at this part of the Isle of Purbeck has an orientation of 047°, the model meshes cut a profile at a bearing of 317°. The converted bedding dip on the mesh is horizontal, and the two other joint sets dip vertically and at 67° to the north in the model. The first model mesh has a step at a height of 17 m, whereas a second model is run to simulate the full cliff height. Both of these scenarios occur in the field at Winspit, because of quarrying of Portland Limestone. For both model meshes, the southern sea cliff face was freed at 6,000 steps of the model run to allow failures to develop.

6.3 Results

Output from the models which were run to simulate characteristics of key field sites in the Portland Limestone cliffs of the Isle of Purbeck was plotted for important stages in the modelling process. A block plot was made for each site after the model mesh had consolidated, and the mesh was plotted at 100,000 cycles for each model, in order to permit comparison. Further block plots highlight important characteristics of the landform development and are individual to each model.

The velocity vectors are also included on the block plots so that the failure mechanism may be clearly identified. The velocity vectors are scaled in units of m s^{-1}, but the model calculation time-scale is not related to real-world time (Section 3.2.2). Thus, the values for velocity are not related to speed at which a block would actually fall away from the cliff face and the representation for disconnected, falling blocks is not accurate (Itasca, 1993). However, it is interesting and possible to make relative comparisons between models for the velocity of failing blocks (Table 6.7).

In the description of model output and discussion which follows, it is possible to make assessment of the relative speed of failure of a particular slope (Table 6.7). Where the velocity of a block exceeds 2.0 m s^{-1}, observations from the modelling process would suggest that a block or more is either falling freely, or failing catastrophically. At less than 2.0 m s^{-1}, blocks are creeping along discontinuity planes.

Model	Figure	Step count	Velocity	Velocity
Winspit quarried			Upper	Lower
	6.38a	10,000	0.7	1.3
	6.38b	30,000	0.1	0.3
	6.38c	90,000	3.9	3.7
	6.38d	150,000	0.016	0.039
	6.38e	190,000	-	0.00038
	6.38f	390,000	-	0.00010
Winspit straight	6.41a	100,000	4.6	
	6.41b	750,000	2.6	
	6.41c	2,400,000	-	
Fossil Forest			Upper	Lower
	6.43a	14,000	1.7	2.0
	6.43b	100,000	-	2.7
	6.43c	200,000	-	0.16
	6.43d	400,000	-	-
	6.45	454,000	-	-
Lulworth pincer	6.47a	10,000	1.3	
	6.47b	20,000	0.5	
	6.47c	40,000	0.16	
	6.47d	60,000	0.63	
	6.47e	100,000	-	
Lulworth mass	6.50a	14,000	2.3	
	6.50b	54,000	2.2	
	6.50c	200,000	0.23	
	6.50d	600,000	-	
East Durdle	6.53a	10,000	1.3	
	6.53b	20,000	6.7	
	6.53c	50,000	2.5	
	6.53d	100,000	2.4	
	6.53e	200,000	2.0	
	6.53f	470,000	-	
West Durdle			North cliff	South cliff
	6.56a	10,000	3.9	1.3
	6.56b	20,000	1.7	2.3
	6.56c	50,000	1.4	4.6
	6.56d	100,000	0.04	0.043
	6.56e	200,000	-	-
Durdle Door			West face	Arch
	6.59a	10,000	2.0	2.5
	6.59b	20,000	0.9	1.0
	6.59c	60,000	1.5	3.5
	6.59d	100,000	1.6	2.0

Table 6.7: The magnitude of the largest velocity vector, measured in m s^{-1}, in each UDEC block plot of modelled sites from the Isle of Purbeck, Dorset. Where activity occurs in two parts of the model, magnitudes are listed for both.

6.3.1 Winspit

The initial model mesh for the section of the quarried Winspit site at 6,000 steps at equilibrium is plotted in Figure 6.35 (Plate 5.9). The northern, fixed side of the mesh is on the left-hand side of the plot, and the stepped sea cliff is plotted on the right-hand side. Indication that the mesh is at equilibrium is given by considering a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.37). The plot of activity decreases exponentially to very close to zero by 3,000 steps. Once the cliff face is released, failure occurs by the detachment of columns of blocks from the rock slope. Initially, the displacement vectors indicate the commencement of rock mass failure at 9,700 steps and displacement of blocks is visible in the output by 11,200 steps. Figure 6.38a demonstrates the failure mechanism which is evident at 10,000 steps. The velocity vectors throughout the model show a general movement of blocks towards the free face. Failure is starting due to a combination of creeping toppling, and creeping sliding along the joint set dipping at 67° into the free face (Table 6.7). The toppling mechanism is controlled by the horizontal and vertical joint sets and occurs due to a relatively low *b/h* ratio of blocks cut by the two sets (Chapter 4).

By 30,000 steps, two columns of rock at each level of the Winspit cliffs are toppling away from the cliff faces (Figure 6.38b). The toppling columns have increased in stability, due to the sliding of blocks at the back of the columns on the 67° joint set creating a wedge, and the speed of the failure has slowed (Table 6.7). The movement of columns away from the cliff has allowed blocks to settle in the upper part of the cliff profile along the 67° joint set decreasing the smoothness of the top slope. The toppling columns are creeping forward slowly. Toppling columns in the field have been observed to fail over a number of years, such as a topple on the Isle of Portland which has increased its crest from the cliff face by approximately 2 m in three years. The monitoring of such a failure at a modelled location could provide a possible method of temporally constraining the UDEC model output for the Portland Limestone outcrop.

The toppling failure occurring in the UDEC model which includes characteristics from the field site at Winspit is still active at 90,000 steps of the simulation run (Figure 6.38c). However, the toppling mechanism in the first column from each of the cliff faces is proceeding at a much greater rate than at 30,000 steps, and the failure is much more advanced (Table 6.7). The earlier, creeping motion appears to

be replaced by a catastrophic falling mechanism. By the monitoring of sequences of images for the Winspit model using the UDEC movie command, it can be estimated that the creeping toppling failure mechanism for the two columns becomes a catastrophic failure after 56,000 cycles. The 46,300 cycles between the initial block displacement and the onset of blocks falling for the upper column represent a period which could possibly be calibrated from field monitoring if such a column were to fail at Winspit Quarry. The temporal scale for the lower column would be affected by marine activity. Further activity in the model at 90,000 steps (Figure 6.38c) is in the upper part of the cliff face with the initiation of a creeping toppling mechanism in the second column from the cliff face.

At 150,000 cycles of the UDEC model examining characteristics of the Portland Limestone coastal cliffs at Winspit, the debris of the toppled columns which formed the cliff face has stabilised (Figure 6.38d). In the upper part of the cliff profile, the second column which commenced a creeping toppling mechanism has also stabilised, and the velocity vectors indicate a reactionary displacement. In the field at Winspit Quarry, piles of failed blocks provide much evidence of toppling failures and the modelled cliff profile is characteristic of conditions in parts of the Quarry (Plate 6.1). The fact that the column has come to a rest at an inclined angle, as opposed to the vertical starting position, demonstrates the influence of the 67° joint set in the stability of the cliffs at Winspit. The block in the upper cliff profile has stabilised because blocks have slid to the rear of the column forming a wedge between the rotating part of the column and the cliff face. The level of understanding which has been gained from the UDEC output in Figure 6.38d would be difficult to achieve using a kinematic approach based upon stereographic projection. The possibility of restabilisation once a failure mechanism has commenced and caused a change in rock mass geometry can be easily monitored using UDEC output. In the lower part of the cliff profile of the model at 150,000 cycles (Figure 6.38d), further failure is starting in the newly-formed free face of the sea cliff. The column is wide due to the statistical distribution of joint spacings for the vertical joint set. Again the mechanism is a creeping toppling mechanism that acts very slowly in combination with a sliding of blocks to the rear of the column (Table 6.7). However, by 190,000 cycles (Figure 6.38e) and 390,000 cycles (Figure 6.38f), virtually no further displacement of the blocks in the column has occurred. The velocity vectors indicate

very slow activity which would not cause appreciable movement of the blocks. In the history plot for total unbalanced forces against time for the model run (Figure 6.39), it can be seen that the model stabilised at about 200,000 steps and no further activity could be expected. The history plot also demonstrates that the stability of the cliffs is episodic, with displaced blocks acting as a buttress to the rock slope and stress conditions up the face changing through time due to the removal of rock columns.

Given that the column at the sea cliff of the Winspit model which is evident at 150,000 steps is protected from rotation by debris from the preceding topple (Figure 6.38d), it was decided to remodel the sequence from 150,000 steps after removing the debris. In a real situation, some of the debris would be removed by the sea. The removal of debris does promote further movement of the column by 170,000 steps (Figure 6.40a), but by 230,000 steps (Figure 6.40b), the column has again stabilised as indicated by the magnitude of the velocity vectors (Table 6.7). The failure is again an indication of the influence of the 67° joint set which controls the stability of the cliffs at Winspit. Toppling commences for columns of rock defined by the vertical joint set and the horizontal bedding, but blocks restabilise as slip occurs at the rear of toppling columns.

A model was also constructed to represent the field conditions at the Winspit field site which used a profile with a single cliff section. The south-east facing sea-cliff face is on the left-hand side of the output mesh and images are presented from 6,000 steps (Figure 6.36), 100,000 steps (Figure 6.41a), 750,000 steps (Figure 6.41b) and 2,400,000 steps (Figure 6.41c). The first image in the diagram shows the UDEC model at equilibrium, and a mesh which has the same joint dip and spacing characteristics as the mesh in Figure 6.35. Initially the first block column starts to move after 10,900 model cycles. The failure mechanism is the same as that for the other Winspit mesh. Columns of blocks fail by a toppling mechanism, but blocks to the rear of the column fall vertically with the 67° joint set acting as a sliding plane (Figure 6.41a). Compared with the previous model of conditions from the Winspit site (Figure 6.38c), many more blocks are affected by the failure mechanism, because the greater the free face height, the greater the instability of the blocks (Hsu and Nelson, 1995). Also, the initial failure involves three columns of rock. By the third plot in the sequence (Figure 6.41b), much of the failure of the first three columns is complete, although there is movement within the fallen blocks. Once some of the load exerted at the base of the remaining free face is

reduced by movement within the talus, a further two columns topple by the same failure mechanism. The last image plotted in Figure 6.41c shows the cliff at equilibrium. The blocks are balanced precariously which would only occur temporarily in the field, with weathering and erosion leading to further movement within the failed blocks. It reflects the fact that the code can not accurately model the relative juxtaposition of disconnected blocks. If the loose talus were removed, the cliff face would be a stepped profile, although marine erosion at the base would probably maintain the exposure of successive sequences of columns.

6.3.2 Fossil Forest

The initial model mesh for the section of the Fossil Forest site at 6,000 steps after the blocks have settled is plotted in Figure 6.34 (Plate 5.6). Indication that the mesh is at equilibrium is given by a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.42). Once the cliff face is released, the displacement vectors indicate the start of rock mass failure at 8,500 steps and displacement of blocks is visible by 10,100 steps. Figure 6.43a demonstrates the failure mechanism which is evident at 14,000 steps. By this stage, failure of the blocks is well commenced with displacement from the cliff crest and the upper part of the step. Failure is by sliding of blocks entirely along the joint set dipping at 53° to the south as indicated by the velocity vectors, which are plotted consistently at 53° on the mesh. However, in the field, wedge failure would be evident with sliding along the intersection between the 53° joint set and the 74° joint set. It is interesting that there is no element of toppling failure occurring at the same time in the model. At the real Fossil Forest field site, there is no evidence of toppled blocks, which suggests that the b/h ratios for the blocks cut by the bedding at 21° to the north and the joint set at 53° to the south are greater than the b/h ratio that would be needed for toppling to occur (Chapter 4). The model mesh for the simulation of characteristics of the Fossil Forest field site demonstrates good corroboration between the theoretical failure of rock masses and real mechanisms. It provides an indirect validation of the rock mass modelling code and indicates the importance of a consideration of theoretical parameter sensitivity studies in the understanding of landform processes.

Blocks on the mesh of the modelled rock mass fail relatively rapidly by the sliding mechanism, although the movement would still be classified as creep as

catastrophic failure does not set in. By 100,000 steps, the sliding layer in the upper part of the rock mass has reached the step in the profile and stabilised (Figure 6.43b). It is doubtful whether another layer of rock will slide, because the morphometry of the profile prevents movement. The initial sliding layer was bounded by a discontinuity dipping at 53° which is exposed in the free face. No other discontinuities do this, unless exposed by failure in the lower part of the cliff. In the actual sea cliff, failure is occurring by a catastrophic sliding mechanism (Table 6.7), although more material is involved.

There is little change in model conditions by 200,000 cycles (Figure 6.43c). However, activity in the sliding of blocks in the lower part of the profile is much slower (Table 6.7). The load of the sliding layer upon the blocks which have reached the base of the cliff has caused a few of the blocks to topple, although the predominant failure mechanism is still a pure sliding mechanism. The rate of activity decreases within the model until about 370,000 cycles when the slope stabilises. The plot of the mesh for 400,000 steps (Figure 6.43d) shows very few differences from the plot at 200,000 steps (Figure 6.43c) which demonstrates how slowly movements occurred during the later stages of the model run. There is a slight change in the position of the sliding layer of blocks, and blocks which have toppled have settled upon the wave-cut platform. The history plot for total unbalanced forces at this point shows that the UDEC mesh for Fossil Forest has stabilised as the forces are nearly zero (Figure 6.44). A further run of the model was made after removing the debris at the base of the lower sea-cliff. As at Winspit, the exercise simulated the effect of the sea upon the fallen talus to determine whether the removal of the support at the cliff base would initiate further failure of the cliffs. In contrast to Winspit, no further failure occurred, as no further sliding layers remain unsupported (Figure 6.45). The control upon the failure of the cliffs in the model mimicking the Fossil Forest field site appears to be daylighting of discontinuities which dip at 53° in the free face, allowing sliding of blocks. Thus, the occurrence and nature of subsequent activity at Fossil Forest would be determined by sea pressure, which could remove supporting blocks at the base of the slope.

6.3.3 Lulworth Cove

Two model meshes were constructed to simulate characteristics of the Portland Limestone outcrop at Lulworth Cove and will be dealt with separately. The first model simulates a profile of the pincer which occurs at the eastern side of the entrance to the Cove. The initial model mesh for the section of this pincer at 6,000 steps after the blocks have settled is plotted in Figure 6.32. The steeper, southern sea-cliff side of the mesh is on the right-hand side of the plot, while the northern edge of the pincer dips at approximately the same angle as the bedding (Plate 5.1; 5.5). Indication that the mesh is at equilibrium comes from a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.46). The plot of activity decreases exponentially to very close to zero by 2,000 steps. Once both slope faces are released, failure occurs by sliding of layers of blocks on the southern, right-hand side of the rock mass. Initially, the displacement vectors indicate the start of rock mass failure at 7,700 steps and displacement of blocks is visible by 9,700 steps. Figure 6.47a demonstrates the failure mechanism, which is evident at 10,000 steps. The northern side of the pincer is clearly stable as the 29° joint set is less than the joint friction angle and sliding is not possible. Toppling is not evident on the northern side, for two possible reasons. Either the b/h ratio is too great for blocks in the model, or a limit is imposed by the slope inclination of less than 30°. On the sea-cliff southern side of the pincer, creep sliding is occurring (Table 6.7). The orientations of the velocity vectors reflect sliding on both the 66° dipping discontinuities and the 48° dipping discontinuities. Again sliding is occurring in layers along discontinuities which are exposed in the cliff face acting as shear planes. An element of a toppling failure mechanism is precluded from the south-facing slopes at Lulworth Cove as the b/h ratio of the blocks is too great.

The slow rate of sliding in the sea-cliff on the Lulworth Cove pincer model is indicated in the plot for 20,000 steps (Figure 6.47b). Compared with the plot for 10,000 cycles (Figure 6.47a) there has been a large displacement of rock blocks at the crest of the cliff. The sliding mechanism is occurring over both southerly-dipping joint sets. By 40,000 cycles (Figure 6.47c), the rate of block displacement has decreased (Table 6.7): the blocks at the base of the sliding layers are acting as key-blocks by countering the force exerted from the motion of blocks above. However, the upward force of the three blocks at the base of the sliding layer has been overcome by 60,000 steps (Figure

6.47d), the resisting blocks have been overturned, and sliding failure has continued. Key-blocks at the base of a failure mechanism explain why fluctuations occur in the total unbalanced forces plotted as the model is run (Figure 6.48). The creeping sliding behaviour of the rock mass continues as periods of sliding, interspersed with periods of low activity as forces build up on certain key-blocks in the mesh.

Block displacement decreases in the model of characteristics of the pincer at Lulworth Cove until the block mesh stabilises at about 90,000 cycles. The final mesh was plotted at 100,000 steps (Figure 6.47e). Further activity would not occur unless one of the southerly dipping joint sets is exposed in the cliff face. Thus, the model signals that the pincer develops through the successive removal of parallel layers on the southern side of the rock mass. Morphometric shape is maintained, but the cliff crest is reduced in height. Periods of activity would be separated by periods of stability until the sea exposes the southerly-dipping joint sets.

The second model mesh constructed to simulate characteristics of the Portland Limestone outcrop at Lulworth Cove included the full cliff profile height which is found next to the pincers at the entrance to the Cove. The model mesh was set at the same orientation as for the Lulworth Cove pincer model, and the joint geometry was defined using the same data. The initial model mesh for the section of the Lulworth Cove cliff at 6,000 steps after the blocks have settled is plotted in Figure 6.33. Indication that the mesh is at equilibrium comes from a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.49). Once the southern cliff face was released, failure occurred by the sliding of layers of blocks. Initially, the displacement vectors indicate the start of rock mass failure at 7,300 steps and displacement of blocks at 8,300 steps.

The plot for the Lulworth Cove cliff model taken at 14,000 steps indicates that the cliffs are failing rapidly (Table 6.7) (Figure 6.50a). The failure mechanism is the same as for the pincer model with a pure sliding mechanism developing along the joint sets dipping at 66° and 48° to the south in the model. However, the failure occurs for a much greater volume of blocks and at a greater depth in the upper part of the cliff profile. The larger failure can be explained by considering that the greater cliff height causes more sliding planes to be exposed in the sea-cliff face. By 54,000 steps (Figure 6.50b), there is much change in form. The cliff takes on the appearance of a stepped profile due to the different displacement of individual sliding layers. At the base of the

profile, the motion of sliding layers from above has caused the toppling of a few blocks. As with the pincer, change in cliff face form slows after rapid initial activity. The plot for the Lulworth Cliff model at 200,000 steps (Figure 6.50c) shows few changes from that at 54,000 steps (Figure 6.50b). Motion of blocks is still occurring, but has taken the form of very slow creep (Table 6.7). By 600,000 steps, the blocks which have slid down the cliff face have settled at the base of the profile preventing further sliding (Figure 6.50d). The plot of total unbalanced forces for the run indicates that stability has been reached (Figure 6.51). Amid the fluctuations in the plot, two periods of increased force activity can be identified. The model appears to be close to stabilisation at approximately 250,000 steps, but resisting forces are obviously overcome and further movement occurs. The end cliff form profile is stepped and is inclined at angles reflecting both 48° and 66° joint sets (Figure 6.50d). However, compared with the pincer model, the mesh for the entire cliff took considerably longer to stabilise because of the volume of material involved in the failure mechanism.

6.3.4 Durdle Door Promontory

Three model meshes were constructed to simulate important characteristics of the Portland Limestone outcrop at the Durdle Door Promontory and will be dealt with separately. The first model simulates a profile of the cliffs which occur at the eastern end of the Promontory. The initial model mesh for the section of the eastern Durdle Promontory cliffs at 6,000 steps, after the blocks have settled, is plotted in Figure 6.30. The steeper, southern sea-cliff side of the mesh is on the right-hand side of the plot, and the northern edge of the pincer dips at approximately the same angle as the bedding. Indication that the mesh is at equilibrium comes from a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.52). Once both slope faces are released, failure occurs by sliding and rotational movement of blocks on the southern, right-hand side of the rock mass. Initially, the displacement vectors indicate the start of rock mass failure at 9,100 steps and displacement of blocks is visible in the output by 11,000 steps. Figure 6.53a demonstrates the failure mechanism within the rock mass at 10,000 steps: the velocity vectors concentrate towards the top of the southern face of the mass and point out of the free face.

The initial block displacement in the model which simulates characteristics of the eastern part of the Durdle Promontory is one of block rotation. The plot for the mesh at 20,000 cycles shows that there is a bending of bedding planes caused by the toppling mechanism rotating individual blocks (Figure 6.53b). Due to the changing rock mass conditions, there is a distinct region of toppling which defines the line of the bend in the beds of the mass. The bending of bedding planes is evident in the field at the Durdle Promontory (Figure 6.13) and in the model reduces the height of the rock mass. The toppling mechanism has been controlled by the bedding which dips at 47° to the north and the joint set which dips at 35° to the south in the mesh. The 35° joint set acts as a base plane, and the blocks topple because the b/h ratio of the small blocks at the Durdle Promontory is sufficiently small. It is interesting to note from the plot (Figure 6.53b) that the toppling blocks defined by the bedding and the 35° joint set which are cut by the 21° joint set have remained intact. At the southern cliff edge two further mechanisms are evident. There is the catastrophic failure of rock blocks which have toppled to the extent that they are now falling (Table 6.7), and there is some sliding of blocks. The sliding plane is the 35° joint set which has steepened due to the rotation of the rock columns. The image presented from 20,000 cycles (Figure 6.53b) is taken during a rapid phase of rock mass activity.

By 50,000 steps for the model from the eastern end of the Durdle Promontory, the toppling failure mechanism has largely ceased (Figure 6.53c). This change in the stability conditions of a rock mass is because the toppling mechanism has caused the blocks to rotate, thus changing the rock mass joint geometry and the b/h ratio. At the same time, the rate of rock mass movement on the southern sea-cliff of the model mesh at 50,000 steps is still high as blocks are sliding and falling. The catastrophic falling of rock blocks is not accurately modelled, but those sliding over other block surfaces are well replicated. At the modelled stage of 100,000 cycles (Figure 6.53d), rock displacement activity is concentrated at the cliff-face part of the model on the right-hand side of the plot. Virtually all of the displacement is by sliding of previously rotated rock beds. The velocity vectors indicate a complex movement pattern within the failing material, which could be due to the irregular geometry of the sliding layers that have previously rotated.

After 100,000 steps the displacement of blocks in the model mesh simulating characteristics of the eastern part of the Durdle Promontory is much reduced. For the plot at 200,000 cycles (Figure 6.53e) the profile form is very similar, although the sliding mass of blocks has been displaced to a lower part of the cliff. Block movement is now entirely confined to the sliding and falling mass of blocks on the right-hand side of the model mesh. The creep of blocks on the cliff edge continues with little change in profile until 470,000 steps, when the rock mass stabilises (Figure 6.53f). The final profile is slightly stepped in form with blocks balanced precariously upon each other. Also, the height of the cliff has been reduced by about 5 m, although the rock mass landform has maintained its shape. Stability occurs when the blocks at the base of the cliff on the southern side of the mesh have finally provided a resistance to the motion of blocks above. The balance between resistance and block motion is reflected in the plot of total unbalanced forces for the model (Figure 6.54). It is interesting to note that the unbalanced forces were greater in the early part of the model run, when block displacement in the model was greatest.

The second model from the Durdle Door Promontory simulates a profile of the cliffs which occurs at the western end of the Promontory. The initial model mesh for the section of the eastern Durdle Promontory cliffs at 6,000 steps after the blocks have settled is plotted in Figure 6.31. The model mesh reflects the field conditions for the profile cross-section which is located close to the Durdle Door sea-arch. The two differences from the previous simulation are that the northern cliff is inclined at a much steeper angle, and that the bedding dips at 73° as opposed to 47°. Indication that the mesh is at equilibrium comes from a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.55). Initially, the velocity vectors indicate the start of rock mass sliding on the northern cliff at 6,500 steps and displacement of blocks is visible in the output by 6,700 steps. On the southern cliff, the velocity vectors indicate that the rock slope is toppling at 7,900 steps and that block displacement is visible in the plot by 9,500 steps. The timings in the model for the initial failures confirm observations from the field site which suggests that Durdle Door fails initially by sliding on the northern side of the Promontory. Figure 6.56a demonstrates the two failure mechanisms which are in operation at 10,000 steps. On the northern side of the profile, rapid sliding movement is already occurring along the bedding planes which are exposed in the cliff

face (Table 6.7). The close-to-vertical orientation of the velocity vectors within the sliding layers is a consequence of the removal of the support from the remainder of the rock mass which is being displaced by a creep toppling failure. The toppling of the blocks is controlled by the bedding planes which cut the side of the blocks and the 35° joint set which cuts the base of the blocks. It would not be kinematically possible for a single block with the same dimensions as these in the model to topple, as the centre of gravity of the blocks does not overhang the pivot points.

The initial failure observed for the rock mass at the western end of the Durdle Promontory is perhaps the most rapid and spectacular of all the Isle of Purbeck field sites modelled. At 20,000 steps (Figure 6.56b) rapid failure of blocks is occurring and a large displacement from the original joint mesh (Figure 6.31) can be identified. On the northern, left-hand side of the model, sliding of two rock layers is well advanced, with individual block displacements being observed of over 5 m from the initial position. However, the main part of the rock mass is failing by a large topple which is fast-moving and on the point of being catastrophic with blocks free falling under gravity (Table 6.7).

By 50,000 cycles, much of the collapse due to the toppling mechanism is completed (Figure 6.56c). The beds in the central part of the rock mass have bent over during the failure event and become more stable. Failure is confined to the falling rocks on the southern side of the rock mass and creep sliding of debris on the northern side at this point in the model run. Compared with the initial plot (Figure 6.31), the modelled rock mass for the western end of the Durdle Promontory at 50,000 steps has decreased in height, but increased in width. As with the other UDEC models of failure events in different parts of the Isle of Purbeck coastline, failure is rapid initially, but it takes a comparatively long time for the blocks to settle towards the end of the failure event. At 100,000 model steps (Figure 6.56d) all the modelled mesh has settled, apart from the southern slope of the Promontory where blocks are continuing to creep. There is evidence for the force-resistance situations which can be imposed by certain key blocks at the base of the profile. Stability finally occurs in the model at approximately 190,000 cycles. Comparing the output for 200,000 steps (Figure 6.56e) with that for 100,000 steps (Figure 6.56d) shows very little change. Once again, the blocks are precariously positioned in the final output, and sea erosion on both sides of the Promontory would

initiate further failure. The modelled profile has decreased in height by approximately 17 m, but increased in width by 9 m. Figure 6.57 confirms that the unbalanced forces for the model run from the western part of the Durdle Promontory decreased in activity.

The third model from the Durdle Door Promontory simulates a profile of the Durdle Door sea-arch which occurs at the western end of the Promontory. The initial model mesh for the section at 6,000 steps after the blocks have settled is plotted in Figure 6.29. The mesh is perpendicular to the model which was designed to simulate characteristics of the western part of the Durdle Promontory. The plot in the UDEC output is viewed from the south and the western, left-hand side of the mass, as well as the arch boundary, were freed to allow failures to develop. The eastern, right-hand side of the plot remains fixed as this part of the model represents the zone of the rock mass which is attached to the rest of the Promontory. Indication that the mesh is at equilibrium comes from a plot of the total history of unbalanced forces for the first 6,000 steps (Figure 6.58). Once the boundaries of the model are released failures occur in several parts of the arch rock mass. A sliding mechanism beginning on the western, left-hand side of the rock mass, and associated with the movement are blocks falling from the upper part of the rock arch. A further small sliding and toppling failure occurs within the arch on the eastern wall. Initially, the velocity vectors indicate the start of rock mass failure at 7,000 steps and displacement of blocks is visible by 8,200 steps. Figure 6.59a demonstrates the failure of the Durdle Door sea-arch at 10,000 steps. Very rapid activity is taking place on the western part of the rock mass associated with a sliding failure and collapse of the upper part of the arch (Table 6.7). It is questionable whether the real-world Durdle Door is unstable and would collapse as rapidly as the three-dimensional stress distribution between the blocks would provide support. However, the modelling exercise does serve the purpose of demonstrating the nature of the arch failure.

The failure of the Durdle Door sea-arch is rapid as a number of catastrophic, free-falling rock blocks are removed from the modelled rock mass. By 20,000 steps (Figure 6.59b) the western slope of the rock mass is failing by sliding-and-toppling along the 50° joint set. At the same time, rocks are falling from the roof of the arch, and there is some toppling in the eastern part of the arch, leaving a rock overhang. The rapid failure continues in a fluctuating manner (Table 6.7). There has been much change in

the landform profile by 60,000 steps (Figure 6.59c). Periods of increased stability occur when the failing material from the two supports on each side of the arch meets and counteracts in forces. At 60,000 steps, elements of the arch cavity still remain, but the activity on the western boundary of the rock mass has decreased. However, blocks are still falling into the central part of the arch. By 100,000 steps, blocks are still falling into the arch, but blocks are also settling close to the boundaries of the two arch supports (Figure 6.59d). There are only a few unstable blocks left on the model mesh. The whole model stabilises at some time before 110,000 cycles (Figure 6.59e; 6.60). In a short number of modelled steps there has been much change in the landform profile. The roof of the arch has been completely eroded and a small sea stack, which would be uncovered at high tide, remains detached from the main part of the Durdle Promontory (Plate 6.1). The eastern spur of the rock arch remains relatively intact.

6.4 Discussion

The models which represent characteristics from the field site at Winspit demonstrate that the cliffs fail slowly during periods of instability which involves the creeping of toppling columns before catastrophic block falling (Table 6.7). The cliffs retreat parallel along the planes of the vertical joint set, but the joint set which dips at 67° into the free face controls cliff failure at the site. The blocks to the rear of the toppling columns slide along the 67° joint set to form wedges which block the rotation of the columns away from the cliff face. Output from the computer simulations of the Winspit model reflects conditions which are seen in the field (Plate 5.9). The model which was constructed to simulate characteristics of the Fossil Forest field site fails by a creeping, sliding mechanism which is controlled by the exposure of the 53° joint set plane. The cliff form of the field site is relatively stable, although the sea pressure at the base of the cliff might promote further failure. The failure mechanism is the same in the models which represent characteristics of the field site at Lulworth Cove, and stability is again controlled by the exposure of sliding planes in the sea cliff. However, the presence of both 48° and 66° joint planes creates a greater sliding potential and activity is more rapid than at Fossil Forest. The models from Lulworth Cove highlight the differences in the volume of failed material created by differences in cliff height. Models which involve a large volume of material take a long time to settle. In models designed to

simulate characteristics of the Durdle Promontory, rapid failure occurs by a variety of complex mechanisms, which vary depending upon the location of the modelled profile within the Promontory. A large number of rock mass blocks are displaced on the model meshes, and much change in form occurs.

The modelling exercise which emphasised characteristics of the Durdle Door sea-arch has indicated the nature of the failure and the changes to the landform. Corroboration for the UDEC final output comes from the field site where across the bays to the east and west of the Durdle Promontory, sea stacks are exposed at low water which have similar form to the model final output (Plate 5.2). It is plausible that the Portland Limestone outcrop at this part of the Isle of Purbeck is initially penetrated through sea arches which collapse to form successive sea-stacks. However, the modelling exercise has not replicated the stable strength of three-dimensional stress distributions in an arch. A three-dimensional form of the landform development can be considered by plotting co-ordinates from the two perpendicular meshes which were constructed for the western end of the Durdle Promontory. A variety of angles can be used to view the pairs of three-dimensional plots from the first and last UDEC cycles (Figures 6.61a to 6.63b). The crude diagrams illustrate the form change and it can be clearly seen that the landform is reduced in height but increased in width. In contrast to the other sites modelled from the Isle of Purbeck, the rock mass at Durdle Door does not retain its shape through the run and allometric change occurs.

For the failure events which were modelled at each site along the Isle of Purbeck coastline, all seemed to start rapidly, with a large displacement of blocks occurring, and then take a long part of the model run to settle (Table 6.7). All modelled cliffs, apart from those simulating Durdle Promontory, retreated parallel with landform shape being retained. It can also be suggested on the evidence of the modelling exercise that there is an increase in the rate of cliff failure and retreat from east to west along the Isle of Purbeck coast. A number of indicators add credence to this observation. The UDEC output shows that the failures at Durdle Door and Lulworth Cove involved a greater number of blocks than the failures at Winspit and Fossil Forest. The failing block velocity (Table 6.7) also seems to increase for the modelled sites at the western part of the Isle of Purbeck. Tentative observations of a variation in cliff retreat and activity can be considered by comparing model response from different sites at two key stages. If

step counts are used as indicators for the start of failure events in the different Isle of Purbeck sites, there is a clear trend along the coast (Table 6.8). Failure occurs first in the Durdle Promontory models and last in the Winspit models. If output is compared at approximately 100,000 cycles for each model, clear differences can be observed. At Winspit (Figure 6.38c), failure is restricted to a couple of columns of rock material. The cliff crest has retreated by a few metres, but the modelled mass has retained its relatively stable form. At Fossil Forest (Figure 6.43b) failure has occurred for a couple of thin sliding layers which are slowly creeping downslope. For the Lulworth Cove pincer model (Figure 6.47e), the model has stabilised, but there is a slight increase in the sliding rate and volume of material involved. A very large volume of material is involved for the full Lulworth Cove cliffs (Figures 6.33 and 6.50c), and the cliff crest has retreated by approximately 8 m. Finally, for the Durdle Promontory models (Figures 6.53d, 6.56d and 6.59d), a very large change in cliff form has occurred by 100,000 cycles due to rapid and complex failure mechanisms.

	Velocity Vectors	Block Displacement
Winspit quarried	9,700	11,200
Winspit Cliff	9,400	10,900
Fossil Forest	8,500	10,100
Lulworth Cove Pincer	7,700	9,700
Lulworth Cove Cliff	7,300	8,300
Eastern Durdle Promontory	9,100	11,000
Western Durdle Promontory	6,500	6,700
Durdle Door	7,000	8,200

Table 6.8:   The model cycle counts for the commencement of failure events at each site.

The count is noted for when the velocity vectors are initially oriented within a failure mechanism and it is noted for the stage at which block displacement first occurs.

The modelling exercise for each site along the Isle of Purbeck has identified a number of similar themes with major geomorphological implications. A particularly interesting possibility illustrated by using UDEC was the restabilisation of rock masses due to a change in morphometric form. At Winspit, a creep rotation of columns was observed which restabilised by sliding of blocks to the rear of the column (Figure 6.38d). The slid blocks formed a wedge. A kinematic analysis would suggest that the rotation of such columns would lead to greater instability, but note that partially toppled stable blocks exist in the field at Winspit (Plate 6.2). Creep toppling and stabilising failure of rock slopes was demonstrated in the theoretical exercise for a mass with two joint sets (Section 4.3.1). It is more likely that such behaviour occurs for rock masses with three joint sets, such as at Lulworth Cove, because it is possible for blocks to slide on a joint set at the rear of a toppling column.

All of the models of the Portland Limestone outcrop along the Isle of Purbeck coast exhibited fluctuations in activity (e.g. Figure 6.51). This was often because key blocks prevented the movement until the force build-up was sufficient. By the monitoring of a real failure event, it may be possible to calibrate model time. It has been possible to identify the creeping movement period of a failure mechanism during a model run, before blocks fall catastrophically (Table 6.7). The start of a second failure event at each of the sites modelled would be controlled by the marine erosion of the blocks. Although the exercise has demonstrated that the cliffs develop most rapidly at the Durdle Promontory and least rapidly at Winspit, a further complication the possibility of marine activity. A second failure event at Winspit occurs after the debris from previous events is eroded by the sea (Figure 6.40a). However, second failure events at Fossil Forest and Lulworth Cove will only occur when further sliding planes are exposed in the cliffs (Figure 6.45). Far more marine erosion would be needed at the two sites to initiate further failure events than at Winspit. Although it has not been possible to model the marine erosion of cliffs accurately, or to ascribe real time bases to the model output, such insights and relative comparisons between sites provided by the modelling do aid geomorphological understanding of the landforms.

The theoretical exercise indicated the importance of accurate discontinuity geometry and spacing characteristics, as well as information such as the cliff morphometric profiles and joint friction angle. However, the simulations maintained a

simple modelling approach as characteristics such as joint stiffness, model boundary conditions, joint pore water pressure and discontinuity variance could be identified as having little control. An explanation of the different failure mechanisms in Isle of Purbeck coastal cliff models could be based on the *b/h* ratios of failing blocks which were cut by critical joint sets and the dip of the failure planes. The possibility of restabilisation of rock masses could be realised by accounting for the change in *b/h* ratio as blocks were displaced. Also, the role of cliff height was evident, which has been identified in other theoretical studies (Hsu and Nelson, 1995; Jiang *et al.*, 1995). Stability in the modelled cliffs at Lulworth Cove and Fossil Forest was attained when sliding planes were not exposed in the free-face. The greater the height of the cliff, the likelihood of the exposure of such planes increases. Overall, the modelling completed to gain an understanding of the different cliff forms along the Isle of Purbeck coast benefited greatly from the background analysis.

6.5 Conclusion

It has been shown that the UDEC computer program can simulate geomorphological characteristics of cliff failure mechanisms and landform development in the Portland Limestone outcrop along the coastline of the Isle of Purbeck, Dorset. The discontinuity pattern in the outcrop has been isolated as a major control which can determine the nature of cliff failure and bedding steepens from east to west along the coastline as a consequence of the position of the Purbeck Monocline. At the same time, average block size decreases to the west and intact rock strength increases. By grouping field sites together, based upon the analysis of the geotechnical data, the four sites of Winspit, Fossil Forest, Lulworth Cove and Durdle Door were identified for the modelling exercise. A variety of failure mechanisms were evident in the models which represented characteristics of the four sites and explanation is related to joint geometrical control. Comparison between the model outputs emphasised that there is an increase in the rate of simulated cliff retreat from Winspit in the east to Durdle Door in the west. The importance of UDEC as a simulation tool is highlighted by a consideration of images preserved in the output which provided a valid comparison with the real field sites.

Further themes which became evident during the exercise included insights on the restabilisation of rock slopes once failure has changed the mass geometry. For all

sites, failure events were rapid initially, before slow creeping activity became dominant for a large part of the model runs as blocks settled. The plots of unbalanced forces indicated fluctuations in rock slope activity, and could well be associated with the occurrence of key-blocks in the model mesh. Displaced blocks could act as key-blocks, but also could initiate the stabilisation of rock slopes. At all sites, other than the Durdle Promontory, facets of parallel retreat were demonstrated and final rock forms were of the same shape as the initial masses. However, the most important conclusion is that differences in cliff development can be understood by comparing the behaviour of rock landforms between sites along the Isle of Purbeck coastline. Lulworth Cove and the Durdle Promontory, at the western end of the coast, have failure events involving large numbers of rock blocks, and retreat rapidly. At Winspit, failure occurs by the gradual removal of rock columns.

Chapter 7: The mechanisms of failure and behaviour of sandstone hard rock steep slopes on the Colorado Plateau, USA

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7.1 Data analysis of sandstone rock slope properties

Work was undertaken in the Canyonlands Region and at the Colorado National Monument analysing the joint geometry and properties of the cap-rock Kayenta Formation and the cliff morphometry to model the mechanisms of failure and behaviour of sandstone scarps on the Colorado Plateau. Both mesas studied are formed from horizontal layers of sandstone and vary in plan-form development so that headlands and embayments can be identified. Eleven sites were selected at Dead Horse Point (Figure 5.14) and fifteen sites were selected at the Colorado National Monument (Figure 5.17). The sample points reflect the varying extent of cliff development at each site, are geographically spread and include different cliff morphometric forms (Table 7.1; 7.2).

Location	Site number	Situation	Description
607900, 4261700	DH1H	Headland	NW/S facing site with low vertical cliff.
608700, 4261300	DH2E	Embayment	NW facing with short basal section.
607800, 4260700	DH3H	Headland	N/SW facing. Sharp headland point.
608100, 4259900	DH4E	Embayment	W facing. Lots of debris from active failure.
607700, 4259700	DH5H	Headland	SE/NW facing. Sharp point. Butte attached.
608800, 4258400	DH6H	Headland	SSW facing. DH Point. Broad.
608700, 4259000	DH7E	Embayment	NE/SW neck. 2 cliffs, high basal section.
609500, 4259600	DH8H	Headland	S/E/N broad headland. High vertical cliff.
609200, 4260700	DH9E	Embayment	E facing. High cliff, low angled base.
610100, 4261100	DH10E	Embayment	S/E small headland within large embayment.
610500, 4260300	DH11H	Headland	E/S/W broad headland. Butte attached.

Table 7.1:   Field site location and description at Dead Horse Point State Park. The grid reference used is the 1000 m Universal Transverse Mercator System.

Location	Site number	Situation	Description
696500, 4330900	CO12H	Headland	N/E facing site. Steep basal section.
696600, 4330700	CO34H	Headland	NE/SE facing. Short basal section.
696600, 4330400	CO56H	Headland	NE/S facing. Short basal section.
696200, 4330300	CO78E	Embayment	S facing. High vertical cliff.
695600, 4329800	CO9E	Embayment	SE facing. High vertical / short basal cliff.
695600, 4329400	CO10E	Embayment	N facing. Long basal section.
696500, 4329800	CO11H	Headland	NW/SE facing. Sharp point. Spire attached.
695900, 4329700	CO12E	Embayment	N. Long basal section/short cliff. Neck.
696200, 4329700	CO13E	Embayment	SE facing. Very short basal section. Neck.
696200, 4329300	CO15E	Embayment	NE facing. Short vertical cliff.
696800, 4329000	CO16H	Headland	N/SE facing. Broad. Short base.
696600, 4329400	CO17H	Headland	NW/SE facing. Sharp point. Butte attached.
697700, 4328400	CO18H	Headland	NW/E. Long vertical cliff. Spire attached.
695800, 4330600	CO19E	Embayment	NW facing. Very long, gentle basal section.
696100, 4330900	CO20E	Embayment	NW facing. Very long basal section

Table 7.2:   Field site location and description at the Colorado National Monument.
The grid reference used is the 1000 m Universal Transverse Mercator System.

7.1.1 Dead Horse Point State Park

An initial understanding of the differences in composite sandstone cliff retreat at Dead Horse Point State Park can be gained by examining the pattern of discontinuities as measured in the cap-rock Kayenta Formation at each of the field sites. By the contouring of pole positions on equal area stereographic projections, mean joint set characteristics at all eleven sites can be identified (Hoek and Bray, 1981; Priest, 1985) (Table 7.3). Stereoplots from the sites of DH4E, DH5H, DH7E and DH11H are presented (Figures 7.1 to 7.4) as well as stereoplots for all of the headland sites (DH1H, DH3H, DH5H, DH6H, DH8H, DH11H) and for all the embayment sites (DH2E, DH4E, DH7E, DH9E, DH10E) (Figures 7.5 and 7.6). If the representative data for the joint sets are considered (Table 7.3), a strong link can be seen between the sites at Dead Horse Point State Park. For all but one site, the strongest concentration of poles represents a close-to-vertical joint set which strikes at an average bearing of 124°. A second close-to-vertical joint set strikes at a bearing of 040° for the headland sites (Figure 7.5) and at

020° for the embayment sites (Figure 7.6). In all but one instance there are two joint sets. The two joint sets which were measured at sites at Dead Horse Point State Park can be compared with the nine vertical joint sets measured by Bergerat *et al.* (1992) for the whole of the Colorado Plateau. It is suggested that a joint set of 25-30° strike is related to a Basin and Range extensional event and that a 125-130° set is due to a second Laramide compressional event (Bergerat *et al.*, 1992). The dominance of the 120-130° joint sets in the stereoplots from Dead Horse Point could be because the joints were formed in the second of two periods of rock deformation and discontinuity genesis. The existing pattern of 20-40° joints may have become slightly displaced by the forces which controlled the formation of the 120-130° joint set.

Site	Set A	Set A Spacing	Set B	Set B Spacing	Set C	Mean Spacing	Mesh
DH1H	129°/89°SW	11.41 m	049°/89°SE	26.10 m		16.0 m	020°
DH2E	120°/89°SW	5.46 m	021°/82°W	8.54 m		6.8 m	102°
DH3H	120°/90°	7.33 m	038°/86°NW	16.00 m		11.1 m	100°
DH4E	125°/88°NE	2.18 m	011°/87°E			2.18 m	060°
DH5H	130°/90°	6.81 m	040°/87°SE	7.52 m		7.14 m	019°
DH6H	129°/89°SW		040°/85°SE				179°
DH7E	130°/90	1.99 m	022°/88°W			1.99 m	068°
DH8H	121°/89°SW	6.78 m	034°/90			6.78 m	078°
DH9E	120°/89°SW	3.10 m	018°/89°W	2.74 m		2.93 m	266°
DH10E	122°/88°SW	6.13 m	039°/89°NW	9.14 m		7.68 m	316°
DH11H	120°/89°NE		032°/88°W		149°/89°NE		010°

Table 7.3: Mean joint set characteristics from Dead Horse Point State Park. The last digit in the site name denotes whether the site has a headland (H) or embayment (E) plan position. Where the joint spacing readings are not available, average site data are used in analysis. The mesh orientation bearings given are taken at 90° to the trend of the cliff at each site.

At nine of the sites on the cliffs at Dead Horse Point State Park, spacings between individual discontinuities were recorded. It has already been demonstrated that the failure mechanisms of rock masses depends upon the geometrical pattern of

discontinuities which is largely controlled by joint spacing between the joint sets. Results of statistical analysis of the joint spacing data from each site at Dead Horse Point are presented in Table 7.3. The mean of all of the joint spacing readings from Dead Horse Point is 6.93 m and the median is 5.75 m. The data have a standard deviation of 5.44, skewness of 1.59 and kurtosis of 5.96, indicating a right-skewed distribution as has been observed elsewhere for joint data (Figure 7.7) (Mohajerani, 1989).

The joint spacing measurements from each site show lower values for the embayments (Table 7.3). Also, landslide deposits in the Kayenta Formation, Wingate Sandstone and Chinle Formation cliffs in the Canyonlands region are significantly longer and larger in area beneath embayments, suggesting that there is a greater degree of jointing (Butler and Nicholas, 1989). Difference of mean Student t-tests demonstrate that there is a significant difference between each adjacent headland and embayment site on the cliff plan at Dead Horse Point State Park (Table 7.4). For instance, at embayment site DH2E, the joint spacing data can be seen to be similar to the average for the total readings from Dead Horse Point State Park (Figure 7.8). However, when the readings from DH2E are compared graphically with readings taken at site DH3H, it is obvious that there are differences. The quantile-quantile graph plots the ordered values of joint spacing for one site against the ordered values for another site. If the data sets are of the same size and distribution, then data would plot along the line $y = x$, which is shown. As the data for total joint spacings for DH3H against DH2E plot below the line of equality, it can be concluded that joint spacings are greater for DH3H (Figure 7.9). If all the readings taken are combined for the embayment sites, and for the headland sites, it can be seen that the average values are greater at the headland sites. The mean for the headland sites is 9.43 m and the median is 7.50 m, whereas the mean for the embayment sites is 4.27 m, with a median of 3.15 m. The Student t-test gives a probability of very close to zero of obtaining sample results as extreme or more extreme than those obtained if the means were equal. The difference is confirmed by considering the quantile-quantile plot, with headland data plotted against embayment data (Figure 7.10).

	DH1H	DH2E	DH3H	DH4E	DH5H	DH7E	DH8H	DH9E
DH2E	H > E							
DH3H		H > E						
DH4E			H > E					
DH5H				H > E				
DH7E					H > E			
DH8H						H > E		
DH9E							H > E	
DH10H								H > E

Table 7.4:   Results of Student t-tests for difference of means of joint spacing measurements.

Eight tests are recorded. The '>' symbol is used to denote a result where the mean joint set spacing for a site is statistically greater than adjacent site. Where means are indicated to be different, the result is statistically significant at the 5% level.

There are many locations at Dead Horse Point State Park where the joint pattern is exposed through a large area. It can be seen that the joint pattern is highly persistent and that the joint sets are repeatable. Stereoplots from sites at Dead Horse Point State Park (Figures 7.1 to 7.4) show high concentrations of poles for each joint set, with very little dispersion. In the field, the joints were obvious at the sites chosen, and so the measurement of joint spacings was straightforward. Indeed, Dead Horse Point State Park proved to be a good location for the study of rock mass joint geometries.

Some explanation of scarp development at Dead Horse Point State Park can be given by linking the discontinuity pattern at sites with the cliff plan-form. By tracing the break in slope at the crest of the cliffs and plotting the joint geometry which cuts the horizontal plane for the field sites (Figure 7.11), interesting conclusions can be made. A plan-form analysis of the discontinuity pattern occurring in the Kayenta Formation cap-rock is possible as the joints measured in the field were close to vertical. The sequence of headlands and embayments which form in the scarp plan give a similar shape to coastal cliffs with embayments having retreated further than the headland locations (Nicholas and Dixon, 1986). Headland scarps may become detached from the main cliff

to become isolated islands or buttes, as at DH11H. The southernmost end of Dead Horse Point, at DH6H, is close to becoming isolated, with attachment being maintained at a 30 m wide neck at DH7E.

Three observations from the cliff plan diagram (Figure 7.11) may help to explain cliff development. The spacing between discontinuities is greater at headland sites, a result which has been observed elsewhere on the Colorado Plateau (Nicholas and Dixon, 1986). The importance of joint spacing as a control on rock mass failure mechanisms and overall stability has been demonstrated in Chapter 4. The narrower spacing of discontinuities in embayments suggests that the rock mass is not as strong, and therefore embayments have retreated further. However, it also appears from the cliff plan diagram that the issue of rock cliff development at Dead Horse Point State Park is more complex than merely being related to joint spacing differences. Second, as discussed above, it is clear from Figure 7.11 that at each site there is one joint set which consistently strikes at 125° to north. The second joint set strikes at 20° from north at the embayment sites and at 40° from north at the headland sites. Both geometric situations result in the same shape of block, with internal angles of 100° and 80°, but it is difficult to see why a rock mass cut by 120° and 40° joint sets should be more resistant. The variation in the 20°/40° joint set may result from its occurrence before a second deformation event. A third observation from the diagram is that the strike of the scarp face is consistent with the strike of one of the two joint sets at each site. This indicates that failure at most sites occurs along a controlling joint plane, with the cliff undergoing parallel retreat to subsequent joint planes.

Further differences between the field measurement sites at Dead Horse Point State Park can be observed if the cliff profile form is considered. At each site, the rock mass is composed of a vertical, cliff-forming section in the Kayenta Formation and Wingate Sandstone, and a tilting base in the Chinle Formation. Statistics have been gained at each site for the height of the vertical cliff face, height of the tilted base, the gradient of the basal section and the altitude of the upper and lower boundary of the scarp (Table 7.5). The height of the vertical cliff sections varies between 97 m and 207 m and the height of the lower, angled base varies between 146 m and 292 m. For all but one site, the height of the base is greater than the height of the upper section, and the ratio between the upper and lower height values varies between 0.44 and 1.06. The

average angle of the basal cliff unit is a relatively steep 40°, and the altitude of the sites at Dead Horse Point is close to 1,800 m. However, there are some differences between headland and embayment sites in profile form. The mean headland ratio is 0.66 and the mean embayment ratio is 0.79, whilst the mean headland angle is 39.7° and the mean embayment angle is 40.0°. But there is a relationship between ratio and angle of base, with angle = 49.9 − 14.1 ratio (r value = -0.51). Thus, the higher the proportion of the cliff base, the lower the angle that the base is inclined at. This observation could be related to the fact that the strength of a rock mass is reduced with increased height (Selby, 1993).

Site	Upper	Lower	Ratio	Angle	Top	Base
DH1H	97 m	176 m	0.55	48°	1755 m	1481 m
DH2E	116 m	146 m	0.79	40°	1774 m	1512 m
DH3H	125 m	220 m	0.57	41°	1783 m	1438 m
DH4E	140 m	146 m	0.96	47°	1798 m	1512 m
DH5H	116 m	219 m	0.53	45°	1798 m	1463 m
DH6H	128 m	268 m	0.48	38°	1810 m	1414 m
DH7E	128 m	292 m	0.44	46°	1810 m	1390 m
DH8H	207 m	195 m	1.06	32°	1816 m	1414 m
DH9E	168 m	170 m	0.99	34°	1801 m	1463 m
DH10E	165 m	219 m	0.75	33°	1798 m	1414 m
DH11H	165 m	219 m	0.75	34°	1798 m	1414 m

Table 7.5: Cliff profile statistics from Dead Horse Point State Park.
Values are given from each site for the height of the upper, vertical cliff section (upper), the height of the lower, angled cliff section (lower), the ratio between the two height values (ratio), the angle of inclination of the lower cliff section (angle), the altitude of the cliff crest (top) and the altitude of the bottom of the cliffs (base).

At ten sites in the Kayenta Formation cap-rock of the cliffs at Dead Horse Point State Park, 100 readings were taken on five stones with the Schmidt hammer and the rebound values noted. Although some difficulties are acknowledged when rebound

values are used to correlate with other rock strength indices (Allison, 1990; Kolati and Papadopoulus, 1993), Schmidt hammer results can be used in comparative studies. Seven of the sites for which data were collected are on headland plan-form features and three of the sites are in embayments. If the frequency distribution of the results from Dead Horse Point State Park is plotted along with a Gaussian or normal fitted curve, it can be seen that the data, which has a skewness of -0.19 and kurtosis of 2.65, are close to Gaussian in shape (Figure 7.12). The mean of the data set is 40.3, the standard deviation is 8.2 and the median is 40.

From the Schmidt hammer results taken at each site, it is clear that there are no statistical differences in the rebound value for intact rock between adjacent headland and embayment sites at Dead Horse Point State Park (Table 7.6). For instance, for the 100 Schmidt hammer readings taken at DH1H, the data were less than the average for all the data recorded at Dead Horse Point (Figure 7.13). When compared using a quantile-quantile plot with the rebound values for DH2E, the data plot below the line of equality, indicating that the embayment site values are greater (Figure 7.14).

	DH1H	DH2E	DH3H	DH4E	DH5H	DH6H	DH7E	DH8H	DH10H
DH2E	E > H								
DH3H		H > E							
DH4E			H > E						
DH5H				H = E					
DH6H									
DH7E					E > H	H > E			
DH8H							E > H		
DH10H								E > H	
DH11H									E > H

Table 7.6:   Results of Student t-test for difference of means of Schmidt hammer rebound values.

Nine tests are recorded. The '>' symbol is used to denote a result where the mean rebound value for a site is statistically greater than adjacent site. Where means are indicated to be different, the result is statistically significant at the 5% level.

Of the Student t-test comparisons made between headland sites and adjacent embayment sites (Table 7.6), it is apparent that five embayment sites have greater Schmidt hammer readings, and three headland sites have greater readings. If all of the readings taken at the embayment sites are combined, and the same for the headland sites, it can be seen that the average values are similar with each other and the overall data set. The mean of the headland sites is 39.3 and the mean for the embayments sites is 42.9, but the t-test statistic suggests that there is a significant difference in mean. The lower strength of intact rock at the headland sites could be due to the tectonic situation of the rock. The rock at the embayment sites occurs in a more confined position and tectonic hardening may occur. The fact that the embayment sites have developed far further than the headland sites, despite having stronger intact rock, indicates the role of joint spacing in the strength of rock masses. The difference in Schmidt hammer rebound values between headland and embayment sites is confirmed by considering the quantile-quantile plot (Figure 7.15). Overall, it can be said that the Schmidt hammer readings of rock strength show that the intact rock is slightly stronger at embayment locations at Dead Horse Point State Park.

Close by to the Dead Horse Point State Park, at six sites at Canyonlands National Park, 100 readings were taken over five rock surfaces with the Schmidt hammer. Four of the sites were at headlands along the cliff front, and two of the sites were in embayments. The mean of the total data set is 37.8 and if the frequency distribution of the data is considered, it can be seen that the sample is again close to normal (Figure 7.16). The skewness of the data set is 0.09 and the kurtosis is 2.82. If the total data for the four headland sites is combined, the mean is 38.6, but for the two embayment sites, it is 36.3. Again, the Student t-test does give a significant probability that the Schmidt hammer rebound value means are different, and this is confirmed by the quantile-quantile plot (Figure 7.17).

The use of the Schmidt hammer as a means of testing the strength of rock has been the source of debate (Allison, 1990; Kolati and Papadopoulus, 1993). Generally the debate has been restricted to the correlation of mean Schmidt hammer rebound values with other rock strength indices. But, questions of sampling arise when using the Schmidt hammer, as with other methods of rock strength testing. Schmidt hammer testing in this study followed guidelines set out by Day and Goudie (1977), and

recorded values from five different boulders in the Kayenta Formation at each site. By moving the hammer across the rock surface, problems of rock anisotropy could be reduced. However, there may be differences between stones due to variations within the Kayenta Formation or difficulties with the Schmidt hammer. It was felt during testing that anomalous values would occasionally occur, although, when large data sets are considered, the distribution is close to a normal fit (Figure 7.16).

If the rebound values from the five rock surfaces are considered individually from Canyonlands site CA1H, the median and interquartile range for the twenty readings taken from each surface are consistent with the overall range (Figure 7.18). There are a few outlying readings recorded on individual blocks, but the overall frequency distribution for the site is close to normal (Figure 7.19). However, if box and whisker plots of Schmidt hammer rebound values are considered for the five stones selected at site CA6H, it can be seen that the stones contribute to different parts of the overall spread of data (Figure 7.20). The section of the overall frequency distribution of high Schmidt rebound values at CA6H is composed entirely of rebound values for block 3 (Figure 7.21). If block 3 had not been sampled, the total data spread from the site could have been very different. The fact that the data from rock surface 3 is consistently greater than the other rebound values from site CA6H would suggest that the difference is not a consequence of problems associated with the Schmidt hammer which may occasionally produce an inaccurate result, but because block 3 is stronger.

In summary, the results from the Schmidt hammer do occasionally include some anomalous values, but if a large sample of data are recorded, statistical comparisons can be made. Difficulties with correlation with other rock strength indices could well be associated with the sampling problem from individual rock units. The Schmidt hammer accounts for anisotropy within individual rock blocks, and large differences in rock strength may occur between blocks within a geological unit. In comparison with other methods of strength testing, the Schmidt hammer is a cost-effective method of gaining an indication of rock strength, as the sampling of several rock cores within blocks and between different blocks would be expensive.

7.1.2 Colorado National Monument

The joint characteristics from fifteen sites in the Kayenta Formation at the Colorado National Monument, Colorado were measured to gain an initial understanding of differences in composite sandstone cliff retreat. By the contouring of pole positions on equal area stereographic projections, mean joint set characteristics for each site can be identified (Table 7.7). Stereoplots from the sites of CO19E, CO9E, CO56H and CO12E are presented (Figures 7.22 to 7.25).

Site	Set 1	Set 2	Set 3	Spacing (m)	Mesh
CO12H	049°/87°SE	110°/86°S		1.47	211°
CO34H	056°/84°S	093°/87°N	120°/88°E	2.41	249°
CO56H	080°/84°S	167°/82°E		1.60	275°
CO78Es	039°/83°S	079°/84°N		1.64	143°
CO9Es	021°/85°E	081°/83°N		1.74	291°
CO10En	011°/83°W	161°/82°E		1.48	164°
CO11H	081°/86°N	153°/81°NE	020°/81°E	2.46	235°
CO12En	059°/82°NW	079°/84°N			140°
CO13Es	050°/85°SE	081°/83°N		2.86	288°
CO15En	136°/81°NE	029°/80°SE			186°
CO16H	049°/80°SE	072°/83°N	120°/82°N	1.17	230°
CO17H	130°/84°NE	030°/86°SE		3.17	208°
CO18H	139°/82°NE	089°/87°N	010°/89°W	4.59	221°
CO19En	0°/90°	079°/84°N		1.46	105°
CO20En	079°/88°N	041°/82°N	010°/86°W	1.83	144°

Table 7.7:   Mean joint set characteristics from sites at the Colorado National Monument.

The site name indicates its plan-form position, with headlands (H), north facing embayment sites (En) and south facing embayment sites (Es). The Set 1 joint set is the most dominant contoured joint set on the stereographic projection for each site, and Set 3 the least. The mean joint spacing for all measured joints at the site is given and the bearing of the UDEC mesh perpendicular to the cliff face is given.

If the representative joint set data is considered from the Colorado National Monument (Table 7.7), much more complexity appears in the joint pattern than at Dead Horse Point State Park. As before, most of the joint sets are very close to vertical in the Kayenta Formation cap-rock. An attempt has been made to categorise and label the joint sets for each site (Table 7.8).

Site	Set A	Set B	Set C	Set D	Set E
CO12H		049° (1)	110° (2)		
CO34H	093° (2)	056° (1)	120° (3)		
CO56H	080° (1)				167° (2)
CO78Es	079° (2)	039° (1)			
CO9Es	081° (2)			021° (1)	
CO10En				011° (1)	161° (2)
CO11H	081° (1)			020° (3)	153° (2)
CO12En	079° (2)	059° (1)			
CO13Es	081° (2)	050° (1)			
CO15En		029° (2)	136° (1)		
CO16H	072° (2)	049° (1)	120° (3)		
CO17H		030° (2)	130° (1)		
CO18H	089° (2)		139° (1)	010° (3)	
CO19En	079° (2)			0° (1)	
CO20En	079° (1)	041° (2)		010° (3)	

Table 7.8:   Labelled joint sets for each of the Colorado National Monument site. The figures in parenthesis indicate the dominance of that joint set on each of the stereographic plots as listed in Table 7.7.

Joint Set A strikes at between 072° and 093° and is evident at 11 of 15 sites. However, it is the most dominant joint set at only 3 sites. Joint set B strikes at between 29° and 59°, is evident at 9 sites and is the dominant joint set at 6. Joint set C strikes at between 110° and 139° and is the dominant joint set at 3 of the 6 sites at which it occurs. Joint set D

strikes at between 0° and 21°, and, again, is dominant at 3 of the 6 sites at which it occurs. Joint set E strikes at between 153° and 167° and is the secondary joint set at 3 sites. The ranges for the joint set labels at the Colorado National Monument are too great for comparison to be made with the nine vertical joint sets identified for the whole Colorado Plateau (Bergerat *et al.*, 1992).

At 13 of the sites in the Kayenta Formation at the Colorado National Monument, spacings between the individual discontinuities were recorded. Given the nature of the joint pattern in the Kayenta Formation, it was difficult to measure the distance between successive joints from the same set. However, because joint spacing is so important, measurements were made by laying tapes along two transects. The distance between all joints intersecting a north-south and east-west transect were noted, and the data provide sufficient basis from which comparisons between sites can be made. The mean joint spacings from each site are presented in Table 7.7. The mean of all the joint spacing readings from the Colorado National Monument is 2.19 m and the median is 1.40 m. The data have a standard deviation of 2.19 m, skewness of 3.04 and a kurtosis of 19.2. The frequency distribution of joint data from the Colorado National Monument is not dissimilar to a log-normal distribution (Figure 7.26) (Mohajerani, 1989).

As with the joint spacing data at the Dead Horse Point State Park, it is interesting to compare the sites. Difference of mean Student t-tests demonstrate that there is a significant difference between the mean of the joint spacing data for five headland sites and the adjacent embayment site (Table 7.9). The other Student t-test comparisons between sites demonstrated similarity in the readings. The graph of distribution of joint spacing readings for CO12H demonstrates that there are slightly fewer readings above the median for all of the sites (Figure 7.27). When compared with results from CO19E, the adjacent embayment, the quantile-quantile plot shows the data close to the line of equality, confirming the impression that there is no great difference (Figure 7.28). Of all the quantile-quantile plot comparisons testing differences between adjacent sites, seven show that the headland spacings are greater and one plot shows embayment data which are greater.

	CO34H	CO56H	CO78E	CO9E	CO11H	CO13E	CO15E	CO17H	CO19E	CO20E
CO12H									H = E	H = E
CO34H			H > E							
CO56H				H = E						
CO78E		H = E								
CO9E	H = E									
CO10E					H > E					
CO11H						H = E				
CO13E								H = E		
CO15E					H > E					
CO17H							H > E			
CO18H										
CO19E		H > E								

Table 7.9: Results of Student t-test for difference of means of joint spacing measurements.

Twelve tests are recorded. The '>' symbol is used to denote a result where the mean joint set spacing for a site is statistically greater than adjacent site. Where means are indicated to be different, the result is statistically significant at the 5% level.

If all the joint spacing readings taken at the embayment sites are combined, and the same for the headland sites, it can be seen that the average values are similar with each other and the overall data set. The mean of the headland sites is 2.49 m, whereas the mean for the embayment sites is 1.80 m. However, the Student t-test statistic of 4.11 gives a probability of very close to zero that the means are different. On a quantile-quantile plot, with the headland readings on the y-axis, the joint spacings data from the embayments plot lower (Figure 7.29). From the statistical evidence given, it could be said that there is a conclusive difference in joint spacing readings taken at headlands or embayments at the Colorado National Monument.

In relative contrast to Dead Horse Point State Park, the joint data are not only more complex, but also more variable at each site. Several concentrations of poles represented less than 10% of the total of plotted poles and there was large spatial variability on some of the stereoplots (Figures 7.22 to 7.25). Observations from sites in the field suggested some causes of the variability. Some sites contained blocks at the

cliff edge which were actively failing by a toppling mechanism. The zone of creeping blocks may occur at some distance into the rock mass and joint planes which are measured may have been slightly displaced. More weathering of blocks in the Kayenta Formation was noted at the Colorado National Monument. The result was rounding of the corners and sides of the blocks. To measure the strike and dip of a partially rounded surface required some estimation to be made. It was difficult to identify individual discontinuities at a site within a joint set, so spacing measurements had to be made for the overall site along two transects. Thus, spacing measurements can be consistently compared between sites at the Colorado National Monument, but comparison is not possible with Dead Horse Point.

The plan-form of the Chinle Formation, Wingate Sandstone and Kayenta Formation composite scarps at the Colorado National Monument resembles a coastal cliff with headlands and embayments (Nicholas and Dixon, 1986). By tracing the 1730 m contour that corresponds to the break in slope at the upper crest of the cliffs and plotting the joint geometry which cuts the horizontal plane for the field sites, an insight is given to the scarp behaviour (Figure 7.30). Several buttes and spires are seen on the plan which were connected to the main scarp, for instance at CO17H. A neck appears to be forming between CO12E and CO13E, which could lead to the detachment of the CO11H headland. It is apparent that joint geometry and relation to plan-form are much more complex than at Dead Horse Point State Park (Figure 7.11). However, there are still observations to be noted. As has been noted before, there is a reduction with rock mass strength with a reduction in joint spacing which is related to the extent of development of embayments. As at Dead Horse Point State Park, the strike of the scarp face is consistent with the strike of one or more of the joint sets for each site. For instance, at CO19E, the angle of the north and west embayment face is consistent with two approximately north and west joint sets. At CO56H, the headland which is created at the angle between north-west and south-west trending faces is consistent with approximately north-west and south-west striking joint sets. But it is difficult to identify zones on the plan where certain joint sets are dominant. The complexity of joint geometry and relation to cliff form at the Colorado National Monument cannot be explained by statistical consideration alone.

Further differences between the field measurement sites at the Colorado National Monument can be observed if the cliff profile form is considered. Statistics have been gained at each site for the height of the vertical cliff face, height of the tilted base, the gradient of the basal section and the altitude of the upper and lower boundary of the scarp (Table 7.10). The height of the vertical cliff sections varies between 22 m and 134 m and the height of the lower, angled base varies between 19 m and 124 m. The ratio between the upper and lower height values varies between 0.18 and 7.11, with six sites having a ratio of less than one. The average angle of the basal cliff unit is 35° and the average altitude of the cap-rock for the sites is 1754 m.

Site	Upper (m)	Lower (m)	Ratio	Angle	Top (m)	Base (m)
CO12H	64	67	0.96	51°	1710	1579
CO34H	96	36	2.67	27°	1720	1588
CO56H	95	39	2.44	29°	1728	1594
CO78Es	91	49	1.86	31°	1746	1606
CO9Es	134	21	6.38	46°	1795	1640
CO10En	49	91	0.54	29°	1780	1640
CO11H	128	18	7.11	47°	1746	1600
CO12En	36	98	0.37	33°	1764	1630
CO13Es	113	19	5.95	33°	1758	1627
CO15En	39	79	0.49	30°	1764	1646
CO16H	113	28	4.04	42°	1777	1636
CO17H	97	19	5.11	37°	1752	1636
CO18H	134	30	4.47	36°	1813	1649
CO19En	28	115	0.24	26°	1728	1585
CO20En	22	124	0.18	33°	1722	1576

Table 7.10:   Cliff profile statistics from the Colorado National Monument.

There are differences in profile form related to plan location (Table 7.10). The mean headland ratio is 3.83 and the mean embayment ratio is 2.00; the mean headland angle is 38° and the mean embayment angle is 33°. Thus, the headland sites have larger

vertical cliff sections, with shorter, steeper bases. In contrast to the profiles from Dead Horse Point State Park, the lower the proportion of cliff base, the higher the angle that the base is inclined at. Increased differences are noted if the embayments are subdivided into those that face north and those that face south. The mean south-facing embayment ratio is 4.73 and the mean north-facing embayment ratio is 0.36, and the mean south-facing basal angle is 37° and the mean north-facing basal angle is 30°. This accounts for the occurrence of north-facing embayment composite scarp, which is composed of a gentle basal section for almost all its height.

At fourteen sites in the Kayenta Formation cap-rock of the cliffs at the Colorado National Monument, 100 rebound readings were taken on five surfaces with the Schmidt hammer. Eight of the sites for which data were collected are on headland planform features and six of the sites are in embayments. The frequency distribution of the results from the Colorado National Monument is again close to Gaussian or normal (Figure 7.31). The 1,400 rebound values have a skewness of 0.10 and kurtosis of 2.52, indicating a close to Gaussian sample. The mean of the data set of 34.9 is less than at Dead Horse Point, and the median is 35.

Examining the Schmidt hammer results taken at each site shows no statistical differences in rebound for intact rock between adjacent headland and embayment sites at the Colorado National Monument (Table 7.11). Perhaps there is less difference between strength values than at Dead Horse Point, because there is less difference in joint spacings between headland and embayment sites. For instance, for the 100 Schmidt hammer readings taken at CO12H, the data were less than the average for all the data recorded at the Colorado National Monument (Figure 7.32). When compared using a quantile-quantile plot with the rebound values for CO19E, the adjacent embayment site, the data plot below the line of equality, indicating that the embayment site values are typically greater (Figure 7.33). In the Student t-test comparisons made between headland sites and adjacent embayment sites (Table 7.11), seven embayment sites have greater Schmidt hammer readings, and five headland sites have greater readings. If the embayment sites are combined, and similarly the headland sites, the average values are similar with each other and the overall data set. The mean of the headland sites is 35.6 and the mean for the embayments sites is 34.6, but the t-test statistic suggests that there is a significant difference in means. However, a difference is

not apparent when considering the quantile-quantile plot (Figure 7.34). From the statistical evidence given, it can be said that there is not a conclusive difference in Schmidt hammer readings recorded at headland or embayment sites at the Colorado National Monument and intact rock strength will not be varied between sites for the modelling exercise.

	CO12H	CO34H	CO56H	CO78E	CO9E	CO10 E	CO11 H	CO13 E	CO15 E	CO17 H	CO19 E
CO34H					E>H						E>H
CO56H				E>H							
CO78E		E>H									
CO9E			E>H								
CO10E											
CO11H						H>E			H>E		
CO13E							H>E				
CO15E										H>E	
CO17H								H>E			
CO19E	E>H										
CO20E	E>H										

Table 7.11: Results of Student t-test for difference of means of Schmidt hammer rebound values.

Eleven tests are recorded. The '>' symbol is used to denote a result where the mean joint set spacing for a site is statistically greater than adjacent site. Where the means are indicated to be different, the result is statistically significant at the 5% level.

7.1.3 Rock strength data

One advantage of studying rock slopes on the Colorado Plateau is that large data sets of sandstone geotechnical properties are available. Large data sets for the Kayenta Formation cap-rock from the Colorado Plateau have been provided by P. Fisher, Consulting Engineering Geologist, North Carolina. One data set was analysed for the US Corps of Engineers (Table 7.12), which states that the Kayenta Formation has a bulk density of 2.34 g cm^{-3}, and a friction angle, as calculated from triaxial tests, of 34°. But there is no information regarding the location of the samples, the number of samples collected or the range of results. The second data set was provided by the US Bureau of Reclamation from tests taken in 1974 as part of the Colorado River Storage Project at

Rainbow Bridge National Monument. Three blocks of Kayenta Formation and two blocks of Navajo Sandstone were collected.

Property	Value
Bulk density	2.34 g cm^{-3}
Porosity	14%
Young's modulus	49.6 GPa
Unconfined compressive strength	22.8 MPa
Friction angle .	34°
Cohesion	6.00 MPa

Table 7.12:   Material property data for the Kayenta Formation (from US Corps of Engineers).

The use of secondary data sources for gaining parameters of intact block strength avoids many of the problems associated with the collection of accurate material property data (Chapter 2). Large variations in rock strength can occur between blocks and within rock blocks, and a large sample of rock cores need to be collected for laboratory testing. For the work undertaken on the Colorado Plateau as part of this study, it was not practical to collect the rock samples for several reasons. First, a large number of samples would be required which would involve expensive and time-consuming collection, transport and analysis. At the same time, the restrictions of the data accuracy and the limited control of intact rock properties on rock slope behaviour (Chapter 4) suggest that work spent on strength testing would not be worthwhile.

The US Bureau of Reclamation used the angle-envelope method of analysing shear tests, which measured the shear stress at three values of normal stress. Equations of Mohr's envelope were also calculated from triaxial stress relationships. Based on 44 tests, the mean bulk density was calculated to be 2.37 g cm^{-3}, with little variation in results (Table 7.13). The mean Young's Modulus from ten tests was 59.5 GPa, with relatively wide variation and the friction angle a very high 39.2°, based upon eight tests. The friction angle calculated from the triaxial tests was an even higher 48°. However, problems apparent with the data for the Kayenta Formation are related to problems

associated with the accurate representation of intact rock strength properties (Chapter 2). Based upon the results from other sandstones and the advice of P. Fisher, it is suspected that a friction angle of 34° is more realistic. In contrast, the values of density, Young's Modulus and cohesion are consistent between the two data sets.

Property	Mean value	No. of tests	Range
Bulk density	2.37 g cm^{-3}	44	2.33 to 2.43
Absorption	4.48%	44	3.70 to 5.11
Porosity	10.62%	44	8.99 to 11.91
Young's modulus	59.5 GPa	10	29.7 to 75.8
Poisson's ratio	0.059	10	0.023 to 0.085
Unconfined compressive strength	97.7 MPa	10	64.4 to 122.7
Friction angle	39.2°	8	32.5 to 44.8
Cohesion	6.7 MPa	8	3.3 to 10.8
Triaxial friction angle	48°	14	
Triaxial cohesion	7.24 MPa	14	

Table 7.13:   Material property data for the Kayenta Formation (from US Bureau of Reclamation).

If the mean Schmidt hammer rebound values from the three locations on the Colorado Plateau are correlated with unconfined compressive strength and Young's Modulus, it is possible to make comparisons of the geotechnical properties of the caprock in each park (Table 7.14). A bulk density value of 2.37 g cm^{-3} was used in correlation. It can be seen that the results are comparable to those of the US Bureau of Reclamation. The values of both the unconfined compressive strength and Young's modulus are slightly lower than those recorded from shear tests (Table 7.13), but they are within the range of results. This shows that the Schmidt hammer gives results consistent with other more complex methods of measuring rock strength.

	Mean Schmidt 'r' value	Unconfined compressive strength	Young's Modulus
Dead Horse Point State Park	40.3	70.9 MPa	34.4 GPa
Colorado National Monument	34.9	54.7 MPa	28.7 GPa
Canyonlands National Park	37.8	62.9 MPa	31.8 GPa

Table 7.14: Correlation of recorded Schmidt hammer rebound values with unconfined compressive strength and Young's Modulus using the equations of Deere (1966) and a bulk density of 2.37 g cm^{-3}.

Also analysed from the results of the US Bureau of Reclamation were data for the strength of the Navajo Sandstone at Rainbow Bridge National Monument. This massively jointed Jurassic aeolian sandstone occurs above the Kayenta Formation in the Canyonlands region and has geotechnical properties similar to the massively jointed Triassic aeolian Wingate Sandstone which is of relevance to this study (Fisher, pers. comm.). The rock has a lower density than the Kayenta Formation of 2.19 g cm^{-3}, and higher absorption and porosity proportions (Table 7.15). The Young's modulus, unconfined compressive strength and cohesion are all lower, and the friction angle is over 4° less than for the Kayenta Formation. The analysis provides useful information for the assignation of material properties for the modelling of cliffs which are partly composed of Wingate Sandstone.

Property	Mean value	No. of tests	Range
Bulk density	2.19 g cm^{-3}	14	2.15 to 2.21
Absorption	8.10%	14	7.65 to 8.83
Porosity	17.70%	14	16.91 to 18.96
Young's Modulus	34.2 GPa	6	27.0 to 42.1
Poisson's Ratio	0.12	6	0.08 to 0.18
Unconfined compressive strength	41.7 MPa	6	37.4 to 48.5
Friction angle	34.9°	9	31.7 to 39.5
Cohesion	2.4 MPa	9	1.8 to 3.8

Table 7.15: Navajo Sandstone material property data (US Bureau of Reclamation).

7.1.4 Differences in cliff form between locations studied on the Colorado Plateau

From the initial analysis of sites in the Canyonlands region of the Colorado Plateau, including Dead Horse Point State Park, and from the Colorado National Monument, differences in cliff form are apparent (Sections 7.1.1 and 7.1.2). At the Dead Horse Point State Park, differences between plan-form headlands and embayments can be accounted for by differences in joint geometry, whereas at the Colorado National Monument, differences are related to differences in profile. In all areas, differences in joint spacing in the Kayenta Formation cap-rock occur between headland sites and embayment sites. However, cliff form is a much more complex issue and many factors may account for differences (Table 7.16).

Although the geology of the cliffs in the Canyonlands region consists of the same Kayenta Formation, Wingate Sandstone and Chinle Formation combination as at the Colorado National Monument, the cliff height is far greater (Table 7.16). When the mean values for the upper vertical cliff sections and lower angled base are compared between locations, it can be seen that most of the height difference is accounted for by the height of the gently angled base. A possible explanation of the differences could be a thickening of deposits in the central part of the Colorado Plateau (Luttrell, 1987). If the mean ratio values are compared, it can be clearly seen that at the Colorado National Monument, a greater proportion of the cliffs are composed of the upper, vertical cliff sections. However, in the Canyonlands region, the composition of vertical and angled elements is consistent between the plan headland and embayment sites. At the Colorado National Monument, there is a large difference in profile form between headland and embayment sites and some north-facing embayment sites have an exceptionally low ratio. Also, the mean angle of the base for the embayment sites is lower than headland sites at the Colorado National Monument and is again consistent between plan locations in the Canyonlands region. However, if mean angles are compared (Table 7.16), it can be seen that the greatest angles occur at Dead Horse Point and the lowest angles occur at Canyonlands National Park.

	DH			CO			CA		
	Overall	Heads	Embays	Overall	Heads	Embays	Overall	Heads	Embays
Mean height	348 m	356 m	338 m	138 m	138 m	139 m	321 m	331 m	297 m
Mean upper	141 m	140 m	143 m	83 m	104 m	64 m	151 m	156 m	139 m
Mean lower	206 m	216 m	195 m	56 m	34 m	75 m	171 m	175 m	159 m
Mean ratio	0.72	0.66	0.79	2.85	3.83	2.00	0.94	0.93	0.96
Mean altitude	1795 m	1793 m	1796 m	1753 m	1749 m	1767 m	1868 m	1872 m	1858 m
Mean angle	39.8°	39.7°	40.0°	35.3°	38.4°	32.6°	33.6°	33.8°	33.0°
Mean spacing	7.0 m	10.3 m	4.3 m	2.1 m	2.4 m	1.8 m			
Mean Schmidt R	40.3	39.3	42.9	34.9	35.6	34.6	37.8	38.6	36.3
Av. an. high temp.							19°C		
Av. an. low temp.							4°C		
Precipitation				250 mm			233 mm		

Table 7.16: Comparative cliff form statistics from different sites on the Colorado Plateau.

Seven sites from Canyonlands National Park have been analysed for the purposes of comparison. The seven sites are situated on a different part of the Island in the Sky mesa from Dead Horse Point State Park.

Two large differences in characteristics of the Kayenta Formation between the Canyonlands region and the Colorado National Monument are demonstrated by looking at the statistics for the mean site spacing and the mean block strength as measured with the Schmidt hammer (Table 7.16). The mean joint spacing from sites at Dead Horse Point State Park is 7.0 m and the mean Schmidt rebound value is 40.3, whereas at the Colorado National Monument, the mean joint spacing is 2.1 m and the mean Schmidt rebound value is 34.9. This would suggest that the Kayenta Formation cap-rock is stronger at Dead Horse Point, although the cliffs are not necessarily more stable as the mean height at the Colorado National Monument is lower. In the Canyonlands region, high cliffs are formed with an even distribution of profile form, and a strong cap-rock where the joint geometry controls the variation in plan-form. At the Colorado National Monument, there are differences in cliff profile form depending upon plan location, and the lower cliffs have a complex joint geometry with narrower spacings.

Differences are difficult to explain when it is considered that the cliffs at the two locations on the Colorado Plateau are at a similar height and subjected to similar climatic influences (Table 7.16). In the Island of the Sky district of Canyonlands

National Park (Figure 5.11), the average annual precipitation is 233 mm and the Colorado National Monument falls on the 250 mm isohyet, although Grand Junction has a recorded average annual precipitation of 210 mm. However, Schmidt and Meitz (1996) plotted isolines representing the sum of present precipitation and estimated Wisconsin moisture gain and the Colorado National Monument is situated in a slightly wetter region. The associated increase of freeze-thaw activity over the Colorado National Monument may have led to the variability and complexity of joint geometries in the Kayenta Formation. It was also apparent in the field that more weathered blocks were seen at the Colorado National Monument. The cliff profile form which occurs at north-facing embayment sites at the Colorado National Monument may not have adjusted from the cliff form which occurred during the wetter and cooler Wisconsin period. Schmidt and Meitz (1996) identified Colorado Plateau cliff forms which have a gentle, stabilised base and a Wingate Sandstone cliff that is segmented into a vertical section and angled base. It is suggested that such forms occur in a contemporary environment at an altitude of 2,300 m and annual precipitation exceeding 400 mm.

7.1.5 Sites selected for modelling

After data analysis, similarities and differences can be identified between the field sites at Dead Horse Point State Park. Based on plan-form and discontinuity geometry, sites can be grouped as either headland or embayment features. There are no major differences in profile form and intact rock strength between sites. It was decided to select typical headland and embayment sites for modelling in order to examine differences in failure and retreat rates. Consideration was also given to the behaviour of cliffs where plan-form headlands on a mesa become detached from the main escarpment to form resistant buttes, which develop into spires. Thus, the neck site at Dead Horse Point State Park was modelled and links were attempted by modelling a butte, which is associated with site DH11H. The profile meshes which were constructed for modelling are drawn on the cliff plan (Figure 7.11).

Embayment site DH4E was chosen as being typical for the category. It has a vertical cliff section with a height of 140 m, a free face bearing of 164°, and mean joint spacing of 2.18 m. The intact blocks at site DH4E have a compressive strength of 79 MPa as determined by Schmidt hardness readings. The adjacent headland, DH5H, has a

wider joint spacing (and base-height ratio) in the Kayenta Formation caprock of 7.14 m and an overall cliff height of 408 m. The two clearly defined joint sets striking at 130° and 40° have spacings of 6.18 m and 7.52 m respectively. The neck at Dead Horse Point, embayment site DH7E, warrants specific examination because of its critical position on the plan. A distance of 27 m across the neck separates the two vertical cliff sections, and the site has a joint spacing of 1.99 m. The examined profile for site DH7E was oriented at 062° and the intact rock has a compressive strength of 89 MPa. If failure of the cliffs were to occur at DH7E to break the neck, the headland at DH6H to the south would become isolated as a butte. The butte which has become detached from the headland at DH11H has been identified for modelling, as it is a classic landform with cliffs as wide as they are high. However, it is impossible to gain data readings from the summit of the butte, so joint characteristics from the adjacent headland, DH11H, were assumed to be the most relevant for the modelling exercise. Here there were three joint sets occurring in the Kayenta caprock, striking at 120°, 149° and 032°. The cliffs have an overall height of 384 m, with a vertical section of 165 m.

Characteristic sites were identified for the modelling exercise, after the data analysis was completed for the fifteen field sites at the Colorado National Monument. Sites were selected for modelling a headland cliff profile, a south-facing embayment cliff profile, a north-facing embayment cliff profile and a cross-section of a neck feature. Headland site CO56H was chosen as being typical for the category. It has a vertical cliff section with a height of 95 m and a total cliff height of 134 m. The examined profile cuts the headland at a magnetic bearing of 275°, the mean joint spacing is 1.6 m and the intact blocks at the site have a compressive strength of 41 MPa as determined by correlation with Schmidt hammer harness readings. An adjacent embayment site on the mesa, CO9E, has a larger vertical cliff section of height 134 m and a short basal section with height 21 m. It is typical of south-facing embayment profiles at the Colorado National Monument which have a large vertical cliff section above a short and steep base. The mean joint spacing at site CO9E is 1.74 m, and the two joint sets which strike at 021° and 081° are consistent with the strike of the cliff face at this location. The intact rock blocks have a correlated compressive strength of 60 MPa.

In contrast to site CO9E, site CO19E, which is situated on the same part of the mesa at the Colorado National Monument, is a north-facing embayment site. The profile

at CO19E has a long, gentle basal section, and a short vertical cliff section. The examination of the two contrasting embayment features which are typically found at the Colorado National Monument is designed to provide an insight into the aspect control on cliff form. Site CO19E has a vertical cliff section with a height of 28 m and a basal section which is angled at 26° and has a height of 115 m. The two joint sets which strike at 000° and 079° are consistent with the strike of the embayment cliff faces, and the mean joint spacing is 1.46 m. The intact rock blocks have a correlated compressive strength of 60 MPa. The 135 m wide neck which is connected to headland site CO11H deserves specific examination because of its critical position on the cliff plan at the Colorado National Monument (Figure 7.30). There are two embayment sites which are situated on the Kayenta Formation at each side of the neck and are contrasting in profile form due to aspect. Site CO12E is a north-facing cliff with a vertical section with a height of 35 m and a gentle base with a height of 98 m. In contrast, site CO13E is a south-facing cliff with a vertical section with a height of 113 m and a base height of 19 m. Both have similar joint set geometries, one set striking between 050° and 059° and a second set striking between 079° and 081°. The profile mesh across the neck feature is oriented with a magnetic bearing of 315°.

7.1.6 UDEC input

The modelling methodology used to simulate geomorphological slope evolution on different parts of the Colorado Plateau used current cliff profiles as a starting point. The profiles are stable initially at a point when the mesh has consolidated, and models are based upon real site conditions. Relevant parameters for model input sub-divide into the groups of rock cliff morphology, discontinuity characteristics and intact rock block properties. Based on the study of background considerations (Chapter 4), importance is attached to model accuracy in parameters such as cliff dimensions, discontinuity geometry and joint friction angle. A level of simplicity is maintained and some factors, such as the weathering of blocks, which is difficult to replicate, have been assumed constant between sites. It is important to note that the models have been designed to replicate important characteristics which differentiate different sites on Chinle Formation, Wingate Sandstone and Kayenta Formation cliffs. Data collected for each

site were formatted as appropriate for model input (Table 7.17) and the actual input files are in Appendices 7.1 to 7.9.

	Units	DH4E	DH5H	DH7E	DH11H	CO56H	CO9E	CO19E	CO12/13E
Free face height	m	140	120	128	165	95	134	28	113/36
Toe slope height	m	220	219	280	219	39	21	115	19/98
Toe slope gradient	x°	47	45	46/44	34	29	46	26	33/33
UDEC mesh bearing	x°	074	033	082	011/101	275	291	105	315
Bedding: mesh dip	x°	0	0	0	0	0	0	0	0
Bedding: spacing	m	4	4	4	4	4	4	4	4
Joint set A: mesh dip	x°	-88	90	90	89/87	-68	-85	90	85
Joint set A: spacing	m	2.43	6.81	1.99	7.00	1.6	1.7	1.5	2.9
Joint set B: mesh dip	x°	-86°	81°	-87°	88/88	-82	-76	84	-82
Joint set B: spacing	m	1.97	7.52	2.31	7.00	1.6	1.7	1.5	2.9
Joint set C: mesh dip	x°				-85/-88				
Joint set C: spacing	m				7.00				
Kayenta bulk density	kg m^{-3}	2370	2370	2370	2370	2370	2370	2370	2370
Kayenta bulk modulus	GPa	23	23	23	23	23	23	23	23
Kayenta shear mod.	GPa	28	28	28	28	28	28	28	28
Kayenta friction angle	x°	34	34	34	34	34	34	34	34
Kayenta height	m	50	50	50	50	34	35	28	36
Kayenta cohesion	MPa	6.7	6.7	6.7	6.7	6.7	6.7	6.7	6.7
Wingate bulk density	kg m^{-3}	2190	2190	2190	2190	2190	2190	2190	2190
Wingate bulk mod.	GPa	15	15	15	15	15	15	15	15
Wingate shear mod.	GPa	15	15	15	15	15	15	15	15
Wingate friction angle	x°	29	29	29	29	29	29	29	29
Wingate height	m	90	70	78	115	61	99	80	77
Wingate cohesion	MPa	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
Joint normal stiffness	GPa m^{-1}	10	10	10	10	10	10	10	10
Joint shear stiffness	GPa m^{-1}	10	10	10	10	10	10	10	10

Table 7.17:   UDEC model input parameters.

Individual blocks were defined as rigid units for simplicity, because the cliffs modelled contain hard rock with failure occurring along the discontinuities, rather than as deformation of the intact material. UDEC model meshes were oriented perpendicular to the trend of each cliff face and the discontinuity strike, dip, and spacing data were converted by computer program into the appropriate orientation for the mesh (Table 7.17). Joints in the vertical, massive Wingate Sandstone were impossible to measure in the field because of the location of the central part of the cliffs. However, because of the regularity and wide spacing of these discontinuities, reasonable estimates were made by

enlarging and scaling photographs. For simplicity, the joint geometry in the central, Wingate Sandstone, was the same for each model, with spacing set to 15 m.

Because there were few differences between sites of the strength data of the Kayenta Formation and the strength properties have little control on the behaviour of rock slopes, US Bureau of Reclamation data provided by P. Fisher were used for all of the models. The height on the vertical cliff where the divide between the Wingate Sandstone and Kayenta Formation occurs was estimated from photographs and knowledge of the thickness of geological layers (Lohman 1974; 1981). In order to assess the control of the tightly jointed Chinle Formation base of the cliffs a variety of scenarios were run, simulating the Chinle Formation alone and with a rock mass overburden. In every situation, with various geotechnical properties, the Chinle Formation part of the cliffs was stable due to the gentle angle of the rock mass free face. Thus, to reduce model complexity and computing time, the Chinle Formation base to the cliffs was modelled as a fixed rock block base. It is important to emphasise that such a simplification can be justified by running a set of scenarios and that model accuracy is maintained as the important controls on the rock mass behaviour are included. The parameter sensitivity testing (Section 4.5.2) suggested that the control exerted on the UDEC models by the joint stiffness parameters is not great. Furthermore, it is very difficult to accurately measure joint stiffness properties and adequate data exists in the literature which were used in this study. It was not necessary to include water flow parameters in the simulations of the composite scarps, as the Navajo Formation acts as the major water transmitting unit (Zhu *et al.*, 1998) and the parameters have little control upon slope failure (Section 4.5.1).

Once the model mesh geometry and material properties were assigned, a gravitational acceleration was set at 9.81 m s^{-2} and compressive stresses were set to act vertically through the model based on weight of the overburden. A horizontal compressive stress gradient was set at the recommended half of the vertical value (Herget, 1988). However, by the time the model reaches its initial equilibrium stage, stresses are natural throughout the rock mass. The basal and vertical boundaries were fixed to nullify movement to allow the blocks to settle and consolidate. For each of the sites the model was run for 10,000 iterations for consolidation, indicated by the model activity output reaching equilibrium. At this point the vertical boundaries of the model

representing the free cliff face were released to allow for displacement of blocks in the cliff. Blocks which became detached from the cliff face were automatically deleted as they are of no consequence to the rate of cliff retreat. UDEC does not accurately model material in free fall and by that point the failure mechanism is known. Blocks which settle on the angled slopes of the Chinle Formation disintegrate quickly and little talus accumulates. Output from the UDEC modelling process was logged and saved every 5,000 cycles.

7.2 Results

Output from the models which were run to simulate characteristics of the sandstone cliffs at key field sites on the Colorado Plateau was plotted for important stages in the modelling process. A block plot was made for each site after the model mesh had consolidated, and the mesh was plotted at 100,000, 200,000 and 500,000 cycles for each model, in order to permit comparison. The whole of the cliff profile form which includes the Chinle Formation, Wingate Sandstone and Kayenta Formation is shown on the first plot at each site. The subsequent plots focus upon the upper part of the cliff profile of the vertical cliffs composed of Wingate Sandstone and Kayenta Formation. The velocity vectors are also included on the block plots in order for the failure mechanism to be clearly identified. The velocity vectors are scaled in units of m s^{-1}, but the model calculation time-scale is not related to real time (Section 3.2.2). Thus, the values for velocity are not related to the speed at which a block would actually fall away from the cliff face and the representation for disconnected, falling blocks is not accurate. However, it is interesting and possible to make relative comparisons between models for the velocity of failing blocks (Table 7.18). Therefore, in the description of model output and discussion which follows, it is possible to make assessment of the relative speed of failure of a particular slope. Where the velocity of a block exceeds 2.0 m s^{-1}, observations from the modelling process would suggest that a block or more is either falling freely, or failing catastrophically. At less than 2.0 m s^{-1}, blocks are creeping along discontinuity planes.

Model	Figure	Step count	Velocity
DH4E	7.35b	100,000	8.1
	7.35c	200,000	0.011
	7.35d	500,000	0.00013
DH5H	7.36b	100,000	-
NE/SW	7.36c	200,000	-
	7.36d	500,000	-
DH5H	7.37b	100,000	-
NW/SE	7.37c	200,000	-
	7.37d	500,000	-
DH7E	7.38b	100,000	4.2
	7.38c	200,000	6.2
	7.38d	300,000	0.0037
DH11Hb	7.40b	100,000	0.58
N/S	7.40c	200,000	0.48
	7.40d	500,000	1.2
DH11Hb	7.39b	100,000	0.8
E/W	7.39c	200,000	0.028
	7.39d	500,000	-
CO56H	7.41b	100,000	0.7
	7.41c	200,000	9.1
	7.41d	500,000	2.8
CO9E	7.42b	100,000	4.3
	7.42c	200,000	6.8
	7.42d	500,000	-
CO19E	7.43b	100,000	4.0
	7.43c	200,000	0.24
	7.43d	500,000	-
CO12E/	7.44b	100,000	0.15
CO13E	7.44c	200,000	0.13
	7.44d	500,000	-

Table 7.18:   The magnitude of the largest velocity vector, measured in m s^{-1}, in each UDEC block plot of modelled sites from the Colorado Plateau.

7.2.1 Dead Horse Point State Park

Four sites were chosen at Dead Horse Point State Park for simulation. At the embayment site, DH4E (Figure 7.35a), equilibrium is reached before 10,000 steps and the joint pattern in the Kayenta Formation is formed by two joint sets and a horizontal bedding set (Plate 5.17). After conversion, one joint set dips at 85° into the free face, and the other set is vertical. The velocity vectors in the second plot show the start of a catastrophic toppling failure (Table 7.18) (Figure 7.35b). The blocks which are in free-

fall close to the cliff face are soon to be deleted as UDEC is not able to accurately model detached blocks. The narrow spacing between the two joint sets gives a *b/h* ratio well below the value required for stability and the blocks fail rapidly. Initial development of the cliffs is rapid, but activity has subsided by the third and fourth plots (Figures 7.35c and 7.35d) (Table 7.18). Activity is concentrated further back in the cliff profile, and there is little movement of the blocks at the cliff edge. The rate of retreat of the modelled cliffs is very slow compared with the Isle of Purbeck, Dorset cliffs.

At the adjacent headland site, DH5H, the joint configuration in the Kayenta Formation leads to a much higher *b/h* ratio of blocks (Figure 7.36a) (Plate 5.16). When the free face is released after equilibrium, the initial movement vectors again indicate a toppling failure process. By 100,000 steps, it is apparent that there has been some displacement of blocks in the Kayenta Formation cap-rock. However, the rate of displacement has slowed considerably, and at this stage in the model run is practically zero (Table 7.18). But displacement vectors are still consistently oriented out of the cliff face, and tension cracks have developed in the top of the profile (Figure 7.36b). One such crack has occurred between blocks which are failing and blocks which are stable. Tension cracks have been observed in other field locations and are indicative of a toppling failure mechanism (Bovis and Evans, 1996; Ishida *et al.*, 1987). The tension crack also propagates into the massive Wingate Sandstone mass and again some small movement of the large blocks occurs under the stress exerted by caprock movement. After the early phase of movement the blocks restabilise and settle. By the third and fourth plots in the sequence the displacement vectors have a much smaller magnitude and are randomly oriented within the rock mass (Figures 7.36c and 7.36d). This pattern of vectors indicates that the rock mass has stabilised again. Stability occurred soon after 100,000 steps (Table 7.18). The model mesh for site DH5H cut into the centre of the headland at a bearing of 033°. A model was also run for a second mesh at 90° to this, across the headland (Figure 7.37a). A similar extent of block movement occurs with initial toppling failure leading to tension cracks in the profile, before stability (Figures 7.37b, 7.37c and 7.37d). Conclusions made about the relative rate of retreat for headland sites have to account for failure occurring in several directions. But the overall rate of retreat is less than that at the embayment site.

When the profile of the narrow neck at Dead Horse Point State Park, DH7E, is considered (Figure 7.38a), it can be seen that the UDEC modelled cross-section reflects real conditions at the site. The bedding in the Kayenta Formation is horizontal and there are two close-to-vertical joint sets. One of the two sets dips slightly towards the west, the left hand side of the plotted modelled profile. The spacing between the discontinuities is typical for an embayment site at Dead Horse Point. Once stress equilibrium was reached in the model, both sides were released to allow failure. By the second plot in the sequence, taken at 100,000 steps, there is a rapid rate of cliff retreat by a toppling failure mechanism (Figure 7.38b). The toppling failure on the eastern side of the neck is just changing from a creeping mechanism to a catastrophic failure as the column has rotated past a threshold angle (Table 7.18). On the western side of the neck, loose blocks are falling away from the cliff. At this point in the modelling process there has been less removal of material than had occurred at the embayment site, DH4E, although there is a much greater retreat than at the headland site, DH5H. It is clear in the third plot of the sequence that there is a much greater rate of retreat on the eastern side of the neck because of the geometric configuration of the joint sets (Figure 7.38c). The fact that one joint set dips very steeply towards the west probably increases the stability on the western side of the neck. There is, however, a slight movement in the large Wingate Sandstone block on the western side of the neck by 200,000 cycles. By the fourth plot, the few remaining blocks on the eastern side of the profile have stabilised, but there is still some slow activity on the western side (Figure 7.38d). The highest column in the plot might appear to be unstable, but stability is probably controlled by the blocks supporting the base (Table 7.18). The activity is much greater at the neck than for the headland sites and the model suggests that the headland at Dead Horse Point will soon become isolated as a butte.

To gain an understanding of the three-dimensional nature of a butte rock mass landform, two model meshes were constructed: one oriented approximately north to south (Figure 7.40a) and the second perpendicular to the first (Figure 7.39a) (Plate 7.1). The north-south profile reflects field conditions in that the butte is connected to the main cliff by a dormant neck at the base of the Wingate Sandstone (Plate 5.18). Discontinuity data were collected from the adjacent headland, DH11H, and assumed to be similar for the butte. Greater block displacement occurs on the narrower east-west

model and is well developed by 100,000 steps (Figure 7.39b). Initial failures occur on the western (left) side of the butte with toppling blocks in the Kayenta Formation even 'pulling' over a large Wingate Sandstone block. Perhaps it is a toppling event involving a large number of blocks that is required in the Kayenta Formation in order for modelled failure of Wingate Sandstone to occur. The third plot in the sequence shows large toppling failures on the eastern side (Figure 7.39c). By the last step in the sequence, the large block has been removed and there are few remaining blocks of the Kayenta Formation (Figure 7.39d). The stable profile (Table 7.18) is similar to the profiles of other, more disintegrated buttes on the Colorado Plateau, with a reduction in height and width.

At the same time there is a displacement of blocks in the north to south axis of the butte (Figure 7.40b). By comparing the number of failed blocks at 100,000 steps, it can be seen that the amount of cliff retreat is much greater for the east-west model, although the actual rates of movement involved are similar (Table 7.18). Also, it is interesting to note that the joint configuration leads to a greater extent of failure at the northern end of the butte close to the headland at DH11H. By the third and fourth plots, columns of Kayenta Formation at the northern end of the butte are toppling, but there is no movement at the southern end (Figures 7.40c and 7.40d). At the southern end of the butte it may be assumed that there was once a headland connected to the main mesa as at Dead Horse Point and that the neck of the headland occurred to the north of the current butte.

7.2.2 Colorado National Monument

As with the results from Dead Horse Point State Park, output from the UDEC modelling process of simulations from the Colorado National Monument was logged and saved every 5,000 cycles. The output plots are for the same stage of the modelling, with the whole of the model mesh being shown on the first plot and subsequent plots focusing on the Wingate Sandstone and Kayenta Formation upper part of the cliff profile. The first image is taken at 10,000 cycles, after the blocks in the model had consolidated, and further images are printed after 100,000, 200,000 and 500,000 steps.

The initial cliff profile for the headland site CO56H is plotted as the first in the sequence representing conditions at equilibrium (Figure 7.41a). The cliffs are lower than

at Dead Horse Point State Park, and the joint spacings narrower (Plate 5.22). Although the two joint sets dip at 84° and 82°, the converted dip angles on the mesh are 82° and 68° respectively, and both dip into the cliff face with the mesh striking into the headland (Table 7.17). By 100,000 cycles it is clear that a creeping toppling failure mechanism is operating (Figure 7.41b). The blocks, which are bounded by the horizontal bedding and close-to-vertical joint sets, have a *b/h* ratio that is sufficiently low for a toppling mechanism to occur. The activity is greater than for the simulation of the headland site from Dead Horse Point (Figure 7.36b) and is a consequence of the lower joint spacing and *b/h* ratios at the Colorado National Monument and the specific intersection between the joint geometry and headland orientation at site CO56H. The velocity vectors indicate that the rock mass failure event starts at 19,100 cycles and displacement starts at 23,300 cycles. The rate of failure of the cliffs is in contrast to the coastal cliffs at the Isle of Purbeck, Dorset, where large displacements have occurred in modelled rock cliffs by 100,000 steps.

At 200,000 steps in the modelled sequence of cliffs at site CO56H at the Colorado National Monument (Figure 7.41c), more rapid toppling failure is occurring, as indicated by the magnitude of the velocity vectors (Table 7.18). At this point, some of the blocks are undergoing catastrophic failure and are in free-fall. However, there is no movement in the vertical, cliff forming Wingate Sandstone as the force of the toppling blocks is not sufficient to rotate a Wingate block. By 500,000 steps (Figure 7.41d), the blocks in the Kayenta Formation have stabilised, although there is still motion within the debris at the base of the cliff. The edge of the free-face Wingate Sandstone block is exposed to the effects of weathering. Further activity and failure events would be related to the removal of weathered material. Such a modelled arrangement of blocks in the Kayenta Formation would possibly not occur in the real-world as weathering would initiate the consolidation of the pile.

The characteristics of the model mesh for the site CO9E are similar to that of headland CO56H. The vertical part of the cliff face is higher and there is a shorter toe-slope section (Figure 7.42a). As before, the intersection between the strike of the modelled cliff profile and the two close-to-vertical joint sets results in the two joints dipping into the modelled cliff face. The joint spacing is similar to site CO56H. This results in a similar failure mechanism occurring at 100,000 steps of the model sequence

(Figure 7.42b). A slightly greater amount of block failure has occurred at a greater rate at the embayment site, and some catastrophic failure has occurred (Table 7.18). The model for the headland site (Figure 7.41b) was showing an entirely creeping failure mechanism at 100,000 cycles. The velocity vectors indicated the commencement of toppling failure at 11,800 steps and block displacement was evident by 14,400 steps, nearly 10,000 steps before failure commenced in the model representing characteristics of the headland site. By 200,000 steps, failure has ceased in the modelled Kayenta Formation cap-rock in the simulation of characteristics of the CO9E site (Figure 7.42c). Some blocks are failing rapidly, but as the UDEC simulation software does not accurately model rock free-fall, these blocks are to be deleted. At 500,000 cycles, all activity in the model has ceased as indicated by the velocity vectors on the plot which are very small and randomly oriented (Table 7.18) (Figure 7.42d).

In contrast to the embayment site, CO9E, the site at CO19E is a north-facing embayment and a different profile form occurs (Figure 7.43a). The long basal section is inclined at 26° and is 115 m high. The upper part of the cliffs is composed entirely of the Kayenta Formation cap-rock and is 28 m high (Table 7.17). After conversion, the joints on the modelled profile section have a dip of 90° and 84° out of the cliff face in the Kayenta Formation. After the model has reached equilibrium and the cliff face freed to allow for movements, the velocity vectors indicate failure commencement after 11,800 steps. Block displacement is observed after 13,400 steps. Failure occurs by a combination of toppling of blocks bounded by the horizontal and vertical joint sets, and by sliding along the 84° joint set. Movement has occurred by 100,000 steps, when a few blocks are failing catastrophically (Figure 7.43b).

By 200,000 cycles, block movement in the model representing conditions from site CO19E is confined to the settling of failed Kayenta Formation blocks on the angled Wingate Sandstone slope (Figure 7.43c). There is no evidence of further activity in the Kayenta Formation rock mass. There are two factors which will control further activity in the Kayenta Formation. The initial failure of blocks occurred along a sliding plane created by the exposure of an individual joint exposed in the free-face of the set which dips at 84°. No further joints of this set are exposed in the cliff face. The control exerted by the exposure of sliding planes in the free face of a rock mass has been observed for models of field sites at the Isle of Purbeck. Once movement had begun for the failure of

blocks in the model of site CO19E, there was some overturning of rock columns. However, further toppling of the Kayenta Formation rock mass will not occur as the cliffs do not have sufficient height (Hsu and Nelson, 1995). Confirmation that the cliffs have stabilised is given in the final plot of the modelled sequence (Figure 7.43d), after 500,000 steps. Less activity has occurred in the north-facing embayment site than in the south-facing site, due to the profile form of the cliffs.

The cliff neck location in the cliff plan at the Colorado National Monument provides an interesting situation for investigation (Figure 7.30). The embayment site of CO13E occurs on the south side of the neck and the embayment site of CO12E occurs on the northern side of the neck. The south-facing cliff of the plan-form neck has a vertical cliff section of 110 m and an angled toe slope height of 19 m (Figure 7.44a). In contrast, the north-facing side of the neck has a vertical cliff section of 36 m and a gentle base with a height of 103 m (Plate 5.21). The width of the neck at the location of the modelled section is 135 m and the two joint sets dip at 82°S and 85°N on the profile. On the model mesh at equilibrium, the south-facing cliff is on the left side of the plot (Figure 7.44a). After the two cliff faces had been freed to allow for block displacement to occur, the velocity vectors indicated the start of failure after 12,500 steps on the south-facing cliff, and after 13,400 steps on the north-facing cliff. Block displacement was evident by 13,900 steps on the south-facing cliff and by 15,000 steps on the north-facing cliff. As has been noted in the previous models, activity is greater in the south-facing embayment sites, as is true for the two sites modelled across the neck.

By 100,000 steps, slow block sliding of one column of blocks has occurred along a joint dipping at 82° on the south side of the neck (Figure 7.44b). Further sliding will not occur on this side of the neck as no further joints are exposed in the cliffs. The movement of the blocks within the Kayenta Formation has been sufficient to cause movement of a large Wingate Sandstone block below. On the north side of the neck, the blocks are stable. At 200,000 cycles (Figure 7.44c), there is movement of a second column on the south side of the neck. There is still no movement on the north side. The neck model from the Colorado National Monument has largely stabilised by 500,000 cycles (Figure 7.44d). Comparison with the initial joint mesh (Figure 7.44a) shows some modification of the south-facing (left side) cliff, but the north side of the neck has remained stable.

7.3 Discussion

7.3.1 Comparisons between modelled sites at Dead Horse Point State Park

In comparing the output from the models at Dead Horse Point State Park, there are clear differences between headlands and embayments. Much activity took place at the embayment location due to the smaller joint spacings and there was further retreat into the rock mass. Attempts have been made to estimate the rate of retreat of cliffs in the Canyonlands region by cosmogenic nuclide dating (Nishiizumi *et al.*, 1993). The exposure ages of two sides of a large toppled block in the Wingate sandstone differ by 10,000 years. For the models presented here, the modelled time steps are unrelated to elapsed clock time. The approach here is to compare relative rates of retreat between models, but there is further potential in dating surfaces and calibrating with real time. At DH4E, there was some movement in the large cliff-forming blocks of Wingate Sandstone, but there was insufficient removal of Kayenta material above this for the top of the block to be exposed. However, it is confirmed that the motion of the toppling blocks above the cliff face did not force the larger cliff blocks to topple. The piles of Kayenta Formation blocks which remain at the top of the cliffs in the second and third plots would be unrealistic in the field as weathering would promote further toppling movement. At the scale of this study, weathering effects are assumed the same for all sites, but despite this, conclusions can be made about the relative rate of cliff retreat between sites.

At the adjacent headland site, DH5H, less activity was observed in the Kayenta Formation due to a higher *b/h* ratio. It is interesting that some rotation of the blocks occurs between 10,000 and 100,000 cycles and tension cracks form in the top of the slope. Despite the change in rock mass joint geometric conditions, the blocks have stabilised. The restabilisation of rock masses also occurred for the Isle of Purbeck models (Chapter 6) and would be difficult to observe using other geotechnical analysis techniques. In the case of the Isle of Purbeck, the changing joint geometrical situation which controls the restabilisation can be identified. For the model of DH5H, it is difficult to identify the cause of stability, as very little block displacement has occurred. Freeze-thaw weathering would possibly initiate further block displacement, but to

facilitate accurate comparison between modelled sites, weathering processes have not been included in this study.

The plots for the embayment and the headland sites at Dead Horse Point after 100,000 steps (Figures 7.35b and 7.36b) show a large difference in the rates of cliff retreat. The scale of the displacement vectors indicates much more activity at the embayment sites: material has been removed by up to 45 m into the profile. A tension crack occurs at 60 m into the top of the profile for the embayment sites, whereas the crack occurs at 40 m into the profile at the headland site. The neck site, DH7E, undergoes a much greater retreat than at the headland site, but there is slightly less removal of material than at the embayment site, DH4E. Although quantitative differences between the development of cliffs at headland and embayment sites cannot be defined, clear differences exist in rates of cliff retreat at particular points during modelling.

Most activity in the models at Dead Horse Point was observed in the east-west transect of the butte at DH11H (Figure 7.39b). The cliffs were unstable due to the form of the narrow butte. It is interesting that the southern tip of the butte on the north-south profile was stable as it was a remnant headland feature. Greater displacement at the northern end could be associated with the rock mass zone of weakness at the remnant neck. As the modelled butte had no variation of spacing in the caprock, the complex joint geometrical configuration is responsible for variations in rates of cliff retreat in this case. From the space / time methodology used to analyse the composite sandstone scarps at Dead Horse Point State Park, a model of slope form evolution may be proposed. Scarps at different stages of development were simulated and the links between forms compared. Headland sites are formed where the joint spacing is wide and cliff retreat rates are low. Where two embayment sites coalesce at a neck feature, the headlands become isolated as a butte. A butte feature is not as stable as a headland, and fails rapidly if narrow and well developed. More failure occurs on the sides of the remnant headland, leading towards the elongated plan-form of the butte. It is striking that greater failure occurs towards the remnant neck. Once the Kayenta Formation has been removed, the Wingate Sandstone blocks are rapidly weathered. The exercise provided little evidence for the toppling of Wingate Sandstone blocks. Only by using

the distinct element approach and constructing simple, representative landform models can such an insight into the processes of slope development be obtained.

7.3.2 Comparisons between modelled sites at the Colorado National Monument

The activity in the model for the headland site at the Colorado National Monument (Figure 7.41b) is greater than for the simulation of the headland site from Dead Horse Point, as a consequence of the lower joint spacing and b/h ratios and the specific intersection between the joint geometry and headland orientation. However, given that the cliffs at Dead Horse Point are higher, the difference in retreat rates is not as great as it would be if the cliffs were the same height. By rigorously examining parameters in conjunction, the UDEC simulation of cliffs can consider the controlling effects of different parameters. At the headland site, CO56H, it takes 13,300 cycles before movement is evident in the cliffs. Failures between periods of stability have previously been identified as the mode of cliff retreat on the Colorado Plateau (Koons, 1955). For the sites modelled at the Colorado National Monument there was little difference in the extent of failure between headland and embayment sites, due to the similarity in joint geometric conditions. However, despite slightly wider joint spacing, the UDEC output demonstrated that the embayment site CO9E retreated slightly faster (Table 7.18). Slightly more block failure occurred at the embayment site. Also, a greater extent of failure at the embayment site is suggested by an earlier start of failure. Clearly, jointed cliff development along mesas on the Colorado Plateau is more complex than just being entirely controlled by variations in joint spacing.

At the Colorado National Monument, differences in cliff form between modelled sites appear to be an important control affecting cliff retreat. The difference in profile form may be accounted for by differences in weathering rates during colder periods, although it would be expected that such a profile form would occur at higher altitudes (Schmidt and Meitz, 1996). Scarp aspect may affect the number of annual freeze-thaw cycles, rates of evaporation and the growth of salt crystals (Butler and Nicholas, 1989). The greatest retreat, as determined by the movement of blocks (Table 7.18), comparison between output and the examination of the start of failure events, was for the south-facing embayment cliffs. More activity occurred at the modelled headland site than for north-facing embayment sites, which are gentle, with a short vertical cliff section.

However, in the Canyonlands region, over 30% of mapped landslide deposits occur in the north-west octant, which is attributed to instability caused by high rates of evaporation and subsequent salt crystal growth (Butler and Nicholas, 1989). The modelled north-facing embayment cliff, CO19E, has a gentle basal section and is relatively stable, although the effects of weathering are not included in the simulation. The activity that does occur at such sites causes the cliffs to decline further along sliding planes. The control exerted by the exposure of sliding planes in the free face of a rock mass has been observed for models of field sites at the Isle of Purbeck (e.g. Figure 6.49d). Once movement started for the failure of blocks in the model of conditions from site CO19E, there was some overturning of rock columns. However, further toppling of the Kayenta Formation rock mass will not occur as the cliffs do not have sufficient height (Hsu and Nelson, 1995).

In the models which simulated sections of the cliffs at the Colorado National Monument, there was no rotation of the large cliff-forming Wingate Sandstone blocks. This is consistent with field observations at the Colorado National Monument, where there are examples of Wingate Sandstone blocks, exposed by the removal of the Kayenta Formation and rounded by weathering processes (Plate 5.20). It is also interesting how the blocks in the Kayenta Formation for the embayment site at 200,000 steps have restabilised (Figure 7.42c), broadly as in the theoretical exercise (Section 4.3.1). The joint geometry which has resulted after failure in the rock mass would, it might be expected, be more prone to failure. But a rotation of blocks leads to a change in the *b/h* ratio, as the former block height dimensions are converted to the block basal dimension. The appearance of the settled rocks in the Kayenta Formation could well be controlled by a few key blocks in the rock mass. The weathering by freeze-thaw on a key block at the base of the failed Kayenta Formation blocks could lead to further toppling failure.

Important observations have been made when modelling neck plan-form features. The limited amount of block displacement in the model of the neck feature at the Colorado National Monument (Figure 7.44b) contrasts with the movement at the narrower neck at Dead Horse Point (Figure 7.38b). Stability of the cliffs at the Colorado National Monument is promoted by the decreased proportion of vertical cliff sections in the profile. Differences in cliff development between locations at the Colorado National

Monument are related to differences in profile form between sites, as opposed to joint geometric differences between sites at Dead Horse Point. Given the large differences in joint spacing in the Kayenta Formation between Dead Horse Point State Park and the Colorado National Monument, it may have been expected that in the UDEC models for sites at the Colorado National Monument slopes would have retreated further. The UDEC output has indicated that cliff form is an important control and that greater failure occurs at Dead Horse Point, as the cliffs are higher. Joint spacing is not such an obvious control. This, once again, shows the advantage of taking a geomorphological approach using UDEC to combine cliff morphometric data with rock mass geotechnical data.

7.4 Conclusion

An initial understanding of the differences in development of composite scarps on the Colorado Plateau can be gained by examining the joint geometry and cliff morphometry at different sites. Mesas formed from the Kayenta Formation, Wingate Sandstone and the Chinle Formation are divided into plan-form headlands and embayments (Nicholas and Dixon, 1986). At many of the sites investigated in the Canyonlands Region and at the Colorado National Monument, the cliff face is consistent with the strike of a controlling joint set and joint spacing is wider at headlands. Specifically, at Dead Horse Point State Park, further geometrical differences occur between headland and embayment sites. At the Colorado National Monument, greater complexity occurs in the joint pattern.

An investigation into the profile form of cliffs on the Colorado Plateau also yielded intriguing results. The height of the cliffs is greater at Dead Horse Point, but a greater proportion of the upper, vertical cliff section occurs at the Colorado National Monument. Based on joint spacing and Schmidt hardness, it would be expected that the cliffs are stronger at Dead Horse Point. However, the modelling of cliff profiles demonstrated that the lower cliffs at the Colorado National Monument are more stable. At the Colorado National Monument, weathering of rock becomes important. Linking with rates of activity based on the dating of the cliffs (Nishiizumi *et al.*, 1993), it can be stated that the landforms were largely developed in past climates when there was a greater amount of freeze-thaw activity (Schmidt and Meitz, 1996). Differences in profile

form related to cliff aspect and weathered blocks of the Wingate Sandstone apparent in the field suggest that it is an important issue. Also, the modelling exercise suggests that weathering is important once the Kayenta Formation cap-rock has been removed as Wingate Sandstone blocks were observed to be stable.

In summary, UDEC simulations of characteristics of sites from the Colorado Plateau have led to increased understanding of the development of jointed rock cliffs. The UDEC methodology provides a further insight into the behaviour of cliffs by considering more than just differences in individual joint parameters such as mean joint spacing. By combining relevant rock mass information into computer simulations, specific failure planes and processes can be highlighted. For instance, the specific joint geometric interaction with cliff form in the north-south transect of the butte at Dead Horse Point (Figure 7.40c) demonstrated that failure occurs towards the dormant neck feature on the landform and that the former headland is stable. In the Canyonlands region of the Plateau, the joint geometric conditions in the Kayenta Formation cap-rock exert a major control on rock mass failure and retreat. Insight has been gained into the continuum of form, starting with a mesa with zones of weaker rock mass due to joint spacing developing headland and embayment plan-form features. The headlands become detached from the main mesa to form a butte by the coalescence of two embayments at a neck. Failure of the butte is concentrated towards the dormant neck, and the former headland location remains relatively stable. From the modelling of cliffs at the Colorado National Monument, this broad model of evolution also occurs, but there are differences in slope retreat depending upon the orientation of the scarp face. Thus, factors of cliff profile form as well as joint geometric conditions have been demonstrated as controls on cliff retreat on the Colorado Plateau.

Chapter 8: Conclusion

Chapter 8: Conclusion

This thesis has presented the results of a study of the failure mechanisms of jointed rock masses and the behaviour of steep slopes using the UDEC rock mass computer simulation code. Theoretical simulations have identified the limiting boundary conditions between different rock mass failure mechanisms. Particular attention has been paid to the Portland Limestone coastal cliffs of the Isle of Purbeck, central southern England and the Chinle Formation, Wingate Sandstone and Kayenta Formation composite scarps of the Colorado Plateau, south-western USA.

8.1 Original contribution to knowledge

This study includes the following work that makes an original contribution to knowledge.

The UDEC software has been introduced as a geomorphological research tool in the study of jointed rock cliff landforms. UDEC works by linking the forces acting upon individual blocks in a pre-defined mesh with displacement for successive model time-steps. The advantage of the code is that it can combine geological, geomorphological and rock geotechnical properties such as joint geometrical, cliff morphometric and intact rock strength data in a rigorous, scientific manner. Blocks can fail along discontinuities and there is no limit to the extent of displacement or the length of the model run. Examples throughout this thesis have demonstrated the use of UDEC and the increase in knowledge of rock slope understanding that can be gained from it.

The limiting boundary conditions between the failure mechanisms of toppling, sliding and toppling-and-sliding have been defined for a theoretically modelled limestone rock mass based on discontinuity geometry. The UDEC simulation software demonstrates that the boundary conditions for a rock mass that responds to the dynamic forces of the interaction of multiple blocks are very different from the boundary conditions for the failure of a single block that can be defined kinematically. The two main differences to be noted are that it is possible for rock masses which have horizontal bedding to fail by a toppling mechanism and that only under certain circumstances will failure occur by

toppling-and-sliding in conjunction. The definition of the limiting conditions can be used as a template for rock mass parameter sensitivity studies. The shape of the curve gives a background understanding for real-world rock slopes. Furthermore, examples of failure in the real-world rock slopes studied for this thesis confirm the theoretical results.

Comparisons between UDEC models designed to simulate geomorphological characteristics of cliff failure mechanisms and landform development in the Portland Limestone outcrop along the coastline of the Isle of Purbeck, Dorset, highlight that there is an increase in the rate of cliff retreat from east to west. The position of the Purbeck Monocline and the decrease of joint spacing in the outcrop to the west determine the mechanism of cliff failure at different sites along the coastline. Explanation can be related to a consideration of the joint geometrical control, as highlighted in the theoretical simulation study. At all of the four modelled sites, Winspit, Fossil Forest, Lulworth Cove and Durdle Door, failure events occur rapidly. All modelled cliffs, apart from the profiles simulating characteristics of Durdle Door, retreat parallel with landform shape being retained. The significance of UDEC as a simulation tool is highlighted by a consideration of output images which provide a valid comparison with observations from the field sites.

The UDEC modelling exercise using data from the Chinle Formation, Wingate Sandstone and Kayenta Formation scarps on the Colorado Plateau, south-western USA leads to an insight into the linkage of rock cliff form. Large mesas contain zones of weaker rock mass due to joint spacing in the cap-rock which erode more rapidly, leading to an embayed cliff plan-form. Resistant headlands become detached from the mesa to form buttes due to the coalescence of two embayments at a neck. The rapid failure of a butte proceeds with greater erosion towards the dormant neck feature. In the Canyonlands region of the Colorado Plateau, joint spacing and cliff height exert the major controls on cliff retreat. At the Colorado National Monument, which was wetter during the last glacial, cliff profile-form is also a control on the behaviour of cliffs. The UDEC modelling provides more insight into the behaviour of cliffs than would have been gained by consideration of joint geometry and cliff morphometry alone.

It is possible to identify links between the behaviour of cliffs from both the Isle of Purbeck and the Colorado Plateau and controlling rock mass parameters. At both locations, classic jointed rock slope landforms were studied using a space / time substitution methodology. Rock mass landforms at different stages of development can be identified in the field. At both locations, the major control on cliff behaviour is the joint set geometry and cliff morphometry. For the Colorado Plateau, variations in slope development can be explained by considering differences in the base-to-height ratio of blocks at different locations. For the Isle of Purbeck, variations in slope development can be explained by considering differences in the dip of bedding along the Portland Limestone outcrop.

The parameter sensitivity study into the influences of rock mass failure mechanisms using UDEC identified joint set geometry as exerting the greatest control. The exercise highlights the need to use accurate joint dip and spacing measurements in real-world rock slope simulations. Other strong influences of rock slope behaviour include the joint friction angle and joint persistence, although the latter is difficult to measure in the field and represent in models. Factors that do not have such a large control include other intact rock strength characteristics, the deformability of blocks in hard rock masses, statistical variation in the joint geometric parameters and the strength of joints. The consideration of all these factors makes an original contribution to the knowledge of rock slope behaviour and has proved to be a prerequisite for explanation of rock mass landforms modelled as part of this thesis.

Throughout the UDEC modelling, the activity of rock mass failures was observed to occur in distinct pulses. When studying different failure mechanisms, the pattern of pulses was noted to reflect the failure mode. For simulation runs, the pattern was often more complex. The observation of pulsed event sequences between stability and movement in landforms is difficult to attain in geomorphology. Analysis of the pulses from the UDEC output suggests that activity increases almost exponentially to high peaks. An explanation may be related to the occurrence of key blocks in a rock mass, which are stable in position, and so inhibit movement of surrounding blocks. Forces

from surrounding blocks eventually overcome the key block and slope activity commences.

Related to the observations about rock slope activity and the occurrence of key blocks in the mass is the mechanism of rock slope restabilisation. Restabilisation was observed for models from both Dorset and the Colorado Plateau. The actual process of stabilisation was common between all. Initially a column of rock would creep in rotation towards a toppling failure. The movement would create a void between the toppling column and the slope which further blocks slide into. The blocks that have slid into the void form a wedge and act as key blocks preventing further rotation. Observations of rock slope restabilisation are difficult to achieve using other rock mechanics analysis techniques. The toppling column, which would have been vertical originally, now rests at an inclined angle, and the new rock mass joint geometry would suggest that failure is more likely to occur.

There are many geomorphological situations where a hard jointed rock mass overlies a softer, argillaceous base (Brunsden *et al.*, 1996; Steger and Unterberger, 1990). UDEC was used to model a theoretical limestone rock mass using a discontinuum formulation overlying a clay basal unit simulated with plasticity formulations. For all combinations of joint geometry modelled in the cap-rock, clay extrudes and bulges from the base. If the model was allowed to run, the velocity vectors indicate processes of clay consolidation and the development of shear planes. Predictions such as these have never been simulated before in a geomorphological context. In turn, it was possible to consider the extent to which the clay base affected the boundary conditions between the failure mechanisms of the limestone rock mass. The UDEC simulations confirmed that the rock mass was more likely to fail when underlain by a soft base.

Work completed as part of this thesis demonstrates that UDEC model simulations can be designed to model classic, but complex, rock mass landforms such as a sea-arch and a butte. The two perpendicular mesh cross-sections used to simulate characteristics of the Durdle Door sea-arch in Dorset indicate that very rapid failure occurs. Initial failure is by the sliding of rock layers on the northern side of the arch and by collapse from the

roof of the arch. Two perpendicular profiles were also used to simulate characteristics of a butte on the Colorado Plateau. On the wider, north-south section, failure is concentrated towards the dormant neck feature to the north. On the narrower, east-west section, greater displacement occurs on the eastern side, as controlled by the joint geometry. Corroboration for the model results from both the butte and arch come from more developed landforms exhibited in the field, which are similar in form to the final model output.

A morphometric study of the cliff forms from sites on the Colorado Plateau demonstrates that relationships exist between different parts of cliff dimensions. When the cliff plan is considered in conjunction with joint geometry data, the orientation of the cliff face is often consistent with the strike of a joint set and there are geometrical differences between plan-form headland and embayment features. At the Colorado National Monument, relationships can be seen between headlands and embayments and profile form ratios.

8.2 Extension to previous studies

The work presented here has extended previous studies in the following respects.

Brunsden and Goudie (1981) note that Lulworth Cove and its neighbouring bays are probably the most frequently visited, poorly described and least understood of all the famous geological and geomorphological teaching sites on the British coastline. Allison (1986; 1989) has conducted thorough geomorphological studies into the erosion of the coast with particular reference to the Portland Limestone outcrop. This study has enhanced knowledge on the specific landform development differences evident along the coast and the relation to geological structure. Often the current situation at Lulworth Cove is considered as a less developed version of the bays surrounding Durdle Door. However, by modelling the joint geometries at the two locations, it can be seen that very different failure mechanisms and cliff behaviour are exhibited in the rampart Portland Limestone outcrop.

The work presented here has demonstrated that the UDEC rock mass computer simulation software has more extensive application than merely being an engineering rock mass stability analysis tool. An important part of justifying the geomorphological use of UDEC has been the theoretical consideration of input parameters. Some engineering parameter sensitivity study had previously been completed. Hsu and Nelson (1995) demonstrated the control of a cliff height parameter in soft rock mass stability. In the explanation from the models of both the Portland Limestone outcrop, Dorset and the Colorado Plateau, the significance of cliff height has been noted. The form and joint geometry statistics from Dead Horse Point State Park indicate that the cliffs are more stable than at the Colorado National Monument. However, the UDEC models from both locations indicated greater failure at Dead Horse Point, showing that cliff height is an important rock mass behavioural control.

There has been much debate concerning the appropriate technique for gaining intact rock block strength data (Amadei, 1996; Brown, 1981; Crïstescu, 1989; Litwiniszyn, 1989). The results of the shear tests taken on the Kayenta Formation and Navajo Sandstone demonstrate some inconsistencies. As the deformation moduli are most relevant to understanding the geomorphological response of the material, sonic wave propagation methods are perhaps the best means to gain representative properties (Allison, 1988; 1991; Davis and Salvudurai, 1996). The rock strength data used for input for the models for the Portland Limestone outcrop of Dorset were consistent. However, where it is not practical to use sonic wave testing equipment in the field, geomorphologists often use the Schmidt hammer. There has also been much debate concerning the accuracy of the Schmidt hammer (Allison, 1991; Campbell, 1991; Day and Goudie, 1977; McCarroll, 1987). However, this study has demonstrated that where it is possible to take large samples, relatively small differences in mean rebound values can be statistically significant. By taking many readings both upon and between blocks, problems of rock anisotropy are overcome. While there are still problems with many methods in gaining accurate and representative intact rock strength indices, it is recommended that use should be made of the cheap and portable Schmidt hammer.

DeFreitas and Watters (1973) presented the results of a study of the kinematic failure of a single block resting upon an inclined plane. The UDEC simulation of a block on an inclined plane has confirmed the results of that study. In turn, the limiting boundary conditions for the kinematic failure of a single block has acted as a source to verify the UDEC simulation code.

Nicholas and Dixon (1986) suggested that the spacing between joints in the cap-rock of Colorado Plateau escarpments is the dominant control of scarp form and that rock strength plays a minimal role. The theoretical sensitivity study conducted as part of this thesis has confirmed the control of joint spacing, and sets of joint spacing data collected from the Kayenta Formation cap-rock have all statistically established that the joint spacing is greater at plan-form headland sites.

8.3 Recommendations for further research

It is possible to make recommendations for further research.

It has been demonstrated in this thesis how the UDEC rock mass computer simulation software has a great potential for geomorphological landform studies. The most obvious direction which further research can take is in establishing a temporal base to the model output. The UDEC model run time-steps are not related to real-time, and this thesis has considered relative rates of retreat by comparing output from different models. The exposure dating techniques that are being developed, such as the dating of cosmogenic isotopes (Nishiizumi *et al.*, 1993), may provide a means of constraining model output. At present, such techniques are complex and expensive. Problems that may be linked with the dating of output include the timing of failure events from cliff faces. One possible solution is to isolate failure of model material by a purely creeping motion, such as a slowly toppling block, relating the output to real-world creeping events. However, once a modelled rock mass has stabilised, the initiation of a further event is controlled by erosion and weathering factors, such as freeze-thaw processes or sea erosion. The dating of rock slopes from different sites may provide an indication in the differences in weathering rates between sites. Such information for the Colorado Plateau field sites could provide a further contribution to understanding.

Linked with the development of a temporal constraint to the model output is further research into the marine erosion at the base of sea cliffs. Modelled blocks that have fallen at the base of the cliff can prevent further movement and act to stabilise the cliffs. Fallen blocks in the real-world may be removed by the sea and the sea may also act to undercut the rock mass at the base of a cliff and expose further failure planes. In this study, the effects of marine erosion have been assumed to be identical for all the Isle of Purbeck site models. The compromise is satisfactory when considering relative differences in rates and mechanisms of cliff retreat. It would be interesting to understand the control on coastal cliff form exerted by the sea, but much of the calculation of sea pressure is complex and experimental (Allsop and Bray, 1994; Allsop et al., 1996; Komar, 1998). The key influences on the pressure are wave height, period, water depth at the cliff, average sea bed slope, local wave length, and breaking wave height. Because of the limitations imposed by the lack of understanding of wave pressures, the modelling exercise completed for this study has been confined to the simulation and comparison between sites of one failure event. Further failure and landform change, after the cliffs have stabilised, would be controlled by the time lapse before undercutting, or other processes, exposes a failure plane. At present, it is difficult for the methodology to include a consistent routine for the determination of further failure and, at the same time, make comparison between the Isle of Purbeck sites.

On the Colorado Plateau, it has been assumed that weathering occurs evenly between sites modelled. But, at sites at the Colorado National Monument, aspect appears to be linked to the form of cliffs. Much of the preceding development of cliff form will have occurred in past climates when freeze-thaw weathering processes may have affected some locations more than others (Ahnert, 1960). Also, the development of the main cliff-forming Wingate Sandstone is affected by weathering processes once the cap-rock Kayenta Formation has been removed to expose the massive unit below. Further work could investigate the weathering rate of exposed Wingate Sandstone blocks and incorporate the removal of material within simulations. However, it is emphasised that the models have been designed in this study to represent important conditions at field

sites, and that an increased understanding of rock mass behaviour on the Colorado Plateau has been achieved by simulating controlling parameters.

The theoretical study which considered the failure mechanisms of a rock mass above a soft base demonstrated that UDEC has the potential to model more complex geomorphological situations. Brunsden *et al.* (1996) suggested an idea for the cliff behaviour on the Isle of Portland, Dorset. The behaviour of Portland Limestone that forms the cap-rock of the Isle of Portland cliffs is well understood as a result of this thesis and would provide a useful link. Further data would need to be collected for the morphometric shape of the cliffs of the Isle and the geotechnical properties of the Kimmeridge Clay for a model profile to be designed.

Other environments where jointed rock slopes occur include formerly glaciated environments. The UDEC software could model the unloading of a rock mass below a glacier, simulating deglaciation. Potential links exist with studies completed on the morphometry of formerly glaciated landforms (Evans and Cox, 1995).

The modelling of complex rock mass landforms such as a Colorado Plateau butte and the Durdle Door sea-arch has been achieved by the consideration of two, perpendicular profile sections. Stability behaviour of the landforms may be maintained by the three-dimensional stress distribution within the rock mass. Although the two-dimensional models provide a useful insight into the failure mechanisms, a further understanding of the development of such landforms could be gained by using a three-dimensional simulation code. However, accurate geotechnical input data would be necessary. A consequence of modelling rock masses is that the problem is data-limited and that some assumptions need to be made. By using a three-dimensional matrix, assumptions and inaccuracies would increase by an order of magnitude.

This thesis is part of a sequence of ongoing research. Its contribution can be assessed both in terms of the details presented for the study areas and also in the conclusions which have much wider general application. The results provide a clear guide to where additional work is required.

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