PILLAR STABILITY AND LARGE-SCALE COLLAPSE OF ABANDONED ROOM AND PILLAR LIMESTONE MINES IN SOUTH-LIMBURG, THE NETHERLANDS

PILAAR STABILITEIT EN GROOTSCHALIGE INSTORTING VAN VERLATEN "ROOM AND PILLAR" KALKSTEENMIJNEN IN ZUID-LIMBURG

PROEFSCHRIFT

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Printed in The Netherlands
Berg in duisternis gehuld,
Van kracht van eeuwen en zee vervuld,
Die op ons wacht met oeroud geduld.
Zeg eens hoe lang je zo wachten zult.

Bergcanon (opschrift in de Jezuïetenberg)
CONTENTS

1 INTRODUCTION

PART I : OUTLINE OF THE MINE CHARACTERISTICS AND METHODS AVAILABLE OF PILLAR STABILITY ASSESSMENT

2 GENERAL DESCRIPTION OF THE MINES AND THEIR GEOLOGY 6
   2.1 Geological setting 6
   2.2 The Maastrichtian chalk of South-Limburg 9
      2.2.1 Lithostratigraphic subdivision 9
      2.2.2 The Formation of Maastricht 10
      2.2.3 The "Maastricht facies" limestone 11
      2.2.4 Microstructure 14
   2.3 Characteristics of the room and pillar mines 15
      2.3.1 Mined levels 17
      2.3.2 Geological structures affecting exploitation 17
         2.3.2.1 Joints 17
         2.3.2.2 Earthpipes 19
         2.3.2.3 Flints 19
         2.3.2.4 Faults 19
      2.3.3 Dimension and shape of pillars and galleries 19
      2.3.4 Overburden composition 22
      2.3.5 Climate and weathering 22
   2.4 Mining history 22
      2.4.1 Flint mines 22
      2.4.2 Origin of room and pillar mines 22
      2.4.3 Mining techniques 23
   2.5 Use of the mines 24
      2.5.1 Non-touristic purposes 24
      2.5.2 Touristic purposes 25
      2.5.3 Access to the mines 26

3 OUTLINE OF STABILITY PROBLEMS OF THE MINES 27
   3.1 Introduction 27
   3.2 Local and large-scale instability 28
   3.3 Types of local instability 30
      3.3.1 Instability of pillars 30
      3.3.2 Instability of the roof 30
3.3.2.1 Observations in the mines 30
3.3.2.2 Consequences of roof instability 31
3.3.2.3 Assessment of roof stability 32
3.3.3 Earth inflow from organ pipes 35
3.3.4 Impact on large-scale stability 36
3.4 Individual pillar instability, large-scale pillar instability, general mine instability and collapse 36
3.5 Accounts of large-scale collapses 38
3.5.1 The Fallenberg collapse of 1705 38
3.5.2 The St.Pietersberg collapses of 1794 and 1809 39
3.5.3 The Gemeentegrot collapse of 1845 39
3.5.4 The Gemeentegrot collapse of 1886 42
3.5.5 The Fallenberg collapse of 1920 42
3.5.6 The Muizenberg collapse of 1926 43
3.5.7 The Roosburg collapse of 1958 44
3.5.8 The Heidegroeve collapse of 1988 46
3.5.8.1 Subsidence features at the surface 46
3.5.8.2 Collapse features underground 51
3.5.8.3 Evidence of long-term deterioration 52
3.6 Hazards and general characteristics of large-scale collapses 53

4 REVIEW OF MODERN CONCEPTS OF PILLAR AND MINE STABILITY 55
4.1 Introduction 55
4.2 Individual pillar stability 55
4.2.1 The concept of the safety factor 55
4.2.2 Pillar strength 56
4.2.2.1 General expression of pillar strength 56
4.2.2.2 Background of the size effect 57
4.2.2.3 Background of the shape effect 58
4.2.2.4 Empirical size and shape formulae 59
4.2.2.5 Adjustments of shape formulae for irregular pillar outlines 61
4.2.2.6 The squat-pillar shape formula for pillars of large width/height ratio’s 63
4.2.2.7 The influence of discontinuities on pillar strength 64
4.2.2.8 Determination of UCS values 65
4.2.2.9 Time-dependent decrease of pillar strength 65
4.2.2.10 Residual strength of failed pillars 66
4.2.3 Pillar stress 67
4.2.3.1 The tributary area method 67
4.2.3.2 Complications in determining overburden stress 67
4.2.3.3 Pillar stiffness and local mine stiffness 69
4.2.3.4 Actual stress distribution over individual pillars 72
4.2.4 Visual assessment of pillar stability 73
4.3 Large-scale pillar stability 73
4.3.1 The concept of yielding pillars and a pressure arch 74
4.3.2 Models of large-scale collapses 75
4.3.3 Criterion of general mine instability in terms of post-peak pillar stiffness and mine stiffness 76
4.3.3.1 Introduction 76
4.3.3.2 The application of local mine stiffness coefficients $k_i$ 78
4.3.3.3 The concept of mine structural stiffness with coefficients $k_{ij}$ 78
4.3.3.4 Critical stiffness 80
4.3.3.5 Experimentally determined post-peak pillar stiffness 80
4.3.3.6 An example of numerically and analytically determined critical mine stiffness 80

4.4 Conclusions: approach to the research 82

PART II: LABORATORY EXPERIMENTS

5 GEOTECHNICAL PROPERTIES OF THE CALCARENITE 86
5.1 Introduction 86
5.2 Experimental procedures 87
  5.2.1 Sampling and sample preparation 87
  5.2.2 Testing 89
5.3 Deformation behaviour of the calcarenite under uniaxial compression 91
  5.3.1 General characteristics 91
    5.3.1.1 Stress-strain behaviour 91
    5.3.1.2 Characteristics of the measured strain 93
    5.3.1.3 Evidence of deformation mechanisms 93
  5.3.2 Class II post-peak behaviour 95
5.4 Influence of moisture content on UCS and deformation moduli 97
  5.4.1 Introduction 97
  5.4.2 Tests and results for the calcarenite 97
  5.4.3 Discussion 100
5.5 Transverse anisotropy 101
  5.5.1 Introduction 101
  5.5.2 Tests and results for the calcarenite 105
5.6 Tensile strength and triaxial strength 106
  5.6.1 Tests and results 106
  5.6.2 Derivation of parameters for Hock-Brown and Mohr-Coulomb failure criterions 109

6 STRENGTH, STIFFNESS AND SAFETY FACTORS OF CALCARENITE PILLARS 111
6.1 Introduction 111
6.2 Experimental and actual loading path 112
6.3 Experimental configuration 113
6.4 Experimental procedure 115
  6.4.1 Sampling and sample preparation 115
  6.4.2 Testing 115
  6.4.3 Lateral expansion of the specimen ends 115
6.4.4 Tilting of the machine platens
6.5 General features of compressive strength tests on calcarenite prisms
   6.5.1 Three phases regarding the stress-strain relationship and observations on macroscopic deformation behaviour during the test
   6.5.2 Additional observations on macroscopic deformation structures after the test
      6.5.2.1 W/H ratio's of one and more
      6.5.2.2 W/H ratio's of less than one
   6.5.3 Rotation of shear planes during infinitesimal axial strain
6.6 The size effect on shear planes
6.7 The influence of W/H on the strength of square-based prisms
   of W/H between 0.33 and 4
      6.7.1 Deformation behaviour and stress-strain diagrams
      6.7.2 The shape function for peak strength
      6.7.3 The shape function for residual strength
6.8 The influence of length/width ratio on the shape effect
6.9 L-shaped prisms and alternative shape formulae
6.10 Shape functions for prisms of W/H ratio's exceeding four
      6.10.1 Introduction
      6.10.2 Peak strength
      6.10.3 Residual strength
      6.10.4 Discussion
6.11 An analytical model of residual strength
6.12 Post-peak modulus
      6.12.1 Results of laboratory experiments
      6.12.2 The influence of shear fracture geometry on post-peak behaviour
6.13 Numerical experiments to assess the effect of end constraints on stress distribution and displacement pattern
      6.13.1 Introduction
      6.13.2 Outline of numerical experiments
      6.13.3 Results
         6.13.3.1 Horizontal displacements and interface slip
         6.13.3.2 Stress distributions
         6.13.3.3 Hoek-Brown safety factors
         6.13.4 Discussion and conclusions
6.14 Application of results in the mines
      6.14.1 Pillar classification
         6.14.1.1 A modified pillar classification system
         6.14.1.2 Influence of joints
         6.14.1.3 Influence of small-scale irregularities in pillar outline
         6.14.1.4 Other classification systems
      6.14.2 Determining peak strength and residual strength
      6.14.3 Evaluation of errors and validation of calculated pillar safety factors
### 7 TIME-DEPENDENT DEFORMATION OF THE CALCARENITE

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.1 Introduction</td>
<td>174</td>
</tr>
<tr>
<td>7.2 General background</td>
<td>174</td>
</tr>
<tr>
<td>7.3 Creep laws</td>
<td>176</td>
</tr>
<tr>
<td>7.3.1 Creep laws derived from deformation mechanism theories</td>
<td>176</td>
</tr>
<tr>
<td>7.3.2 Creep laws based on rheological analogs</td>
<td>177</td>
</tr>
<tr>
<td>7.3.3 Empirical creep laws</td>
<td>177</td>
</tr>
<tr>
<td>7.4 Experimental procedure</td>
<td>179</td>
</tr>
<tr>
<td>7.4.1 Sampling and sample preparation</td>
<td>179</td>
</tr>
<tr>
<td>7.4.2 Creep tests on cylindrical cores</td>
<td>180</td>
</tr>
<tr>
<td>7.4.3 Creep tests on prismatic samples</td>
<td>183</td>
</tr>
<tr>
<td>7.5 Experimental results</td>
<td>185</td>
</tr>
<tr>
<td>7.5.1 Cylindrical cores</td>
<td>183</td>
</tr>
<tr>
<td>7.5.2 Prismatic samples</td>
<td>188</td>
</tr>
<tr>
<td>7.6 Data analysis</td>
<td>188</td>
</tr>
<tr>
<td>7.6.1 Cylindrical cores</td>
<td>188</td>
</tr>
<tr>
<td>7.6.1.1 Type of decelerating creep law</td>
<td>188</td>
</tr>
<tr>
<td>7.6.1.2 Type of accelerating creep law</td>
<td>189</td>
</tr>
<tr>
<td>7.6.1.3 Separation of decelerating and accelerating creep</td>
<td>191</td>
</tr>
<tr>
<td>7.6.1.4 Assessment of creep law parameters</td>
<td>194</td>
</tr>
<tr>
<td>7.6.1.5 General law, describing decelerating and accelerating creep</td>
<td>195</td>
</tr>
<tr>
<td>7.6.1.6 Incorporation of stress in decelerating creep laws</td>
<td>197</td>
</tr>
<tr>
<td>7.6.1.7 Incorporation of stress in accelerating creep laws</td>
<td>203</td>
</tr>
<tr>
<td>7.6.1.8 The relationship between stress level and time to failure</td>
<td>206</td>
</tr>
<tr>
<td>7.6.1.9 The relationship between time to failure and other creep parameters</td>
<td>207</td>
</tr>
<tr>
<td>7.6.1.10 The assessment of long-term strength</td>
<td>209</td>
</tr>
<tr>
<td>7.6.2 Prismatic samples</td>
<td>213</td>
</tr>
<tr>
<td>7.7 Long-term strength of mine pillars</td>
<td>213</td>
</tr>
<tr>
<td>7.7.1 Delineation of the problem</td>
<td>213</td>
</tr>
<tr>
<td>7.7.2 Fracture mechanics and creep rupture of calcarenite</td>
<td>215</td>
</tr>
<tr>
<td>7.7.3 Evaluation of long-term strength of pillars relative to residual strength</td>
<td>217</td>
</tr>
<tr>
<td>7.7.4 Verification of long-term strength of pillars</td>
<td>217</td>
</tr>
<tr>
<td>7.8 Application of results in the mines</td>
<td>218</td>
</tr>
<tr>
<td>7.9 Creep deformation of a whole mine</td>
<td>220</td>
</tr>
</tbody>
</table>

## PART III: OBSERVATIONS IN THE MINES AND APPLICATION OF RESULTS TO ASSESS MINE STABILITY

### 8 THE GEULHEMMER GROEVE

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.1 Introduction</td>
<td>223</td>
</tr>
<tr>
<td>8.2 Structural analysis of the Geulhemmer Groeve</td>
<td>223</td>
</tr>
</tbody>
</table>
10.4 Analysis of the collapses
   10.4.1 The 1920 collapse
       10.4.1.1 Mechanisms of collapse termination
       10.4.1.2 Insufficient large-scale pressure arching
       10.4.1.2 Deformation behaviour of a thin rock overburden
       10.4.1.4 Deterioration of pillars near the limit of collapse
       10.4.1.5 Location of the collapse
   10.4.2 The 1705 collapse

10.5 Pillar stability analysis according to strength formulae
   10.5.1 Individual pillars
   10.5.2 Large-scale pillar stability and general mine stability

11 OTHER INSTABLE OR COLLAPSED MINES; SYNOPSIS OF COLLAPSE POTENTIAL
   11.1 Introduction
   11.2 The collapses in the Gemeentegrot
   11.3 The collapse area of the Muizenberg
   11.4 The collapse area of the Roosburg
   11.5 The area of failed pillars in Groeve De Schenk
   11.6 The area of failed pillars in the Grooberg
   11.7 The area of failed pillars in the southeastern part of the Gemeentegrot
   11.8 Two types of collapses
   11.9 Analysis of general mine stability and collapse potential
       11.9.1 General mine stability in terms of mine span and rock overburden height
       11.9.2 General mine stability in terms of pillar and strata stiffnesses
       11.9.3 Synopsis of large-scale collapse

12 CONCLUSIONS AND RECOMMENDATIONS
   12.1 Application of the system to assess pillar stability and large-scale collapse potential
   12.2 General principles of support measures
   12.3 Recommendations towards improvements of the system

APPENDIX
REFERENCES
LIST OF ABBREVIATIONS AND SYMBOLS
SAMENVATTING
SUMMARY
ACKNOWLEDGEMENT
DANKWOORD
CURRICULUM VITAE
CHAPTER 1

INTRODUCTION

In South Limburg, The Netherlands, and in adjacent parts of Belgium extensive room and pillar mines have been excavated in calcarenites of Maastrichtian age at depths of 50 m at most. The average porosity of these rocks is 45% and their uniaxial compressive strength (UCS) generally does not exceed 5 MPa. Nevertheless the soft rock served as a durable building stone for centuries. At present, mining is undertaken at only one location for restoration purposes. The abandoned mines are of great social-cultural value and their touristic exploitation, particularly those near Maastricht and Valkenburg, is economically important for the region.

In several mines stability problems have arisen. Instability of the roof forms a hazard for people underground, and earth inflow from solution pipes into the mine may eventually result in sinkhole formation at the surface. However, these types of instability are only of local importance. This does not apply to instability of pillars, which may give rise to a catastrophic collapse and destruction of a whole mine. Therefore, pillar stability and large-scale collapse, which represent the stability problem with the most widespread consequences, form the theme of this thesis.

Stability of individual pillars depends on the ratio of pillar strength and pillar stress, which is known as the safety factor. Strictly spoken, a value of one or less means immediate failure, while the risk of failure will decrease at an increase of the safety factor above one. Failure of an individual pillar, i.e. decrease of its load carrying capacity due to fracturing, does normally not represent an acute danger. But, if a pillar fails and starts to fracture, load will be transferred to the surrounding pillars. These pillars may also start to deteriorate. Due to such a domino effect all pillars of a mine section may eventually fail. However, a seriously deteriorated mine section will not necessarily collapse. In such a case also the general mine stability should be considered: if the arching capacity of the overburden is insufficient, a sudden large-scale collapse of up to several hectares may result and the whole mine section is destroyed within a few seconds. Such events have occurred until recent times. Evidence exists that mine pillars may gradually deteriorate in the course of time due to creep, which is time-dependent deformation at a more or less constant stress.
Such a sudden, large-scale pillar collapse is not only a lethal hazard for people who happen to be inside the collapsing area. The event is always accompanied by a strong air blast, which has proven to be fatal to people inside the rest of the mine and even outside, near the entrance. The most tragic collapse until now occurred in 1958, killing 18 people and severely injuring 3. Additionally, a collapse is almost immediately followed by subsidence at the surface, including serious faulting and sinkhole formation.

It should be noted that instability of ancient room and pillar mines is not limited to South Limburg and its surroundings but occurs world-wide. For example, in France thousands of such excavations exist, undermining almost 10% of the metropolitan communities.

As noted previously, this thesis concentrates on pillar stability, and attempts to describe and analyze the phenomena related to large-scale collapse of a calcarenite mine. On the basis of experiments in the laboratory and field studies in several mines a system is developed to assess pillar stability and to predict the possibility of a large-scale collapse.

This thesis is divided into three parts. Part I (Chapter 2 to 4) is mainly a literature study. Part II (Chapter 5 to 7) gives an account of experimental work. Part III (Chapter 8 to 11) concerns field studies of collapse areas and pillar stability, and a validation of the experimentally derived methods of pillar stability assessment. In Chapter 12 guidelines are given as how to apply the results of this thesis into practice and recommendations as to how the method could be improved.

In Chapter 2 an outline is given of the regional geology and the characteristics of the calcarenite mines. In Chapter 3 the general stability problems of the mines are delineated. Accounts of major collapses are studied, discussed and analyzed, to reveal their main characteristics and consequences as known at the start of the research. In Chapter 4 the existing methods of pillar stability assessment are presented. Regarding individual pillar stability, formulae of pillar strength as a function of material strength, size and shape are described and methods of determining pillar stress are outlined. This is followed by an evaluation of the concepts for large-scale stability and collapse potential. Finally shortcomings of the existing techniques of stability assessment are analyzed and it is evaluated which data should be acquired for this study. This results in a definition of the further approach of this study.

Chapter 5 gives an account of the mechanical behaviour of the calcarenite, measured during uniaxial and triaxial compression experiments. Also the influence of the moisture content is determined. Chapter 6 discusses the size effect and the influence of pillar shape on deformation behaviour, strength and deformation modulus, before and after failure. Not only peak strength but residual strength of failed pillars is evaluated as well. Numerical experiments are performed to compare the stress distribution in rock prisms, tested in the laboratory, and in actual pillars in a mine. On the basis of the results thus far, practical methods are developed as how to
classify visually observable pillar damage and as how to determine a safety factor for an individual pillar and a total safety factor for a whole mine. The errors of such calculations are evaluated and a method of validation is proposed, which is based on a comparison of calculated safety factors with the degree of observed pillar damage. Chapter 7 deals with the creep behaviour and the long-term strength of the calcarenite. Recommendations are given as how to incorporate the creep behaviour in short-term pillar strength formulae, in order to estimate their long-term strength and long-term safety factor.

In Chapter 8 the Geulhemmer Groeve serves as a first case study. The variation of UCS throughout this mine is measured and correlated to Schmidt hammer (PT) values. Pillar safety factors are calculated and validated according to the methods of Chapters 6 and 7. By establishing the age of the various parts of the mine also the factor time could be incorporated in this analysis. In Chapter 9 the Heidegroeve, where the most recent large-scale collapse occurred, is studied. The collapse area underground and the ground movements are extensively described and analyzed. A comparison between safety factors and damage to pillars, observations at the collapse boundary and additional data result in an explanation of this collapse, which was totally unexpected some years before. Chapter 10, dealing with the Fallenberg, comprises a study of the phenomena inside the collapse area and at its perimeter. The comparison between damage and safety factor for the pillars inside the adjacent Jezuïetenberg serves to investigate the impact of the collapse on this part of the mine. Chapter 11 analyzes 6 additional collapsed areas and 4 areas which did not collapse despite widespread pillar failure. Different types of large-scale collapses could be distinguished. Additionally, this analysis results in a rough guideline as how to determine general mine stability, i.e. if a large-scale collapse is possible of a seriously deteriorated mine or mine section.

Chapter 12 summarizes how to perform a stability assessment, i.e. how to determine whether a mine may collapse in the future or not. Guidelines are formulated as to which actions should be undertaken according to the outcome of the analysis. Finally, recommendations are given towards improvements of stability assessment.
PART I

OUTLINE OF THE MINE CHARACTERISTICS AND METHODS AVAILABLE OF PILLAR STABILITY ASSESSMENT
CHAPTER 2

GENERAL DESCRIPTION OF THE MINES AND THEIR GEOLOGY

2.1 GEOLOGICAL SETTING

The area of South Limburg is situated to the north of the Hercynian Ardennes-Rhenish Massif and at the eastern margin of the Caledonian Brabant Massif (Fig. 2.1) The direct vicinity of these two massifs greatly influenced sedimentation. From the Upper Carboniferous onwards sedimentation was also significantly affected by the NW-SE trending faults of the Roer Valley Graben. Fig. 2.2 shows a simplified geological map of South Limburg.

In the area no sediments from the Upper Carboniferous (Stephanian) until the beginning of the Upper Cretaceous can be found (Kuyl, 1980). During the worldwide transgression period of the Upper Cretaceous the Brabant Massif was flooded (Ziegler, 1982). Also the Ardennes-Rhenish Massif was reached by this transgression. In the area studied, Upper Cretaceous sedimentation started with the deposition of lagoonal clays and sands, the Formation of Aachen, unconformably on top of the folded Carboniferous. Subsequently the shallow marine glauconitic sands of the Formation of Vaals and the fine-grained mudstones of the Formation of Gulpen were deposited in a clearly marine environment (Kuyl, 1980). It was already recognized in the 19th century by Binkhorst van den Binkhorst that the Upper Cretaceous sediment succession is a transgressive-regressive sequence (Zijlstra, 1994). This regression is characterized by the shallow marine carbonate sands of the Formation of Maastricht and by the marly Formation of Kunrade. The regression is considered by various authors to be brought about by epi-orogenetic movements (Zijlstra, 1994). During the Tertiary the Upper Cretaceous sediments were uplifted above sea-level and the limestones became affected by dissolution and karstification. At the beginning of the Tertiary Southern Limburg became situated at the margin of the Cenozoic North Sea Basin. Only between Valkenburg and Maastricht, Lower Paleocene sediments, the Limestone of Houthem, can be found.

No Eocene sediments have been found in the area. A period of transgression resulted in the deposition of Lower Oligocene sands (Formation of Tongeren) and Middle Oligocene sands and clays (Formation of Rupel). Then a new period of non-deposition occurred. The sands and browncoal layers of Middle Miocene age (Breda
Fig. 2.1 Location of study area near the Brabant and Ardennes-Rhenish Massifs, and the Rhine-Ruhr fault system (reproduced from Price & Verhoef, 1988).

Formation) are the last sediments in South Limburg deposited in a coastal marine environment. The period from Upper Miocene to present time is characterised by the deposition of alluvial sediments of the river Maas and its tributaries. Commonly Upper Pleistocene loess deposits are to be found at the topographic surface.

Table 2.1 Classification according to grain-size of detrital limestones (Geological nomenclature of the Royal Geological and Mining Society of The Netherlands, 1980).

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<thead>
<tr>
<th>Classification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calciultite</td>
<td>Consisting predominantly of carbonate grains of silt and clay-size, i.e. smaller than 64 μm</td>
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<tr>
<td>Calcisiltite</td>
<td>Consisting mainly of carbonate grains of silt-size, i.e. between 2 and 64 μm</td>
</tr>
<tr>
<td>Calcarenite</td>
<td>Consisting for more than 50 % of carbonate particles of sand-size, i.e. between 64 and 2000 μm</td>
</tr>
<tr>
<td>Calcirudite</td>
<td>Consisting for more than 50 % of carbonate particles larger than 2 mm</td>
</tr>
</tbody>
</table>
Fig. 2.2 Pre-Quaternary geological map of South Limburg (reproduced from Felder et al., 1984). The faults in the northeast form the southwestern boundary of the Roer Valley Graben.
Reviews of the regional geology are to be found in Kuyyl (1980), Bless (1983) and Felder (1989). An overview of the interaction of sedimentation and tectonics in Limburg and its vicinity is given by Geluk et al. (1994).

2.2 THE MAASTRICHTIAN CHALK OF SOUTH LIMBURG

During the Upper Cretaceous widespread deposition of chalk occurred in Northwest Europe (Fig. 2.1). The chalk generally contains numerous flint layers. In most parts true chalk s.s. can be found, which is a light-coloured, friable calcilutite (Table 2.1) mainly composed of coccoliths\(^1\) and their fragments. This very fine-grained limestone is well known, for example, from the white cliffs of Dover and is used for writing on the blackboard. Locally a much coarser-grained, tuffaceous chalk (French: tuffeau, Dutch: tufkrijt) has been preserved, which is actually not a true chalk as defined above. Tuffaceous\(^2\) chalk is a designation for weak, porous limestones which can be characterized according to grain-size as calcarenites and calcisiltites (see Table 2.1). This chalk is of Maastrichtian age and occurs in South Limburg and in the adjacent area of Belgium towards the west as well as in SW France.

An extensive study of the sedimentology of the (tuffaceous) chalk of South Limburg, particularly of authigenesis\(^3\) of flints and hardgrounds, is presented by Zijlstra (1994).

2.2.1 Lithostratigraphic subdivision

Fig. 2.3 shows the lithostratigraphic subdivision of the Upper Cretaceous chalk of South Limburg (Felder, 1975). This subdivision has been established on the basis of laterally continuous flint layers and erosion surfaces. Two Upper Cretaceous limestone formations are distinguished: the Formation of Gulpen and the younger Formation of Maastricht. Apart from the lowermost part of the Formation of Gulpen, the Upper Campanian Limestone of Zeven Wegen, both formations are considered to be of Maastrichtian age. The coarsening upwards sequence formed by the Maastrichtian carbonate rocks starts with coccolithic, partly clayey calcilutites, such as the Limestone of Vijlen. Here the true "blackboard" chalk is represented. The overlying Lixhe 1,2,3 and Lanaye Limestones of the Formation of Gulpen are mainly coccolithic calcisiltites with numerous flint layers.

\(^1\) Minuscule calcite plates originating from unicellular planktonic algae

\(^2\) Note that, strictly speaking, tuffaceous means "containing volcanic ash". Obviously a tuffaceous chalk differs completely from volcanic tuff, which is an indurated, porous volcanic ash deposit.

\(^3\) The producion of constituents of sedimentary rocks in place.
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Unconformity

Fig. 2.3 Lithostratigraphic subdivision of the Upper Cretaceous and Paleocene limestones in South Limburg (after Felder, 1975). Black rectangles indicate major mining levels.

2.2.2 The Formation of Maaschtiand

The commonly coarser grained limestones of the Formation of Maaschtiand were deposited in a shallow sea and their lateral uniformity is limited. Two different facies developments can be recognized. In the western part of Southern Limburg it is developed as the "Maaschtiand facies", in the eastern part as the "Kunrade facies". The distribution of the Formation of Maaschtiand in South Limburg and the facies boundary are displayed in Fig. 2.4. The maximum overall thickness of the formation is about 100m. As a result of the uplift of the Arden-Heinsh Massif and the subsidence of the Rhine-Ruhr Valley graben the Formation of Maaschtiand dips about 1°30' towards the northwest (Felder, 1989).

In the eastern part of Southern Limburg the upper part of the formation is not present. The lower part consists of alternating hard, relatively well cemented limestones and soft limestones with some clay content, representing the "Kunrade" facies of the Maaschtiand Limestone.
2.2.3 The "Maastricht facies" limestone

The building stone mines were almost exclusively excavated in the "Maastricht facies" limestone (Felder, 1973). All members of this facies comprise predominantly carbonate bioclasts of sand-size. This sequence of calcarenites is also known as the tuffaceous chalk of Maastricht. The low concentration of terrigenous siliciclastics is probably due to a low relief of the continent of Northwest Europe. The chalk was deposited at rates of centimeters to decimeters per 1000 years (Zijlstra, 1994). The limestones are characterized by fining upwards cycles with a coarse-grained basis of bioclastic sand that may contain even gravel-sized fossil fragments, while the finer-grained top consists of silt-sized bioclasts which are generally homogeneously bioturbated. The bioturbated zone is lithified in various degrees, producing

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4 It should be noted that in both popular and scientific literature the Dutch word "mergel" (English: marl, French: marne, German: mergel) is often used for the nearly pure limestones of the "Maastricht" facies limestones, probably because of their comparable softness. Since marl is defined as a mixture of clay and fine-grained carbonate material, this designation is not correct.
hardgrounds\textsuperscript{5} at various levels in the formation. The top of the cycle represents a time of non-deposition and erosion. Although the thickness of the cycles varies considerably, a general increase in thickness from a decimeter at the bottom towards several meters at the top of the formation can be observed.

Six members are distinguished according to the subdivision of Felder (1975). The units, light-yellow to light yellow-grey in colour, are bounded at both top and bottom by "horizons" (Fig. 2.3). These horizons commonly contain hardgrounds at the top of the lowermost unit, covered by one or more thin beds of fossil fragments forming the base of the next unit.

\textsuperscript{5} Calcareous sediments are often indurated due to cementation at the surface during periods of non-deposition. This rocky seabottom, frequently penetrated by boring organisms, is known as a hardground.
In the lower three units lithification is often poorly developed and layers of flint nodules\(^6\) occur instead. In the upper part of the formation flints are largely absent. With the exception of the lowermost unit, the CaCO\(_3\) content is high and rarely less than 96 % (Felder, 1973). Apart from the hardgrounds the calcarenites are soft, weakly cemented and highly porous. Porosities of 40 to 52 % have been measured (Price & Verhoef, 1988). Additionally, the grains are loosely bonded and evidence of pressure-solution or compaction is hardly observed. As a consequence strength is low, i.e. the unconfined compressive strength amounts only to a few MPa. Apart from hardgrounds, layers of fossil grit and flint layers sedimentary bedding is generally not visible, even under the optical microscope. However, geomechanical

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\(^6\) Flint denotes silica concretions often occurring in chalks and other calcareous rocks of Cretaceous and Tertiary age. According to Zijlstra (1994) the flint nodules were formed after deposition by concentration of dissolved silica skeletons of micro-organisms. They occur in layers as a result of the vertical variation in silica concentration. Zijlstra concluded that during periods of a low (carbonate) sedimentation rate, deposits formed with a high silica concentration, which became the later flint layers. During periods of fast carbonate sedimentation the silica concentration in the deposits was low. The rhythmic variation of sedimentation rate is attributed to cyclic climatic changes due to orbital variations.
tests show a transverse isotropy\(^7\), as shown in Chapter 5.

2.2.4 Microstructure

Fig. 2.5 shows a typical microstructure of the "Maastricht facies" Formation of Maastricht. The sample is taken from the upper part of the Emael Limestone in the Sibbergroeve. A well sorted clastic carbonate rock is to be seen, displaying a grain size ranging from 0.1 to 0.4 mm (calcarenite). On the basis of the grain supported texture and the absence of mud it can be classified as a grainstone according to the Dunham (1962) classification system.

The allochems mainly consist of fragments of mesofossils (fossils varying in size from 1 to 2.4 mm). They underwent strong micritization, in most cases resulting in the elimination of observable internal structure. Fresh micrite between the grains is absent. Therefore the micritization could have occurred before transportation of the

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\(^7\) Transverse isotropy of a sedimentary bedded rock means that its mechanical properties, e.g. Young's modulus and compressive strength, are the same in all directions within the bedding plane. However, the mechanical properties in other directions differ from those measured within the bedding plane (see also Section 5.5).
grains. Many particles display very fine (less than 0.02 mm long) "dogtooth" cement overgrowth. Some particles probably representing nuclei of echinoderms show syntactical overgrowth, but as a whole cementation is very poorly developed. As a consequence the porosity is high and the strength is low.

2.3 CHARACTERISTICS OF THE ROOM AND PILLAR MINES

The room and pillar mines have been studied and described by various authors. The contributions of Van Wijngaarden (1967), Felder (1973) and Breuls (1994) give an appropriate general introduction. Van Schaik (1983) described the mines of the St.Pietersberg.

In South Limburg about 180 mine workings are known. Many of these are small but a few represent extensive labyrinths with total gallery lengths of up to 100 km. Fig. 2.6 shows the entrances of the most important mine systems. The majority of the entrances are located at the slopes which rise from the valleys of the river Maas, Geul and Jeker upwards on to the plateaus. Although this thesis concentrates on the mines in the Dutch part of Limburg it might be appropriate to note that most mines in Belgian Limburg are accessible from the plateau by means of a "graet", a winding staircase excavated in the overburden.
The largest mine system existed in the St. Pietersberg (Fig. 2.7) south of Maastricht (250 hectares), but has disappeared for at least 80% due to the open pit mining in the ENCI-quarry, leaving less than 50 hectares of intact mines. The largest mine now is the Sibbergroeve (85 hectares) in the region of Valkenburg. Other large mines are the Gemeentegrot (30 hectares), the Geulhemmergroeve (10 hectares), both also in the Geul Valley, the system of Fallenberg, Boschberg and Cannerberg (in total 31 hectares) excavated in the Cannerberg and the Avergat (20 hectares) near Kanne in Belgium\(^8\). The most extensive mine system in Belgium is the Grote Berg in Zussen (45 hectares).

\(^8\) The Dutch word "berg" officially means mountain or hill. However, at the St. Pietersberg and Cannerberg (the hills to the south of Maastricht) also the mine workings themselves are named "berg", according to local dialect. Also in Belgium mines are often denoted "berg" (e.g. Grote Berg). Additionally, the name "gat" (e.g. Avergat) is used. Most mines in Dutch Limburg are known as "groeve". Some room and pillar mines are named "grot", which is incorrect because "grot" means natural cave.
2.3.1 Mined levels

The Limestone of Maastricht was mined at levels relatively free of flint nodules and promising the best building stone quality (Fig. 2.3). Roof and floor were normally formed by hardgrounds, also named "tauwlagen" (Figs. 8.2, 9.2, 10.2). Mine workings were situated above the watertable. The stratigraphical levels of the calcarenite mines studied here are given by Felder (1979a, 1979b, 1980). The Sibbergroeve has been excavated in the upper part of the Emael Limestone. In the direct vicinity of Valkenburg most mining, including the Gemeentegrot, has been undertaken in the upper part of the Emael Limestone and the lower part of the Nekum Limestone. In the Geul Valley towards the west these layers become situated below the watertable. Here various levels of the Meerssen Limestone were chosen. The mines of the St.Pietersberg and Cannerberg were mainly excavated in the Nekum Limestone. Fig. 2.8 gives an impression of an intact mine system as occurred in the Heidegroeve near Valkenburg before the 1988 collapse. Here mining was undertaken within two levels of the Meerssen Limestone separated by a clearly distinguishable hardground.

2.3.2 Geological structures affecting exploitation

Within a certain mining level the mining direction was determined by the limits of the concession and by the quality and strength of the rock. Furthermore, densely spaced joints, faults, solution pipes and the more rarely occurring vertical flint bodies were avoided. In this section it is shown that, within a certain mining level, the calcarenites can be considered as continuous rocks.

2.3.2.1 Joints

The calcarenites are not jointed at regular intervals. Their spacing is variable and usually tens of meters, and their persistence rarely exceeds 200 m. Their aperture is normally less than 1 mm. The majority of joints are subvertical, but inclined orientations up to 45° also exist. Their strikes show a distribution around two directions. Generally a major joint set, with a strike ranging from 270° to 320°, and a secondary set, striking 0° to 60°, can be distinguished (Fig. 2.9). These joints directions are not related to the topography. The directions of the major joint set correspond well to the present-day crustal stress field in Northwest Europe, which is characterized by a NW-SE oriented maximum horizontal stress (Zoback, 1992; Grünthal & Stromeyer, 1994). This stress field also brings about the still continuing extension of the NW-SE trending rift system of the Roer Valley Graben (Figs. 2.1 and 2.2). The origin of the minor joint set with an average NE-SW orientation is less clear. Van den Berg (1994) proposed presently reactivated crustal buckling generated by NW-oriented compression by the Ardennes. An explanation could be that the minor joint set developed parallel to the axial plane of previously existing folds, which became affected by (further) flexure. This could apply especially to the hinge areas. It is concluded that both joint sets are most likely of tectonic origin.
Fig. 2.10 Earthpipe of about 0.5 m diameter in the Geulhemmer Groeve.

Near mine entrances, commonly created in a topographic slope, joints often show an aperture of several millimeters or even centimeters, and are often filled with clay and affected by karst phenomena. This applies particularly to joints oriented more or less parallel to the topographic slope. Since the overburden thickness is minimal near the entrances, these phenomena rarely have an adverse effect on pillar stability.

Joints represent planes of weakness and may have an adverse effect on pillar- and roof stability, and on the quality of the building stone. When the joint spacing was small no usable building stone could be mined. It was difficult, if not impossible, to penetrate such a zone of narrow joint spacing, without artificial support, to reach other areas of exploitable limestone. A gallery which penetrated such a zone was called a "strafpijp" (a "penal gallery"), because its creation did not yield material of building stone quality and therefore no income.
2.3.2.2 Earthpipes

The upper surface of the Limestone of Maastricht is commonly highly irregular due to solution weathering (karst). Often solution pipes have been formed (Fig. 2.10). They are filled with material from the overlying sediments, usually clay, sand and gravel, and have been observed to penetrate to depths of as much as 40 m. In most mines these earthpipes can be seen, sometimes in isolation but usually in clusters, especially where the calcarenite overburden is thin. This happens mainly near entrances, but sometimes earthpipes can also be seen in clusters in more central parts of a mine system. The diameter of the earthpipes at mining level ranges from a few centimeters to 5 m. Sometimes the soil of the earthpipe has flowed into the mine, resulting in surface subsidence. The rock strength in their immediate vicinity is higher due to a stronger than usual cementation. Accordingly earthpipes often attract miners.

2.3.2.3 Flints

Flint nodules are normally concentrated in certain levels of the Maastrichtian Limestone. They are of irregular shape and are orientated parallel to the bedding if developed as discs or lenses. However, locally vertically orientated flint bodies, called "flint curtains", also occur. Felder (1980) described such a flint curtain which was 7.25 m high.

2.3.2.4 Faults

Faults are rare in the calcarenite mines. The most prominent fault encountered in a mine system is the east-west trending steeply to the north dipping Klauwpipi fault (Fig. 2.2). This normal fault shows a vertical displacement of about 20 m and roughly forms the southern boundary of the Gemeentegrot.

2.3.3 Dimension and shape of pillars and galleries

In most mines the height of the pillars varies between 2 and 3 m. The roof and floor were normally formed by hardgrounds, but often the hardground of the floor was removed to exploit deeper levels. At the St.Pietersberg deepening generally occurred in two or three stages, resulting in pillars averaging 6 to 8 m high. Locally deepening was carried out four to five times and pillars reached a height of 12 to 15 m. In the Avergat a pillar height of 7 to 11 m was produced in usually one stage.

After the blocks had been sawn out large quantities of "waste" rock material remained. After mining, this material, mainly slabs of some cm thickness, was stacked up to the roof in niches and against the pillar walls. Due to these "knabbenhopen" the gallery width was significantly reduced. From the beginning of
the 20th century the calcarenite mines have been used more and more for mushroom growing. It is estimated that this activity has been performed in at least 90% of the mines. This development left its mark on the subterranean landscape. In order to create more surface for the mushrooms, the once piled up slabs were spread over the mine floor and pulverized. As a consequence, the mine floor was raised considerably and the three-dimensional form of the mine became more regular. For example, in parts of the Fallenberg the average pillar height is presently 3 m, but was originally 6 m. In the next chapters it will become clear that this must have stabilized the pillars. However, occasionally the "waste" material was transported out of the mine and used for agricultural purposes, for example in parts of the Gemeentegrot. Sometimes further excavation was carried out exclusively for this reason, for example in parts of the Caestert mines in the St.Pietersberg. Here also existing galleries were deepened to produce pulverized calcarenite.

Sometimes the floor was only locally broken away and subsequently a new mine system was excavated below the first working level. This happened, for example, in parts of the Gemeentegrot, the Fallenberg and the Geulhemmergroeve. In the Barakkengrot near Valkenburg even three storeys exist.

Usually the galleries are 3.5 to 4 m wide and rectangular in vertical cross-section. However, in the Avergat the pillar walls at both sides of a gallery are inclined in such a way that the gallery width increases from 3.5 to 4 m at the roof towards maximally 6 m at the floor. Also, due to "pillar robbing", gallery widths can raise significantly. In parts of the Caestertgroeve, excavated in the southern part of the St.Pietersberg, the cross-section is triangular in shape forming an arch. Mining was
carried out here exclusively by using a chisel, resulting in pulverised rock for agricultural purposes.

The pillars are generally rectangular and show horizontal dimensions typically varying from 4 to 20 m. The width to height ratio is usually about 1 to 2. When the geological conditions were favourable a highly organized exploitation was possible. Then a regular pattern of pillars resulted as in the St.Pietersberg (Fig. 2.11). But more often due to the geology and unskilled mining rather irregular patterns were created. Also due to later pillar robbing irregular pillars with a complex outline resulted (Figs. 2.12 and 2.13). Locally the extraction ratio approached 60 %.
2.3.4 Overburden composition

The overburden above the mine roof is usually calcarenite of at least a few meter, overlain by soil units such as Lower Oligocene sands of the Formation of Tongeren, Quaternary clays, sands and gravels of the alluvial Maas sediments and finally loess deposits. The total overburden thickness is at maximum about 50 m. Examples of overburden composition are to be found in Figs. 8.2, 9.2 and 10.2.

2.3.5 Climate and weathering

The climate in the mines is fairly constant. The temperature is generally $10 \pm 2 \, ^\circ C$ and the air moisture content is 90 to 100%.

Accordingly, weathering inside the mines is generally not serious, except some frost weathering close to the entrance where a zone of some centimeters thickness may be affected. It has been argued that acid vapour resulting from mushroom growing might induce weathering, but this has never been shown indisputably. There might be a slight gradient in moisture content in the outermost centimeter of pillar walls and roof surfaces due to constant evaporation into the mine opening. But slaking, general deterioration and breakdown at the rock surface due to exposure by excavation, has not been observed.

2.4 MINING HISTORY

2.4.1 Flint mines

Underground mining within the Upper Cretaceous limestones has taken place over a considerable period. Already 3000 years before Christ flint was mined in the upper level of the Gulpen Limestone (Bosch, 1979). Near Ryckholt, to the southeast of Maastricht, many flint mines have been found (Fig. 2.6). The flint layers were reached from a vertical shaft and exploited in irregular galleries radiating from the shaft base.

2.4.2 Origin of room and pillar mines

The oldest building stone mines in the calcarenites were claimed by some to have been made by the Romans. At one locality near a Roman villa a mine was discovered which consisted of a shaft giving access to four chambers arranged in the form of a clover-leaf (Engelen, 1989). The same locally exploited limestone was probably used for the foundation of the villa. This discovery and other findings have brought about the idea that the earliest extensive room and pillar mines near Maastricht and Valkenburg would be of Roman origin. However, there is no solid evidence at all for this and it is generally believed that the room and pillar mines were excavated in a later period. Historical documents and inscriptions in the mines
of the St. Pietersberg suggest that the room and pillar workings came into being in the 13th century (Westrenen, 1988). The first exploitation in the St. Pietersberg was probably carried out by monks from a nearby monastery. In later times the limestone was mined by day-labourers.

The limestone was primarily mined to produce building stone but was also used for agricultural purposes.

2.4.3 Mining techniques

Right from the beginning of mining, coarse toothed handsaws were used to cut the rock. Picks, and later chisels, served to create space for sawing and to loosen the
blocks. The limestone was exploited in blocks, named "stoelen", of about 50 cm wide, 80 cm deep and 200 cm high. But it must be emphasised that the block size was not always and everywhere the same. As described above, many existing mines were deepened one or more times. Amongst others in the Avergat a gallery height of up to 11 m was reached in one stage. The various methods and the progress and direction of extraction can be reconstructed easily by studying the scars left on the roof and walls. The blocks were transported out of the mine by wagon and cart-horse. The scars of the wheelnaves indicate the mining levels. The mining techniques most frequently applied are described by Van Wijngaarden (1967), Bochman & Hillegers (1984) and Breuls (1985, 1994).

From the second half of the 19th century onwards the Maastrichtian limestone became more and more exploited in open pit mines for the cement industry (e.g. the ENCI-mine in the St.Pietersberg). In other open pit mines, for example near 't Rooth, limestone is extracted mainly for the chemical industry. In the beginning of the 20th century the glass industry was an important consumer of limestone.

The exploitation of the building stone mines continued well into this century but has been reduced significantly since 1955. Presently building stone is mined only in the Sibbergroeve and mainly for restoration purposes. Chisel and handsaw have been replaced by electric chainsaw (Fig. 2.14) and the calcarenite is transported by adapted tractors instead of horsedrawn wagons.

Reviews of the exploitation history of the Upper Cretaceous limestones are given by Felder (1973), van Westrenen (1988) and Engelen (1989).

2.5 USE OF THE MINES

The room and pillar mines have played an important role in the history of Southern Limburg. Once mining was completed they were used for various purposes.

2.5.1 Non-touristic purposes

For the inhabitants of the region the mines have always served as a refuge in time of war as is witnessed not only by inscriptions on the walls but also by the presence of clandestine chapels (Geulhemmer Groeve) and bakeries (St.Pietersberg). During the Second World War famous Dutch paintings were stored in the St.Pietersberg. During this period some mines, for example the Heidegroeve, were converted by the Germans into bomb proof factories for their war industry (see Section 9.2.3). In the Gemeentegrot and the St.Pietersberg nuclear shelters have been built. The Boschberg was used as a NATO command post from 1954 to 1992.

From about the beginning of the 20th century until the 1960’s many mines have been used extensively for mushroom growing. Also, on a smaller scale, chicory and cardoon were grown (Breuls, 1994). At present mushroom growing is only
performed in parts of the Avergat and the Noordelijk Gangenstelsel of the St. Pietersberg. Cattle were often housed and in some mines (Geulhemmergroeve, St.Pietersberg) even people have lived. In the Geulhemmergroeve a modern house has been built recently. Some mines serve as storage place. Thus, in the Geulhemmergroeve a collection of various coal types is kept. Because of the stable atmospheric conditions in this mine measuring tapes for the coal mines were calibrated. Also seismographs are housed in the Geulhemmergroeve.

2.5.2 Touristic purposes

For centuries the mines have been a curiosity for tourists, who are interested to view not only the workings themselves but also the sculptures and paintings contained within them (Fig. 2.15). The room and pillar mines are of great social-cultural value and their touristic exploitation is economically important for the region of South Limburg. Various inscriptions, some very old, from visitors or from the miners
themselves reveal a lot of the history and the social-economical situation throughout the centuries. Hence a large number of mines are kept open to be visited by tourists, mainly mines near Maastricht and Valkenburg. For the latter town the mines form the main tourist attraction and are of essential economic importance. Some of these are modified especially for tourists and contain replicas of prehistoric monsters, are adapted for "survival" activities, or represent a coal mine or Roman catacombs. A few mines are also used for parties and other social activities.

2.5.3 Access to the mines

The entrances of the mines are locked by fences or walls in order to prevent vandalism and to preclude people entering dangerous areas. Although it is officially prohibited to enter the mines unaided, frequently people manage to break in. In 1993 two boys got lost due to failing illumination and were found dead three weeks after their disappearance.

Since various bat species winter inside the mines, the locked off entrances generally comprise openings especially designed to allow their passage. The boys mentioned above proved to have managed to squirm through these bat openings, which had been considered impossible before. Mine workings of some significance have more than one entrance or ventilation shafts. As a consequence the mines are never completely sealed off from the atmosphere outside and always some ventilation exists.
CHAPTER 3

OUTLINE OF STABILITY PROBLEMS OF THE MINES

3.1 INTRODUCTION

This chapter gives an introduction to the stability problems encountered in the calcarenite mines, especially with regard to large-scale pillar instability and collapse. Accounts of major collapses are compiled and analyzed to reveal their main characteristics and hazards.

Instability of ancient room and pillar limestone mines is not a problem limited to South Limburg and its surroundings. For example, in France thousands of underground mines exist. Almost 10 % of the metropolitan communities are underlain by mines, predominantly excavated in calcareous rocks. Twenty percent of those communities suffered from one or more instability phenomena during the past 30 years (Toulemont, 1995). In 1961 a large-scale pillar collapse brought about the destruction of 20 buildings killing 21 people and injuring 36. Most workings are of the room and pillar type and were excavated for building purposes. Depth and age of these mines are comparable to these in South Limburg. Instability phenomena include local roof falls, pillar fracturing and large-scale pillar collapse, which may result in sinkhole formation, fracturing or slope instability at the surface. A review of the characteristics of the French underground quarries and their instability problems is given by Josien (1995).

Also in England many limestone mines occur, for example in the Silurian limestones of the West Midlands. Roof collapses often resulted in sinkhole formation at the surface and large-scale pillar collapses brought about the development of large crownholes or subsidence troughs (Braithwaite & Seago, 1988). The Cretaceous chalk occurring in the southeastern part of England (Fig. 2.1) has also been mined extensively. Here mainly bellpits and deneholes have been created and relatively few room and pillar mines. Accordingly, sinkholes constitute the major surface hazard (Edmonds et al., 1989).

However, world-wide the majority of room and pillar mines has been excavated in coal. Most of the existing knowledge of pillar and roof stability has been gained from this setting (see Chapter 4).
Fig. 3.1 The three main types of local instability: a) pillar cracking and slabbing, b) failure and collapse of immediate roof layers, c) earth inflow from organ pipes.

3.2 LOCAL AND LARGE-SCALE INSTABILITY

The calcarenite mines of South Limburg are generally reasonably safe, but there are several collapsed areas and unstable zones which might collapse in the future. In the next sections observations of instability phenomena in these mines will be presented as known at the beginning of this project. Local instability affecting only one particular pillar or gallery will be distinguished from large-scale instability which involves the whole mine or a major part of it.
Fig. 3.2 Failed, hour-glass shaped pillar in the Geulhemmer Groeve.

Fig. 3.3 Collapsed roof and hanging roof in the Geulhemmer Groeve (Courtesy of T. Habets).
3.3 TYPES OF LOCAL INSTABILITY

Three major types of local instability can be recognized (Fig. 3.1)

3.3.1 Instability of pillars

If the average vertical stress on a pillar exceeds its strength, pillar cracking and surface spalling of centimeter thick slabs will occur. Cracks normally start to grow near the edges at the top or at the bottom and may propagate through the whole height of the pillar. Eventually a concave fracture plane may form, separating pillar side from pillar core. In this thesis such a pillar is classified as "failed". When such concave fracture planes have been generated on all sides of the pillar an hour-glass shaped pillar core results (Fig. 3.2). Joints oriented (sub)parallel to and situated near the side of a pillar have an adverse effect on stability, but, since joints do not frequently occur, this is not an ordinary situation.

Local pillar deterioration does normally not represent an acute danger to people inside the mine, except when large slabs threaten to fall into the gallery. However, it will be shown later in this chapter that pillar instability may become the most prominent hazard if it affects an extensive part of a mine.

Frequently overmining, i.e. reducing pillar size too much, must have been important in inducing pillar deterioration. A particular case of overmining is "pillar robbing", the extraction of additional calcarenite from existing pillars. This occurred often in an uncontrolled way resulting in irregular pillar shapes. General experience learns that also time-dependent deformation may play a significant role.

3.3.2 Instability of the roof

Since roof instability is not dealt with in detail in the next chapters, a relatively extended outline of this topic is presented below.

3.3.2.1 Observations in the mines

Cracks in the middle of the roof, detached roof beams and roof collapses are not uncommon in the calcarenite mines. The immediate roof is nearly always formed by a hardground of some decimeter thickness. Sometimes, as seen in the major part of the Geulhemmer Groeve, bedding planes in the roof are separated by clay layers. Roof falls are frequent in this mine (Van Steveninck, 1987; Vreugdenhil, 1988; Vink, 1991). Also irregular joints intersecting the hardground facilitate roof falls, as has been observed in the Jezuïetenberg (Orlic, 1990). But usually the horizontally stratified rock mass is almost free of joints.
Fig. 3.4 Compressive arching stresses in a downwards deflecting roof beam.

Generally instability is characterized by immediate roof layers, which tend to detach from the main rock mass and form separate beams or plates. A detaching roof layer can be recognized by downward deflection and the formation of cracks parallel to and in the middle of the gallery. Sometimes, adjacent to a roof previously collapsed, a deflection of the roof can be observed in profile. Detached and cracked roof layers often remain in position for many years. Sometimes it was impossible to extract a threatening roof beam even by means of a crowbar.

Roof collapses can extend over a gallery length of more than 10 meters. They occur preferably at gallery intersections where the roof span is maximal. Normally a bedding plane collapses over more or less the full width of the gallery. Often the roof fall is bounded at the edges by curved cracks of an arch-shaped geometry (Fig. 3.3). After a collapse the broken roof is normally found more or less in its original orientation on the floor. Roof falls are mostly confined to one to three beams with a total thickness of one meter.

Sometimes miners have extracted several roof layers creating a stable arch geometry. At a few locations roof fragments of irregular shape fall down, not bounded by bedding planes and/or not formed over the full gallery span. These collapses are observed where roof stability is affected by intense jointing, by clay layers or lenses, or by earth pipes.

3.3.2.2 Consequences of roof instability

For people inside the mine roof falls represent the major hazard in the short term. Roof stability is difficult to estimate and the moment of roof collapse cannot be anticipated. Particularly during mining of the calcarenite, roof falls occurred. Several miners have been killed or injured in this way. The last casualty dates from 1987 when during the creation of a connection between the Slavante and Zonneberg mines in the St.Pietersberg a 2 by 5 m roof collapse buried both miners working there. One was killed and the other one severely injured (Annual Report State Supervision of Mines, 1987).
Roof instability is commonly only hazardous for people visiting the underlying gallery. Only where the complete calcarenite roof is thin—one meter or less—or significantly affected by karst, earth may invade the mine, with sinkhole formation at the surface as a consequence. This hazard mainly applies to galleries close to the valley slope and to many mines situated under the plateau of Zichen-Zussen-Bolder in Belgium, where the calcarenite roof is generally only a few meters thick.

Roof falls are not considered to reduce pillar stability. Although they induce a small increase of pillar height, a more stable dome shape of the roof usually results (Fig. 3.3).

3.3.2.3 Assessment of roof stability

Regarding mine roofs consisting of unfractured and unjointed bedding planes, which are fairly smooth, flat and weakly bonded, beam theory (Obert & Duvall, 1967; Duvall, 1976) is often useful. Hence this method seems appropriate to the roof conditions in the calcarenite mines. In beam theory it is assumed that a roof layer acts like a clamped beam which flexes elastically under its own weight. For a single beam the maximum deflection $\delta_{\text{max}}$ at the centre of the roof span is equal to

$$\delta_{\text{max}} = \frac{\gamma L^4}{32 E h_r^2}$$  \hspace{1cm} (3.1)

where $L$ is the roof span, $h_r$ the thickness of the beam, $\gamma$ its unit weight and $E$ its Young's modulus. The maximum tensile stress $\sigma_{t,\text{max}}$ is developed at the upper surface of the beam near the ends and is equal to:

$$\sigma_{t,\text{max}} = \frac{\gamma L^2}{2h_r} - \sigma_h$$  \hspace{1cm} (3.2)

In the centre, at the bottom of the beam, the first term becomes $\gamma L^2/4h_r$. If the tensile stress $\sigma_{\text{max}}$ exceeds the tensile strength of the beam, tensile fractures in the middle and at the sides of the roof, parallel to the mine gallery, may develop. According to Eq. 3.2, a moderate horizontal compressive stress $\sigma_h$, oriented perpendicular to the gallery, strengthens the beam. Joints oriented more or less parallel to the gallery, but not intersecting it, may indicate relaxation of such a horizontal stress with a lower roof beam stability as a result. However, a relationship between joint orientation and roof beam stability has not been observed in the calcarenite mines.

At an intersection of two galleries the mechanical behaviour of the immediate roof is not accurately described by the above beam theory. Wright (1973) presents an approximate method to estimate tensile stresses at intersections, which are in excess of these for a single gallery. It has also to be noted that the theory outlined above applies to single roof beams. Obert & Duvall (1967) delineate which configurations of multiple beams enhance roof stability, relative to that of single beams, and if so, to which extent.
Fig. 3.5 Roof beam collapse modes (Potts et al., 1979): a) shear sliding along discontinuities, b) crushing and rotation, c) elastic buckling and rotation, d) shear fracturing inside roof beam elements.

Due to arching effects inside the roof beam, detached and fractured roof layers often do not fall down. Hence Eq. 3.2 does generally not predict collapse of the roof and gives a (too) conservative estimation of roof stability. The collapse of fractured roof beams is described by arching theories (e.g. Wright, 1972; Brady & Brown, 1985). In a roof beam which deflects downwards an arch-shaped zone of lateral compressive stresses develops, which transfers the gravity load of the fractured beam laterally towards the abutments (Fig. 3.4). These stresses increase with increasing deflection. As a result of this "voussoir beam" mechanism the beam, initially failed in tension, can become self-supporting and remains stable.
Fig. 3.6 Roof beam fracturing near the abutments, as frequently observed in the Geulhemmer Groeve. The roof span is 3 m. Arrows indicate the shear and dilational components of fracturing.

The authors cited above distinguished three collapse modes for jointed and/or fractured roof beams (Fig. 3.5). A fourth collapse mode was recognized by Potts et al. (1979):

1. shear sliding along the fractures or joints. Roof collapse occurs by sliding down of roof blocks.

2. crushing in areas of high compressive stress, and subsequent rotation of blocks and collapse.

3. elastic buckling of the roof beam, without compressive failure of the roof material, followed by rotation and collapse.

4. development of shear fractures inside the roof blocks near the abutments. Collapse of the roof bounded by the shear fractures and the upper bedding plane.

Which collapse mode applies, depends mainly on the span/thickness ratio, compressive strength and Young’s modulus of the roof beam and the number, orientation, location and friction coefficient of the joints/fractures. In the calcarenite mines the fourth collapse mode is generally observed. The generally observed fracture pattern (Fig. 3.6) suggests that failure of the roof beam near the abutments is characterized by a combination of simple shear and dilatation (Ramsay & Huber, 1983). The shear component is considered here to result from the lateral compressive stresses in the roof beam, while the component of dilatation is attributed to the roof beam’s own weight. At present no reliable methods exist which take the particular collapse mode into account and describe roof stability in the calcarenite mines accordingly. Such methods could be developed by empirical research in the mines, in combination with numerical and laboratory experiments. The variation of the geomechanical parameters of the roof should also taken into account. Often roof beam thickness is difficult to assess.
3.3.3 Earth inflow from organ pipes

Generally the upper surface of the calcarenites is affected by karst. Solution pipes may extend tens of meters downwards into the rock. These pipes are normally filled with soil material, of a grain-size from clay to gravel, from the overlying layers and are then specified as organ pipes or earth pipes. Once intersected by a gallery, the filling may immediately flow into the mine, or many years later. Due to the earth inflow open space develops over some height of the solution pipe. It cannot be predicted when the opening reaches the top of the calcarenite. At this stage a sinkhole may form at the surface (e.g. Fig. 8.4). Since earth pipes are more or less vertical, the sinkhole can be expected to arise directly above its intersection underground. The earth inflow into the gallery stops when a cone has developed at an angle of repose of about 40° and reaching the roof. The volume of this cone can be used to estimate the volume of an eventual sinkhole. Pre-existing open space in karst structures might raise this volume. It should be noted that, due to the existence of such natural openings, sinkholes are not confined to undermined areas. In general earthflow is stimulated by heavy rainfall.

Evidently, the presence of organ pipes may locally reduce pillar or roof stability. On the opposite, particularly in the Sibbergroeve, the cementation of the calcarenite in a zone of some decimeters thickness around earth pipes is stronger than usual, resulting in a higher rock strength and building stone quality. Since the load carrying
capacity of clay is at least one order of magnitude less than that of calcarenite, it is believed that, overall, organ pipes weaken roofs or pillars.

3.3.4 Impact on large-scale stability

It can be inferred that roof instability and earth inflow from organ pipes represent only hazards which remain of local importance and do not threaten the stability of the rest of the mine. On the contrary, this does not apply to an unstable pillar. Experience has shown that deterioration of one or a few pillars can give rise to deterioration of more and more pillars, with a sudden collapse of a major part of the mine as a result. Because of the large scale and the serious consequences of such a catastrophic event, as is shown in the next sections, pillar instability must be considered as by far the most significant type of deterioration which may occur in a calcarenite mine. Accordingly, this thesis concentrates on the assessment of pillar stability.

3.4 INDIVIDUAL PILLAR INSTABILITY, LARGE-SCALE PILLAR INSTABILITY, GENERAL MINE INSTABILITY AND COLLAPSE

If the strength of an individual pillar decreases due to fracturing, load is partly transferred to the surrounding pillars, which may in turn also start to fail. Eventually this well known domino-effect (Hoek & Brown, 1980) may bring about pillar deterioration in large areas. In this way local pillar instability may induce large-scale pillar instability.

Now it depends on the interaction between pillars and mine environment whether large-scale pillar instability will give rise to a large-scale collapse: are the roof strata strong enough to protect the weakened pillars underneath against destruction. This is a question of general mine stability.

If neither large-scale pillar stability nor general mine stability are sufficient, a sudden, major collapse will occur: within a few seconds the whole area is destroyed. Such a large-scale collapse is obviously the most significant hazard for people inside the mine and also brings about considerable surface subsidence.
Fig. 3.8 Map showing the collapse areas of 1705 (3 ha), 1920 (7 ha) and 1926 (5.5 ha) of the mine workings excavated in the Kannerberg (compilation map from Stevenhagen; in Knubben, 1995).
3.5 ACCOUNTS OF LARGE-SCALE COLLAPSES

A brief description of significant large scale collapses in the past is presented here to reveal the characteristics of these events and their consequences.

3.5.1 The Fallenberg collapse of 1705

The Fallenberg is one of the calcarenite mines excavated in the Kannerberg-hill, to the south of Maastricht. At present five major mine workings can be recognized: the Fallenberg, the Boschberg, the Kannerberg, the Muizenberg and the De Keel mine (Fig. 3.8). On a map produced at the end of the 19th century the outlines of a collapse area of about 3 hectares in the southeastern part of the present Fallenberg is indicated. This area is accompanied by the date of 1705. Legend has it that baroness De Dopff, wife of the governor of Maastricht, and her two children were killed by the collapse, seated in their coach, when they took shelter in the mine from a rain storm (Schreiner, 1960). The age of the collapsed part is difficult to assess. According to Schreiner the exploitation must have started in the 17th century or earlier. Probably long-term deterioration by creep played a part in the collapse. The mine was originally known as the Jufferenberg or St.Lambrichtsberg (De Bruin,
1903) but was named the Fallenberg after this collapse. This collapse is studied in more detail in Chapter 10.

3.5.2 The St.Pietersberg collapses of 1794 and 1809

The collapse of 1794 in the northern mine workings, known as the Noordelijk Gangenstelsel, was generated in an unusual way. During a siege of the city of Maastricht by the French army it was decided to destroy the mine workings beneath the fortification of St.Pieter held by the Dutch and the Austrians. By means of a considerable amount of gunpowder a pillar was blasted triggering an underground collapse of about 3 hectares (Fig. 3.9). Where the pillar had been blown up a cavern was generated. This arch shaped cavern, known as the "Koepelgrot", is about 8 m high and shows a span of about 15 m. The explosion had occurred well to the south of the fortification and the fortification was not damaged. A historical study of Van Schaik (1942) revealed that a later collapse occurred in 1809, possibly due to the damage induced in 1794. The southern part of the Noordelijk Gangenstelsel and the northern part of the mine workings of the Zonneberg were destroyed. In total 9 hectares were affected. According to a French report of 1843 surface subsidence could still be recognized then. In 1813 the French troops placed explosives again, this time in the roof (Vennmans, 1985). Further movements were induced by the blasts in the area already damaged and in 1817 more galleries in the most northern part of the workings collapsed.

In 1929 and 1930 the collapsed area of 1809 within the northern mine workings was thoroughly investigated by Van Schaik for the benefit of the construction of a tunnel. An account of this adventurous study is presented in Van Schaik et al. (1983). The area, accurately mapped by the French between 1794 and 1796 before it collapsed, proved to be difficult to penetrate. It was just possible to clamber or crawl over the debris piles. At many locations the roof proved to have collapsed up to several meters above the original roof level. Sometimes an intact gallery was discovered surrounded by debris. In order to cross the collapsed area completely from west to east it was necessary to remove the debris at a few locations. Pillars had normally failed resulting in an hourglass shape and often the roof had collapsed forming an arch. According to Van Schaik pillars had been modified in shape but he wrongly assumed that they had maintained their original load carrying capacity.

3.5.3 The Gemeentegrot collapse of 1845

Fig. 3.10 shows the present extent of the Gemeentegrot near Valkenburg. A collapse of about 1.5 hectares in the eastern part of the present Gemeentegrot was reported by the mine inspector Büttenbach (Archives of the State Supervision of Mines; Knubbgen, 1995). Some days before the collapse miners had observed inauspicious signs and most of them had left the mine. On the 18th of December the remaining miners were alarmed by a violent cracking sound and fled immediately. A few moments later the collapse occurred. Nobody was injured or killed. A local journal
Fig. 3.10 The Gemeentegrot with the collapse areas of 1845 and 1886 (both of about 1.5 ha) and the northern collapse area of unknown extent (compilation map from Stevenhagen; in Knubben, 1995).
Fig. 3.11 Map depicting the mine workings of the Muizenberg and the deformation structures at the surface (reproduced from the Annual Report of the State Supervision of Mines (1926)).

of the 1st of January 1846 reported that at the surface an area of 800 m² had subsided and that cracks outside this area heralded additional ground movements. Additional subsidence occurred indeed on the 8th of January.

By some it was stated that the collapse had been the result of recent overmining, because at some locations older parts of the mine had been undermined by lower galleries. However, Büttgenbach stressed that due to mining activities in the past also older parts of the mine might have been already deteriorated before the collapse. Not only the recently excavated mine working had collapsed but also the older part of the mine, which had remained intact for more than a century. Hence it is quite possible that the collapse was brought about by both long-term deterioration of old mine pillars and recent overmining. Note that at that time no supervision existed in that mine and most other mines, and that, as a consequence, mine workers were not kept from widening galleries and narrowing pillars too much.
3.5.4 The Gemeentegrot collapse of 1886

Another large-scale collapse of about 1.8 hectares occurred in the southern part of the Gemeentegrot (Fig. 3.10). This collapse was described in the annual report of the State Supervision of Mines of 1886. It was stated that in the collapsed area excavation had ceased at least 30 years ago and that pillars were considered too small. Thus it appears that the collapse resulted from time dependent deformation of highly stressed pillars. Nobody was injured or killed. Damage at the surface was limited to a few small cracks of 2 to 5 cm width which soon disappeared.

3.5.5 The Fallenberg\(^1\) collapse of 1920

This is one of the largest collapses in the history of the calcarenite mines. About 7 hectares collapsed suddenly (Fig. 3.8). The annual report of the State Supervision of Mines (1920) reported extensive fractures with vertical displacements of about 10 cm at the surface above the mine. Several dolines had been formed due to flow of soil into underlying karst structures. People in the vicinity of the mine felt an earthquake and a pronounced thunder-like sound. The collapse must have been accompanied by a strong air blast because the calcarenite dust near the new entrance (Fig. 3.8) was completely flattened out and the numerous foot prints had vanished. The collapse area was blocked at its perimeter by rock debris. All pillars in the direct vicinity of the collapsed area had become heavily fractured. Large slabs had been formed along their walls.

The large-scale collapse was not unexpected. Near the northern entrance (Fig. 3.8) pillars were overstressed due to pillar robbing at the end of the mining period. Many galleries were too closely spaced. Schreiner reports that the Jesuits had observed deterioration of roof and pillars at several locations. A small-scale collapse had been mentioned by a local newspaper of 1903, which attributed the damage to a tram passing the Jeker Valley during that period. In 1911 the State Supervision of Mines had warned of roof instability at several locations (State Supervision of Mines, 1920).

A considerable time span exists between the end of the mining activities in about 1880 (Schreiner, 1960) and the collapse. Hence long-term deterioration must have been an important factor, together with overmining. Schreiner states that the three days preceding the collapse were accompanied by heavy rain fall. The resulting increased water content of the overburden layers and the increase of vertical pillar stress as a consequence might have triggered the collapse. This collapse is extensively studied in Chapter 10.

\(^1\) The intact, southern part of about 4 hectares is presently known as the Jezuïetenberg.
Le tragique éboulement de la champignonnière du Roosberg

Deux nouveaux affaissements de terrain arrêtent les recherches

La fête de Noël a été célébrée dans le deuil à Zichen-Zussen-Bolder; treize cadavres au moins sont encore ensevelis dans la champignonnière des Frères Heynen, après le tragique éboulement de mardi matin.

Comme on sait, des sauveteurs venus d'un peu partout en Belgique et même des Pays-Bas ont travaillé dans des conditions souvent dangereuses pour tâcher de ramener à la surface les corps. En effet, tout indicait que les victimes ont succombé et qu'à moins d'un miracle pas un homme ne sortira vivant des galeries éboulées.

Le Corps national de secours, appelé dès que la catastrophe fut connue, a quitté la commune mercredi, vers 18 h, mais des équipes de la Centrale de sauvetage des mines, de la protection civile et des pompiers continuaient les recherches.

A 20 h, une équipe composée de deux ingénieurs et de six hommes poursuivait ses efforts en descendant sous terre par des bouches d'aération.

A ce moment, des signes de nouveaux affaissements étaient perçus et on envisageait d'arrêter les travaux de sauvetage. Vers 21 heures, une décision dans ce sens était prise.

Entretemps, une nouvelle équipe de sauvetage, venant des mines de Houthalen, était arrivée sur les lieux.

Vers 23 heures, mercredi, un premier affaissement de terrain se produisit à nouveau. Jeudi, vers 2 heures du matin, il y eut un second.

Ces deux éboulements couvrent une superficie de plusieurs dizaines de mètres carrés.

Jeudi midi, les travaux ne s'arrêtaient pas encore repris. Les ingénieurs du Corps des mines et M. Rogge, gouverneur de la province, renvurent sur les lieux, étudiaient les décisions à prendre.

Fig. 3.12 One of the numerous Belgian newspaper articles describing the collapse of the Roosburg mine system in 1958 and its lethal consequences (Courtesy of Staatstoezicht op de Mijnen).

3.5.6 The Muizenberg collapse of 1926

The account of this collapse of about 5.5 hectares is based on the Annual Report of the State Supervision of Mines of 1926. Additional facts can be found in Breule (1984). The mine comprised two storeys. An upper level, 3 to 4 m thick and of modest extent (Fig. 3.11), had been mined more than a century ago and was used for mushroom growing. No map exists of this excavation, only its outline has been approximated. The main part of the mine was excavated in a lower level of 6 to 8 m thickness, separated from the higher level by a "taulwaal" of 1 to 1.5 m thickness. Only part of the lower mine working is indicated on maps (Fig. 3.11). Mine workers were still extracting calcarenite in this storey. The exploitation front at that time was located well to the north of the area about to collapse. Before the large-scale collapse fragments of pillar sides had fallen off and locally the layer separating the upper and lower storeys had collapsed.
The day before the collapse there was a rumour in the nearby village of Kanne that unusual ground movements were occurring in the mine.

The large-scale collapse occurred early in the morning of the 11th of May, when six workers were transporting the last calcarenite blocks out of the mine as fast as they could. At that time seven mushroom growers were standing in a brick shelter just beside the entrance. They heard a thunder-like sound coming out of the interior of the mine. Within one minute this noise suddenly increased tremendously in intensity and was accompanied by an earthquake. Then a strong air blast projected the seven men against the talus of the nearby road. Two of them were killed. Four of the six men inside the mine tried to escape but were stroke by the air blast which killed one of them. Both men who stayed inside the mine were buried together with their cart-horse. About one hectare in front of the entrance had been covered by dust. Bricks and beams from the constructions built near the entrance and even rock fragments from inside the mine had been projected forward and deposited in front of the entrance.

At the surface normal faults and subsidence troughs developed contouring the underground collapse from the southwest to the northeast. Several sinkholes formed due to flow of soil into underlying karst structures (see Fig. 3.11). However, it is not completely certain that these sinkholes did not already existed before the collapse. From the Cannerberg debris piles were observed at points A to F, which reached above the original roof level and blocked the entrance towards the collapsed area. A tunnel was made starting at point C in an attempt to rescue both men inside the collapsed area, but after a month this work had to be given up. Neither of them was found.

Considering that the collapse occurred in the older parts of the mine it can be hypothesized that long-term deterioration of the calcarenite has played a role. According to the report cited above the relatively low uniaxial compressive strength of 1.4 MPa for the calcarenite in the Muizenberg must have been an important factor as well.

3.5.7 The Roosburg collapse of 1958

The Roosburg is situated near the village of Zichen in Belgium, about 8 km to the southwest of Maastricht (Fig. 2.7). The abandoned mine was extensively used for mushroom growing which provided work for about 400 people. The account given below is based on newspaper-articles found in the Archives of the State Supervision of Mines in Heerlen (Fig. 3.12). On the 23th of December 4 hectares of the mine collapsed killing 18 workers and severely injuring 3. The large-scale collapse was preceded by significant roof collapses and fracturing and spalling of pillars. During the days before the collapse cracking sounds had been heard extensively by the workers in the mine. Against their own better judgement they continued harvesting the precious mushrooms. Just before the collapse 20 workers moved to another part of the mine at some distance of the area about to collapse, where cracking sounds
Fig. 3.13 The boundary of the 1958 collapse area in the Roosburg. Access into the area was only possible by means of a new artificially supported tunnel.

were less abundant. This saved their lives. Suddenly the sound of thunder was heard, pillar walls collapsed and dust and stones were projected through the galleries. They run towards the collapsed area and found a fatally injured woman hurtled from her bicycle against a pillar wall by the air blast. Seventeen workers were buried by the collapse. Six of them were found during rescue operations. The following days more collapses occurred affecting at least 2 hectares additionally and impeding the rescue parties. It appears from the accounts of the rescue parties that they could only penetrate through the collapsed area by creating a new open tunnel with extra support (Figs. 3.13 and 3.14). Only locally open space was encountered within the collapse. Some pillars, even in the centre of the collapsed area, were in reasonable state. Pillars were shortened considerably during the collapse and fragments from the pillar walls which had been pushed sideways had filled the galleries. Additional roof falls might have resulted in further blocking of the galleries.

The strong air blast which had killed one person projected the steel gate at the entrance 50 m away. The area in front of the entrance was covered with dust. Surface subsidence occurred over the whole collapsed area. The subsided area was bounded by normal faults.

Mining had ceased long before the collapse. Hence long-term deterioration of the rock must have been important.
Fig. 3.14 At the end of the about 25 m long tunnel, excavated in desperation nearly 40 years ago in an attempt to rescue the mushroom growers trapped by the collapse or recover their bodies.

3.5.8 The Heidegroeve collapse of 1988

In the Heidegroeve, situated near Valkenburg (Fig. 2.7), the most recent large-scale collapse occurred. At this occasion the results of a collapse could be studied, which were not obliterated during the passage of time. Surveys were undertaken after the event, both underground and at the surface. The following description of the immediate effects of the mine collapse is based on the work of Price (1990). A more extensive study of the collapse is presented in Chapter 9.
3.5.8.1 Subsidence features at the surface

On 23 June 1988 the seismograph in the Geulhemmergroeve (about 2.8 km from the mine) recorded an earthquake shock at 18h 13m 34 sec Dutch summertime. The shock lasted less than 45 seconds as indicated by the seismogram (Fig. 3.15). "Strong" motion ceased after about 10 seconds. The shock was also recorded by the seismograph at Epen, about 12.7 km distant, and perhaps at Kerkrade, about 17 km away. Shortly afterwards walkers in the Polferbos observed cracks in the footpaths and raised the alarm. An examination of the mine showed that the southeastern part, about 30 % (0.4 hectares) of the mine, appeared to have collapsed. Fig. 3.16 shows the limits of the collapse as seen underground and the ground shears bounding the surface subsidence. The collapse underground presumably extends to the southeastern limits of the mine. The presence of two shafts facilitated matching of underground and surface maps. Obviously the collapsed zone underground corresponds more or less with the subsidence area at the surface. As a consequence of the collapse rock dust and pieces of rock appeared to have been blown out of the mine to the other side of the Plenkertstraat. The plate steel door at the mine entrance was observed to be bent outwards but later inquiries indicated that this damage could wholly or partially be attributed to vandalism.

At the surface above the collapsed part of the mine normal faults partially bounded an elliptical area about 100 m long and 70 m wide (Figs. 3.17 and 3.18). The outer
Fig. 3.16 Map of the Heidegroeve showing the pillar damage, the limit of the underground collapse and deformation structures at the surface (modified after Price, 1989).
Fig. 3.17 Map of the surface deformation structures over the collapsed Heidegroeve (reproduced from Price, 1989).

Normal faults showed downward displacements of up to 0.8 m towards the centre of the subsidence area (Fig. 3.17). Their heave was up to about 0.25 m. Inner normal faults parallel to the outer faults showed lesser vertical and horizontal displacements. Their downward displacements were generally towards the outer normal faults, resulting in a graben-like structure in between the inner and outer faults. The inner normal faults were probably antithetic relative to the outer main faults.

A fence running alongside the main footpath was damaged in compression. Shearing of one part of the fence indicated horizontal shortening of about 10 cm in that part of the fence (Figs. 3.17 and 3.19).

The faults were fortunately confined to the wooded area, stopping about 30 m short of the restaurant to the south and not reaching the artificial "bob-sleigh run" to the east (Fig. 3.17). The soil exposed in the fault planes is composed of mainly silt (loess deposits) or a mixture of dominantly sand and clay (Tongeren Sands). The overburden in the subsidence/collapse area consists of 21-25 m of calcarenite covered by 15-30 m of soil.
Fig. 3.18 Normal faults with a downward displacement of up to 55 cm on the footpath just north of the restaurant in the background (Courtesy of D.G. Price).

Fig. 3.19 Wooden fence sheared by horizontal compression (Courtesy of D.G. Price).
Fig. 3.20 The boundary of the collapse area of the Heidegroeve blocked by debris.

About 15 m to the NNE of the easternmost shaft a circular depression about 1.5 m deep can be seen. It is not known if this was formed as a result of the collapse or already existed. However, it should be noted that other large-scale collapses, i.e. the collapse of the Muizenberg, were accompanied by the formation of sinkholes at its perimeter (Fig. 3.11). It probably represents a doline, resulted from the flow of soil into a solution pipe of the underlying calcarenite. The solution pipe did not reach the mined level as observed before the collapse by visitors of the mine.

3.5.8.2 Collapse features underground

A pillar survey was undertaken in September 1988 (Fig. 3.16). All galleries at the periphery of the collapsed area were blocked as result of roof falls (Fig. 3.20). In between the collapsed area in the southeast and the intact northwestern part of the mine a transition zone of about 50 m broad existed, in which pillar damage decreased gradually towards the entrance. Especially in the 4.6 m high workings severe pillar deterioration was encountered (Fig. 3.21). Several pillars could be classified as failed. Often brick cladding had parted from the pillar walls and fallen on the floor.
3.5.8.3 Evidence of long-term deterioration

Also here the factor time is important in the process of pillar deterioration preceding the collapse. The last mining activities occurred in the 1930’s. During World War II the Germans used the mine and made additional excavations, possibly reducing the stability in the long term. The Heidegroeve appeared to be safe until the 1980’s when a deterioration of the mine was observed, increasing significantly during the summer of 1987. The State Supervision of Mines inspected the mine weekly and observed additional pillar damage every visit (personal communication, W. Miserê). For example one week a new crack was noticed, a week later some mm. of movement had occurred along this crack, after two weeks the crack was extended and the whole side of the pillar had moved several millimeters into the gallery along that crack, and after three weeks the whole pillar side had fallen off on the floor.
Also fresh calcarenite dust was observed at several locations every week. The major part of the mine was declared unsafe and the mushroom grower who was working in the mine at that time stopped the exploitation. The Heidegroene mine was closed and abandoned. In June 1988, within one year, the southern part of the mine collapsed.

This case shows that a mine, being stable for many years, can deteriorate and collapse within ten years. It is known that there was heavy rainfall during the weeks preceding the collapse. The resulting increase in overburden weight might have triggered the collapse.

### 3.6 Hazards and General Characteristics of Large-Scale Collapses

It can be concluded that a large-scale collapse is accompanied by the following hazards:

- inside the collapsing area pillar and roof fragments fall into the galleries, burying people who happen to be there.

- a strong air blast is generated which may kill people in the mine outside the collapsing area. Also outside the mine, near the entrance, the air blast may be lethal.

- a collapse is almost immediately followed by subsidence at the surface. The perimeter of the subsided area often corresponds with the outline of the underground collapse and is generally at least partly characterized by normal faults and sinkholes. The normal faults may show horizontal and vertical offsets of several decimeters. Obviously such phenomena constitute a hazard to people and buildings.

Other characteristics, revealed by the accounts presented above, are:

- a large-scale collapse might be induced by overmining, i.e. reducing pillar size to much. At one occasion a collapse was generated by severe explosions.

- a considerable time span often exists between the end of the excavation and the large-scale collapse. Hence time-dependent deterioration of the mine system must be an important factor.

- a large-scale collapse of an already unstable mine system might be finally triggered by (additional) overmining or an increase of overburden weight due to a period of heavy rain fall. The importance of the last triggering mechanism has also been shown by Rode et al. (1989) for room and pillar mines in the Gironde, France.
an acceleration of pillar fracturing and at some cases even small scale pillar and roof collapses serve as precursors of a large-scale collapse. The time span between the observation of these signs and the collapse ranges from several years to at least some days.

finally the large scale collapse occurs suddenly, within a few seconds.

a collapse area measures up to several hectares.

inside the mine a collapse area is generally bounded by a pile of rock debris which often blocks access from the intact area of the mine.

intact pillars adjacent to the collapse area are sometimes loaded more severely, finally resulting in an additional collapse.
CHAPTER 4

REVIEW OF MODERN CONCEPTS OF PILLAR AND MINE STABILITY

4.1 INTRODUCTION

Comparing pillar strength ("capacity") with pillar stress ("demand") gives an assessment of pillar stability. In this chapter existing methods and concepts are presented which describe pillar strength as a function of the compressive strength of the rock mass and the size and shape of the pillar. Also the influence of discontinuities is dealt with. It must be stressed that, since these methods are based on short-term experiments, the deformation behaviour in the long term is not incorporated. Subsequently, methods of determining the stress on a mine pillar and their complications are outlined.

The evaluation of individual pillar stability is followed by an outline of the concepts of general mine stability, i.e. the stability of a mine system as a whole and the potential of a large-scale collapse.

Finally the missing data and the shortcomings of the existing techniques of stability assessment are analyzed, in particular with regard to the calcarenite mines. This serves to define the strategy of this study.

4.2 INDIVIDUAL PILLAR STABILITY

4.2.1 The concept of the safety factor

The stability assessment of an individual pillar is usually simplified by comparing its strength with the average vertical stress acting on it. For near-horizontal mine workings pillar strength \( \sigma_p \) (in MPa) is defined here as the maximum vertical load per unit area which a pillar can support. The onset of macroscopic fracturing is generally observed at peak stress or just after peak stress has been achieved. This peak stress is normally considered as strength. A sample which is deformed beyond peak stress is denoted as "failed". A typical stress-strain diagram of a constant strain rate compression test is depicted schematically in Fig. 4.1. Pillar stress \( \sigma \) (in MPa)
is denoted here as the average vertical load per unit horizontal pillar area. The ratio of pillar strength and pillar stress is a measure of pillar stability and known as the safety factor SF:

\[ SF = \frac{\sigma_p}{\sigma} \]  \hspace{1cm} (4.1)

A safety factor of one represents a critical situation. Generally only a safety factor well above one will be regarded as acceptable. Experience suggests that safety factors should lie between 1.5 and 2, but this may vary depending on the accuracy of the calculations and the use of the mine.

4.2.2 Pillar strength

4.2.2.1 General expression of pillar strength

The strength \( \sigma_p \) of a pillar can be expressed as a product of the uniaxial compressive strength on a laboratory scale and a function \( F(W,H,L) \), which relates size and shape of the pillar to those of the specimen tested in the laboratory (Goodman et al., 1980). Here \( W \), \( H \), and \( L \) are width, height and length of the pillar respectively (all in meters). Generally only width and height are taken into account. The uniaxial compressive strength is determined on a cylindrical core, which is at least twice as long as its diameter, or on a cubical sample. These tests give the unconfined compressive strength UCS or the cube compressive strength \( \sigma_c \) respectively (both in MPa). The function \( F \) is often divided in a strength-size relationship \( N_{size}(H \text{ or } V) \) and a strength-shape relationship \( N_{shape}(W,H) \). Here \( V \) denotes specimen volume,
which is equal to $H^3$ for a cubical sample. $F$, $N_{\text{shape}}$ and $N_{\text{size}}$ are dimensionless. If not, correction factors must be included, equal to unity and of the appropriate dimension. Now pillar strength is generally formulated as follows:

$$\sigma_p = UCS \times F(W,H) = UCS \times N_{size}(H) \times N_{shape}(W,H)$$  \hspace{1cm} (4.2)

Size and shape effect are depicted in Fig. 4.2. Experience shows that, in general, the strength of a rock body decreases with increasing size and decreasing width/height ratio $W/H$.

4.2.2.2 Background of the size effect

The strength reduction with increasing specimen size has been shown in experiments on various rock types (e.g. Bieniawski, 1968; Lama & Vutukuri, 1974). The size effect is ascribed to the fact that the number of flaws and other inhomogenities on a micro-scale and hence the probability of failure increases with the volume of the rock body, according to Weibull’s weakest link theory (Weibull, 1939). Additionally, discontinuities at a larger scale like joints and bedding planes affect the mechanical behaviour more prominently with an increase of specimen size. Often the concept of the "critical size" applies (e.g. see Lama & Vutukuri, 1974). With sufficiently large specimens the density and distribution of weakening structures is not affected any more by size. Therefore, from a certain "critical size" onwards a continued
Fig. 4.3 The effect of specimen size on UCS (from Goodman, 1989). For each material a critical size appears to exist such that no strength decrease occurs on further expansion of specimen size.

increase of specimen size does not bring about a further decrease in strength (Fig. 4.3). The types of inhomogenities and their distribution, and therefore also the size effect, vary according to rock type and rock mass structure.

4.2.2.3 Background of the shape effect

With regard to the shape effect, it has been recognized by various authors (i.e. Wilson, 1972) that the vertical pillar load is mainly supported by the central core, which is surrounded and confined by a failing outer zone. Hence pillar strength can be expected to increase with the expansion of the central core relative to the total pillar volume, i.e. with increasing W/H. The validity of this concept was demonstrated convincingly by Wagner (1974). In his experiments coal model pillars were loaded by 25 hydraulic jacks which expanded by a certain amount regardless of the resistance over each pillar section. It became clear that the resistance, measured at mid-height of the pillar, is least in the circumferential portions, particularly at the corners (Fig. 4.4). On approaching the peak load the sides and edges fail while the central core is loaded more intensely. Even when a pillar has failed, i.e. is deformed beyond its peak strength, the central core can withstand very high stresses.
4.2.2.4 Empirical size and shape formulae

Various empirical power law expressions were developed of the form:

\[ F = H^a W^b \]  \hspace{1cm} (4.3)

Since the constants \( a \) and \( b \) are not equal such expressions incorporate the effects of both size and shape. This can easily be shown (Hoek & Brown, 1982) by rearranging Eq. 4.3 as a product of \( W/H \) and volume \( V \):

\[ F = (W/H)^c V^d \]  \hspace{1cm} (4.4)

where

\[ c = (b - 2a)/3, \quad d = (a + b)/3 \]

The values of the pillar strength constants suggested by various authors are listed in Table 4.1. The constants apply to dimensions in feet. A correction factor, not equal to unity, should be applied to obtain an expression for dimensions in meters.
Table 4.1 Constants of Eqs. 4.3 and 4.4 determining pillar strength according to size and shape.

<table>
<thead>
<tr>
<th>Authors</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>Derived from:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greenwald, Howarth &amp; Hartman (1939)</td>
<td>-0.85</td>
<td>0.50</td>
<td>0.73</td>
<td>-0.117</td>
<td>in-situ tests on Pittsburgh coal pillars (USA)</td>
</tr>
<tr>
<td>Holland &amp; Gaddy (1957)</td>
<td>-1.00</td>
<td>0.83</td>
<td>0.83</td>
<td>-0.167</td>
<td>laboratory tests on West Virginia coal (USA)</td>
</tr>
<tr>
<td>Salamon &amp; Munro (1967)</td>
<td>-0.66</td>
<td>0.46</td>
<td>0.59</td>
<td>-0.067</td>
<td>statistical analysis of in-situ pillar failures in South African coal mines</td>
</tr>
<tr>
<td>Bieniawski (1968)</td>
<td>-0.55</td>
<td>0.16</td>
<td>0.42</td>
<td>-0.130</td>
<td>underground tests on coal specimen of 3-18 inch in South African coal mines</td>
</tr>
<tr>
<td>Singh &amp; Hedley (1981)</td>
<td>-0.75</td>
<td>0.50</td>
<td>0.66</td>
<td>-0.083</td>
<td>statistical analysis of in-situ hard rock pillar failures in Canada</td>
</tr>
</tbody>
</table>

Most expressions, such as Eq. 4.4, relate to coal mine pillar design. A considerable scatter exists in the proposed values of constants. Hustrulid (1976) reviewed size-strength relationships for coal suggested by various authors and found that the following general expression described all the data fairly well:

\[
N_{\text{size}} = \frac{1}{\sqrt{H}} \quad \text{for} \quad H \leq 91.5 \text{ cm}
\]

\[
N_{\text{size}} = \frac{1}{\sqrt{36}} \quad \text{for} \quad H > 91.6 \text{ cm}
\] (4.5)

This relationship is in agreement with the critical size concept outlined above. For calcarenite \( N_{\text{size}} \) has been assumed to be equal to unity as an approach (i.e. Price & Verhoef, 1988; Vreugdenhil, 1988), but this has not been confirmed by experiments.

The shape effect can often also adequately be described by an expression of the form:

\[
N_{\text{shape}} = A + B \left( \frac{W}{H} \right)
\] (4.6)

Most authors included the material strength into the strength-shape relationship. Hustrulid (1976) and Van Heerden (1975) normalized these expressions by dividing pillar strength by cube compressive strength \( \sigma_c \). In table 4.2 an overview is given of the major results.
Table 4.2 Constants of Eq. 4.6 determining the strength-shape relationship.

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>A</th>
<th>B</th>
<th>Derived from:</th>
<th>W/H range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baushinger (1876)</td>
<td>0.778</td>
<td>0.222</td>
<td>laboratory tests on sandstone prisms</td>
<td>0.5-2</td>
</tr>
<tr>
<td>Obert et al. (1946)</td>
<td>0.778</td>
<td>0.222</td>
<td>laboratory tests on cores of various diameter to length ratios of marble, limestone, granite, sandstone, slate, greenstone</td>
<td>0.5-2</td>
</tr>
<tr>
<td>Holland (1964)</td>
<td>0.775</td>
<td>0.225</td>
<td>tests on coal; alternative data fit by Hustrulid (1976)</td>
<td>2-6</td>
</tr>
<tr>
<td>Bieniawski (1968)</td>
<td>0.645</td>
<td>0.355</td>
<td>underground tests on coal specimen of 60 inch width in South African coal mines</td>
<td>1-2.5</td>
</tr>
<tr>
<td>Van Heerden (1975)</td>
<td>0.64</td>
<td>0.36</td>
<td>underground tests on coal specimen of at least 60 inch width in South African coal mines</td>
<td>1-3.5</td>
</tr>
</tbody>
</table>

The constants reported in Table 4.2 do not vary considerably. Moreover, they are determined not only for coal but for various rock types. This suggests that the shape effect is mainly a result of geometry and not of the properties of the rock material. Hustrulid analyzed the Eq. 4.6 type expressions and found that in every case the data fitted the following relationship quite well:

\[ N_{shape} = \sigma_p / \sigma_c = 0.778 + 0.222 \frac{W}{H} \]  \hspace{1cm} (4.7)

Goodman et al. (1980) used a revised version of Eq. 4.7 in which pillar strength is not related to cube strength but to UCS:

\[ N_{shape} = \sigma_p / UCS = 0.875 + 0.250 \frac{W}{H} \]  \hspace{1cm} (4.8)

In previous investigations of the calcarenite mines this formula was used (Price & Verhoef, 1988), but it could not be verified accurately by field data. Laboratory experiments on calcarenite prisms had not been performed.

4.2.2.5 Adjustments of shape formulae for irregular pillar outlines

The formulae for the calculation of pillar strength specified above utilize but two dimensions, width and height, and were intended to aid new pillar design. However, completed pillars are generally not square-based. For most pillars length is not equal to width. Often even more irregular and complex outlines exist, particularly in the calcarenite mines. Many authors (e.g. Holland & Gaddy, 1957) considered the minimum horizontal dimension as width. As outlined above, Wagner (1974) showed
that the load carrying capacity of a pillar is mainly achieved by its central core. The resistance of circumferential portions is much less and independent of W/H. Therefore it is not width which should be taken into account but the horizontal cross-sectional area $A$ relative to the pillar circumference $C$.

Accordingly, Wagner defined the effective width $W_e$ for a pillar of irregular shape, which is given by:

$$W_e = 4A/C$$  \hspace{1cm} (4.9)

For a square-based pillar $W$ is $W_e$. It should be noted that for long and narrow rib pillars the effective width $W_e$ approaches a value of twice the minimum horizontal dimension $W$. Bieniawski (1968) had measured that the strength of model coal pillars indeed increases with increasing length. It is however uncertain if the influence of length on strength is actually so strong as inferred by Wagner. No thorough experimental evidence exists on this point.

Note that the smallest horizontal dimension can still be used as pillar width in shape formulae to establish a conservative strength value. In former studies (Van Steveninck, 1987; Vreugdenhil, 1988; Dirks, 1990) pillar strength in the calcarenite mines has been calculated by averaging the width of the pillar, if it is more or less rectangular, or by splitting up the pillar into approximately rectangular units.
Fig. 4.6 Percentage of continuous material strength for aerated concrete prisms with vertical discontinuities at various locations relative to pillar geometry (from Vreugdenhil, 1988).

4.2.2.6 The squat-pillar shape formula for pillars of large width/height ratio's

The classic shape formulae described so far generally apply to pillars of width/height ratio's of less than 4 or 5. These relationships indicate that the pillar strength increases linearly with W/H or according to a power law. In the latter case the rate of strength increase for greater width/height ratio's even decreases. It was recognized by Wagner & Madden (1984) that pillar strength increases more strongly due to the exponential increase of lateral confinement at higher width/height ratio's. They analyzed laboratory tests on sandstone specimens and found that for values of W/H, here denoted as R, less than the so-called critical width/height ratio R₀, the usual formula $\sigma_p = k V^a R^b_0 \left( \frac{R}{R_0} \right)^\varepsilon + 1$ applies. However, for pillars of $R > R_0$ the so-called squat-pillar formula is valid:

$$\sigma_p = k V^a R^b_0 \left( \frac{R}{R_0} \right)^\varepsilon - 1 + 1$$

(4.10)

where

- $R_0 =$ the critical width/height ratio, i.e. the width/height ratio at which the increase in strength starts to accelerate
- $V =$ pillar volume
- $\varepsilon =$ the rate of strength increase ($\varepsilon > 1$, normally $\varepsilon=2.5-4.5$)
- $a = 0.067$
- $b = 0.59$
k = cube compressive strength $\sigma_c$ (in MPa; adjusted to the given geomechanical setting, if necessary; see for example Brady & Brown (1994), Stacey & Page (1984))
All dimensions in meters

By a comparison with Table 4.1 it can be verified that the first part of this equation is similar to the shape and size function of Salamon and Munro. Values of 4 to 4.5 for $R_0$ and 4.5 for $\epsilon$ were found for the model studies, but Wagner & Madden proposed to use more conservative values with $R_0$ equal to 5 and $\epsilon$ as 2.5. In Fig. 4.5 the difference in pillar strength is shown according to the equation of Salamon & Munro and the more recent squat-pillar formula. Measurements in South African coal mines (Madden, 1991) and a probabilistic analysis of collapsed and stable coal mine workings in Australia (Galvin et al., 1995) seem to support the concept of a squat-pillar equation.

4.2.2.7 The influence of discontinuities on pillar strength

The size and shape equations do not incorporate large structural weakness zones like earthpipes, non-periodical joint discontinuities etc. Not much research has been performed on this subject. When joints are present, an orientation at $45^\circ - \phi/2$, which is commonly about $30^\circ$, with the pillar axis is the least favourable. The symbol $\phi$ denotes the angle of internal friction of the pillar rock. But normally it is not necessary to consider this situation in the calcarenite mines because nearly all joints are (sub)vertical. The amount of weakening not only depends on the properties of the discontinuity, like roughness and infill material, but also, for a major part, on the position of the joint relative to pillar geometry.

Vreugdenhil (1988) performed a series of laboratory compression tests on model pillars of aerated concrete. He used square-based prisms of 10*$10*6$ cm (W/H = 1.67), placed between platens of the same material. The prisms had been vertically split in two prior to the experiment at different locations relative to the pillar geometry. The results are summarized in Fig. 4.6. A reduction in strength of about 10 % resulted when the discontinuity was situated at 9/10 of the pillar width. Apparently the thinner segment of 1/10 pillar width spalled from the main pillar on loading and did hardly contribute to pillar strength. Hence the load carrying capacity was finally determined by the remaining 9/10 of the pillar. The location of a discontinuity more towards the centre of the pillar brought about less significant reductions in strength. This can be explained by the strong lateral confinement in the pillar core. It can be imagined that the presence of joints affects pillar strength even less when W/H exceeds the value of 1.67 applied in these experiments. Finally, it should be noticed that this is not a major aspect because the Maastrichtian calcarenite is, relative to the dimensions of most mine pillars, generally unjointed.
Table 4.3 Typical test values for the limestones of the Formation of Maastricht from the quarry of 't Rooth (from Lap et al., 1987).

<table>
<thead>
<tr>
<th>Property</th>
<th>Meerssen</th>
<th>Nekum</th>
<th>Emael</th>
<th>Schiepersberg</th>
<th>Gronsveld</th>
<th>Valkenburg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturated unit weight (kN/m³)</td>
<td>18.0</td>
<td>21.3</td>
<td>18.3</td>
<td>19.3</td>
<td>21.0</td>
<td>19.7</td>
</tr>
<tr>
<td>Porosity (%)</td>
<td>47</td>
<td>40</td>
<td>52</td>
<td>45</td>
<td>48</td>
<td>47</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (MPa) *</td>
<td>1.39</td>
<td>2.53</td>
<td>1.54</td>
<td>2.06</td>
<td>1.93</td>
<td>4.22</td>
</tr>
<tr>
<td>Brazilian tensile strength (Mpa) *</td>
<td>0.22</td>
<td>0.85</td>
<td>0.47</td>
<td>0.56</td>
<td>0.46</td>
<td>1.17</td>
</tr>
<tr>
<td>Secant E-modulus at 50 % of UCS (GPa) *</td>
<td>0.26</td>
<td>1.40</td>
<td>0.38</td>
<td>0.37</td>
<td>0.52</td>
<td>0.96</td>
</tr>
<tr>
<td>Poisson’s ratio (secant modulus at 50 % of UCS) *</td>
<td>0.13</td>
<td>0.07</td>
<td>0.18</td>
<td>0.11</td>
<td>0.11</td>
<td>0.15</td>
</tr>
<tr>
<td>Slake durability index</td>
<td>16</td>
<td>26</td>
<td>40</td>
<td>28</td>
<td>39</td>
<td>29</td>
</tr>
</tbody>
</table>

* tested unsaturated, normal to bedding (moisture content not measured)

4.2.2.8 Determination of UCS values

Whichever formula may be used to determine pillar strength it will incorporate unconfined compressive strength of the pillar rock material as one of the most important parameters. The UCS is determined on cylindrical cores of 4 or 5 cm diameter and with a length/diameter ratio of 2 to 2.5. Geotechnical properties of the Maastrichtian calcarenites including UCS have been reported to show a large variation per lithostratigraphical unit (Table 4.3).

Moreover, the UCS may show significant variation within a certain lithostratigraphical level, both parallel and perpendicular to the bedding. For example, Vreugdhenhil (1988) found that UCS-values from 24 sample locations within the same stratigraphical level in the Geulhemmer Mine range from 2.32 to 3.97 Mpa. This number of locations within an area of 10 hectares represents only 2.4 sample locations per hectare. The variation of UCS on a smaller scale was not known.

Finally, the influence of varying moisture content on the strength of the calcarenite was not known, except for the difference in strength between dry and saturated specimens.

4.2.2.9 Time-dependent decrease of pillar strength

Most functions describing pillar strength are based on short-term experiments at a constant strain rate, which were performed within a couple of minutes. Therefore conclusions can be drawn at most as to whether a pillar will fail or not immediately
after excavation, which lasts several days. In reality a pillar which remains intact immediately after its creation may fail tens or hundreds years later. This may occur because the pillar is continuously exposed to a more or less constant vertical stress. The pillar deforms further, a process known as creep, and, although the stress acting on it is less than its short-term strength, failure may result eventually. Thus, due to time dependent deformation the "true" pillar strength in the long term may be less than its short-term strength. For example Wagner (1974) commented that for coal pillars the classic Salamon & Munro formula (1967) predicts about 50 % lower strength than formulae established on the basis of short-term experiments on model pillars. Therefore the factor time should be incorporated in pillar strength formulae. This aspect will be studied in more detail in Chapters 7 and 8.

4.2.2.10 Residual strength of failed pillars

Pillars which have safety factors of less than one are often considered not to carry any load (e.g. Goodman et al., 1980). In reality such pillars still have a residual strength and carry some of the overburden pressure. No strength formulae have been developed yet for failed pillars. It is known that pillars of small W/H, i.e. one or less, do hardly show any residual strength, while less slender pillars are still able to support some load (e.g. Fig. 4.7). The considerable load carrying strength of the pillar core, even after failure, suggests that for very large width/height ratio's, and relatively large pillar cores as a consequence, residual strength approaches peak strength.
4.2.3 Pillar stress

4.2.3.1 The tributary area method

Pillar stress $S$ is meant to be the average vertical stress within the uppermost horizontal section of the pillar. The pillar stress is normally calculated using the tributary area method:

$$\sigma = \sigma_{ov} \times \frac{A_t}{A_p} \tag{4.11}$$

where $P$ is the vertical stress of the overburden at the level of the roof of the opening, $A_t$ is the tributary area, i.e. the area of overburden supported by the pillar, and $A_p$ is the horizontal cross sectional area of the pillar (Fig. 4.8). The overburden stress is equal to the sum of the vertical stresses exerted by the individual overburden layers:

$$\sigma_{ov} = \sum \gamma_i \cdot h_i \tag{4.12}$$

where $\gamma_i$ and $h_i$ are respectively the unit weight and thickness of successive overburden layers.

4.2.3.2 Complications in determining overburden stress

The overburden stress is often difficult to determine. In South Limburg the overburden is usually composed of some thickness of calcarenite and thence
Table 4.4 Unit weights of overburden materials (compiled from Grabandt et al. (1983) and Kronieger (1988;1989).

<table>
<thead>
<tr>
<th>Formation</th>
<th>Mainly composed of:</th>
<th>Dry Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Range</td>
</tr>
<tr>
<td>Upper Pleistocene Loess</td>
<td>silts</td>
<td>18-20</td>
</tr>
<tr>
<td>Pleistocene Maas Deposits</td>
<td>sands and gravels</td>
<td>18-21</td>
</tr>
<tr>
<td>Lower Oligocene Tongeren Sands</td>
<td>sands and clays</td>
<td>14-18</td>
</tr>
<tr>
<td>Maastrichtian Limestones (Maastricht facies)</td>
<td>calcarenites</td>
<td>13-16</td>
</tr>
</tbody>
</table>

overlying sand, gravel, clay etc. There is a significant contrast between, and to a lesser extent within, the unit weights of the calcarenites and the individual soil units (Table 4.4).

There is evidence from borehole measurements that the relative thicknesses of the individual overburden layers are far from uniform. The density of boreholes is low. Normally not more than five boreholes, in many cases not even one, have been made within a radius of one km from the mine system boundaries. The variation in unit weights of the individual overburden layers combined with the lack of knowledge of the exact nature and thicknesses of the overburden materials can easily give rise to an error in the estimation of overburden stress of ±10%.

In this respect also the moisture content of the overburden should be considered. Due to moisture saturation unit weights of the soils can increase with 10 to 20% relative to the values above. It was shown in Chapter 3 that some large-scale pillar collapses were probably triggered by extreme and continuous rainfall.

The calcarenites have a porosity of between 40 and 50%. The natural moisture content measured on samples taken in the mines is usually of the order of 7 - 10%, but values of up to 20% could occur beyond the vicinity of mined areas. However, Waverijn (1990) concluded that groundwater flow within the calcarenites is concentrated along joints, faults and flint layers. It can expected that, above the water table level, most water is effectively drained through the discontinuities and will not saturate the calcarenite itself. Hence a significant increase in unit weight of the calcarenites above a mine is not probable. Only periods of extreme and continuous rainfall could in theory raise the moisture content of overlying calcarenites for a short time.
Fig. 4.9 Influence of pillar height (a) and pillar width (b) on convergence.

4.2.3.3 Pillar stiffness and local mine stiffness

The actual stress distribution over the individual pillars is more complex than assumed in the tributary area method. The actual load or stress on a pillar is also dependent on the local mine stiffness $\kappa$ and the pillar stiffness $\lambda$ (both in N/m). The latter term describes the amount of convergence $s$ (in m), i.e. pillar shortening, under a certain load $p$ (in N) and is given by (Galvin et al., 1994):

$$\lambda = \frac{p}{s} = \frac{\sigma A_p}{\epsilon H} = \frac{E A_p}{H}$$  \hspace{1cm} (4.13)

where $\epsilon$ is the vertical strain of the pillar and $E$ the Young’s modulus of the pillar rock mass. For a square-based pillar Eq. 4.13 can be written as:

$$\lambda = E \frac{W^2}{H}$$  \hspace{1cm} (4.14a)
Fig. 4.10 The concept of local mine stiffness, by replacing a pillar by a jack (a), and the characteristic force-convergence line (b) (modified after Hoek & Brown, 1981).

Pillar shortening can be written as:

\[ s = \sigma_{ov} \frac{H}{E} \frac{A_f}{A_p} \]  \hspace{1cm} (4.14b)

These equations show that pillar shortening increases with a decreasing W/H. For instance, when a certain pillar is doubled in height it tends to be compressed to a double amount (Fig. 4.9). This also applies to a pillar which is halved in area, if its tributary area remains the same. However for the last case, if the dimensions of the neighbouring pillars remain constant, the tributary area will decrease somewhat as well.

Whether or to which extent this will actually occur depends on the local mine stiffness \( \kappa \). For example, when the stiffness of the surrounding strata is high, the
Fig. 4.11 Schematic interaction diagram after the principle of Brady (1979). The pre-failure load-convergence characteristics of mine and pillar at the central pillar location for various panel widths are based on an interpretation of data from Galvin (1995). It can be observed that local mine stiffness $k$ decreases on increasing the total mine span. As a result, the equilibrium load $p_p = -p_m$ on the central pillar increases.

Load on relatively "soft" pillars tends to be relieved and transferred towards stiffer pillars or abutments. Finally, note that the E-modulus, which is a material parameter, is independent of specimen or pillar shape. This was confirmed experimentally by Wagner (1974). Accordingly, in a stress-strain diagram, as opposed to a load-convergence graph, the pre-failure slope is identical for different shapes.

Starfield & Fairhurst (1968) explained the concept of local mine stiffness by replacing a pillar by a jack (Fig. 4.10a). When, starting from an initial local mine support load $p_{m,0}$ (which is here defined negative), the jack is slowly retreated, the load $-p_m$ will decrease (i.e. $p_m$ will become less negative) and convergence will occur. If the roof remains intact the jack force-convergence curve can be characterized as shown in Fig. 4.10b. If the roof might fail jack force and convergence are described by the dashed line. The slope $-\kappa$ of the force-convergence
curve defines the local mine stiffness $\kappa$ and is determined by local mine geometry, i.e. the total mine span $w$, the total thickness of the roof rocks $h$, the location, size and shape of the adjacent pillars and abutments, the location of the jack relative to the other pillars, and by mechanical properties, particularly Poisson's ratio $\nu$ and elastic modulus $E$, of roof, floor and surrounding pillars. It can be imagined that $\kappa$ will be minimal for a large total mine span. Also a rock overburden which is thin relative to mine span will tend to flex more easily, resulting in a small $\kappa$. Finally, notice that $\kappa$ is truly local, i.e. its value varies throughout a mine. Near a massive, stiff abutment $\kappa$ will be larger than in the middle of an extensive mine working with slender pillars.

4.2.3.4 Actual stress distribution over individual pillars

The lower the local mine stiffness $\kappa$ the more a pillar carries the full overburden load according to tributary area theory. A high ratio of total mine span $w$ to rock overburden thickness $h$ is a situation where tributary area theory must be implemented. This also applies to an extensive mine working comprising pillars of equal size and shape (Salamon, 1970; Galvin, 1995). In a rigid mine environment, e.g. a narrow mine working with a thick rock overburden, the roof strata are capable of arching across the mine and the load on a pillar is partly transferred towards the abutments.

While pillar stiffness $\lambda$ can be relatively easily estimated based on compression tests on model pillars, the assessment of local mine stiffness $\kappa$ is more complex. Analytical methods applicable to relatively simple mine geometries like arrays of long pillars, were developed by Salamon (1970) and Starfield & Wawersik (1972). Local mine stiffness was determined numerically for example by Brady (1979) and Gill et al. (1994). An example from Brady (1979) of how to apply $\lambda$ and $\kappa$ in order to determine pillar stress for pre-failure conditions was presented by Hoek & Brown (1981) by means of an interaction diagram. Such a diagram, modified after a calculation of Galvin (1995) for different pillar arrays, is schematically shown in Fig. 4.11. Both local mine support load $p_m$ and pillar load $p_p$, acting in opposite directions, are considered. The performance of a pillar in the centre of a mine working and its local mine environment is depicted by their respective load-convergence lines. The slope of the pillar load-convergence line, equal to the pillar stiffness $\lambda$, is positive because pillar shortening and local mine convergence increase with $p_p$. The slope of the load-convergence line of the local mine environment, equal to $-\kappa$, is negative (the local mine stiffness $\kappa$ is defined positive) because an increase of $-p_m$ induces a decrease of mine convergence. The intersection of these lines gives the equilibrium condition and thus stress and shortening of the pillar. The total mine span $w$ is varied and accordingly the number of pillars giving different slopes of the mine reaction curve. Pillar and gallery width are similar for each mine span. It can be observed that $\kappa$ is reduced on extending the mine working. As a consequence, pillar load approaches the value predicted by tributary area theory.
Table 4.5 Pillar classification system after Van Steveninck (1987).

<table>
<thead>
<tr>
<th>Class</th>
<th>Description of pillar condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No cracks</td>
</tr>
<tr>
<td>2</td>
<td>Cracks only at the top or at the bottom of the pillar</td>
</tr>
<tr>
<td>3</td>
<td>Cracks in both top and bottom of the pillar</td>
</tr>
<tr>
<td>4</td>
<td>Cracks going completely from top to bottom of the pillar</td>
</tr>
<tr>
<td>5</td>
<td>&quot;Failed&quot; pillar</td>
</tr>
</tbody>
</table>

Gill et al. (1994) showed how to perform such an analysis quantitatively for arrays of multiple and non-uniform pillars. However, he did not take into account that the actual load-convergence curves, particularly for pillars, are not linear. Additionally, the analysis of pillar deformation only relates to a pre-failure situation. When pillars fail, shortening increases substantially and pillar behaviour will be much more complex to calculate (see Section 4.2).

It is generally considered that the tributary area method is correct if the mine working is extensive in all directions and the pattern of pillars is not too irregular (e.g. Salamon, 1970; Abel et al., 1988; Galvin, 1995). In most other cases this method is conservative, i.e. pillar stress is overestimated. On the other hand, pillars which are considerably wider and stiffer than the surrounding pillars, might be stressed more severely than predicted by tributary area theory.

4.2.4 Visual assessment of pillar stability

The only obvious evidence of incipient pillar instability is the presence of fractures in the pillar walls. One of the aims of past research in the calcarenite mines has been to try to match the observed condition of pillars, as determined by fractures, in a semi-quantitative way with the results of a stability analysis. To this end Van Steveninck (1987) developed a pillar classification system with 5 classes (Table. 4.5). The description "failed" was specified to mean that one or more pillar sides have been parted from the pillar core, resulting in an hour-glass shaped pillar geometry. This terminology is somewhat confusing because failure starts at peak strength. Hence class 2 pillars are, strictly spoken, failed as well. The author suggests that a class 5 pillar should be designated "completely failed".

4.3 GENERAL MINE STABILITY

Until here this chapter only dealt with the circumstances which bring about failure of an individual pillar. In Sections 3.2 and 3.4 it was explained that individual pillar
instability may give rise to large-scale pillar instability, i.e. pillar deterioration all over the mine. Methods to assess large-scale pillar stability do not exist yet but will be developed in Chapters 6 and 7. This section deals with general mine stability, the consequences of widespread pillar failure with regard to the stability of the mine system as a whole. As outlined in Section 3.4, a sudden large-scale collapse becomes imminent when both large-scale pillar stability and general mine stability are insufficient.

4.3.1 The concept of yielding pillars and a pressure arch

After failure the load carrying capacity of a pillar decreases to a considerable extent, depending on its geometry. It tends to be shortened significantly and to fail eventually. But when the roof strata are capable of arching across failed pillars, load can be transferred towards the abutments or eventually towards stiffer, intact pillars in their vicinity. This is the concept of a pressure arch which is considered to develop over yielding, i.e. failing, pillars (Adler, 1973; Barrientos & Parker, 1974; Jeremic, 1985). The pressure arch maintains its load transferring ability up to a
certain span known as the maximum pressure arch (Adler, 1973). The maximum pressure arch is observed to increase with depth. Fig. 4.12 depicts the principle of a pressure arch and its maximum width as a function of depth. Unfortunately no data are available for depths of less than 50 m, which apply to the calcarenite mines studied here. According to the pressure arch theory the yielding pillars are only loaded by the rock mass within the arch above them and the weight of the overburden above the arch is transferred to the relatively rigid abutments. Various shapes of a pressure arch are proposed, e.g. ellipsoidal (Adler, 1973; Barrientos & Parker, 1974), triangular or trapezoidal (Jeremic, 1985).

By allowing the yielding of pillars more material can be excavated resulting in an increased profit of the mining activities, but the yielding pillars must retain sufficient residual strength to prevent major inelastic closure, i.e. failure of the overburden and large-scale pillar collapse. In this regard also the ratio of mine span w to depth h and the geomechanical properties of roof and floor have to be taken into account.

4.3.2 Models of large-scale collapses

Esterhuizen (1990) observed that large-scale collapses in South African coal mines of the room and pillar type occurred by caving of the overlying strata up to the ground surface. The caved overburden was bounded by collapse induced faults, which were inclined at so-called cave angles ranging between 60° and 75° (Fig. 4.13). It was shown numerically that the stress on the barrier pillars\(^1\) at the perimeter of the collapse increased at a decreasing cave angle. Esterhuizen also concluded that, when a pillar collapse is not immediately followed by caving, the stress on the adjacent barrier pillars, abutments or intact mined areas in general must increase dramatically. As caving develops these stresses will decrease, because the

\(^1\) Barrier pillars are pillars of such a size that they are capable of withstanding elevated stresses which arise due to an adjacent collapse. In this way mined areas at the other side of the barrier pillar are protected and the domino-effect of pillar collapse is stopped, at least for a certain time interval.
Fig. 4.14 Schematic model of large-scale pillar collapse, based on base friction model experiments performed by Venmans (1985). As a result of pillar failure, the mine roof deflects, with tensitional fractures as a consequence (denoted by 1). Finally the overburden collapses, not along the initially formed tensitional fractures, but along less inclined tensitional shear fractures (2). The thin arrows indicate the shear and extensional components of fracturing.

Load of the caved strata become to rest on the collapsed pillars and debris situated in between. This phenomenon was clearly measured in-situ by Barrientos & Parker (1974). It can be imagined that after a major collapse a stable situation develops because of the increase of load carrying area and a more favourable shape of flattened, collapsed pillars. Moreover, these pillars are at least partly laterally confined and supported by pillar and roof debris. Finally debris material might directly support the roof.

In Fig. 4.14 a model of large-scale collapse is presented, which is established on the base of base friction model tests, performed by Venmans (1985). He observed that initially tensile fracturing occurred of the deflecting mine roof, at the top of the rock overburden near the abutments and at its base in the centre of the mine working. The eventual collapse occurred along less inclined fractures, which showed components of extension and shear. These fractures result from the compressive arching stresses in the calcarenite overburden. However, it should be noticed that base friction experiments have many practical complications and that results must be interpreted carefully.

4.3.3 Criterion of general mine instability in terms of post-peak pillar stiffness and mine stiffness

4.3.3.1 Introduction

It has been outlined above that under certain conditions an area of failed pillars might still be stable, for example if the area is not too wide relative to rock
overburden thickness. If conditions are unfavourable such an area becomes unstable. Suddenly the overlying roof mass comes down and pillars collapse. Whether this happens or not can also be well described in terms of local-mine stiffness and post-peak stiffness of pillars.

It has been recognized by various authors (e.g. Bieniawski et al., 1969; Rummel & Fairhurst, 1970; Hudson et al., 1972) that during an uniaxial compression test the violence and completeness of failure of a rock specimen depends on the relationship between the stiffness of the sample and that of the testing machine. The situation is depicted in Fig. 4.15. The load-convergence characteristics of the rock specimen are described by the function $F(s)$ and the loading machine is depicted as a spring with a spring constant (stiffness) $k$. The function $F(s)$ has a positive slope $\lambda$ until peak-stress, i.e. an increase in strain is associated with an increase in resistance. This behaviour is denoted strain-hardening. After peak-stress the performance of the failed sample is characterized by a negative slope $\lambda$ of $F(s)$. Now an increase in strain is accompanied by a decrease in resistance. This phenomenon is known as strain-softening. It is the post-failure part where instability might occur. Failure will proceed in a stable way only as long as the work done by the spring during an increment of displacement is smaller than the work required to cause the same displacement of the specimen. Otherwise, the excess elastic strain energy released
by the spring will crush the specimen. The condition for non-violent failure is accomplished when the stiffness of the spring (slope \( k \); \( k \) itself is positive per definition) exceeds the minimum, i.e. most negative, slope \( \lambda \) of the post-failure curve:

\[
k + \lambda > 0
\]  

(4.15)

It can be inferred from Fig. 4.15 that \( \lambda \) is positive in the pre-failure range. Since \( k \) is positive per definition Eq. 4.14 indicates that stable equilibrium will always occur before failure of the sample. Stability will also be maintained after failure up to the point at which the load-convergence line of the spring becomes tangent to the post-failure curve of the specimen.

Salamon (1970) used the analogy between a uniaxially tested rock sample and a loaded individual pillar and showed that the same condition applies to stable pillar failure. Now \( k \) becomes \( \kappa \), the local mine-stiffness, and \( \lambda \) is the minimum (most negative) slope of the load-convergence curve of the pillar.

4.3.3.2 The application of local mine stiffness coefficients \( \kappa_i \)

Starfield & Fairhurst (1968) utilized the inequality Eq. 4.15 as a condition for global mine stability. According to this concept the complete mining layout will be stable if for all pillars \( i \):

\[
\kappa_i + \lambda_i > 0
\]  

(4.16)

Note that the value of \( \kappa_i \) at a certain pillar may change in time, for instance when another pillar in the vicinity fails. Hence Eq. 4.16, expressed at a given time, does not necessarily exclude instability at an increase of pillar convergence.

4.3.3.3 The concept of mine structural stiffness with coefficients \( k_{ij} \)

Another approach to mine stability analysis was presented by Salamon (1970). The following relationship between load and convergence at pillars was derived:

\[
P = K (\Gamma - S)
\]  

(4.17)

Here, \( P \), \( \Gamma \) and \( S \) are \( n \times 1 \) column matrices, while \( n \) is the number of pillars. \( P \) gives the loads on the individual pillars, \( \Gamma \) describes the convergence at individual pillar locations which would arise by mining the excavation without pillar support, and \( S \) represents the actual convergence at each pillar. \( K \) is an \( n \times n \) square matrix, named the stiffness matrix. The elements \( k_{ij} \), not equal to \( \kappa_{ii} \), of \( K \) relate the load on an arbitrary pillar \( i \) to the convergence at another pillar \( j \). It can be shown that, as long as roof, floor and abutments continue to behave elastically, the stiffness coefficients \( k_{ij} \) remain constant during increasing convergence and eventual pillar failures. Hence
Fig. 4.16 Post-peak stiffness, $-\lambda$, normalized with respect to the pre-peak elastic modulus, $E$, as a function of width to height ratio $W/H$ (after Ozbay, 1989).

It is not necessary to adapt the coefficients continuously. This makes the approach of Salamon suitable for analytical stability calculations of not too complex mine structures. Additionally, matrix $\Lambda$ is an $n \times n$ diagonal matrix with elements $\lambda_{ii}$, which are the minimum stiffnesses of each pillar. Salamon showed that a stable equilibrium is accomplished if the symmetric matrix $K + \Lambda$ is positive definite:

$$K + \Lambda > 0 \quad \text{\textit{(4.18)}}$$
4.3.3.4 Critical stiffness

A conservative, but nevertheless practical, approach is that of perfect stability. Salamon (1970) demonstrated that uncontrolled collapse will never occur, regardless of the amount of convergence experienced by the pillars, if:

$$\lambda_m > \lambda_c$$  \hspace{1cm} (4.19)

where $\lambda_c$, denoted as the critical stiffness, is the smallest eigenvalue of matrix $K$ and $\lambda_m$ is the minimum (most negative) load-convergence slope of all pillars. The critical stiffness represents the worst case scenario, i.e. all pillars might be failed. The advantage is that, if $\lambda_c$ has been determined, it can immediately be compared with several values of $\lambda$.

4.3.3.5 Experimentally determined post-peak pillar stiffness

Regarding the negative, post-peak, values of pillar stiffness $\lambda$, it can be inferred for example from Fig. 4.7 that the post-failure slope decreases with increasing W/H. This has been clearly demonstrated for coal pillars by Wagner (1974), Van Heerden (1975) and other authors. The results of an inventarisation of this effect for various rock types by Ozbay (1989) are depicted in Fig. 4.16. The post-peak stiffness $\lambda$ was normalized with respect to the initial pre-peak E-modulus and plotted against W/H. It was found that the value of $-\lambda/E$ decreases with increasing W/H at more or less the same rate for all experiments. The constants of this equation, showed in the figure, must apparently be regarded in meters for dimensional correctness. The scatter of the data is attributed to the differences in experimental techniques (e.g. end constraints) and rock types. The most important conclusion is that for W/H ratio's between 4 and 7 $-\lambda/E$ shows a transition from positive to negative values. Hence pillars with a W/H ratio exceeding about 5 do not experience post-failure strain-softening. Such pillars cannot fail in an unstable way.

4.3.3.6 An example of numerically and analytically determined critical mine stiffness

Ozbay (1989) determined numerically, by means of a boundary element computer programm, critical stiffnesses for two-dimensional pillar layouts. An approach comparable to that of Salamon was chosen in that coefficients were defined which relate convergence $s_i$ at a certain pillar $i$ to the forces $p_j$ at all pillars $j = 1, 2, \ldots, n$:

$$s_i = \sum_{j=1}^{n} c_{ij} p_j$$  \hspace{1cm} (4.20)

This equation can also be written in matrix notation as $S = CP$, where $S$ and $P$ are n*1 column matrices of convergences and loads respectively. The influence matrix
Fig. 4.17 The influence of the span to depth ratio w/h, the number of pillars, and pillar width to distance between pillar centres W/l on critical stiffness (reproduced from Ozbay, 1989).

C was determined by applying a force field $p_j$ at the elements occupying the area of the $j^{th}$ pillar, while no other pillars are assumed to exist. The convergences at all pillar locations are recorded, divided by $p_j$ and registered as the $j^{th}$ column of the influence matrix. By repeating this procedure all coefficients are determined. The critical stiffness $\lambda_c$ is equal to the reciprocal of the maximum eigenvalue of $C$.

The results for various 2D (plain strain) mine structures are depicted in Fig. 4.17. The critical stiffness is normalized with respect to the elastic modulus of the strata $E_s$ and Poisson’s ratio $\nu$ in the form $\lambda_\Omega$, where $\Omega = 4(1-\nu^2)/E_s$, and plotted as a function of total mine span to depth ratio w/h. The constant, equal to 4, is probably meant in m$^{-1}$, to achieve a formulation which is dimensionally correct. Each graph
applies to a different ratio of pillar width $W$ to pillar distance $l$. The points marked * refer to purely analytical solutions of Salomon (1970) for mine workings at infinite depth. The major results are:

- for all studied mine layouts strata stiffness decreases considerably (i.e. $-\lambda_c\Omega$ becomes less positive) with increasing $w/h$ ratio until the $w/h$ ratio is about 5. From a value of $w/h$ of more than five $-\lambda_c\Omega$ approaches zero asymptotically. Note that, according to Eq. 4.15, when the strata stiffness is zero, a stable equilibrium is not possible when the load-convergence curve of the pillars becomes negative, i.e. when the pillar fails. Hence pillar failure should not be allowed when the mine span to depth ratio is more than five. Obviously, arching cannot develop sufficiently for such high $w/h$ ratio's. This conclusion is of particular interest regarding the calcarenite mines, because the ratio of mine width to rock overburden thickness of the major mine workings generally ranges between 1 and 15.

- the extraction ratio is equal to $1-(W/l)$. The graphs show that in general extraction ratio does not significantly affect strata stiffness. Only for mine structures comprising less than five pillars does an increase of extraction ratio result in a reduction of strata stiffness. However, notice that smaller pillars, necessarily associated with a higher extraction ratio, have steeper post-failure load-convergence slopes. As a consequence, general mine stability may yet decrease.

- strata stiffness decreases with an increasing number of pillars. Notice that an increase of pillars for a certain extraction ratio involves a decrease of $W/H$ and, accordingly, a steepening of the post-peak load-convergence slope of the pillars.

It should be noticed finally that the methods outlined here apply only to an overburden which remains intact, at least at the start of the collapse. Whether this is true for the calcarenite mines could be revealed by studies of the collapse areas underground.

### 4.4 CONCLUSIONS: APPROACH TO THE RESEARCH

From the preceding paragraphs it can be concluded that various aspects of pillar and mine stability assessment have been studied, particularly for coal mines. However, with respect to the calcarenite mines many important topics are not well described and several concepts have not been applied. Additionally, some aspects are not yet well understood in general.
The shortcomings of stability assessment of the calcarenite mines which will be dealt with in the following chapters of this thesis are listed below:

- some geotechnical properties of the calcarenite are not well described, e.g. elasto-plastic behaviour, the influence of moisture content on strength, transverse isotropy etc. (Chapter 5).

- the size factor for the calcarenite was assumed to be equal to unity, but no experiments have been performed to confirm this. Also the shape effect had not been studied for calcarenite, neither on square-based nor on less regular pillars (Chapter 6).

- the residual strength of failing pillars had not been investigated (Chapter 6).

- time-dependent deformation of calcarenite, which is of major importance with regard to mine stability, must be investigated. Guidelines should be derived which enable to apply the results in practice (Chapter 7).

- a criterion of large-scale pillar stability has not developed yet. A method is needed to determine the total load carrying capacity of all pillars together relative to the total overburden load (Chapters 6 and 7).

- the variation of UCS is only known on a 100 m scale at best. More densely spaced measurements are needed on a pillar scale and even in more detail. Additionally, measuring UCS directly from block samples sawed out of intact pillar walls is time-consuming. Besides possible weakening of the pillar, unaesthetic scars result. Indirect methods have to be found to determine UCS more efficiently and without damaging the pillars (Chapter 8).

- field work in the mines must provide data concerning the nature of pillar deterioration and to compare observations in the mines with experimentally developed methods of stability assessment. If possible, the time-factor should be incorporated (Chapters 8 to 11).

- the nature and geometry of large-scale collapses were not well described, particularly with regard to surface subsidence and the deterioration of adjacent mined areas (Chapters 9 to 11).

- general mine stability of the calcarenite mines had not been investigated yet. The principles of pressure arch, pillar and mine stiffness should be applied in order to improve the understanding why some badly deteriorated mines collapse and others do not (Chapter 11).
PART II

LABORATORY EXPERIMENTS
CHAPTER 5

GEOTECHNICAL PROPERTIES OF THE CALCARENITE

5.1 INTRODUCTION

During the several fieldworks carried out by students since 1983, in both underground mines and open quarries, much data concerning the geotechnical characteristics of the calcarenites have become available (e.g. Grabandt et al., 1983; Lap et al., 1987). Table 4.3 gives an impression of the high porosities and low strength values and deformation moduli of the Maastricht facies type calcarenites. The measured values vary considerably with lithology. It has to be noted that Table 4.3 represents an example of parameters acquired at only one location, the quarry of 't Rooth (Fig. 2.6). In general, lateral variation is considerable. Furthermore, the interlithological differences of certain properties apparent in Table 4.3 are not consistent and might be quite different or even opposite elsewhere. In Table 5.1 an overview is given of the general range of measured geotechnical properties of the calcarenites.

In this chapter an account will be given of experiments on calcarenite which were performed to give insight into deformation characteristics which were not studied before or with less detail. For example, cohesion and friction angle had not been measured by triaxial testing. Although it was known that the presence of water weakens the calcarenite (Table 5.1) the exact relationship between water content and deformation moduli and UCS had still to be determined. Unconfined compression tests had not been described in much detail. Finally, transverse isotropy which was assumed to exist had not been quantified. Evidence of microstructural deformation mechanisms is discussed, based on macroscopic phenomena, microstructures, as far as detectable by optical microscope, and existing data and theories from other authors.

The samples studied here originated just from one location in the Sibbergroeve (Fig. 2.7), where the calcarenites are known to be relatively strong. In this regard it has to be born in mind that the geotechnical properties of the calcarenites vary by location and lithology. But all calcarenites at levels mined by the room and pillar method show comparable texture, chemical composition and mechanical behaviour, e.g. weak intergranular cementation, high porosity, chemically almost 100 %
Table 5.1 General range of measured geotechnical parameters of the calcarenite of the Formation of Maastricht, Maastricht facies.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight (kN/m³)</td>
<td>12.5 - 15.0</td>
</tr>
<tr>
<td>Saturated unit weight (kN/m³)</td>
<td>18.0 - 21.0</td>
</tr>
<tr>
<td>Porosity (%)</td>
<td>40 - 50</td>
</tr>
<tr>
<td>Dry UCS (MPa)</td>
<td>1.5 - 5.5</td>
</tr>
<tr>
<td>Saturated UCS (MPa)</td>
<td>1.0 - 4.5</td>
</tr>
<tr>
<td>Dry Braz. tensile strength (MPa)</td>
<td>0.15 - 0.50</td>
</tr>
<tr>
<td>Dry secant E-mod. at 50 % of UCS (GPa)</td>
<td>0.3 - 1.0</td>
</tr>
<tr>
<td>Sat. secant E-mod. at 50 % of UCS (GPa)</td>
<td>0.2 - 0.6</td>
</tr>
<tr>
<td>Poissons' ratio (secant modulus at 50 % of UCS)</td>
<td>0.1 - 0.2</td>
</tr>
</tbody>
</table>

CaCO₃, and low UCS, E-modulus and Brazilian tensile strength. Hence it can be stated that the general characteristics obtained here are roughly representative for the mechanical behaviour of the calcarenites mined underground. Obviously, the most important parameters regarding stability calculations, particularly UCS and E-modulus, have to determined at each separate investigated location.

While this chapter deals with the short-term behaviour of the calcarenites, the long-term load response of the material will be described in Chapter 7.

In this chapter the geotechnical properties of the hardgrounds are not studied because they do not form part of the pillars. Nevertheless, since they represent a portion of the mine structure, the hardgrounds are of some interest. Kronieger (1989) measured some parameters of the Limestone of Meerssen including hardgrounds at various locations. In Table 5.2 the data are listed which were determined on the Maastricht facies type of this limestone. As expected, the porosity of the hardground is considerably less than that of the "normal" calcarenite. Accordingly, dry unit weight, UCS, Brazilian tensile strength and E-modulus are higher.

5.2 EXPERIMENTAL PROCEDURES

5.2.1 Sampling and sample preparation

The tested calcarenite originated from the northeastern part of the Sibber Groeve (the third niche in the south wall of the gallery in between point 34 and 35). Here the homogeneous and relatively strong (3 to 5 MPa) calcarenite from the upper part of the Emael Limestone was excavated. The calcarenite was mined at the end of a blind gallery where concentrations in vertical stress are expected to be less than at most
Table 5.2 Values of geotechnical parameters of hardground and "normal", weakly cemented calcarenite of the Limestone of Meerssen, Maastricht facies (after data from Kronieger, 1989).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>General calcarenite</th>
<th>Hardground</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight (kN/m³)</td>
<td>12 - 14</td>
<td>21</td>
</tr>
<tr>
<td>Porosity (%)</td>
<td>35 - 50</td>
<td>10</td>
</tr>
<tr>
<td>Dry UCS (MPa)</td>
<td>1.0 - 4.0</td>
<td>28</td>
</tr>
<tr>
<td>Dry Braz. tensile strength (MPa)</td>
<td>0.1 - 0.4</td>
<td>3.5</td>
</tr>
<tr>
<td>Dry secant E-mod. at 50 % of UCS (GPa)</td>
<td>0.5 - 1.5</td>
<td>16</td>
</tr>
<tr>
<td>Poissons’ ratio</td>
<td>0.14 - 0.30</td>
<td>0.20</td>
</tr>
</tbody>
</table>

pillar walls and edges. A study of thin sections under the optical microscope did not reveal microfracture damage, although the strong micritization (Fig. 2.5) hindered the recognition of deformation structures. Since the local pre-mining vertical stress is just about 0.6 MPa, it is considered reasonable to assume the testing material to be unaffected by microfracture damage.

Three calcarenite blocks, called "stoelen" (about 1.8 m high, 0.5 m wide and 0.9 m deep), were mined by using an electric chain saw (Fig. 2.14). During the mining of the "stoelen" small samples were taken at different depths and packed in plastic foil to measure the moisture content relative to the distance from the original rock face. The moisture contents of 9.1, 8.4 and 8.9 % at respectively 1, 0.5 and 0.1 m from the original rock face did not show a significant variation with rock face distance. The "stoelen" were transported out of the mine and sawn into smaller blocks of about 50*40*15 cm. Immediate packing into two layers of plastic foil was undertaken in order to prevent drying out during transport to Delft and during storage. Cylindrical cores were prepared in accordance with the suggested methods of the ISRM (Brown, 1981). Both 40 and 50 mm diameter cores were produced with a length of respectively 80 and 100 mm. In most cases coring took place perpendicular to the bedding except for the samples used for determining the transversal anisotropy. After preparation the cores were immediately packed in plastic foil and tested within a couple of days. The moisture content of the samples, just prior to testing, proved to be about 7 to 8 %. Hence a loss of moisture of about 1 % occurred in spite of the measures against drying out. Determination of moisture content directly after the test showed that no further moisture loss had occurred during the test.
5.2.2 Testing

For the uniaxial tests a servo-controlled compression machine of a capacity of 50 kN was used. To ensure controlled specimen failure the machine was designed to be very stiff, i.e. its stiffness exceeds the post-failure slope of the specimen, regardless which rock type is tested (see Section 4.3.3.1 and Fig. 4.15).

The axial displacement was measured by two LVDT’s, placed between both loading platens, and the radial displacement by a circumferential direct-contact extensometer, situated at mid-height of the core. Steel disks of the same diameter as the sample were inserted between sample ends and machine platens. It should be noted that the specimen has a significantly lower Young’s modulus (about 1 GPa) than the steel disks (about 200 GPa) and therefore tends to expand more, radially, under axial compression. Due to friction between specimen ends and disks, the relatively large expansion of the specimen tends to be prevented. As a consequence, the specimen becomes slightly barrel-shaped and the measured radial strain cannot be considered representative for the whole sample. It is generally known (e.g. Peng, 1970; Vutukuri et al., 1974) that shear stresses develop at the interfaces and that the stresses within the specimen are not uniaxial, particularly near the ends. For this reason a specimen length/diameter ratio of at least two should be used.

The control unit of the compression machine uses the output of the extensometer to adjust the oil pressure. In order to measure the actual post-failure behaviour (see Sect. 5.3.2) the tests were carried out at radial strain control. The output of LVDT’s, extensometer and force was amplified and digitized by a 12-bit A/D converter. All data were finally stored on a floppy disk.

The Brazilian tests were performed using the same compression machine, but with axial displacement control.

The triaxial tests were carried out using a Wykeham Farrance compression machine and triaxial cell body. The established oil pressure was transferred to the flexible sample jacket by water. Axial stress and displacement were displayed by mechanical dial gauges and registered by hand. As a consequence of the low stiffness of the machine and its inability to achieve a constant strain rate with a feedback control system the post-peak portion of the stress-strain curve is affected by unloading of the machine, if the confining stress is such that strain softening occurs. In this case uncontrolled failure occurs. The triaxial tests were performed at axial displacement control.

Except for the low stiffness of the compression machine missing a servo-control system, which was used for the triaxial tests, all tests were performed according to the suggested methods of the ISRM (Brown, 1981).
Fig. 5.1a Typical stress-axial strain curve of a radial strain controlled uniaxial compression test (test "natural 2").

Fig. 5.1b The stress-radial strain curve of test "natural 2".
5.3 DEFORMATION BEHAVIOUR OF THE CALCARENITE UNDER UNIAXIAL COMPRESSION

5.3.1 General characteristics

5.3.1.1 Stress-strain behaviour

Figs. 5.1 a-c show typical curves of stress vs. axial, radial and volumetric strain, resulting from a radial strain-controlled $(d_{\text{radial}}/dt = 6 \times 10^{-6} \text{ sec}^{-1})$ uniaxial compression experiment on the calcarenite at a moisture content of about 6% (test "natural 2" of Table 5.3). The radial strain, measured at mid-height of the specimen, is depicted. Due to the end effects, this strain exceeds the average radial strain for the whole sample to some extent. Therefore the volumetric strain of Fig. 5.1c, determined from the axial strain and the radial strain at sample mid-height, is somewhat less than in reality. This applies especially when the radial strain rate starts to accelerate and the specimen begins to dilate. For this experiment this happens at about 1.8 MPa, which corresponds to about 65% of the UCS.

Failure of calcarenite cylindrical cores is generally characterized by the development of a macroscopic shear fracture at about 30° to the applied axial stress (Fig. 5.2). An axial crack initiating at the sample-loading platen interface is the first macroscopically visible phenomenon denoting the onset of failure. This axial crack could be observed at or just after peak stress and attains a length of up to 5 mm.
Subsequently the axial crack propagates at about 30° with the core axis. This can often be observed as the growth and coalescence of axial cracks of maximally 5 mm long. The shear fracture gradually propagates while the stress is continuously decreasing. The nature of the positive slope of the post-peak stress-strain curve will be dealt with later. Then the axial strain starts to increase again while the axial stress continues to decrease until an almost constant stress level has been attained. Now the stress-strain slope is small and positive. A continuous shear fracture has formed, and all further deformation can be ascribed to movement on the shear plane.
5.3.1.2 Characteristics of the measured strain

By the performance of an unloading-reloading cycle it can be demonstrated that the permanent, non-elastic strain component of the calcarenite is significant and exceeds the component of elastic strain even before the peak strength. At a stress of about 60 % of the UCS the elastic/permanent strain ratio proved to be 35:65. As a consequence the values of the deformation moduli are highly dependent on the way in which they are determined. Three methods of E-modulus determination are known, resulting in respectively the secant-modulus, the tangent modulus and the unloading modulus (see Jaeger & Cook, 1979). The moduli values are equal to the slope of the corresponding dashed lines in Fig. 5.1a. The secant-, tangent- and unloading moduli at 60 % of the UCS are respectively 1.08, 1.30 and 3.70 GPa, corresponding with 390, 460, and 1320 times the UCS value. Obviously purely elastic deformation of the calcarenite is only adequately described by the unloading modulus, because the secant- and tangent moduli also represent a significant amount of permanent strain. For this reason secant and tangent moduli of elasticity should actually be referred to as moduli of deformation, as proposed by Goodman (1989). A more important conclusion is that most deformation in the mines is non-elastic.

Uniaxial tests were carried out at radial strain rates ranging from $10^{-3}$ to $10^{-6}$ sec$^{-1}$. Peak stress was attained in about 3 sec to $3 \times 10^{3}$ sec. It was observed that UCS and deformation moduli were not related to strain rate within this strain rate range. Also the ratio of permanent and elastic strain at certain percentages of UCS was unaffected by the changes in strain rate. This means that the entire amount of permanent strain, measured in the experiments described above, can be produced within a few seconds. Hence the measured permanent strain from this type of test can be considered as quasi-instantaneous.

5.3.1.3 Evidence of deformation mechanisms

Microscopic studies, including fluorescence-techniques, did not reveal information concerning deformation mechanisms operating in the calcarenite before the onset of failure and the development of macroscopic cracks, because of the already mentioned strong micritization of the grains. After completion of the test the sharp, macroscopic shear plane was examined. Under the microscope it was to be seen that the fracture surfaces were covered with loose grains. After impregnating the tested sample with epoxy resin, it was observed that macroscopic fractures were exclusively intergranular (Fig. 5.3).

Costin (1989), who gave an extensive review on the mechanisms of brittle deformation and failure concluded that deformation and failure of brittle rock is dominated by the nucleation, growth and coalescence of microcracks. It is generally acknowledged that not only under tensile- but also under compressive loading this microcracking results from local concentrations of tensile stress at pre-existing flaws and pores. The tensile microcracks tend to propagate in a direction parallel to the greatest principal compressive stress. As the compressive load increases more and
more microcracks develop and existing cracks continue growing until a critical microcrack density evolves and cracks start to interact with each other. Eventually coalescence occurs and macroscopic fractures are formed.

It can be assumed that also for this Maastrichtian calcarenite microcracking is controlling brittle deformation and failure. In this regard it has to be noted that the calcarenite, with its open fabric of grains which are weakly cemented together, constitutes a special rock type. For this material microcracking just corresponds to breakage of the weak intergranular cement bonds. Such a brittle deformation mechanism was indeed observed for calcarenites from the Northwest Shelf of West-Australia, which are of comparable texture, porosity and strength. A microstructural study by Price (1988) also revealed a small elastic response, which was supposed to represent the behaviour of the relatively weak carbonate cement. Permanent strain could be ascribed to rupture of this cement at grain contacts. When this had occurred around a sufficient number of grains, they were free to move. Also for Upper Cretaceous highly porous chalks of Northwest Europe, i.e. calcilutites which show a fabric comparable with the calcarenite of South-Limburg, such a brittle deformation mechanism, governed by intergranular cement bond breakage, was observed (Loe et al., 1992; Petley et al., 1993).

Plastic deformation by mechanical twinning should also be considered. Nicolas & Poirier (1976) summarized research of various authors and concluded that such a
mechanism operates in calcite at all temperatures while it is capable of inducing tens of strain percentages depending on the direction of compression relative to lattice orientation. The significance of this deformation mechanism relative to cement bond breakage remains to be studied.

5.3.2 Class II post-peak behaviour

After the peak stress has been attained, at 2.79 MPa for this test, the post-peak stress-strain curve shows a positive slope. This property is known as class II behaviour, in contrast to class I behaviour which is characterised by a persistent negative post-failure slope. This terminology was developed by Wawersik & Fairhurst (1970), who observed that some rocks under uniaxial compression failed in an uncontrolled way even if a perfectly stiff compression machine was used. They could obtain the complete stress-strain curve only indirectly, from the envelope of unloading-reloading loci, and found that the post-failure slope was positive. Hudson, Brown & Fairhurst (1971) found an easier way to obtain the complete stress-strain curve by performing the experiments using a servo-controlled compression machine at radial displacement control. It is this type of testing arrangement, which was used for the uniaxial compression tests on the calcarenites. It has to be noted that, if the same test setup is used at axial displacement control, the calcarenite sample fails in an uncontrolled way, resulting in a partly vertical post-peak stress-strain curve. This was also actually observed for the calcarenite. He, Okubo and Nishimatsu (1990) describe other, more complex methods to obtain the complete stress-strain curve of rock samples showing class II behaviour.

Hudson, Crouch and Fairhurst (1972) recognized that class II behaviour is not a fundamental material characteristic and hypothesized that the post-peak response depends on diameter/height ratio of the rock sample. He, Okubo and Nishimatsu (1990) found that as failure proceeds the elastic strain continuously decreases. They concluded that, if the non-elastic strain (i.e. slip and propagation of a macroscopic shear fracture) increases faster than the elastic strain decreases, class I behaviour will be observed, and class II behaviour in the opposite case. At an increase in confining pressure the relative portion of non-elastic strain was found to increase resulting in a the post-peak stress-strain curve becoming negative and gentle.

Labuz and Biolzi (1991) stressed that the elastic energy is stored per unit of volume while the fracture energy is dissipated per unit of fracture area. As a consequence, the post-peak response includes a size- and shape effect. They demonstrated experimentally that for cylindrical cores of a diameter/height ratio of 0.5 a critical size exists such that only cores exceeding this size showed class II behaviour. Cores exceeding the critical size exhibited class I behaviour when the diameter/height ratio was increased.
Table 5.3 UCS and deformation moduli at various moisture contents. All samples which are not oven dried are considered as wet.

<table>
<thead>
<tr>
<th>Moisture (weight %)</th>
<th>UCS (MPa)</th>
<th>Unloading constants</th>
<th>Tangent constants</th>
<th>Secant constants</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E (GPa)</td>
<td>ν</td>
<td>E (GPa)</td>
<td>ν</td>
</tr>
<tr>
<td>Saturated 1</td>
<td>24.20</td>
<td>2.76</td>
<td>1.43</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>24.48</td>
<td>2.70 3.25 0.21</td>
<td>1.54 0.37</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>24.67</td>
<td>2.59 3.29 0.19</td>
<td>1.45 0.31</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>24.22</td>
<td>2.68 3.53 0.27</td>
<td>1.50 0.33</td>
</tr>
<tr>
<td>Natural 1</td>
<td>6.42</td>
<td>2.73 3.21 0.26</td>
<td>1.43 0.34</td>
<td>1.28 0.18</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>5.68</td>
<td>2.79 3.19 0.29</td>
<td>1.39 0.31</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>6.43</td>
<td>2.53 3.28 0.23</td>
<td>1.38 0.35</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.45</td>
<td>2.50 3.26 0.33</td>
<td>1.34 0.35</td>
</tr>
<tr>
<td>Dried, 48 h 1</td>
<td>0.87</td>
<td>2.76 3.70 0.20</td>
<td>1.52 0.35</td>
<td>1.48 0.10</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.89</td>
<td>2.73 3.50 0.21</td>
<td>1.41 0.31</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.87</td>
<td>2.65 3.22 0.31</td>
<td>1.50 0.38</td>
</tr>
<tr>
<td>Oven dried 1</td>
<td>0.00</td>
<td>3.67 3.49 0.21</td>
<td>1.64 0.36</td>
<td>1.32 0.12</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.00</td>
<td>3.74 3.44 0.26</td>
<td>1.81 0.36</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.00</td>
<td>3.89 3.41 0.21</td>
<td>1.74 0.37</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.00</td>
<td>3.75 3.25 0.26</td>
<td>1.87 0.35</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.00</td>
<td>3.69 3.56 0.25</td>
<td>1.83 0.33</td>
</tr>
<tr>
<td>Mean (wet)</td>
<td></td>
<td>2.675 3.343 0.250</td>
<td>1.445 0.342</td>
<td>1.315 0.148</td>
</tr>
<tr>
<td>St.Dev. (wet)</td>
<td></td>
<td>0.097 0.172 0.049</td>
<td>0.064 0.022</td>
<td>0.124 0.053</td>
</tr>
<tr>
<td>Mean (dry)</td>
<td></td>
<td>3.748 3.430 0.238</td>
<td>1.778 0.351</td>
<td>1.410 0.142</td>
</tr>
<tr>
<td>St.Dev. (dry)</td>
<td></td>
<td>0.086 0.106 0.026</td>
<td>0.090 0.013</td>
<td>0.123 0.021</td>
</tr>
</tbody>
</table>
It can be concluded that the post-peak behaviour is not only dependent upon the stiffness of the machine and the rock material properties, but that also size- and shape effects appear in the stress-strain response. In Chapter 6 it will be shown that class II behaviour cannot be observed in calcarenite prisms of width/height ratio’s of one or higher.

5.4 INFLUENCE OF MOISTURE CONTENT ON UCS AND DEFORMATION MODULI

5.4.1 Introduction

The data collection of Table 5.1 shows that UCS and secant E-modulus are lower for saturated than for oven dried samples. The lowering of rock strength by moisture is a generally known phenomenon. Vutukuri, Lama & Saluja (1974, Chapter 2) gave an overview of experimental evidence obtained by several authors. The sensitivity of different rock types to moisture proved to be highly variable. The reduction from dry to saturated strength mainly varied between about 20 and 60 %.

Peng (1975) and Van Eeckhout (1976) showed for coal mine shales that with an increased moisture content there was not only a reduction in UCS but also a reduction in E-modulus and an increase in Poisson’s ratio.

Parate (1973) concluded from a study on a compact limestone that the presence of water reduces compressive strength, tensile strength and the Mohr-Coulomb cohesion, while the friction angle was not affected.

Hawkins & McConnell (1992) investigated the influence of moisture on the strength and deformability of 35 sandstone types. The loss in strength on saturation was ranged from 8 to almost 80 %. No relation could be found between strength loss and porosity. The decrease from dry to saturated strength was not gradual, but there was generally an abrupt strength reduction between 0 and 1 % moisture content with only 10 to 20 % of the total strength loss occurring at higher moisture contents. The E-modulus decrease on saturation also occurred mainly between zero and 1 % percent moisture content. Sandstones containing calcite cements were commonly more susceptible to moisture than those with siliceous cements.

5.4.2 Tests and results for the calcarenite

16 calcarenite cores, originating from one block sample, of various moisture contents (Table 5.3) were tested in uniaxial compression. The moisture content is expressed in weight percentage. The saturation of samples was carried out by immersion in water for 24 hours. The saturated moisture content proved to be about 24.5 weight
Fig. 5.4 UCS as a function of moisture content.

Fig. 5.5 E-moduli as a function of moisture content.
Fig. 5.6 Poisson's ratio's as a function of moisture content.

%. Dry samples were obtained by drying in an oven at 95°C, also for 24 hours. Before testing these specimens were allowed to cool down to room temperature under vacuum. As mentioned previously, the more or less natural moisture content was about 6.5 %. An additional intermediate moisture content of about 0.9 weight % was achieved by exposing some of these cores of a natural moisture content during 48 hours to an environment of approximately 15°C and 60 % air moisture content. Stress-axial strain slopes were determined in three ways, resulting in the establishment of an unloading modulus, a tangent modulus and a secant modulus (Fig. 5.1a). These deformation moduli were divided by the respective values of the stress-radial strain slopes, determined in a similar way. These quotients are taken respectively as the unloading, tangent and secant Poisson’s ratio’s. The results are

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1 Drying at this temperature may induce cracks due to anisotropic thermal expansion of calcite. Theoretically, due to this effect, the strength increase on drying could be reduced. On the contrary, according to Hawkins & McConnell (1992) at temperatures of more than 50°C baking of clay minerals can occur, resulting in an increase in strength. However, thin sections of untested, oven-dried samples did not reveal any cracking, neither intergranular, nor intragranular. Neither clay minerals could be detected. But it should be noted that inspection by optical light microscope does not give conclusive evidence. More certainty could be obtained by means of SEM at high magnification or by additional uniaxial tests on samples, which had been dried under vacuum at room temperature. The latter option was chosen. The UCS values were 3.74, 3.84, 3.73, 3.61 and 3.75 MPa (mean value 3.73 MPa, standard deviation 0.08 MPa). Thus no significant difference with the oven dried samples exists.
summarized in Table 5.3 and Figs. 5.4 to 5.6. The data were statistically analyzed using a two-tailed t-test at a 95% confidence level. The following conclusions can be drawn:

- no significant reduction of UCS and elastic moduli exists at an increase in moisture content from 0.9 to 24 weight % (saturation). Therefore, all specimens containing a measurable percentage of moisture are denoted as wet, and the oven dried specimens as dry.

- the UCS is significantly higher for dry than for wet samples (Fig. 5.4). The decrease in UCS for wet samples was about 28.5% on the average. Samples of a moisture content of only 0.9 weight % showed the same reduction in strength as those which were saturated. No strength loss could be observed at intermediate moisture contents. Hence it can be concluded that the strength reduction occurs more or less completely between 0 and 0.8%.

- the tangent E-modulus were more or less similarly moisture dependent as the UCS (Fig. 5.5). A decrease of 18.5% could be observed from 0 to 0.8% moisture content, whereas no further decrease was to be detected at higher moisture contents. The corresponding Poisson's ratio did not show a significant dependence on moisture content (Fig. 5.6).

- the unloading and secant moduli did not show a significant decrease with moisture content (Fig. 5.5). Concerning the secant modulus this is probably obliterated by the large variation at every moisture content as a result of the large variation in strain accumulated during the first increase of the axial stress, the "settling down phase" (Paterson, 1978; Chapter 7). That the unloading E-modulus is more or less independent of moisture content, could be ascribed to the fact that this modulus is almost entirely an elastic parameter, in contrast with the tangent and secant moduli.

5.4.3 Discussion

Van Eeckhout (1976) evaluated several mechanisms which are used to explain the strength reduction of rocks due to the presence of water, i.e. reduction of the required fracture energy, pore pressure increase, chemical deterioration or stress corrosion, capillary effects and reduction of the coefficient of friction of sliding cracks.

Vutukuri (1974), Van Eeckhout (1976) and Hawkins & McConnell (1992) ascribed, for limestone, sandstone and coal respectively, the influence of water on strength to the first mechanism. According to the Griffith fracture criterion the tensile stress necessary to induce crack growth is linearly related to the square root of the surface energy, which is the energy required per unit crack propagation. Since the surface energy is lowered by the adsorption of water, the fracture strength is reduced and hence the strength of the rock material.
The tests results indicate that pore pressure increase cannot play an important role regarding the observed strength reduction of calcarenite. As for the sandstones studied by Hawkins & McConnell, the sudden strength reduction occurs at moisture content values well before saturation. Hawkins & McConnell performed additional tests on sandstone specimens at saturation using an experimental setup which allowed the measurement of pore pressure. However, only extremely small positive, and even negative, pore pressures were measured and no significant effect on rock strength was detected. Obviously, if drainage is impeded pore pressures do become significant regarding rock strength, but this situation does not apply to the experiments evaluated here.

Also stress corrosion is often considered as an important mechanism in the weakening of rocks due to moisture. Stress corrosion crack growth is a process of weakening of the strained bonds at crack tips and the resulting facilitation of crack propagation because of the chemical action of water or other agents. This effect has been observed for silicate glasses and quartz (Atkinson and Meredith, 1989), but the role of stress corrosion in the deformation of calcareous rocks is not yet well understood (Hawkins & McConnell, 1992).

To investigate which mechanisms are actually the most significant for calcarenite would go beyond the scope of this thesis. However, former research seems to indicate that fracture energy reduction might be the most prominent process for this rock type.

5.5 TRANSVERSE ISOTROPY

5.5.1 Introduction

In the absence of shell beds, flint layers, hardgrounds etc. bedding is invisible in the field. Also under the optical microscope no traces of a sedimentary layering were
Table 5.4 UCS and deformation moduli, measured with the core axis perpendicular and parallel to the bedding plane.

<table>
<thead>
<tr>
<th></th>
<th>UCS (MPa)</th>
<th>Unloading constants</th>
<th>Tangent constants</th>
<th>Secant constants</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>E (GPa)</td>
<td>$\epsilon_{\text{circ}}/\epsilon_{\text{axial}}$</td>
<td>E (GPa)</td>
</tr>
<tr>
<td>Perpendicular 1</td>
<td>2.76</td>
<td>3.69</td>
<td>0.33</td>
<td>1.60</td>
</tr>
<tr>
<td>Perpendicular 2</td>
<td>2.57</td>
<td>3.70</td>
<td>0.24</td>
<td>1.49</td>
</tr>
<tr>
<td>Perpendicular 3</td>
<td>2.79</td>
<td>3.64</td>
<td>0.25</td>
<td>1.36</td>
</tr>
<tr>
<td>Perpendicular 4</td>
<td>2.82</td>
<td>3.85</td>
<td>0.37</td>
<td>1.48</td>
</tr>
<tr>
<td>Perpendicular 5</td>
<td>2.63</td>
<td>3.72</td>
<td>0.27</td>
<td>1.12</td>
</tr>
<tr>
<td>Mean</td>
<td>2.71</td>
<td>3.72</td>
<td>0.29</td>
<td>1.41</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.10</td>
<td>0.07</td>
<td>0.05</td>
<td>0.16</td>
</tr>
<tr>
<td>Parallel 1</td>
<td>2.47</td>
<td>3.04</td>
<td>0.24</td>
<td>1.32</td>
</tr>
<tr>
<td>Parallel 2</td>
<td>2.31</td>
<td>2.73</td>
<td>0.26</td>
<td>1.04</td>
</tr>
<tr>
<td>Parallel 3</td>
<td>2.26</td>
<td>3.08</td>
<td>0.28</td>
<td>1.04</td>
</tr>
<tr>
<td>Parallel 4</td>
<td>2.25</td>
<td>2.87</td>
<td>0.30</td>
<td>1.31</td>
</tr>
<tr>
<td>Parallel 5</td>
<td>2.41</td>
<td>2.86</td>
<td>0.27</td>
<td>1.29</td>
</tr>
<tr>
<td>Mean</td>
<td>2.340</td>
<td>2.916</td>
<td>0.270</td>
<td>1.200</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.096</td>
<td>0.143</td>
<td>0.022</td>
<td>0.146</td>
</tr>
</tbody>
</table>

visible. However, this does not necessarily mean that layering is absent and that the rock is mechanically isotropic. The calcarenite might be transversely isotropic, as commonly observed in layered rocks: its mechanical properties, e.g. Young's modulus and compressive strength, are the same in all directions within the bedding plane. However, the mechanical properties in other directions differ from those measured within the bedding plane.

The elastic properties of a transversely isotropic rock are completely described by uniaxial compression tests normal and parallel to the bedding, as delineated below. However, this does not apply to its inelastic behaviour. For example, uniaxial compressive strength varies continuously with the orientation of bedding relative to the core axis. As a consequence, for a complete characterization of this property tests at various angles from $0^\circ$ to $90^\circ$ relative to the layering are necessary (e.g. see Hoek & Brown, 1982; p. 157-162). On the calcarenite only uniaxial tests normal and parallel to the bedding are performed here.
Table 5.5 Compilation of transversely-isotropic parameters; 1) established by Bekendam, samples from Sibbergoewe, 2) from Grabandt et al. (1983), samples from ENCI-quarry.

<table>
<thead>
<tr>
<th>Stratigraphic level</th>
<th>UCS&lt;sub&gt;1&lt;/sub&gt; (MPa)</th>
<th>UCS&lt;sub&gt;2&lt;/sub&gt; (MPa)</th>
<th>UCS&lt;sub&gt;1&lt;/sub&gt;/UCS&lt;sub&gt;2&lt;/sub&gt;</th>
<th>Unloading moduli</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E&lt;sub&gt;1&lt;/sub&gt; (GPa)</td>
<td>E&lt;sub&gt;2&lt;/sub&gt; (GPa)</td>
<td>E&lt;sub&gt;1&lt;/sub&gt;/E&lt;sub&gt;2&lt;/sub&gt;</td>
<td>E&lt;sub&gt;21&lt;/sub&gt;</td>
</tr>
<tr>
<td>Upper Emael&lt;sup&gt;1&lt;/sup&gt;</td>
<td>2.71</td>
<td>2.34</td>
<td>1.16</td>
<td>3.72</td>
</tr>
<tr>
<td>IV-AB&lt;sup&gt;2&lt;/sup&gt;</td>
<td>2.17</td>
<td>1.12</td>
<td>1.94</td>
<td>-</td>
</tr>
<tr>
<td>VI-AB&lt;sup&gt;2&lt;/sup&gt;</td>
<td>2.39</td>
<td>1.14</td>
<td>2.10</td>
<td>-</td>
</tr>
<tr>
<td>VI-CD&lt;sup&gt;2&lt;/sup&gt;</td>
<td>1.97</td>
<td>0.99</td>
<td>2.00</td>
<td>-</td>
</tr>
<tr>
<td>VI-CD&lt;sup&gt;2&lt;/sup&gt;</td>
<td>2.04</td>
<td>1.01</td>
<td>2.02</td>
<td>-</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Tangent moduli</th>
</tr>
</thead>
<tbody>
<tr>
<td>E&lt;sub&gt;t&lt;/sub&gt; (GPa)</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>Upper Emael&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>IV-AB&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>VI-AB&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>VI-CD&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>VI-CD&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Secant moduli</th>
</tr>
</thead>
<tbody>
<tr>
<td>E&lt;sub&gt;s&lt;/sub&gt; (GPa)</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>Upper Emael&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>IV-AB&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>VI-AB&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>VI-CD&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>VI-CD&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

Five independent elastic moduli can be recognized which can be defined as follows (Lekhnitskii, 1981) if the x-z plane is the plane of isotropy (Fig. 5.7):

\[
E_1 = E_x \quad \text{modulus of elasticity in the direction normal to the plane of isotropy}
\]

\[
E_2 = E_y \quad \text{modulus of elasticity in the plane of isotropy}
\]

\[
\nu_{21} = \nu_{yx} \quad \text{Poisson's ratio for the normal strain in the x-direction related to the normal strain in the y-direction due to uniaxial stress in the y-direction}
\]

\[
\nu_{23} = \nu_{yz} \quad \text{Poisson's ratio for the normal strain in the z-direction related to the normal strain in the y-direction due to uniaxial stress in the y-direction}
\]

\[
G_{12} = G_{xy} \quad \text{shear modulus in the plane of isotropy}
\]

For this material the following relations apply:

\[
E_2 = E_3 \quad \text{or} \quad E_y = E_z
\]
Table 5.6 Results of tensile, uniaxial and triaxial tests.

<table>
<thead>
<tr>
<th>Confining stress (MPa)</th>
<th>Axial peak strength (MPa)</th>
<th>Axial residual strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tensile strength</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brazilian</td>
<td>&quot;True&quot;</td>
<td></td>
</tr>
<tr>
<td>-0.458</td>
<td>-0.366</td>
<td>0</td>
</tr>
<tr>
<td>-0.473</td>
<td>-0.378</td>
<td>0</td>
</tr>
<tr>
<td>-0.481</td>
<td>-0.384</td>
<td>0</td>
</tr>
<tr>
<td>-0.463</td>
<td>-0.368</td>
<td>0</td>
</tr>
<tr>
<td>-0.489</td>
<td>-0.392</td>
<td>0</td>
</tr>
<tr>
<td>-0.472</td>
<td>-0.376</td>
<td>0</td>
</tr>
<tr>
<td><strong>UCS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>2.32</td>
<td>0.22</td>
</tr>
<tr>
<td>0</td>
<td>2.15</td>
<td>0.15</td>
</tr>
<tr>
<td>0</td>
<td>2.28</td>
<td>0.15</td>
</tr>
<tr>
<td>0</td>
<td>2.21</td>
<td>0.20</td>
</tr>
<tr>
<td>0</td>
<td>2.36</td>
<td>0.21</td>
</tr>
<tr>
<td>0</td>
<td>2.26</td>
<td>0.19</td>
</tr>
<tr>
<td><strong>Triaxial strength</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>2.40</td>
<td>0.61</td>
</tr>
<tr>
<td>0.1</td>
<td>2.65</td>
<td>0.86</td>
</tr>
<tr>
<td>0.2</td>
<td>3.48</td>
<td>1.76</td>
</tr>
<tr>
<td>0.2</td>
<td>3.03</td>
<td>1.47</td>
</tr>
<tr>
<td>0.3</td>
<td>3.46</td>
<td>1.72</td>
</tr>
<tr>
<td>0.4</td>
<td>3.70</td>
<td>2.30</td>
</tr>
<tr>
<td>0.5</td>
<td>3.87</td>
<td>2.87</td>
</tr>
<tr>
<td>0.6</td>
<td>4.14</td>
<td>3.03</td>
</tr>
<tr>
<td>0.8</td>
<td>4.16</td>
<td>3.80</td>
</tr>
<tr>
<td>0.8</td>
<td>4.83</td>
<td>3.77</td>
</tr>
<tr>
<td>0.8</td>
<td>4.46</td>
<td>3.89</td>
</tr>
<tr>
<td>1.1</td>
<td>5.25</td>
<td>4.59</td>
</tr>
<tr>
<td>1.1</td>
<td>4.98</td>
<td>4.18</td>
</tr>
<tr>
<td>1.4</td>
<td>5.12</td>
<td>5.12</td>
</tr>
<tr>
<td>1.4</td>
<td>5.36</td>
<td>5.36</td>
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<td>1.4</td>
<td>5.20</td>
<td>5.20</td>
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<tr>
<td>1.4</td>
<td>4.57</td>
<td>4.57</td>
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<tr>
<td>1.8</td>
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<td>1.8</td>
<td>4.82</td>
<td>4.82</td>
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<tr>
<td>2.4</td>
<td>4.91</td>
<td>4.91</td>
</tr>
<tr>
<td>2.4</td>
<td>5.10</td>
<td>5.10</td>
</tr>
<tr>
<td>2.4</td>
<td>5.18</td>
<td>5.18</td>
</tr>
</tbody>
</table>
\[
\nu_{23} = \nu_{32} \text{ or } \nu_{yz} = \nu_{zy} \\
\nu_{12} = \nu_{13} \text{ or } \nu_{xy} = \nu_{xz} \\
G_{12} = G_{13} \text{ or } G_{xy} = G_{xz} \\
G_{32} = E_2 / 2(1 + \nu_{32}) \text{ or } G_{zy} = E_y / 2(1 + \nu_{zy}) \\
\nu_{xy} = \nu_{yx} \frac{E_x}{E_y}
\]

The tests on the cores with the bedding normal to their symmetry axis allow to determine \(E_i = E_x = \sigma_x / \epsilon_x\) and \(\nu_{ij} = \nu_{xy} = -\epsilon_y / \epsilon_x\). The experiments on cores with the bedding parallel to their symmetry axis made it possible to establish \(E_2 = E_y = \sigma_y / \epsilon_y\). It can be shown that the second independent Poisson's ratio can be approximated by \(\nu_{23} = \nu_{yz} = -\{2 \epsilon_{\text{circ}} / \epsilon_y\} - \{\nu_{xy} E_y / E_x\}\). According to Lekhnitskii \(1/G_{12} = 1/G_{xy} = \{(1 + 2\nu_{xy})/E_y\} + 1/E_x\).

5.5.2 Tests and results for the calcarenite

From one block sample 10 cores were produced, 5 perpendicular to the bedding, as usually, and 5 parallel to the bedding. The bedding orientation was known from the orientation of the hardground layer in the roof above the sample location in the mine. The uniaxial tests were performed on the cores at the natural moisture content of about 6% and at circumferential strain control (\(d\epsilon_{\text{circ}} / dt = 6 \times 10^6 \text{ sec}^{-1}\)). In this section the term circumferential strain is used instead of radial strain because in experiments on cores with the bedding orientated parallel to the direction of applied uniaxial stress the radial strains respectively in the plane of isotropy and perpendicular to the plane of isotropy are not necessarily of the same magnitude. The deformation moduli were established in three ways, as in Sects. 5.3 and 5.4.

The results are summarized in Table 5.4. A two-tailed t-test at a 95% confidence level revealed that UCS and unloading- and tangent E-moduli of the cores loaded perpendicular to the bedding plane were significantly higher than those of the specimens stressed parallel to the bedding. Regarding \(\epsilon_{\text{circ}} / \epsilon_{\text{axial}}\) and secant E-moduli, no significant distinction was determined between these moduli measured on cores with the bedding orientated respectively perpendicular and parallel to the axial stress.

Using the relations outlined above, E-moduli and Poisson's ratio's were calculated and summarized and compared with data from Grabandt et al. (1983), which were acquired in the ENCI-groeve near Maastricht (Table 5.5). These authors measured a much higher degree of anisotropy, affecting both UCS and E-moduli (since strain could only be measured axially Poisson's ratio was not determined), than observed for the calcarenite from Sibbe. The E-moduli in the ENCI-groeve are remarkable low, even compared with the UCS. It can be concluded that also the degree of anisotropy varies considerably, at least for different stratigraphical levels and different locations at a kilometer scale.
Fig. 5.8 Stress-strain curves at various confining pressures.

The characterization of the transverse isotropy can be completed by uniaxial compression tests on cores at more angles relative to the bedding plane. Transverse isotropy could be described in terms of microstructure by X-ray tomography.

5.6 TENSILE STRENGTH AND TRIAXIAL STRENGTH

5.6.1 Tests and results

The experiments were performed on 50 mm diameter cylindrical cores with a length of about 30 mm for the Brazilian tensile tests and of about 100 mm for the other tests. The displacement rate was 0.20 mm/min. All samples were tested at their natural moisture content. The results of the tensile, uniaxial and triaxial tests are shown in Table 5.6.

Six Brazilian tensile tests were performed. Vutukuri, Lama and Saluja (1974, p. 134) compared the results obtained by direct tensile tests and several indirect methods on various rock types, including clastic limestones and sandstones. The directly measured tensile strength proved to be $80 \pm 15\%$ of the Brazilian tensile strength. The percentages are normally distributed and $15\%$ denotes two times the standard distribution. The ratio is used here as a rough approximation, resulting in an average "true" tensile strength of $0.377 \pm 0.073$ MPa. The average UCS, measured on 6 samples, is $2.26 \pm 0.15$ MPa. Again the values following the ± symbol signify two
Fig. 5.9 Major and minor principle stresses at failure. The Hoek-Brown curves for peak and residual strength apply only to the brittle range, to minor principle stresses of up to 0.6 MPa.

times the standard deviation.

22 triaxial tests were carried out at confining pressures ranging from 0.1 to 2.4 MPa, corresponding with 4 to 106 % of the UCS. At confining pressures lower than 1.4 MPa a peak stress was reached and then the load resistance decreased with increasing strain until it reached a more or less constant value (Fig. 5.8). Residual strength is defined here at 1.5 % strain. It must be noted that at these confining pressures, due to the softness of the testing machine, the measured post-peak stress-strain curve is not truly representative for the rock material itself, as already outlined in Section 5.2.2. Hence the actual residual strength is probably somewhat higher than measured. For confining pressures of 1.4 to 2.4 MPa no post-peak strength reduction or strain-softening could be observed and residual strength and peak strength are given the same value (Fig. 5.9). The ability to sustain permanent deformation without a decrease in load resistance is generally defined as ductile behaviour (Hoek and Brown, 1982, p. 133; Brady and Brown, 1985, p. 103; Goodman, 1989, p. 73). At low confining pressures brittle failure should occur which is defined as a decrease in load resistance with increasing deformation. According to these definitions the brittle-ductile transition for the tested calcarenite should be reached at a confining pressure of between 1.1 and 1.4 MPa.

Samples which failed at confining pressures of up to 0.6 MPa showed a sharp macroscopic shear plane at angles of about 30° to the specimen axis. At confining
Fig. 5.10 Pairs of major and minor principle stress used for the establishment of the Hoek-Brown failure criterion.

Fig. 5.11 Pairs of major and minor principle stress used for the establishment of the Hoek-Brown criterion for residual strength.
pressures of 0.8 to 1.1 MPa a 2 mm wide zone of shear failure formed at an angle of 50 to 54° with the sample axis. At confining pressures equal to or exceeding 1.4 MPa "barrelling" occurred without the development of macroscopic fractures or shear zones. Paterson (1978; Ch. 8) defined ductility as the capacity of significant change of shape without major macroscopic fracturing and characterized brittle failure mainly by a single macroscopic shear fracture. Thus, according to Paterson, these macroscopic phenomena also indicate a brittle-ductile transition at a confining stress in between 1.1 and 1.4 MPa. The broadening of the zone of shear failure represents the upper end of the brittle range.

It can be concluded that the brittle-ductile transition for the tested calcarenite occurs at a confining pressure of 22 to 27 % of the axial peak stress. This is in agreement with the findings of Paterson, who concluded that for carbonate rocks the transition is reached at confining pressures of about 25 % or a somewhat less of the axial peak stress.

Regarding the deformation mechanisms operating at increasing confining pressure, Wawersik & Fairhurst (1970) showed for marble that at higher confining pressures the distributed microcracking prior to macroscopic, brittle shear failure continued up to larger strains. According to Olsson (1974) the transition to ductile behaviour for marble at increasing confining pressure is determined by the capacity of microcracking to develop in a stable way, distributed throughout the sample, without coalescence into a macroscopic shear plane. Also the contribution of mechanical twinning will certainly increase. However, for chalk pore collapse was observed to be the deformation mechanism, responsible for the ductile behaviour and the increasing homogeneity of strain within the sample at increasing confining pressures (see Loe et al., 1992). The same applies to the West-Australian calcarenites (Price, 1988). With regard to the deformation behaviour of the calcarenite of South-Limburg, which is also characterized by a high porosity and weak intragranular cementation, pore collapse becomes probably also significant at increasing confining stress.

5.6.2 Derivation of parameters for Hoek-Brown and Mohr-Coulomb failure criterions

For numerical calculations and other purposes the use of a failure criterion is necessary. For various reasons the well known Coulomb's shear strength criterion and modifications of Griffith's criterion do not satisfactory predict peak strength (see Brady & Brown, 1985; Jaeger & Cook, 1979). Based both on these theories and on experimental data Hoek and Brown (1982) developed an empirical failure criterion. On the basis of more than ten years of experience this criterion was modified for discontinuous rock but has proven to be successful for massive rock (Hoek et al., 1992). The classic, original criterion is formulated as follows:

\[
s_1 = s_3 + \sqrt{\frac{m s_c s_3 + s s_c^2}{s}}
\]  

(5.2)

where:
\( \sigma_1 \) is the major principle stress at failure.

\( \sigma_3 \) is the minor principle stress applied to the specimen.

\( \sigma_c \) is the uniaxial compressive strength of the intact, unjointed rock material in the specimen.

\( m \) and \( s \) are constants which depend upon the properties of the rock and upon the extent to which it has been jointed or broken before the test. The constant \( s \) is equal to unity for intact, unjointed rock and decreases towards zero with increasing extent of jointing or fracturing.

Like most failure criteria Eq. 5.2 only refers to the brittle range of rock behaviour. This literally means that the failure criterion can be applied for the tested calcarenite if the confining pressure does not exceed 1.1 to 1.4 MPa. However, pillar failure, as observed both in laboratory experiments (see Chapter 7) and in the field (Chapter 3), is always characterised by the development of sharp shear planes at about 30° with the pillar walls. Also numerical experiments show that stress distributions in the mined calcarenite levels cannot be such that ductile pillar failure would be possible (Chapter 7). Hence it is decided to use only the data from the triaxial tests performed at confining pressures of up to 0.6 MPa. Also the results from the uniaxial data were included in the calculation.

A weighted regression analysis with \((\sigma_1 - \sigma_3)^2\) as dependent and \(\sigma_3\) as independent parameter gave \(m = 5.34\) and \(\sigma_c = 2.36\) MPa with \(R^2 = 0.85\). The constant \(s\) is equal to one because the calcarenite can be considered as unjointed. The measured \(\sigma_1, \sigma_3\) -observation pairs and the Hoek-Brown fit are depicted in Fig. 5.10. Substitution of \(\sigma_1 = 0\) in Eq. 1 gave a tensile strength of 0.43 MPa. The Hoek-Brown values and the directly measured values of tensile and uniaxial strength did not differ significantly. The Mohr-Coulomb friction angle \(\phi\) and cohesion \(c\) were also calculated, because they are still often used. Application of the methods outlined by Hoek & Brown (1982, App. 5) gave \(\phi = 31°\) and \(c = 0.68\) MPa.

Residual strength was described by the Hoek-Brown criterion with \(m_r = 3.90\) and \(s_r = 0.01\) \((R^2 = 0.95; \text{Fig. 5.11})\). Corresponding Mohr-Coulomb parameters were \(\phi_r = 39°\) and \(c_r = 0.11\) MPa. Fig. 5.10 shows that in the ductile range the established Hoek-Brown criteria for peak strength and residual strength do not fit the experimental data.
CHAPTER 6

STRENGTH, STIFFNESS AND SAFETY FACTORS OF CALCARENITE PILLARS

6.1 INTRODUCTION

In Chapter 4 the factors which determine pillar strength and stiffness, including size and shape effects, have been outlined in general. The determination of the UCS, an important parameter for the assessment of pillar strength, was dealt with in the previous chapter. Here it is investigated how the strength and stiffness of calcarenite pillars of various sizes and shapes can be predicted from UCS tests on cylindrical samples and additional laboratory experiments on prismatic samples. Additionally, for different prism shapes the stress-strain relationship well beyond failure and the macroscopic deformation behaviour, including fracture geometry, are studied and analyzed. The latter aspects had not been studied in much detail previously, neither for calcarenite nor for other rock types. Residual strength as a function of shape had neither been investigated quantitatively for any rock type. This characteristic is determined for calcarenite, accompanied by an analytical model. This chapter concentrates on short-term pillar behaviour. The effect of creep deformation is dealt with in Chapter 7. Finally, the determination of pillar safety factors is outlined and the possible errors of the calculation. It is proposed how to adjust calculated safety factors, in order to match the observed pillar condition in the mine.

Most experiments were performed by Dirks (1990) and Vink (1991) in cooperation with the author. Dirks carried out compression tests on prisms of various sizes and width/height ratio’s. The author performed additional tests to measure the size effect using larger prisms. Vink varied the length to width ratio. All their tests were performed on prisms of a width to height ratio of up to four, which generally agrees with the range of pillar shapes in the calcarenite mines. The stress-strain curves resulting from these tests are analyzed in Sections 6.5 to 6.8. In 1995 the author carried out additional tests on prisms of ratio’s of up to ten. The complete data set is then analyzed in Sections 6.9 to 6.12, including the presentation of analytical models. In Section 6.13 the results of numerical experiments are dealt with. Finally, in Section 6.14 the application of all results to the situation in the mines is considered.
Fig. 6.1 The actual change in the average stresses at the pillar boundaries due to excavation.

6.2 EXPERIMENTAL AND ACTUAL LOADING PATH

Strictly speaking, the loading path for the specimens tested in the laboratory is different from that for actual mine pillars. During laboratory experiments the vertical stress on prismatic model pillars changes while the horizontal stresses on the side walls are always zero (Fig. 6.1). When a real mine pillar is created, a change in an already existing in-situ stress state occurs, in that the average vertical stress significantly increases while the external horizontal stresses on the side walls decrease to zero. Further, the actual loading path depends on the excavation method and the sequence of extraction of the rock around the pillar. During pillar excavation the horizontal stresses are relieved from the side walls in a short time span of just some days or a few weeks at most. However, the in-situ horizontal stresses inside the body of the pillar may not dissipate completely immediately after excavation. This phenomenon is known as anelasticity or delayed elasticity (see Jaeger & Cook, 1979): part of the initial horizontal elastic compression, according to the former horizontal in-situ stresses, decreases exponentially in time. It takes some time before the pillar has expanded horizontally, in accordance with the zero stress at the pillar side walls. Theoretically this results in a temporary stress state of the pillar after its creation which is somewhat more favourable than the final elastic stress state. However, since the calcarenite mines are generally tens to hundreds of years old, it is unlikely that anelastic stress relaxation is important if present pillar stability is considered. As a matter of fact, the significance of the effects described above are difficult to quantify because in-situ stress states were never measured in the calcarenite mines. Often a horizontal stress, equal to the vertical stress multiplied by $\nu/(1-\nu)$ is taken, which corresponds to an elastic stress state with perfect horizontal confinement. It should also be noted that joints (Fig. 2.9) bring about relaxation of the in-situ horizontal stresses.
Fig. 6.2 The different end constraints for experimentally tested prisms and actual pillars (A and B); deformed specimen shape without (C) and including (D) lateral constraint at specimen-platen interface; shear stress $\tau$ and non-uniform normal stress $\sigma$ at the specimen end due to lateral constraint (E); Figs. C-E after Hoek & Brown (1995).

6.3 EXPERIMENTAL CONFIGURATION

Until present laboratory experiments for the assessment of pillar strength have been performed by compressing rock prisms or cylinders of various sizes and shapes, which were in direct contact with the steel machine platens (Section 4.2.2.4). There are, however, some differences of geometry and stress distribution between the experimental setup and the actual configuration in the mine. This mainly applies to the end constraints, which are characterized by a specimen-machine platen interface in the laboratory and by a pillar-roof/floor, both calcarenite, contact in the mines (Fig. 6.2 a,b).

It was outlined in Section 5.2.2 that friction between platens and specimen ends and different elastic properties of rock and steel tend to prevent expansion of the specimen at its ends. Due to this lateral constraint shear stresses arise at the specimen-platen contact and stresses within the specimen are not uniaxial, particularly near the ends. (Fig. 6.2 c,d,e). This affects the compressive strength significantly.

The stress distribution varies as a function of specimen geometry. The larger the volume of the sample which is laterally constrained relative to its total volume the
higher the sample strength will be. This explains the increase of prism strength with increasing W/H ratio. It has been experimentally shown that the shape effect disappeared when the specimen ends were not constrained and allowed to deform laterally (e.g. Hoek & Brown, 1994; Sect. 4.3.3). In this case prisms of arbitrary W/H ratio all showed the same strength, equal to the value for a W/H ratio of 0.5 to 1. Since most pillars in the mines show W/H ratio’s exceeding 1 the disappearance of a shape effect would result in a lowering of pillar strength.

In the mine pillars are bounded by a roof and a floor of approximately the same rock material instead of steel platens. The stiffness contrast between the mined calcarenite (about 1-2 GPa) and the overlying roof (tauwlaag; about 1-20 GPa) is much lower than between calcarenite and steel (about 200 GPa). Because the calcarenite roof and floor represent "platens" of much lower stiffness than steel, constraining shear stresses at the pillar ends tend to be lower than those resulting at the experimental specimen-platen interface. Note that this applies when no slip occurs between pillar and roof layers and between prism and steel platens respectively. On the other hand, it should also be noted that the calcarenite roof in the mine is horizontally constrained. This also limits horizontal expansion of the actual mine pillar near its ends. All in all, the shape effect for actual pillars might be less pronounced than for prisms tested in the compression machine. Accordingly, shape formulae based on laboratory experiments on prisms might slightly overestimate strength of real mine pillars.

Thus the most accurate way to assess pillar strength is constituted by performing compression tests on model pillars of an actual mine configuration, i.e. comprising a calcarenite roof and floor. However, such experiments will be complicated. The thickness of roof and floor must be such that the upper and lower boundary of the model on the one hand and the pillar on the other hand are outside one another's "zones of influence", i.e. the disturbance of the stress field by the pillar does not extend towards the model boundaries and vice versa. To fulfil this requirement the thickness of the model roof/floor for pillar width/height ratio’s from 1 to 4 must be at least about 1.5 to 6 times the pillar height, according to an analysis of Bray (1986). Since the maximum separation between the machine platens is about 20 cm, it is hardly possible to satisfy this condition, particularly for pillars of a W/H of 4 or more. Additionally, the appropriate confining stress, corresponding with the actual in-situ horizontal stress, with lateral restraint (no lateral movement possible) should be applied to the parts of the model which represent roof and floor. A zero in-situ horizontal stress can be modelled relatively easily by the application of a steel constriction around roof and floor, but non-zero horizontal in-situ stresses are much more complicated to create.

Considering the complications outlined above it was decided to perform experiments on prismatic samples, in direct contact with the machine platens. This method also allows to compare the results directly with those from previous experiments. In Section 6.13 the influence of the end constraints on the stress distribution in the pillar are analyzed by means of numerical experiments.
6.4 EXPERIMENTAL PROCEDURE

6.4.1 Sampling and sample preparation

The sample material originated from the same location and was transported and stored in the same way as described in Section 5.2.1. The blocks of about 50*40*20 cm were sawn into samples of the desired shape and size. The upper and lower surfaces of the samples, in agreement with the in-situ orientation, were ground flat such that the smoothness did not depart from a plane for more than the grain size, i.e. about 0.1 mm. These surfaces did not depart from perpendicularity to the vertical axis by more than 0.1 mm per 100 mm of end surface length. From each block 5 to 6 cylindrical cores of 40 mm diameter were produced according to the ISRM standards (Brown, 1981) in order to be able to relate prism strength to UCS. After preparation each sample was immediately packed in plastic foil and tested within 2 weeks. The moisture content just prior to testing was about 7 to 8 %.

6.4.2 Testing

The cores were tested in the 50 kN servo-controlled compression machine as described in Section 5.2.2. and the prisms in a similar machine but now of a 500 kN capacity. The upper and lower prism surfaces were in direct contact with the machine platens, which extended beyond the horizontal specimen perimeter. Axial displacement during testing of prisms was measured similarly as for cores, i.e. by 2 LVDT's placed between the loading platens on either side of the specimen. The average value measured by the LVDT's was taken to calculate the vertical strain. The tests on cores were performed at radial strain rate and the experiments on prisms at axial strain rate control. The constant radial strain rate for the cores and the constant axial strain rates for the various prisms ranged from about 4 * 10^-6 s^-1 to 1.5 * 10^-5 s^-1 to obtain a time to failure of about 10 minutes for all samples. No loss of moisture during the tests was measured.

Three cubic samples which were too large for the compression machines described above were tested at a servo-controlled machine at the Faculty of Civil Engineering. The vertical strain was measured in the same way. Also the strain rates were within the same range.

6.4.3 Lateral expansion of the specimen ends

Unfortunately it was difficult to measure directly to which extent the specimen was allowed to expand laterally. The LVDT could not be placed closer to the loading platen than at a distance of 1 cm. A series of loading cycles on a 5 cm high and 10 cm wide prism, from 5 to 50 % of the peak strength, were performed with the LVDT placed at 50 and 80 % of the sample height. The lateral strain at 80 % of the sample height was about 75 % of the maximum value at mid-height of the prism.
To quantify the end friction, a series of 15 shearbox experiments, using a Wykeham & Farrance machine, on a 10 by 10 cm steel-calcarenite (at natural moisture content) surface were performed at a displacement rate of \(3.0 \times 10^6\) m/s. The cohesion proved to be negligible and the angle of friction ranged from 13° to 17°. In Section 6.13 it will be shown numerically that, during uniaxial compression tests on calcarenite prisms, slip at the interface of sample and platen is more or less negligible and that the specimen ends can be considered to be perfectly confined.

6.4.4 Tilting of the machine platens

During closure the upper platen could rotate slightly to ensure accurate contact between platens and sample ends, along a spherical joint with springs along its perimeter between the upper platen and the machine. The springs had a two-fold function. At the one hand they continuously exerted tensional forces between platen and machine and ensured in this way that rotation of the platen indeed occurred if the orientation of the upper sample end deviated from a direction perpendicular to the vertical machine axis. At the other hand the springs tended to pull the upper platen into a horizontal position which should restrain excessive tilting of the platen upon failure of the specimen.

During the experiments of Dirks and Vink on specimens of W/H ratio's ranging from 1 to 4 the difference in strain, measured by the pair of LVDT's placed at both sides of the specimen, amounted to 2 millistrain maximally at the onset of failure. This corresponds to a maximum relative difference in strain of 30 % and a maximum tilt angle of 0.1°. The original alignment of the samples (see Section 6.4.1) was such that the tilt angle did not exceed 0.05°. The strain difference and tilt angle developed gradually with increasing vertical strain. Thereafter, the absolute strain difference did not change considerably any more and increased or decreased just slightly. No relation could be found between difference in strain and tilt angle at the one hand and prism strength at the other hand.

However, more serious tilting occurred during the tests on samples of smaller W/H. A closer look at the data of Dirks even revealed that during the tests on prisms of a W/H of 0.25 and 0.33 the vertical strain at one end was even decreasing again to zero after failure while at the other side strain increased. This strong amount of tilting of "slender" prisms is due to the different geometry of shear fractures which develop at failure in comparison with "flatter" specimens (see Section 6.5.2).
Fig. 6.3 Stress-strain diagram of compression test D12d on a cubical calcarenite prism of 150 mm width, accompanied by drawings showing crack development.

Fig. 6.4 Stress-strain diagram of compression test D21h on a calcarenite prism of 150 mm width and 50 mm height, accompanied by drawings showing crack development.
6.5. GENERAL FEATURES OF COMpressive STRENGTH TESTS ON CALcARENITE PRISMS

6.5.1 Three phases regarding the stress-strain relationship and observations on macroscopic deformation behaviour during the test

Figs. 6.3 and 6.4 show stress-strain plots from tests on 150 mm wide prisms of W/H ratio's of respectively 1 and 3. At or just after peak stress one or more axial cracks appeared. These cracks developed from the upper or lower prism surface close to the edges. Generally fracturing started from the upper loading platen. The pre-peak portion of the experiment is denoted here as phase I, which is characterized by a positive slope of the stress-strain curve and the absence of macro-cracking.

Subsequent to phase I stress decreased and the stress-strain curve showed a relatively steep slope. This slope is negative for all specimens of W/H ratio's of 0.5 and more, but more or less vertical for three of the samples of a W/H of 0.33 and all samples of a ratio of 0.25. Possibly class II behaviour would be measured at lateral strain rate control (see Section 5.3.2). During the decrease of stress the cracks grew in a more or less vertical direction at increasing strain. Additionally, more axial cracks developed, mainly close to the prism edges but sometimes also at a greater distance. Eventually one or more cracks extended from top to bottom of the prism. Then, at a certain strain, the slope of the stress-strain curve became significantly less negative. This change in slope marked the end of the deformation stage which is designated here phase II. It was generally observed that the first crack had grown from top to bottom before the termination of phase II (Figs. 6.3 and 6.4).

During the next stage, named here phase III, the stress hardly decreased any more or, depending on W/H, remained more or less constant, i.e. a residual stress was reached. During the beginning of this phase it could be observed that cracks started to open. At the scale of the experiments crack widening is visible when the width of the opening is about equal to grain size, i.e. about 0.1 mm. For all tests this happened before a strain of four times the strain at failure was reached (Figs. 6.3 and 6.4). Obviously the widening of the cracks was a result of the pillar sides or parts of those being pushed outwards from the pillar core. Along an axial crack often the most central part of a pillar side was pushed outwards relative to the part in the vicinity of a prism edge. At a further increase of strain new cracks were hardly formed. Instead, existing cracks expanded continuously.

6.5.2 Additional observations on macroscopic deformation structures after the test

6.5.2.1 W/H ratio's of one and more

After the completion of a test the platens parted and the prism was withdrawn from the machine and placed on a table. Generally some force was needed to separate the specimen from the platens. A thin layer of fine-grained calcarenite always remained at the platen surfaces.
Fig. 6.5 Failed calcarenite prism of 150 mm dimension (W/H of one) after the test. The final strain is more than three times the strain at failure.

Fig. 6.6 Failed calcarenite prism of 50 mm height and 150 mm width (W/H of three) after the test. The final strain is more than three times the strain at failure.
This material probably resulted from crushing of protruding calcarenite grains.

The failed prisms, deformed beyond three times the strain at failure, proved to be separated into an hourglass shaped prism core and parted prism sides. The prism sides, now relieved from the previous vertical constraint in the machine, could be overturned manually towards the horizontal plane without any exertion of force.

The fracture planes had always developed at an angle $\beta$ of $30 \pm 5^\circ$ relative to the vertical prism sides, no matter which prism shape (Figs. 6.5 and 6.6). The general fracture orientation suggests that failure occurred in shear, as has been observed for cylindrical cores which fail in uniaxial compression (Section 5.3.1). Additional evidence for pillar failure by the generation of shear planes at such an angle is given by numerical experiments (Section 6.13).

The geometry of shear fractures is depicted in Fig. 6.7. It can be inferred that the amount of shear movement tends to decrease from the prism edges towards the mid-plane, where the amount of movement is zero. However, this shear movement, and the resulting outward movement as well, are hindered because the pillar sides are compressed by the machine platens by the same amount as the central core. Some minor movement is only possible if the shear fracture has started just in the sidewall and not exactly at the edge or in the upper or lower surface. Shear movement is also allowed if crushing occurs of the thin and susceptible upper and lower ends of the sides. This crushing has been actually observed during experiments and at failing pillars in the calcarenite mines. At increasing strain the central core is continuously shortened vertically and expands in the horizontal direction. Additionally, the angle $\beta$ may decrease due to rotation of the shear planes. This aspect will be dealt with
into more detail in Section 6.5.3. As a consequence the prism sides tend to be pushed sideways.

In the mines it has often been observed that sides of failed pillars had rotated into the gallery, which generally occurs by toppling (Fig. 6.8) but sometimes by a sliding movement (Fig. 3.21). These phenomena can mainly be noticed inside collapse areas or in their direct vicinity, i.e. in areas of large strains. But pillar sides have also fallen into the gallery in areas of relatively modest strains. In these cases one or more joints were present or the shear plane intersected the pillar wall at some distance from the roof and/or floor. During the laboratory experiments the prism sides did not fall apart from the core, even at vertical strains exceeding three times the strain at failure. Possibly the strain was insufficient. Moreover, scale effects play a role here. The sideward collapse of a failed pillar/prism side is promoted by gravity, which is proportional to volume, but is restrained by friction, for example
Fig. 6.9 General geometry of failed prisms of width/ratio's of one and more.

at the contact between the upper edge of the pillar/prism side and the mine roof/machine platen, which is proportional to area. Since volume increases ten times as much as area at increasing the pillar/prism size, actual pillar sides, which are 100 times as high on the average as the sides of the tested prisms, are more likely to topple or slide sidewards.

Within the range of strain applied in these tests the central core proved to be unaffected by macroscopic, visible fracturing. However, impregnated thin sections might show relatively homogeneous deformation by pore collapse (see Loe et al., 1992) as a result of the high confining stress inside the prism core.

A three-dimensional drawing of the geometry of a failed pillar is presented in Fig. 6.9. The near-vertical cracks in the vicinity of the pillar edges, which are the first visible signs of pillar deterioration, form part of planes, still developing or complete, denoted A in the figure. Each such plane separates two separating pillar sides. Fracture planes B separate the central core from the pillar sides. The intersections of fracture planes B with the outer pillar surfaces develop close to the upper and lower pillar edges. Cracking of the pillar side away from the edges (C) occurs after the formation of cracks close to the edges. Small scale spalling (D) develops simultaneously with the crack formation near the edges.

It can be imagined that the lateral movement of the prism sides results in the observed widening of the near-vertical cracks A close to the pillar edges during phase III (see also Figs. 6.3 and 6.4).
Fig. 6.10 Observed geometries of shear planes in failed prisms of W/H ratio's of less than one.

A number of prisms which had been compressed just till the beginning of phase III also showed the formation of complete A and B shear planes. Hence it is concluded that the shear plane geometry is fully developed at the onset of phase III.

6.5.2.2 W/H ratio's of less than one

The prisms of W/H ratio's of less than one also failed in shear, but the geometry of the shear planes is different. In Fig. 6.10 some observed geometries are depicted. Generally only one major shear plane developed, often accompanied by one or more subsidiary planes of less extent and coalescing with the main plane. The variety of shear plane geometry for these prisms, as opposed to the uniform geometry for prisms of higher W/H ratio's, probably accounts for the relatively high variation of strength and stiffness values dealt with later in this chapter.

A prism will fail more or less along a single shear plane when that plane intersects the prism surface only at the sidewalls and not at the top or bottom. This is the case when W/H is less than tan \( \phi \). For a \( \phi \)-value of 30° this W/H is 0.58. The prisms of W/H ratio's of 0.25, 0.33 and 0.5 failed along a single plane.

6.5.3 Rotation of shear planes during infinitesimal axial strain

Considering prisms of W/H exceeding 0.58, at increasing vertical (axial) strain the central core and prism sides are shortened. Since shear movement between platens and sample ends can be considered negligible, the central core cannot expand in a horizontal direction at its upper and lower surfaces. But horizontal expansion tends to develop more towards the mid-plane where horizontal constraint by platens is not
Fig. 6.11 Geometry of rotating shear surface at increasing vertical strain for prisms of W/H ratio's of one and more. Strain and angle of rotation are exaggerated.

present. If the prisms sides do not resist this horizontal expansion of the core the angle $\beta$ will change (Fig. 6.11).

Here the modification of $\beta$ as a function of W/H is determined for the prism core shear surface AC in case that the remaining parts of the prism, i.e. the prism sides,
do not exist. The angle $\beta$ and half the prism height $H/2$ define the triangle ABC, which is not occupied by rock material. All strains and displacements considered here are assumed infinitesimal. This agrees with reality as long as the vertical strain is of the order of millistrains. The horizontal displacement $dx_{\text{core}}$ of the core at the mid-plane level is equal to:

$$dx_{\text{core}} = \frac{\varepsilon_{x,\text{core}}}{2} (W - H \tan \beta)$$

where $\varepsilon_{x,\text{core}}$ is the horizontal strain of the central core at mid-height. Since $\varepsilon_{x,\text{core}} = \nu \varepsilon_{y,\text{core}}$ and $\varepsilon_{y,\text{core}} = \varepsilon_{y,\text{AB}} = \varepsilon_{y}$, $dx_{\text{core}}$ can be rewritten as:

$$dx_{\text{core}} = \frac{\nu \varepsilon_{y}}{2} (W - H \tan \beta) \quad (6.1)$$

This expression also describes the horizontal displacement of point C forming part of the triangle ABC. For the horizontal strain $\varepsilon_{x,\text{BC}}$ of the segment BC we get:

$$\varepsilon_{x,\text{BC}} = \frac{dx_{\text{core}}}{BC} = \nu \varepsilon_{y} \left( \frac{W}{H \tan \beta} - 1 \right) \quad (6.2)$$

According to general theory of strain and the principle of superposition (i.e. Obert & Duvall, 1967; Section 2.3), the increase of $\beta$ is equal to:

$$d\beta = (\varepsilon_{y} - \varepsilon_{x,\text{BC}}) \sin \beta \cos \beta \quad (6.3)$$

Combination of Eqs. 6.2 and 6.3 give:

$$\frac{d\beta}{\varepsilon_{y}} = (1 + \nu) \sin \beta \cos \beta - \nu \cos^{2} \beta \frac{W}{H} \quad (6.4)$$
Fig. 6.12 $d\beta / \varepsilon_y$ as a function of $W/H$ for $\phi = 30^\circ$ and different values of $\nu$, according to Eq. 6.4.

Fig. 6.13 $d\beta / \varepsilon_y$ as a function of $W/H$ for $\nu = 0.15$ and different values of $\phi$, according to Eq. 6.4.
For calcarenite prisms with $\beta = 30^\circ$ and $\nu = 0.15$, Eq. 6.4 can be written as:

$$\frac{d\beta}{\epsilon_y} = 28.7 - 6.5 \frac{W}{H}$$

(6.5)

Note that $\beta$ is given in radians for Eqs. 6.3 and 6.4. For Eq. 6.5 $\beta$ is given in degrees.

Since crushing of the upper and lower ends of prism sides occurs (Fig. 6.11) the vertical stress on the side parts will be at least partially relieved and the orientation of the shear surfaces forming the boundary of the prism side parts tend not to change or change less than the orientation of those at the margin of the core, as determined above. Since $\beta$ changes more for the core than for the side parts, the prism elements do not fit properly. If the core surface rotates more than the surface of the side part a lack of space develops and the pillar core tends to push the side part outwards. Depending on the resistance of the side parts a confining stress develops. This confinement of the prism core by the side parts increases the prism strength, according to the confined core concept of Wilson (1972).

In Figs. 6.12 and 6.13 $d\beta/\epsilon_y$ is depicted as a function of $W/H$ according to Eq. 6.5. For $\nu = 0.15$ this confining effect develops for $W/H$ ratio’s exceeding about 4. Confinement is greatly influenced by the value of Poisson’s ratio but hardly affected by the value of the angle of friction $\phi$.

### 6.6 THE SIZE EFFECT ON PILLAR STRENGTH

Experiments were performed on cubic specimens ranging from 5 to 30 cm width. Fig. 6.14 shows no evidence for a size effect with regard to peak strength $\sigma_p$. Residual strength $\sigma_r$ was evaluated at about three times the strain at failure. At least for specimen widths of up to 15 cm this property was neither affected by size (Fig. 6.15). The post-failure behaviour was not measured for the 30 cm cubes. Previous experiments on various rock types (Section 4.2.2.2) showed that the critical size is generally about 1 m and that strength is affected by size predominantly up to an edge width of 30 cm, which corresponds with the range tested here. Since for calcarenite even within this range no size effect was measured it must be assumed that for this rock type the critical size is less than 5 cm, the minimal edge length. Such a low value may be explained by the continuous and homogeneous character of the calcarenite at this scale compared with the rock types tested by other authors. It can be concluded that an $N_{size}$ of 1 can be used for the calcarenite.
Fig. 6.14 The relationship between normalized cube strength and size.

Fig. 6.15 The relationship between normalized residual cube strength and size.
Fig. 6.16 Stress/UCS-strain diagrams for compression tests on prisms of various width to height ratio's. The graph shows the curves for tests which relate to W/H ratio’s of 0.5 to 4.

6.7 THE INFLUENCE OF W/H ON THE STRENGTH OF SQUARE-BASED PRISMS OF W/H BETWEEN 0.33 AND 4

6.7.1 Deformation behaviour and stress-strain diagrams

Figs. 6.3 and 6.4 revealed that the stress-strain curve varies for different W/H ratio’s. In Fig. 6.16 curves for W/H ratio’s ranging from 0.5 to 4 are depicted. Here experiments of Dirks are chosen which were performed up to relatively large strains. Each curve shows the representative behaviour for a certain prism shape.

At increasing W/H the following can be observed:

- peak strength and residual strength increase.

- residual strength increases as a percentage of peak strength. Pillars with a W/H exceeding 4 are almost indestructible, i.e. they may fail and fracture, but maintain more or less their initial load carrying capacity.

- the pre-failure, phase I, stress-strain slope is more or less identical, in agreement with expectation (Section 4.2.3.3).
Fig. 6.17 The relationship between peak strength, normalized by UCS, of square-based calcarenite prisms and width/height (0.25-4) ratio.

Fig. 6.18 The relationship between residual strength, normalized by UCS, of square-based calcarenite prisms and width/height (0.25-4) ratio.
the strain developed during phase II increases and the reduction of strength is diminished. Accordingly, the stress-strain slope during phase II increases, i.e. becomes less negative. This has been observed by various authors (see Section 4.3.3.5).

- the stress-strain slope during phase III increases, i.e. becomes less negative.

- additional crack development at some distance from the vertical pillar edges becomes more likely.

6.7.2 The shape function for peak strength

The shape function presented here is mainly based on the data of Dirks, i.e. for square-based prisms of W/H ratio's 0.25, 0.33, 0.50, 2.0, 3.0 and 4.0. For prisms of a W/H of 1 the results of Vink were chosen, because these experiments, although giving values of peak strength within the same range, were performed up to larger strains allowing a more complete establishment of residual strength.

In Section 4.2.2.4 shape functions were reviewed, which could be divided into power law and linear strength-width/height ratio relationships. The data presented here do not fit power law functions but can be well described by the linear relationship, based on weighted regression, $\sigma_p/\text{UCS} = 0.865 + 0.240 \ W/H$ with $R^2 = 0.93$. This equation is almost exactly similar to the Hustrulid-Goodman formula:

$$N_{\text{shape}} = \frac{\sigma_p}{\text{UCS}} = 0.875 + 0.250 \ W/H \quad (6.6)$$

This shape function was also advanced by Bekendam & Dirks (1990) and has been used for pillar stability calculations since then. Eq. 6.6 will be applied in this thesis as well. It should be stressed that it only relates to prisms of a W/H equal to or less than four. This is also the range of W/H ratio's in the mines studied. The data and the shape function are depicted in Fig. 6.17.

6.7.3 The shape function for residual strength

In Section 4.2.2.10 it was commented that strength formulae do not exist for residual strength of failed pillars. Obviously, a failed prism can still carry a certain percentage of its peak stress, depending on its geometry. As a consequence, it seems useful for pillar stability calculations to derive also formulae which describe residual strength. Figs. 6.3, 6.4 and 6.17 show that for a certain prism one unique value of residual strength does not exist. Also during phase III strength continues to decrease with increasing strain, although just slightly and less and less intensively with continuing deformation. It is decided to consider the stress at three times the strain at failure as the residual strength. The prisms of width/ratio's of less than one were deformed up to lower strains. Here two times the strain at failure is taken as the residual strength. At these strains, generally ranging from 15 to 25 millistrain for
Fig. 6.19 The relationship between peak strength, normalized by UCS, of rectangular calcarenite prisms of W/H = 1 and length/width ratio.

Fig. 6.20 The relationship between peak strength, normalized by UCS, of rectangular calcarenite prisms of W/H = 1 and length/width ratio.
W/H ratio's between 1 and 4, cracks had started to open in all samples. This range of W/H ratio's applies to almost all actual pillars in the calcarenite mines.

Weighted regression yields that the residual strength measured on the calcarenite samples can be described by:

\[
\sigma_r / UCS = 0.118 + 0.350 \frac{W}{H} \quad (6.7)
\]

with \( R^2 = 0.95 \) (Fig. 6.18).

### 6.8 THE INFLUENCE OF LENGTH/WIDTH RATIO ON THE SHAPE EFFECT

Generally calcarenite mine pillars are not square-based. Using the smallest horizontal dimension as width results in an underestimation of pillar strength. Therefore experiments were undertaken by Vink (1990) on prisms of a length/width ratio not equal to one. The calcarenite prisms were 50 mm high and 50 mm wide (W/H = 1) and their length \( L \) varied between 50 and 200 mm. Peak strength and residual strength at three times the strain at failure were measured (Figs. 6.19 and 6.20). From the 44 tests 19 were performed up to sufficient strain to measure this last characteristic. An attempt was undertaken to establish linear relationships:

\[
\sigma_p / UCS = 1.03 + 0.086 \frac{L}{W} \quad (6.8)
\]

\[
\sigma_r / UCS = 0.401 + 0.046 \frac{L}{W} \quad (6.9)
\]

Combination with Eqs. 6.6 and 6.7 give respectively:

\[
\frac{\sigma_p}{\sigma_{p, W/H=1}} = 0.92 + 0.076 \frac{L}{W} \quad (6.10)
\]

\[
\frac{\sigma_r}{\sigma_{r, W/H=1}} = 0.87 + 0.10 \frac{L}{W} \quad (6.11)
\]

The correlation coefficients were just 0.50 and 0.31 respectively. Although a certain increase in strength seems to accompany an increase in \( L/W \), the data do not allow the establishment of a quantitative relationship.

### 6.9 L-SHAPED PRISMS AND ALTERNATIVE SHAPE FORMULAE

Most pillar basal planes are not quadrangular but of more irregular shape. Some L-shaped prisms were tested in uniaxial compression by Dirks (1990). Their height was 50 mm. From the originally 150*150 mm basal plane, for 6 specimens 50*50 mm
was removed from one of the edges. For 6 other samples 100*100 mm was removed. Width was taken to be the shortest prism side flanked at both sides by more or less parallel sides (Fig. 6.21). Figs. 6.22 and 6.23 show, respectively for peak strength and residual strength, a compilation of all test results including those from L-shaped prisms. The rectangular prisms of L/W ratio's unequal to one, dealt with in the previous section, plot at one W/H coordinate and generally show a higher peak strength than predicted by the general shape formulae. This can be expected because an increased length offers an extra lateral support to the prism. The L-shaped samples of both W/H = 1 (100*100 mm removed) and W/H = 2 (50*50 mm removed) also plot above the general strength curves, particularly regarding residual strength. It can be concluded that by taking the smallest horizontal dimension as width for non square-based prisms the general shape equations underestimate peak- and residual strength.

Alternative shape formulae were developed to describe the strength of irregular prisms better. First attempts were made by Vink (1991) and refined by the author. In this regard it has to be noted that for prisms and pillars failure occurs at the sides leaving a more or less intact hourglass-shaped core. For all shapes it has been observed that the hourglass profile is characterized by two 30° fractures (see Sections 6.5.2 and 6.5.3). Hence for a certain height one unique value results for the width of the fractured zone at the prism/pillar perimeter. Obviously the strength of the prism sides is less than that of the prism core. It can be imagined that the larger the basal plane circumference C relative to the basal plane area A the higher the percentage of the pillar or prism will be fractured and part from the central core lowering its strength. Accordingly, the ratio A/C should be used in shape formulae instead of W. By substituting W by 4A/C the strength of square based prisms will not be affected and relation 6.6 becomes:

\[ \sigma_p / UCS = 0.875 + A/CH \]  

(6.12)
Fig. 6.22 The relationship between normalized prism strength of various basal plane shapes and width/height ratio.

Fig. 6.23 The relationship between normalized residual prism strength of various basal plane shapes and width/height ratio.
**Fig. 6.24** The relationship between normalized prism strength of various basal plane shapes and A/CH.

**Fig. 6.25** The relationship between normalized residual prism strength of various basal plane shapes and A/CH.
and relation 6.7 becomes

\[
\sigma_r / UCS = 0.118 + 1.40 \frac{A}{CH}
\]  

(6.13)

Later the author found that taking 4A/C as effective width was already proposed by Wagner (1974). Eq. 6.12 generally shows a better fit for the peak strength of rectangular prisms of L/W ratio not equal to 1 (Fig. 6.24). However, the peak strength of L-shaped prisms is sometimes overestimated by Eq. 6.12. Residual strength is generally slightly better characterized by Eq. 6.13 than by Eq. 6.7 (Fig. 6.25), but Eq. 6.13 may overestimate residual strength of not square-based prisms.

6.10 SHAPE FUNCTIONS FOR PRISMS OF W/H RATIO'S EXCEEDING FOUR

6.10.1 Introduction

While Eqs. 6.6 to 6.13 describe peak and residual strength reasonably well for prisms of W/H ratio's of up to 4, it is uncertain in how far these formulae apply to "flatter" prisms. According to Eqs. 6.6 and 6.7 residual strength becomes equal to peak strength at a W/H of 7.57. At higher values of W/H residual strength would even exceed peak strength. Since this is obviously impossible it can be concluded that these equations may not be used for prisms of W/H ratio's exceeding 4. The combination of a power law equation for low W/H ratio's and the squat-pillar shape formula for W/H ratio's exceeding a so-called critical W/H ratio (Section 4.2.2.6) predicts a peak strength which increases less and less strongly with W/H ratio up to the critical value and then accelerates (Fig. 4.5). In the next sections the strength of squat calcarenite prisms will be investigated.

6.10.2 Peak strength

The results of additional experiments on prisms of larger W/H ratio's were combined with those from previous tests on square-based prisms. The new series comprised 14 experiments on square-based prisms of a W/H of up to 9.72. The general trends with increasing W/H ratio listed in Section 6.7.1 were also observed for the new series of tests.

Neither power law functions nor linear relationships appeared to describe the data well. The best fit by far was achieved by a logarithmic equation, albeit only for W/H ratio's of at least one (Fig. 6.26):

\[
\sigma_p / UCS = 1.150 + 0.962 \log (W/H)
\]  

(6.14)

with R² = 0.94. For W/H ratio's of less than one this equation underestimates prism strength. In Fig. 6.27 both the linear function 6.6, valid for W/H ratio's of up to four, and the logarithmic function, valid for W/H ratio's of one and more, are depicted in a graph with linear axis. Clearly the linear function greatly overestimates
Fig. 6.26 The relationship between normalized prism strength of square-based prisms and width/height ratio (up to nine; log-normal scale).

Fig. 6.27 The relationship between normalized strength of square-based prisms and W/H. The "classic", linear shape function and the "new" equation, valid for W/H exceeding four, are depicted.
Fig. 6.28 The relationship between normalized residual prism strength of square-based prisms and width/height ratio (up to nine; log-normal scale).

Fig. 6.29 The relationship between normalized residual strength of square-based prisms and W/H. The linear shape function and the "new" equation, valid for W/H exceeding four, are depicted.
Fig. 6.30 The relationship between normalized residual prism strength at the phase II-III transition of square-based prisms and width/height ratio (up to nine; log-normal scale).

Fig. 6.31 The ratio of residual strength to peak strength as a function of width/height ratio.
prism strength for W/H ratio's exceeding four.

6.10.3 Residual strength

Regarding residual strength power law and linear relationships neither fitted the data. Analogous to peak strength, residual strength at about three times the strain at failure is well described by a logarithmic function from a W/H ratio of one (Fig. 6.28):

$$\sigma_r / UCS = 0.450 + 1.600 \log (W/H)$$ (6.15)

with $R^2 = 0.97$. This equation underestimates residual strength for W/H ratio's of less than one. Fig. 6.29 shows that the linear function 6.7 overestimates residual strength for W/H ratio's exceeding four.

Finally, residual strength $\sigma_r'$ was measured at the marked change in the post-failure stress-strain slope, which is denoted as the transition from phase II to phase III deformation (Figs. 6.3 and 6.4). A relationship comparable to Eq. 6.15 was found:

$$\sigma_r' / UCS = 0.573 + 1.508 \log (W/H)$$ (6.16)

with $R^2 = 0.96$ (Fig. 6.30).

6.10.4 Discussion

For W/H ratio's exceeding four both peak strength and residual strength increase less strongly with W/H ratio as predicted by linear equations, developed for W/H ratio's of up to one. The slopes of the strength-W/H ratio curves continuously decrease, at least up to a W/H ratio of nearly ten. Accordingly, the critical W/H ratio, if it exists, must exceed the maximum value tested here. Residual strength is approaching peak strength at increasing W/H ratio (Fig. 6.31). For W/H ratio's exceeding ten a stress drop at failure probably hardly exists. This agrees with the geometrical consideration that from the total horizontal prism area the percentage of failure zone area at the perimeter of a prism decreases with increasing W/H ratio.

The reason that Eqs. 6.14 to 6.16 do not describe the peak and residual strength of prisms of W/H ratio's of less than one can be attributed to the difference in failure and shear zone geometry. In Section 6.5.2.2 it was outlined that calcarenite prisms of a W/H ratio of less than 0.58 failed more or less along one single shear plane (Fig. 6.10), while only for higher W/H ratio's the typical hour-glass shaped prism core develops, which is separated by shear surfaces from prism side parts (Figs. 6.7 and 6.9). As a consequence, it can be imagined that prism strength equations will be different according to shear zone geometry.
6.11 AN ANALYTICAL MODEL OF RESIDUAL STRENGTH

In the previous section it was found that residual strength approaches peak strength at increasing $W/H$ ratio. Here an analytical model is presented to explain this effect for square-based prisms of $W/H$ ratio’s which are characterized by the hour-glass shaped core geometry, i.e. of $W/H$ ratio’s exceeding 0.58. As residual strength $\sigma_r$ is taken. This value corresponds with the deformation stage where the shear zone geometry has just completed. Peak strength $\sigma_p$ agrees with the deformation stage at the onset of shear zone development.

The model is based on the individual load carrying capacities of prism core and prism sides. Fig. 6.32 shows the geometry of a failed square-based prism from above. The width of the core $W_c$ is equal to:

$$W_c = W - H \tan \beta$$

(6.17)

It can be derived that the ratio of prism area $A_p$ and core area $A_c$, both in horizontal cross-section, is equal to:

$$\frac{A_c}{A_p} = 1 - \frac{2 \tan \beta}{W/H} + \frac{\tan^2 \beta}{(W/H)^2}$$

(6.18)

Notice that this ratio depends on shape, but not on size. The total load $p$ is divided over the prism core ($p_c$) and the prism sides ($p_s$). The total load $p_p$ at the onset of failure equals:

$$p_p = \sigma_p A_p$$

(6.19)
When the residual strength \( \sigma_p' \) has been attained the load on core and sides deviates considerably (e.g. Wagner, 1974). The total prism load \( p_r' \) is the sum of the residual loads on the core \( p_{r,c}' \) and on the sides \( p_{s,s}' \):

\[
p_r' = p_{r,c}' + p_{r,s}'
\]

or

\[
\sigma_r' A_p = \sigma_{r,c}' A_c + \sigma_{r,s}' A_s
\]  
(6.20)

where \( \sigma_{r,c}' \) and \( \sigma_{r,s}' \) are the mean residual stresses on core and sides respectively. The total horizontal area of the sides is denoted by \( A_s \). Thus, the ratio of peak strength and residual strength is:

\[
\frac{\sigma_r'}{\sigma_p} = \frac{A_c}{A_p} \frac{\sigma_{r,c}'}{\sigma_p} + \frac{A_s}{A_p} \frac{\sigma_{r,s}'}{\sigma_p}
\]  
(6.21)

It is tentatively assumed that the residual strength \( \sigma_{r,s}' \) of the prism sides is a certain, fixed percentage of peak strength, no matter the prism shape. As a consequence, since peak strength increases with W/H ratio, this also applies to \( \sigma_{r,s}' \). This corresponds with the effect outlined in Sect 6.5.3, that the confining effect of the side parts might increase with W/H ratio. This confining action would then appear as an increase of the residual strength of the side part.

Since \( A_p = A_c + A_s \), Eq. 6.21 can be reformulated as follows:

\[
\frac{\sigma_r'}{\sigma_p} = \left( \frac{\sigma_{r,c}'}{\sigma_p} - \frac{\sigma_{r,s}'}{\sigma_p} \right) \frac{A_c}{A_p} + \frac{\sigma_{r,s}'}{\sigma_p}
\]  
(6.22)

The model can be tested by plotting \( \sigma_r'/\sigma_p \) as a function of \( A_c/A_p \). A linear relationship should result. Fig. 6.33 shows such a function indeed, which describes the ratio of residual strength to peak strength as:

\[
\frac{\sigma_r'}{\sigma_p} = 0.63 \frac{A_c}{A_p} + 0.39
\]  
(6.23)
Fig. 6.33 $\sigma_{t}/\sigma_p$ as a function of $A_c/A_p$ for square-based prisms of W/H ratio's from one to nine.

Fig. 6.34 The relationship between $\sigma_{t}/\sigma_p$ ratio and W/H ratio, including the analytically derived Eq. 6.23.
Thus, the assumption made above proves to be valid and ratio’s of the residual strength of respectively the core and the sides to peak strength are:

\[
\frac{\sigma_{r,c}'}{\sigma_p} = 1.02 \quad , \quad \frac{\sigma_{r,s}'}{\sigma_p} = 0.39
\]  

(6.24)

The residual strength of the core \(\sigma_{r,c}'\) is more or less equal to the peak strength \(\sigma_p\), which is measured over the whole pillar. The residual strength of the sides \(\sigma_{r,s}'\) is considerably lower. The general concept of the shape effect outlined in Section 4.2.2.3 is confirmed here for the post-peak stage. This concept says that the pillar load is mainly supported by the central core. If W/H is elevated, the area of the central core increases relative to that of the sides. As a result, the residual strength increases according to width/height ratio.

These equations show that the ratio of residual to peak strength is dependent on shape and not on size. The shape function is depicted in Fig. 6.34 and fits the measured data well. Also the function, which considers \(\sigma_{r}'/\sigma_p\) equal to the \(A_c/A_p\) ratio, is shown. Such a function infers that residual strength would completely be attributed to the core. While it plots below function 6.23, it is illustrated that the prism sides indeed seriously contribute to residual strength.

That prism sides remain carrying some load is supported by the observation of additional fracturing during phase III deformation at some distance from the prism edges.

6.12 POST-PeAK MODULUS

6.12.1 Results of laboratory experiments

Regarding global mine stability post-peak pillar stiffness is an important parameter (see Section 4.3). Stiffness is a function of E-modulus, width and height, according to Eq. 4.14a. The steepest negative post-failure slope is always to be measured during phase II deformation. Accordingly, this phase is considered here. For each experiment the minimum (most-negative) and average values of phase II post-failure modulus \(E_{pf}\) were determined. Since the prisms did not originate from the same calcarenite sample, the values of \(E_{pf}\) were normalized by the pre-failure tangent E-modulus. Fig. 6.35 shows the relation of average \(-E_{pf}\) as a function of W/H ratio. The values for W/H ratio’s of less than one are considerably higher than for the remaining shapes. For W/H=0.25 \(-E_{pf}\) is not depicted because the slope is more or less vertical. In Fig. 6.36 only the values for W/H ratio’s of at least one are depicted. \(-E_{pf}/E\) decreases with W/H, which also seems to apply for the variation of measured values. It should be noted that for a given specimen the maximum value of \(-E_{pf}/E\) can amount to twice the average value.
Fig. 6.35 The relationship between normalized post-peak modulus and width/height ratio (from 0.33 to 10).

Fig. 6.36 The relationship between normalized post-peak modulus and width/height ratio (from 1 to 10).
Fig. 6.37 The relationship between normalized post-peak stiffness and width/height ratio.

Fig. 6.38 The relationship between normalized post-peak modulus and cube size (W/H = 1).
Fig. 6.39 Geometry and diagram of mechanical elements for prisms of W/H ratio's of less than tan φ (A) and more than tan φ (B).

Since post-peak stiffness λ should be considered for the assessment of general mine stability, this value was evaluated as well from the average values of post-peak moduli, using Eq. 4.14. However, Fig. 6.37 shows just scattered data. No correlation of λ/E with W/H can be delineated. Thus unfortunately no practical data could be acquired for the assessment of post-pillar stiffness.

In previous experiments (i.e. Wagner, 1974, 1995; Van Heerden, 1975) only the height was varied while the width remained constant. Thus it was not known whether a change in post-failure modulus was due to a change in height or in width/height ratio. Width and height of the tested calcarenite prisms are indicated in Fig. 6.35. For W/H ratio's between 0.33 and 1 only height is varied while for W/H ratio's between 1 and 4 this is only the case for width. In both cases the modulus changes. Hence post-peak modulus is both affected by width and height.

In an attempt to achieve more certainty about a possible size effect on post-failure modulus values for differently sized cubical prisms were evaluated (Fig. 6.38). There is no evidence of a size effect, but due to the large amount of scatter this effect can certainly not be excluded. Therefore it is concluded that the post-failure modulus is at least for an important part determined by shape, as far as the W/H ratio is such that the geometry of the failed prism is characterized by an hour-glass shaped core and sides, i.e. W/H > 0.58. It was noted previously that for smaller W/H ratio's the shear fracture geometry is different. Here a strong size effect has been actually
Fig. 6.40 Failed pillar in the Geulhemmer Groeve. The front side has collapsed and the pillar side at the right has rotated somewhat. The pillar height is 3 m.

demonstrated by Labuz & Biolzi (1991). In the next section both separate geometries of failure will be analyzed in order to explain the observed differences in post-peak behaviour.

6.12.2. The influence of shear fracture geometry on post-peak behaviour

Prisms of a W/H ratio of less than 0.58 fail more or less along one shear plane (Fig. 6.39). At least at one end of the plane shear movement is not constrained by an end platen. If some crushing at the top of the left prism part (Fig. 6.39a) occurs and/or some rotation of the upper end platen develops, for the sliding movement of both parts of the prism past each other only the resistance at the shear surface itself has to be overcome. If the end-platens are of the same diameter as the specimen, as in UCS tests, crushing at the sample ends is not involved at all. The same amount of
shear movement can develop over the whole plane. The situation for prisms of higher W/H ratio's is quite different (Fig. 6.39b). Now shear movement is everywhere constrained by end platens and decreases towards zero at the prism mid-plane.

As a result of the more or less free, unconstrained shear fracturing in the situation of Fig. 6.39a, the specimen becomes unloaded for a major part and the elastic strain decreases, i.e. both parts of the prism expand elastically in the axial direction. This effect has been measured by He et al. (1990; see also Section 5.3.2). This axial expansion is converted into non-elastic strain, i.e. slip and propagation of the shear fracture. As a simplification, the prism can be described by two mechanical elements in series, one representing non-elastic strain and the other one elastic strain. The increase of non-elastic strain is for a certain part compensated by a decrease of elastic strain. As a consequence, the net increase in axial strain during phase II deformation is limited. This is observed as a steep post-peak slope. The decrease of elastic strain can even be faster than the increase of non-elastic strain. In this case, if the experiment is performed under certain conditions, a net decrease of strain can be measured, which is denoted as class II post-peak behaviour (see also Section 5.3.2). Additionally, the strong tilting for prisms of small W/H ratio's, dealt with in Section 6.4, can be understood by considering this geometry.

In the situation of Fig. 6.39b the geometry is considerably less favourable for shear movement. As a consequence, non-elastic strain is not localized at one shear plane but develops at the complete prism perimeter. Elastic strain energy stored in the prism can hardly or not be released. Both central core and prism sides remain stressed by the platens. Hence, upon failure the net-strain is hardly reduced due to a decrease of elastic strain. Accordingly, it can be expected that the post-peak slope is less steep and class II behaviour must be considered impossible. Indeed, class II behaviour for prisms of W/H ratio's of more than 0.58 (more or less, depending on the angle of friction) have not been reported. Furthermore, in the situation of Fig. 6.39b the prism cannot be just represented by mechanical elements in series as in Fig. 6.39a. Now a more complex system applies, comprising one or more elements for the sides at the one hand and one or more elements for the central core at the other hand, located in parallel. Even if some elastic unloading would occur in the prism sides the effect on the total stress-strain curve would be moderate due to the presence of the core. This applies more and more with an increase of W/H.

6.13 NUMERICAL EXPERIMENTS TO ASSESS THE EFFECT OF END CONSTRAINTS ON STRESS DISTRIBUTION AND DISPLACEMENT PATTERN

6.13.1 Introduction

It has been observed that actual pillars fail in the same way as prisms in the laboratory. Observations of crack development and of the geometry of prism failure in the laboratory match those performed on actual pillars in the mine (Figs. 6.5 and 6.40). However, in Section 6.3 it became clear that differences of stress distribution,
displacement pattern and peak strength may exist between the experimental setup and the configuration in the mine. Particularly the amount of lateral confinement at the pillar/prism ends proved to be important. For the experimental setup the amount of this confinement depends on the extent of slip which is possible at the calcarenite-steel interface.

To study these aspects numerical experiments were performed. The two-dimensional finite difference program FLAC (Fast Lagrangian Analysis of Continua; version 3.30) was used. FLAC is an explicit program: for small calculation steps a disturbance at a given gridpoint is only experienced by the gridpoints in the direct vicinity. During the successive calculation steps the model approaches an equilibrium situation. At each step out-of-balance forces are determined at each gridpoint from zone stresses. These give rise to gridpoint displacements and strain rates, resulting in new zone stresses etc. The "maximum unbalanced force" expresses in how far equilibrium has been attained. During the calculation this unbalanced force approaches zero. In FLAC Lagrange-equations are used. This means that the model is not geometrically linear, i.e. changes in geometry during the calculation affect the stress situations.
6.13.2 Outline of numerical experiments

Here only the elastic behaviour of pillars/prisms is studied. Still the distribution of Hoek-Brown safety factors throughout the model was established, to get an impression of where the stress state is least favourable and where failure may occur. This safety factor at a given point is defined as the ratio of the major principle stress at failure, according to the Hoek-Brown failure criterion, and the local major principle stress at that point. Despite the consideration of these safety factors, non-elastic behaviour is not applied to the model.

By symmetry, only the right upper quarter of the pillar/prism was modelled. The pillar and prism quarters are of a W/H ratio of one, and each consists of the same number of 40 × 40 gridzones, to avoid grid-size effects. Plain strain conditions were applied corresponding with infinite pillar/prism length, i.e. the dimension perpendicular to the paper.

According to the test results of Chapter 5, the following material properties were used for the calcarenite: bulk modulus = 1000 MPa, shear modulus = 450 MPa, density = 1500 kg/m³, and Hoek-Brown constants m = 5.34 and s =1 with UCS = 2.36 MPa. The steel platen was characterized by: bulk modulus = 175 GPa, shear modulus = 80 GPa and density = 7800 kg/m³, according to the specifications of the manufacturer.

In test 1 the deformation behaviour of a uniaxially loaded calcarenite prism was modelled, which was in direct contact with the steel platens. It was assumed that no slip is possible at the steel-calcarenite interface, resulting in perfect lateral confinement at the specimen ends. The model comprised a 40 × 51 grid (Fig. 6.41). The lower 40 rows of gridzones represent the calcarenite prism and the upper 10 rows the steel platen. The single row in between constitutes a "glued" interface, where any slip or separation is prevented. For the row of interface zones a zero thickness was modelled. The top of the steel platen was treated as a rigid boundary and was modelled by coupling vertical displacement of all gridpoints at the top and by prescribing zero horizontal displacement. Also along the vertical symmetry plane on the left no horizontal displacement was allowed. Along the horizontal symmetry plane at the bottom zero vertical displacement was prescribed. The upper boundary of the model was subjected to a constant vertical "velocity" (i.e. displacement per calculation step), imitating a constant strain rate experiment.

Test 2 was similar to test 1, except for the modelling of a true steel-calcarenite interface, characterized by the experimentally determined zero cohesion and the 15° average angle of friction (see Section 6.4.3). If interface slip is allowed according to these values of cohesion and friction angle, the amount of slip is determined by the stiffness contrast of the materials on both sides.

In test 3 actual pillar configuration was considered, albeit without an immediate hardground roof (Fig. 6.42). The quarter of the pillar measured 40 × 40 gridzones
and was overlain by a roof of 200 rows of gridzones. Previous experiments had shown that for this ratio of overburden thickness and pillar height, equal to 2.5, pillar and upper model boundary are outside one another's zones of influence. Boundary conditions and symmetry planes were identically defined as for both previous tests, except for additionally prescribing zero horizontal displacement at the right boundary of the model, in agreement with horizontal confinement of the calcarenite roof.

6.13.3 Results

6.13.3.1 Horizontal displacements and interface slip

Fig. 6.43 depicts the horizontal displacements of the calcarenite for test 1 (perfect confinement) and test 2 (calcarenite-steel contact) at the upper boundary of the prism (i.e. for \( y = H \)) from the centre (\( x/W = 0.5 \)) to the prism edge (\( x/W = 1.0 \)), normalized by the vertical displacement of the steel platen at \( y = H \). As for the following graphs, a positive value of normalized horizontal displacement corresponds to displacement to the right, i.e. away from the pillar centre. According to expectation these horizontal displacements are insignificant for test 1. The maximum value of 0.25 % at the prism edge results from minor elastic deformation of the steel platen. For test 2 horizontal displacements diverge from those of test 1, i.e. interface slip occurs, only near the prism edge. But even then the maximum slip at the prism edge does not exceed 3.5 %.

In Fig. 6.44 a similar comparison is made between test 2 (calcarenite-steel contact) and test 3 (pillar geometry). Horizontal displacements at the upper pillar boundary are maximally 6 % at about 0.87 pillar width, but strongly decrease to about 1 %.
Fig. 6.43 Horizontal displacements of the calcarenite for test 1 (perfect confinement) and test 2 (calcarenite-steel contact) at the upper boundary of the prism (i.e. for $y = H$) from the centre ($x/W = 0.5$) to the prism edge ($x/W = 1.0$), normalized by the vertical displacement of the steel platen at $y = H$.

Fig. 6.44 Horizontal displacements of the calcarenite for test 2 (calcarenite-steel contact) and test 3 (pillar geometry) at the upper boundary of the prism/pillar (i.e. for $y = H$) from the centre ($x/W = 0.5$) to the prism edge ($x/W = 1.0$), normalized by the vertical displacement at the upper prism/pillar boundary.
Fig. 6.45 Normalized horizontal displacements of the vertical prism walls (i.e. for x = W), from the mid-plane of the prism (y/H = 0.5) to its upper boundary (y/H = 1.0), for test 1 (perfect confinement) and test 2 (calcarenite-steel contact).

Fig. 6.46 Normalized horizontal displacements of the vertical prism/pillar walls (i.e. for x = W), from the mid-plane (y/H = 0.5) to the boundary (y/H = 1.0), for test 2 (calcarenite-steel contact) and test 3 (pillar geometry).
Fig. 6.47 The distribution of vertical stress (upper right quadrant), normalized by the mean vertical stress, for test 1 (A), test 2 (B) and test 3 (C).
Fig. 6.48 The distribution of horizontal stress (upper right quadrant), normalized by the mean vertical stress, for test 1 (A), test 2 (B) and test 3 (C).
Fig. 6.49 The distribution of Hoek-Brown safety factors (upper right quadrant) for test 1 (A), test 2 (B) and test 3 (C).
at the pillar edge. The small amount of horizontal expansion for test 3, which is even less than for test 2, is a result of the horizontal confinement of the calcarenite roof.

In Fig. 6.45 the normalized horizontal displacements of the vertical prism walls (i.e. for \( x = W \)), from the mid-plane of the prism (\( y/H = 0.5 \)) to its upper boundary (\( y/H = 1.0 \)), are shown for tests 1 and 2. Except exactly at the pillar corner, over the whole prism height hardly any difference in horizontal prism expansion is to be detected. The ratio between horizontal and vertical strain, which is equal to the ratio between horizontal and vertical displacement because \( W = H \), is about 0.44 at midheight of the pillar. This value, in excess of the Poisson’s ratio of 0.30, can be explained by the plain strain condition. More or less the same applies to the comparison between the horizontal displacements for test 2 and 3 (Fig. 6.46).

6.13.3.2 Stress distributions

Contours of \( \sigma_v/p \) and \( \sigma_h/p \) for the parts of the models which correspond to prism or pillar are drawn in Figs. 6.47 and 6.48 respectively. Here \( p \) is the mean vertical stress across the horizontal mid-plane of the pillar/prism, and \( \sigma_v \) and \( \sigma_h \) denote the vertical and horizontal stress respectively. Clearly hardly any difference exists between the stress distributions of test 1 and 2. However, the stress distribution for the prism configuration (test 1 and 2) diverges from that for the pillar geometry (test 3).

The prism shows \( \sigma_v/p \) values exceeding 1 only in the upper corner, while for the pillar this is the case in a much more extensive area. The stress concentration in the corner is more significant for the pillar than for the prism. This relatively strong stress concentration in the upper corner for the pillar could be explained by arching effects. As a compensation, within the upper part of the core \( \sigma_v/p \) is generally less for the pillar than for the prism.

\( \sigma_h/p \) is for both pillar and prism more or less zero at the pillar edge and increases gradually towards the core. The pillar, however, shows a marked stress concentration of about 1 in the upper corner, which is compensated by smaller stresses, relative to the prism, in the upper part of the core.

6.13.3.3 Hoek-Brown safety factors

Since all tests were run until \( p \) was equal to 1.40 MPa, Hoek-Brown safety factors, calculated using principle stresses \( \sigma_v \) and \( \sigma_h \) as variables, could be compared (Fig. 6.49). At this value of \( p \) safety factors of less than 1 just appeared in the extreme upper corner of the pillar, indicating the onset of local failure (for the chosen grid size). As for the vertical and horizontal stresses, the distribution of safety factors is more or less identical for test 1 and 2. But this distribution for these prism configurations diverges from that for the pillar model (test 3). Inside the prisms the minimum safety factors also exist in the upper corner, but these are always greater.
than for the pillar. This is important considering brittle failure and pillar/prism strength. In the core the safety factors for prisms and pillars are somewhat different as well, but this is less relevant because pillar failure does not occur in that area.

6.13.4 Discussion and conclusions

The numerical experiments indicate that, during uniaxial laboratory compression experiments on calcarenite prisms, slip along the specimen-platen interface is small, relative to the vertical platen displacement, and limited to the vicinity of the prism edge. This interface slip is so small that stress distributions, Hoek-Brown safety factors and horizontal prism expansion at different vertical levels of the prism are more or less similar to those for a configuration of perfect horizontal confinement at the specimen ends.

According to the distribution of Hoek-Brown safety factors, failure of pillars and prisms occurs within the brittle regime and in shear. Failure starts at the pillar corners. This agrees with laboratory and field observations. The pattern of safety factors appear to indicate shear fractures at angles of 10° to 30° to the pillar walls. However, since the initial, local failure brings about a redistribution of load, these experiments, which just studied elastic deformation, cannot predict the actual geometry of the eventual shear fracture.

For prisms the safety factors near the corner are somewhat higher than for pillars. Thus, the elastic stress distribution is slightly more favourable for the laboratory setup than for the actual mine situation. Hence strength of pillars might be slightly less than that of prisms and the shape formulae developed in the preceding sections might overestimate pillar strength to some degree. But it should be noted again that these experiments do not give a clue of what happens when one or more gridzones fail. To predict actual differences in strength, more sophisticated constitutive models, including plasticity and strain-softening, should be applied. Such models, which describe the complete stress-strain behaviour are difficult to establish. Nevertheless the obtained elastic stress distributions do indicate qualitatively whether differences in strength can be expected between experimentally tested prisms and mine pillars.
Table 6.1 Modified classification of pillar damage.

<table>
<thead>
<tr>
<th>Class</th>
<th>Description of pillar condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No cracks</td>
</tr>
<tr>
<td>2</td>
<td>Superficial spalling and/or cracks only at the top or at the bottom of the pillar</td>
</tr>
<tr>
<td>3</td>
<td>Cracks going completely from top to bottom of the pillar</td>
</tr>
<tr>
<td>4</td>
<td>&quot;Completely failed&quot; pillar: one or more sides parted from the central pillar mass (hour-glass geometry)</td>
</tr>
</tbody>
</table>

6.14 APPLICATION OF RESULTS IN THE MINES

6.14.1 Pillar classification

6.14.1.1 A modified pillar classification system

The relation between fracture development and stress-strain diagrams (Section 6.5) can be utilized to evaluate and eventually redesign pillar classification systems. Since this relationship applies, strictly speaking, to constant strain rate experiments on prisms in the laboratory, classification is analyzed initially on the basis of these tests.

Obviously for unfractured prisms the stress $\sigma$ has not reached the peak strength $\sigma_p$. For these prisms, which can be rated class 1 according to the classification system of Van Steveninck (Table 4.5), formulae of peak strength apply. Prisms which are fractured, no matter to which extent, have been stressed beyond $\sigma_p$ and are considered as failed. After failure the stress decreases. Thus for a fractured prism the stress ranges somewhere between peak strength $\sigma_p$ and residual strength $\sigma_r$. If fractures have not developed from top to bottom (class 2 and 3 of Van Steveninck), deformation is in phase II (Figs. 6.3 and 6.4) and prism stress has decreased to a value of $\sigma_p$ to $\sigma_r’$. Prisms which are fractured from top to bottom (class 4 of Van Steveninck) are deformed within phase II if the fractures have visibly opened. If cracks have visible apertures, phase III deformation applies and the prism stress is equal to $\sigma_r’$. Completely failed prisms (class 5 of Van Steveninck), characterized by an hour glass geometry, are all deformed beyond phase II and prism stress is less than $\sigma_r’$, but in excess of $\sigma_r$. Since the difference between class 2 and class 3 is not important in terms of deformation phase and stress level, it was chosen to withdraw class 3 from the system. Experience in the mines has also revealed that such pillars hardly ever occur. Now the classification scheme, presented initially by Price & Bekendam (1991), comprises four classes (Table. 6.1; Fig. 6.50).
Fig. 6.50 The modified classification system used for describing pillar damage in the calcarenite mines (modified after Price & Bekendam, 1991).

Additionally, the classification allows an estimation of the stress levels at present relative to peak and residual strength:

class 1: \[ \sigma < \sigma_p \]
class 2: \[ \sigma' < \sigma < \sigma_p \]
class 3: \[ \sigma < \sigma_p \]
class 4: \[ \sigma < \sigma'_r \]

This relationship between pillar class and present stress level applies in general, also if creep is considered. However, due to creep the estimation of stress is less precise. Actually long-term strength, here denoted as \( \sigma_h \), should be regarded in stead of short-term strength \( \sigma_p \). For example, for a class 1 pillar two possibilities arise: \( \sigma < \sigma_h \), i.e. the pillar will never fail (unless unfavourable load redistribution occurs), \( \sigma_h < \sigma < \sigma_p \), i.e. the pillar will fail in the future.
6.14.1.2 Influence of joints

Also the influence of joints should be considered. Cracks sometimes develop along pre-existing joints, i.e. the joint plane is opened at the pillar margin. In this way even continuous cracks from the top to the bottom of the pillar can be formed and the pillar should be characterized as class 3 according to the classification system. But the pillar as a whole did not suffer the amount of vertical shortening, fracture damage and loss of load bearing capacity which are typical for a class 3 pillar. Accordingly, if no continuous cracks are observed elsewhere, the pillar will be characterized as class 2, despite the apparent class 3 damage. The same applies to an apparent class 2 pillar with fractures at the top or the bottom of the pillar along joints. Such pillars are denoted class 1 if no other damage has developed.

6.14.1.3 Influence of small-scale irregularities in pillar outline

Finally attention should be paid to small pillar projections (Fig. 6.51a). Sometimes fractures only occur at these protrusions while the rest of the pillar is intact. If the protrusion forms a relatively insignificant volume part of the whole pillar then it can still be reasonable to apply class 1 for that pillar. Fracturing of the projection can be considered as a warning for eventual later damage of the main pillar mass.

6.14.1.4 Other classification systems

In 1993 another classification system was introduced by Pakalnis et al. For sulphide pillars they distinguished five classes. Their classes 1, 2 and 5 agreed with classes 1, 2 and 4 of Price & Bekendam. Their class 3 is characterized by fracture lengths of less than one-half pillar height and fracture apertures less than 5 mm, and class 4 by fracture lengths greater than one-half pillar height and fracture apertures between 5 and 10 mm. It is probably a good idea to subdivide class 3 of Price & Bekendam in the future because it is not clear whether such pillars are deformed beyond phase II or not.

6.14.2 Determining peak strength and residual strength

Since no size effect on pillar strength has to be considered for calcarenite only shape factors are involved. As noted previously, creep deformation is not taken into account at this stage of the study. Hence the formulae for peak strength presented here apply to the situation immediately after excavation.

Since the majority of calcarenite mine pillars shows a W/H ratio equal or less than four it is recommended to use the formulae applicable to these range of pillar shapes.
Fig. 6.51 The selection of pillar width.

Taking width to describe pillar shape Eqs. 6.6 and 6.7 apply:

For peak strength: \[ \sigma_p / UCS = 0.875 + 0.250 \text{ W/H} \]

For residual strength: \[ \sigma_r / UCS = 0.118 + 0.350 \text{ W/H} \]

It was shown in Section 6.9 that these equations may underestimate strength if the basal plane is not square. Hence, a conservative estimation of pillar strength is obtained.

A drawback of these formulae is the difficulty of choosing width for irregularly shaped pillars. An example is given in Fig. 6.51a. Which horizontal dimension should be taken as width? Strictly speaking, width is the minimum horizontal dimension, i.e. possibility 6. It can be judged intuitively that this does not result in a reasonable estimation of pillar strength.
The following guidelines are proposed here regarding the selection of width:

- small projections, not representative for the pillar as a whole, are not considered.

- for each pillar the maximum "enveloping" horizontal dimensions can be easily determined (Fig. 6.51a). The largest of both is taken as enveloping length, the smallest as enveloping width. The width for strength assessment should be measured perpendicularly to the enveloping length.

- a horizontal dimension should be selected which is representative along the enveloping length.

Obviously also the use of these guidelines does not ensure a strictly objective assessment of pillar width. However, the method presented here may be considered reasonable.

Some pillars comprise a narrowing somewhere along the enveloping length (Fig. 6.51b). Considering option 1 or 3 as width will result in an overestimation of strength, because the "inlets" at opposites sides of the narrowing do not contribute to carrying load. The opposite applies to option 2. In such cases the best solution is to subdivide the pillar in two, including the tributary areas. The narrow part is then neglected and widths 1 and 2 are used for pillars I and II respectively. Notice that the narrow part is neither be accounted for regarding pillar and tributary area for the calculation of pillar stress.

Also A/CH can be taken to describe the pillar shape:

For peak strength: \( \sigma_p / UCS = 0.875 + A/CH \)

For residual strength: \( \sigma_r / UCS = 0.118 + 1.40 A/CH \)

These equations have the advantage that the problem of choosing width is avoided. Furthermore, these formulae fit the experimentally derived data slightly better than Eqs. 6.6 and 6.7. Also here subdivision of pillars and small, non-representative pillar projections should be considered, as outlined above.

For pillars of W/H ratio's exceeding four the formulae presented above overestimate pillar strength considerably. Here Eqs. 6.14 and 6.15 must be utilized:

For peak strength: \( \sigma_p / UCS = 1.150 + 0.962 \log (W/H) \)

For residual strength: \( \sigma_r / UCS = 0.450 + 1.600 \log (W/H) \)

For irregular pillars W/H can be replaced by 4 A/CH.
Fig. 6.52 Provisional scheme of the determination of individual pillar strength, ignoring creep deformation and statistical complications.

It has to be noted that, as a result of the difference between the experimental setup in the laboratory and the actual pillar geometry in the mines, all experimentally derived equations might slightly underestimate pillar strength (Section 6.13). However, Hustrulid (1976) reviewed some field verifications of experimentally derived shape formulae, which did not reveal such an effect. The shape formulae will be validated with field data of the calcarenite mines in Chapters 8 to 11.

The determination of individual pillar strength, ignoring creep deformation, is summarized in Fig. 6.52. Note that statistics are not involved yet, i.e. spatial distribution of UCS, measuring accuracies etc. are not considered.

In a few mines clay seams of up to several centimeters thickness are interbedded within the direct roof layers, for example in the Geulhemmer Groeve (see Chapter 8). It is well possible that shear movement at the pillar ends is allowed here due to this natural insert. The resulting reduction of lateral confinement at the pillar ends will have an adverse effect on strength. As a consequence, if such clay seams are present, the shape formulae might overestimate pillar strength.

The applied mining methods resulted in linear incisions in the pillar walls made by a chisel. For example continuous horizontal incisions are to be found just below the roof (Figs. 2.8 and 8.5). As a consequence, the horizontal pillar dimensions are reduced. But, since the chisel marks are just a few centimeters deep, they are neglected with respect to the meter scale pillar dimensions. It should be noted that the space resulting from the incisions could facilitate the rotation and translation of isolated sides of failed pillars.

6.14.3 Evaluation of errors and validation of calculated pillar safety factors

The stress and strain paths of an actual mine pillar are more complicated than those of a prism which is compressed at a constant strain rate in the laboratory. After excavation, a mine pillar is exposed to a more or less constant stress. The pillar, exposed the a stress of less than $\sigma_p$, may still fail in the long term due to creep
deformation, i.e. deformation may still proceed at a constant stress. Thus actually a "long term" strength, which is less than $\sigma_p$ (see also Section 4.2.2.9), should be considered. This long term strength $\sigma_n$ is not yet known and will be studied in the next chapter. Additionally, pillar stress may increase or decrease due to interaction with surrounding pillars. Comparable to constant strain rate tests, pillar stress is reduced upon fracturing. For pillars this occurs due to load transfer towards other pillars and/or to the abutments. Otherwise the individual pillar would collapse. Note that equilibrium requires that the stress on a pillar is equal to its strength at each stage of deformation.

With these considerations in mind, the classification system can also be applied to mine pillars. This system is used in this thesis to describe pillar damage and enables a validation of pillar strength formulae, established in the laboratory, and tributary area theory in the mines. Peak strength $\sigma_p$ can be determined according to the formulae of Section 6.14.1, and the tributary area method gives an estimation of pillar stress $\sigma$ (Section 4.2.3.1). The $\sigma_p/\sigma$ ratio represents a short-term safety factor just after excavation, denoted here as the initial safety factor $SF_0$:

$$SF_0 = \frac{\sigma_p}{\sigma}$$  (6.25)

For each individual pillar this safety factor can be established and compared with the pillar class.

For several reasons the calculated safety factors may not match the observed pillar condition, when applied to individual pillars. In the first place systematic errors will result. Each systematic error in the calculation procedure induces a certain general, systematic deviation from actual values of safety factors, which is either positive or negative and of the same magnitude for all pillars involved. Three types of systematic errors can be recognized:

I A. Systematic errors, resulting in an overestimation of safety factors, may arise due to:

- 1. creep deformation. When pillar class is observed, the pillar has been exposed to creep since its excavation. If $\sigma > \sigma_n$, the pillar may be fractured already. Thus, due to creep a pillar with a $SF_0$ value well in excess of one may yet be characterized as class 2 or more.

- 2. a strong domino effect. If conditions are such that a strong domino effect occurs, i.e. more and more pillars become fractured due to successive load redistributions, calculated safety factors give a general overestimation.

- 3. end constraints, which may be less favourable for an actual mine pillar than for a prism tested in the laboratory (Section 6.13).
I B. A systematic error with an opposite effect, i.e. with an underestimation of safety factors as a consequence, may result from the calculation of pillar stress by means of tributary area theory. In this way possible pillar stress reduction due to arching towards the abutments is ignored.

I C. Also systematic errors exist which give either a general overestimation or a general underestimation of safety factors. Which of the two possibilities applies is not known:

1. a systematic over- or underestimation for the whole mine of the thickness of a certain overburden unit, relative to the thicknesses of the remaining units.

2. a systematic over- or underestimation for the whole mine of unit weight of one or more overburden units.

II. In the second place the calculation procedure produces random, non-systematic errors. Due to a random error calculated safety factors show a certain distribution around the actual values. A random error results in deviations from the actual value, which vary from pillar to pillar within the analyzed mine system. For each individual pillar the deviation may be positive or negative. The following random errors are expected on the basis of:

1. variation of UCS between pillars.

2. the different extent of the effect of creep, arching and end constraints on pillars, which vary in shape and location relative to the abutments and which consist of calcarenite of different creep behaviour. This random error represents the variation of sources which induce systematic errors outlined above.

3. locally confined interaction between pillars and redistribution of pillar load. The load on a given pillar may increase and exceed its tributary area load when the load carrying capacity of a nearby pillar decreases due to fracturing. On the opposite, relatively strong pillars in the vicinity may bring about a reduction of load on a given pillar, which is less than its tributary area load.

4. local variations of UCS within a pillar. Fractures grow at relatively weak spots. If the calcarenite is relatively weak at the usual potential failure zones, safety factors, using the average UCS of the pillar, might signify an overestimation. The opposite situation, relatively strong rock at the "critical" zones, may result in an underestimation of safety factors.

5. the incapacity of the shape formulae to predict the precise shape effect for pillars with a highly irregular outline.

6. inaccuracy of the measured pillar dimensions.
Table 6.2 Input parameters for total safety factor example.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$\mu_i$</th>
<th>$\sigma_i$</th>
<th>$\sigma_i / \mu_i * 100%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS</td>
<td>2.0 MPa</td>
<td>0.3 MPa</td>
<td>15.0</td>
</tr>
<tr>
<td>W</td>
<td>6.0 m</td>
<td>0.1 m</td>
<td>1.67</td>
</tr>
<tr>
<td>H</td>
<td>3.0 m</td>
<td>0.1 m</td>
<td>3.33</td>
</tr>
<tr>
<td>$A_p$</td>
<td>36.0 m$^2$</td>
<td>1.0 m$^2$</td>
<td>2.78</td>
</tr>
<tr>
<td>$A_t$</td>
<td>81.0 m$^2$</td>
<td>2.0 m$^2$</td>
<td>2.47</td>
</tr>
<tr>
<td>$\sigma_{ov}$</td>
<td>0.50 MPa</td>
<td>0.10 MPa</td>
<td>20.0</td>
</tr>
<tr>
<td>$K_0^*$</td>
<td>0.50</td>
<td>0.05</td>
<td>10.0</td>
</tr>
<tr>
<td>$N_{shape}$</td>
<td>1.375</td>
<td>0.15</td>
<td>10.9</td>
</tr>
<tr>
<td>$\sigma_p$</td>
<td>2.75 MPa</td>
<td>0.51 MPa</td>
<td>18.5</td>
</tr>
<tr>
<td>$S = \sigma_p A_p$</td>
<td>99.0 MN</td>
<td>18.57 MN</td>
<td>18.8</td>
</tr>
<tr>
<td>$P = \sigma_{ov} A_t$</td>
<td>40.5 MN</td>
<td>8.16 MN</td>
<td>20.2</td>
</tr>
</tbody>
</table>

- 7. Inaccuracy of isopach contour lines.
- 8. Variation of unit weight of one or more overburden units.

The systematic errors can be eliminated by a general correction factor. According to the analysis of Salamon & Munro (1967), pillar fracturing can be expected when the safety factor is critical, i.e. the safety factor is about equal to one. Accordingly, fractured pillars (class 2-4) should be characterized by such safety factors. Due to the systematic errors outlined above the calculated safety factors will probably exceed the true values in general, i.e. it is expected that the systematic errors under A exceed those under B. A general correction factor $K_0^*$ is proposed:

$$SF_{0}^* = K_0^* \cdot SF_0$$  \hspace{1cm} (6.26a)

or

$$SF_{0}^* = K_0^* \cdot \frac{\sigma_p}{\sigma_{ov}} \cdot \frac{A_p}{A_t}$$  \hspace{1cm} (6.26b)

Due to the random errors it is likely that even the corrected safety factors are still not the same as the true safety factors. Hence the corrected safety factors for fractured pillars are either smaller or greater than one, and are characterized by a certain frequency distribution around $SF_{0}^* = 1$. Thus $K_0^*$ should be chosen such that the values of $SF_{0}^*$ for fractured pillars are densely distributed around one. In general it is preferable to choose the location of the distribution with the median at $SF_{0}^* = 1$. The median is the value which divides the area under the distribution curve into two
Table 6.3 Mean value and standard deviation of $SF_{\text{tot,0}}$ for various numbers of pillars $N$ in the example.

<table>
<thead>
<tr>
<th>Output parameter</th>
<th>$\mu_l$</th>
<th>$\sigma_l$</th>
<th>$\mu_l / \sigma_l \times 100%$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$SF_{\text{tot,0}}$, $N = 1$</td>
<td>1.22</td>
<td>0.336</td>
<td>27.5</td>
</tr>
<tr>
<td>$SF_{\text{tot,0}}$, $N = 10$</td>
<td>1.22</td>
<td>0.106</td>
<td>8.7</td>
</tr>
<tr>
<td>$SF_{\text{tot,0}}$, $N = 30$</td>
<td>1.22</td>
<td>0.061</td>
<td>5.0</td>
</tr>
<tr>
<td>$SF_{\text{tot,0}}$, $N = 50$</td>
<td>1.22</td>
<td>0.048</td>
<td>3.9</td>
</tr>
<tr>
<td>$SF_{\text{tot,0}}$, $N = 100$</td>
<td>1.22</td>
<td>0.034</td>
<td>2.8</td>
</tr>
<tr>
<td>$SF_{\text{tot,0}}$, $N = \infty$</td>
<td>1.22</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

halves. By choosing the median, the analysis also applies to asymmetric distributions.

Care should be taken when correction factors are applied to other mines. Systematic errors may vary somewhat per mine, for example due to differences in pillar geometry, mine span and age. Such a variation may be quantified when various mines are analyzed. A comparison of $SF_0$ with pillar class in a number of mines may allow an estimation of a general safe correction factor, which can be applied to assess individual pillar stability in other mines of comparable outline and age. The validation of safety factors will be dealt with in more detail in Chapters 7, 8, 10 and 11.

6.14.4 Load carrying capacity and safety factor for a whole mine: the assessment of large-scale pillar stability

The total load carrying capacity $S_{\text{tot}}$ of a mine (section) is given by:

$$\sum (\sigma_p \times A_p)$$

A safety factor for the area can be achieved by dividing the total load carrying capacity $S_{\text{tot}}$ by the total overburden load $P_{\text{tot}}$:

$$SF_{\text{tot}} = S_{\text{tot}} / P_{\text{tot}} = \sum (\sigma_p \times A_p) / \sum (\sigma_{ov} \times A_t)$$  \hspace{1cm} (6.27)

These equations apply to a situation of exclusively intact pillars, which could occur directly after the mining activities when pillars are hardly affected by creep deformation. Hence this safety factor can be designated $SF_{\text{tot,0}}$:

$$SF_{\text{tot,0}} = \sum (\sigma_p \times A_p) / \sum (\sigma_{ov} \times A_t)$$  \hspace{1cm} (6.28)
The total safety factor $SF_{tot,0}'$ can be applied as an indicator of short-term large-scale stability directly after excavation. A long-term total safety factor, which considers the systematic errors, including that due to creep, outlined in Section 6.14.3, can be expressed as follows:

$$SF_{tot,0}' = K_0' \sum (\sigma_p * A_p) / \sum (\sigma_{ov} * A_i) \quad (6.29)$$

Such a safety factor represents a tool to establish large-scale pillar stability.

When calculating a total safety factor, the random errors, e.g. those due to variation of UCS and due to pillar interaction, cancel each other out for an important part. To get an idea in how far random errors are eliminated in the total safety factor, the following analysis is performed. The standard deviation $\sigma_f$ of the output parameter $f(i)$, i.e. $SF_{tot,0}'$, is calculated as follows from the means $\mu_i$ and variances $\sigma_i$ of the $n$ input parameters $i$:

$$\sigma_f^2 = \sum_{i=1}^{n} (\sigma_i \frac{\partial f}{\partial i}) \quad (6.30)$$

This simple and often used approach is known as the first order second moment method (FOSM). Its accuracy proved to be close to that of for example Monte-Carlo simulation methods for this kind of application (Pine & Thin, 1993). It should be noted that the FOSM-method applies to normal distributions of input and output parameters.

A mine system is considered which consists of $6*6$ m pillars of $3$ m height, separated by $3$ m wide galleries. The overburden stress is $0.5$ MPa. The input parameters are given in Table 6.2. The standard deviations $\sigma_i$ are obtained by evaluation of the random errors, dealt with in the previous section. Of course in a real mine pillars have varying dimensions, but this does not affect the relevance of the analysis. However, to include all possible errors, the values of $\sigma_i$ are estimated as if the pillars do have various sizes and shapes:

- Errors 1,4: affect UCS, estimation of $\sigma_i$ at $0.3$ MPa
- Errors 2,5,6: affect $N_{shape}$, estimation of $\sigma_i$ at $0.15$
- Errors 2,3,7,8: can be considered to affect $\sigma_{ov}$, $\sigma_i$ is about $0.1$ MPa
- Error 6: affects $A_p$ and $A_i$, estimation of $\sigma_i$ at $1.0$ and $2.0$ m$^2$ respectively

In this example $K_0'$ is estimated at $0.50$. Since $K_0'$ only results from systematic errors, it is per definition not affected by random errors.
Applying Eq. 6.30 gives the following results:

For $\sigma_p$:

$$\sigma_i = \sqrt{ (0.3 \times 1.375)^2 + (0.15 \times 2)^2 } = 0.510 \text{ MPa}$$

Then for $\sigma_pA_p$:

$$\sigma_i = \sqrt{ (0.510 \times 36)^2 + (1.0 \times 2.75)^2 } = 18.565 \text{ MN}$$

And for $\sigma_{ovA_t}$:

$$\sigma_i = \sqrt{ (0.1 \times 81)^2 + (2.0 \times 0.50)^2 } = 8.161 \text{ MN}$$

Since $SF_{tot,0}' = K_0' \times \sum S / \sum P$, application of Eq. 6.30 yields, for a mine system of N pillars, a standard deviation $\sigma_f$ for the output variable $SF_{tot,0}'$:

$$\sigma_f = \frac{\sqrt{N}}{N} K_0' \sqrt{ \frac{\sigma_s^2}{P^2} + \frac{\sigma_p^2}{P^4} + \frac{S^2}{P^4} }$$

(6.31)

where $\sigma_s$ and $\sigma_p$ represent the standard deviations of $S$ and $P$ respectively. For this example the standard deviation $\sigma_f$ of $SF_{tot,0}'$ significantly decreases at increasing number of pillars $N$ (Table 6.3). Here a satisfying precision is achieved for $N$ is 30 or more: at a 95 % confidence level, the value of $SF_{tot,0}'$ does no deviate more than 10 % , i.e. 2 $\sigma_f$ from the actual value.

In Chapter 8 it will be shown that the distribution of safety factors is more or less log-normal, which was also found by Salamon and Munro (1967). Pine & Thin (1993) showed that also for such a distribution the FOSM-method is applicable.

Now, if $F = \log f$, then, according to Eq. 6.30, $\sigma_F = (dF/df)\sigma_f = \sigma_f / \mu_f$. A drawback of the FOSM-method is the need to assume a given output-distribution. This disadvantage does not apply to Monte-Carlo simulation methods.
CHAPTER 7

TIME-DEPENDENT DEFORMATION OF THE CALCARENITE

7.1 INTRODUCTION

As outlined in the previous chapters (e.g. Sections 4.2.2.9 and 6.14) pillar deformation in the calcarenite mines is time dependent. Examples of field evidence were given in Chapter 3. The observed deterioration of calcarenite mine pillars in the course of time is partially a result of increased pillar stress due to later pillar robbing, but it is believed that the process is dominated by true time-dependent deformation or creep. Many years after excavation, continuing deformation of calcarenite mine pillars can eventually result in pillar failure and large-scale collapse. As a consequence of creep deformation pillars fail at a stress below their short-term strength. Existing pillar strength formulae, including those presented in Chapter 6, do not comprise a factor accounting for creep. It is the goal of this chapter to describe the creep behaviour of the calcarenite at various stresses and to indicate how the results can be applied in order to incorporate time-dependent deformation in the assessment of pillar stability.

Constant stress or creep tests were carried out to investigate, for example, how an increase of the applied uniaxial stress affects the time to failure and how strain and strain rate develop as a function of time and stress. The creep tests were performed on cylindrical cores from two calcarenite blocks of different UCS in order to get an idea of the variation of creep deformation characteristics. Most tests were executed on samples at natural moisture content (6-7 %) but also on dry specimens. Additionally, creep tests on prisms of a W/H ratio of about 2 were performed to give insight into the effect of geometry. Finally, the practical consequences of the results will be discussed.

7.2 GENERAL BACKGROUND

The time-dependent stress-strain behaviour of a material is usually studied by a creep test, in which an increment of stress is applied quickly and then held constant while the increasing strain of the specimen is measured with time. The classical idealised creep curve for rock at constant compressive stress, given by various authors (e.g.
Fig. 7.1 Classical creep curve with three phases of creep deformation (after Lama & Vutukuri, 1978).

Lama & Vutukuri, 1978; Farmer, 1983; Goodman, 1989), is depicted in Fig. 7.1. This curve seems to be based on early creep tests on metal wires under tension. Three distinct creep phases can be recognized. Immediate elastic and plastic strain is followed by primary or transient creep in which the strain rate decreases with time (strain hardening). Sudden destressing will result in instantaneous and then delayed recovery, where the strain asymptotically tends to zero. This time-dependent elasticity is also referred to as anelasticity. If the stress is maintained beyond the phase of primary creep, secondary or steady state creep occurs. The strain rate is constant and the specimen accumulates permanent deformation. If the specimen is destressed after the onset of secondary creep, an amount of permanent strain will persist. If the specimen is not destressed, tertiary or accelerating creep may set in: the strain rate increases with time until failure occurs.

However, Cruden et al. (1971, 1987) stated that creep curves of rock under compression generally have an appearance different from Fig. 7.1. For most rocks at room temperature, some even at elevated temperatures, and under uniaxial compression a phase of steady state creep is difficult to detect, and, if secondary creep can be identified, it is of short duration. Instead, for rock under brittle conditions the complete creep curve appears to be described more adequately by two phases, a decelerating phase and an accelerating phase, which pass into each other through an inflexion point (Fig. 7.2).

Fig. 7.2 also illustrates that with an increasing stress the strain rate at a given time increases and the time to failure decreases. According to many authors (e.g. Bieniawski, 1967a,b; Singh et al., 1977; Helal et al., 1988) there is a particular stress below which the rock will not fail, no matter how long the stress be applied. This stress is known as the long-term strength and generally equals 60% to 90% of
Fig. 7.2 The two phases of creep in brittle rock at different uniaxial stresses.

the (short-term) uniaxial compressive strength, depending on rock type. Apparently at stresses below the long-term strength the time-dependent deformation is confined to the decelerating creep stage and the accelerating creep stage finally resulting in failure is not reached.

In Chapter 5 it was demonstrated that the (short-term) uniaxial compressive strength of the calcarenite decreases significantly if a few percent of water is added, a phenomenon not uncommon for most rock types. Experiments of several workers showed that for a wide range of rock types also time-dependent behaviour is affected by moisture content (e.g. Singh et al., 1977; Lama & Vutukuri, 1978; Lajtai et al., 1987; Hadizadeh & Law, 1991).

7.3 CREEP LAWS

A creep law describes the (creep) strain or strain rate of a material as a function of time, and preferably also as a function of stress history, microstructure and pore fluid chemistry. Three possibilities exist to establish a creep law:

7.3.1 Creep laws derived from deformation mechanism theories.

Such theories of creep, also named structural models, explain the deformation in terms of physical processes which are typical for that particular material studied. In rocks at elevated temperatures and high pressures and in metals, crystal plastic mechanisms (dislocation glide, dislocation creep) and diffusional flow mechanisms (Nabarro-Herring creep, Coble creep, pressure solution) are dominating. Deformation-mechanism maps show which mechanism is dominant at a certain
combination of temperature and deviatoric stress.

On the other hand, at room temperature and low confining pressure both short-term and creep deformation of most rocks is brittle and dominated by the mechanism of microcracking (Lama & Vutukuri, 1978; Costin, 1983; Lajtai & Bielus, 1986). Costin (1989) reviewed various structural models of microcrack growth. Some of these models proved to be in good agreement with experimental data on creep and other rate-dependent effects for certain rock types. In this regard continuing damage theories are promising. But the establishment of such a structural model for the calcarenite, of a porosity of up to 50% and comprising extensively micritized and loosely bonded grains, would be outside the scope of this study.

7.3.2 Creep laws based on rheological analogs

Rheological analogs comprise a combination of several mechanical elements, like springs, dashpots and masses resting on a plane, connected in series and/or parallel. In such models elastic behaviour is represented by springs, viscous deformation by dashpots and plastic responses by a mass resting on a plane with a frictional force preventing the movement of the block under the action of the forces below the yield limit. These systems only describe the mechanical behaviour under certain testing conditions, but nevertheless visualize well the mutually influencing mechanical processes within a material. Unfortunately, the application of rheological analogs often leads to exponential terms which did not agree well with the actual calcarenite rock behaviour in preliminary modelling attempts. Moreover, these models only approximate the primary and, if existing, the secondary creep stages. More about rheological analogs can be found in Price (1964), Obert & Duval (1967), Lama & Vutukuri (1978), Jaeger & Cook (1979) and Goodman (1989).

7.3.3 Empirical creep laws

Empirical models are, like rheological models, not based on any deformation mechanism theories. The advantage of empirical models is that no simplifications are needed, as the material is described as it is found in nature (Haupt, 1991). However, since such laws do not relate to any microstructural deformation mechanism, they only apply to the specific test condition. An empirical creep law only applies to a constant stress and does not give insight in the relationships between stress, strain and time for other circumstances, e.g. a change in stress level, an imposed strain rate, a variation of temperature.

Cruden (1971) gave an overview of various empirical laws for primary and secondary creep on several rock types. The primary creep laws for cylindrical samples could be divided into two main types, an exponential law

\[
\frac{dc}{dt} = A_1 \exp (-A_2 t)
\]  

(7.1)
and a power law

\[ \frac{de}{dt} = B_1 t^{B_2} \]  

(7.2)

de/dt is the axial strain rate at time t. As for the rest of this chapter, \( \epsilon \) denotes axial strain. \( A_1 = (de/dt)_{t=0} \) and \( B_1 = (de/dt)_{t=1} \), i.e. the axial strain rate at one time unit. \( A_2 \), greater than zero, and \( B_2 \), usually lying between zero and -1, determine the rate of decrease of de/dt with increasing time and can be considered as strain-hardening parameters. The exponential law of Eq. 7.1 can also be written as

\[ \epsilon_{cr} = \left( \frac{A_1}{A_2} \right) \left( 1 - \exp \left( -A_2 \ t \right) \right) \]  

(7.3),

the creep of a Kelvin body. Eq. 7.2 can be written as

\[ \epsilon_{cr} = \left( \frac{B_1}{(B_2 + 1)} \right) t^{B_2 + 1} \]  

(7.4)

When both creep laws are compared, it becomes evident that at large values of t the rate of decrease in axial strain rate (strain hardening) in a power law will be less than in an exponential law. When t approaches zero, the rate of strain hardening in a power law is infinite. In an exponential law the axial strain rate equals \( A_1 \) for t=0. At intermediate values of t the rates of strain-hardening are about the same.

From Eqs. 7.1 and 7.2 it can be seen that in an exponential law the log axial strain rate vs. time plot should be linear, while in a power law plotting log axial strain rate vs. log time and, if \( B_2 \) is not -1, also log creep strain vs. log time should result in a straight line. If \( B_2 = -1 \) equation 7.2 can be written as

\[ \epsilon_{cr} = \epsilon_{cr, t=1} + B_1 \log t \]  

(7.5)

and the axial creep strain vs. log time plot will be linear. Note that a power law can describe the decelerating creep only for values of t exceeding one time unit.

Cruden calculated the parameters of best fit for both exponential and power laws for 48 creep experiments at several stresses and on various rock types, including anhydrite at 15° and 600°C, olivine at 750°C, and marble and sandstone at room temperature. After applying several criteria of goodness of fit, it could be concluded that exponential laws, also when a secondary creep component was added, are not generally appropriate descriptions of the data. Power laws of the Eq. 7.2 type appeared to fit all data adequately. There was no evidence for secondary creep in any of the experiments, hence the whole part of the creep curves considered by Cruden can also be referred to as decelerating creep.

Cruden (1971) also observed that the strain-hardening parameter \( B_2 \) is usually about -1. This is in agreement with the conclusions of Misra and Murrell (1965), who found that at temperatures below about 20% of the melting temperature the creep behaviour of rocks could be described by a creep law of the form \( \epsilon_{cr} = A + B \log t \).
Cruden et al. (1987) and Okubo et al. (1991) attempted to establish complete creep curves including the accelerating or tertiary creep region by means of combinations of power laws. The creep laws of Okubo et al. comprised the sum of primary, secondary and tertiary creep components. The calculated results gave a satisfactory fit for five experiments on an andesite, a tuff and a cement-mortar at room temperature. However, their experimental data had not shown a discrete phase of secondary creep. Cruden et al. used the sum of decelerating and accelerating creep power laws to fit their creep experiments on coal at room temperature. The result was an equation of the form:

\[ \frac{de}{dt} = A t^b + C t^p \]  

(7.6)

Their analyses were carried out by means of a series of computer programs. Some criteria of goodness of fit were applied to match both individual creep components and to separate the regions of dominating decelerating and accelerating creep. The double power law (Eq. 7.3) appeared to be an adequate description, but only one experiment at one single stress was considered.

All power laws established by the authors mentioned above are expressions of axial strain rate or axial strain as a function of time and apply to experiments performed at one single stress. There were no successful attempts to include also applied stress as a variable of the power law. In Sections 7.6.1.6 and 7.6.1.7 this will be attempted for the experiments on the calcarenite samples.

7.4 EXPERIMENTAL PROCEDURE

7.4.1 Sampling and sample preparation

The calcarenite tested originated from the location in the Sibber Groeve, described in Section 5.2.1, and was transported and stored as outlined in that section. Cylindrical cores were prepared in accordance with the suggested methods of the ISRM (Brown, 1981). The sample diameter was 39.6 mm and the length ranged from 97.1 to 101.8 mm. Their moisture content was 6 to 7 %, which is the natural moisture content, referred to as wet in this chapter. After preparation the samples were immediately packed in plastic foil to prevent desiccation.

Three calcarenite blocks of about 50*40*15 cm, numbered 1 to 3, were used, which were taken from three subsequent stratigraphical levels at the same location. From each block a series of specimens originated for the creep experiments. Each series was numbered according to the respective block sample.

Cores for creep testing were taken from blocks 1 and 2. Several cores from each block were used to establish the UCS-value (Table 7.1). The wet UCS-values of block 1 and block 2 are 3.18 respectively 3.49 MPa (both with a standard deviation of 0.06 MPa). For block 2 also the dry UCS-values were determined: 5.04 MPa with a standard deviation of 0.14 MPa). Creep experiments of series 1 were
Table 7.1 UCS values of sample material for creep tests.

<table>
<thead>
<tr>
<th>Cores Series 1, wet (MPa)</th>
<th>Cores Series 2, wet (MPa)</th>
<th>Cores Series 2, dry (MPa)</th>
<th>Prisms W/H=2 Series 3, wet (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.18</td>
<td>3.52</td>
<td>4.86</td>
<td>4.36</td>
</tr>
<tr>
<td>3.17</td>
<td>3.40</td>
<td>5.21</td>
<td>4.25</td>
</tr>
<tr>
<td>3.22</td>
<td>3.56</td>
<td>5.16</td>
<td>3.99</td>
</tr>
<tr>
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<td>3.95</td>
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<td>3.27</td>
<td>3.44</td>
<td>4.96</td>
<td>3.95</td>
</tr>
<tr>
<td>3.14</td>
<td>3.50</td>
<td></td>
<td>4.05</td>
</tr>
</tbody>
</table>

mean = 3.18 mean = 3.49 mean = 5.04 mean = 4.10

\( \sigma_{n-1} = 0.06 \) \( \sigma_{n-1} = 0.06 \) \( \sigma_{n-1} = 0.14 \) \( \sigma_{n-1} = 0.16 \)

n = 6 n = 6 n = 5 n = 7

performed on cores from block 1 at natural moisture content (code cw1). The series 2 tests comprised both wet (code cw2) and dry (code cd2) creep experiments on samples from block 2.

The series 3 tests were carried out on prismatic samples with a square base and a with/height ratio of about 2 (code 3). The samples were created from block 3 according to the procedure outlined in Section 6.4.1. The sample size was about 100*100*50 mm. W/H varied between 1.96 and 2.05. The moisture content was 6 to 7 % and the short-term prism strength \( \sigma_5 \) is 4.10 MPa (with a standard deviation of 0.16 MPa; Table 7.1), corresponding with a UCS value of 2.98 MPa according to Eq. 6.6.

7.4.2 Creep tests on cylindrical cores

Because of the low strength of the calcarenite the creep tests on cylindrical cores were performed in a device normally used for consolidation tests on soils (Fig. 7.3). The loading beams C and D amplify the weight B to a force ten times as high on to the specimen. A rubber sleeve around the specimen and the loading platens prevented the water-containing specimen from drying out. The stress was applied manually within 30 seconds. The axial strain was measured by means of a LVDT (linear variable differential transformer). The output was amplified and continuously displayed by an Y-t-recorder. The accuracy on the electronic display was 10^-3 volt,
which corresponded to about $10^{-2}$ millistrain. The display was read on a regular base and noted together with the time of the observation. Time and strain were also recorded by the Y-t plotter. This proved to be particularly useful during the final stage of accelerating creep and subsequent failure, because these events were not observed personally but registered afterwards. The accuracy of the plotted graph was also about $10^{-2}$ millistrain. During the test the paper feed was adjusted a few times to ensure an adequate resolution. Depending on the paper feed the accuracy was 1 second to 1 minute. The possibility of electronic drift was examined by loading a stainless steel core of the same size and shape as the calcarenite specimens. No drift could be detected during a period of 26 days.

It was decided to perform the tests on wet samples in a closed part of the Geulhemmer Mine (Fig. 7.4). At the test location no vibrations due to traffic etc. could influence the results and the temperature was fairly constant at 10 ± 2 °C. Because of the 90-100 % air moisture content in the mine the electronic equipment was placed in a plastic container with a plexiglass lid, and a moisture absorber inside. After the experiment the decrease in moisture content of the specimens during the creep test was determined, which proved to be less than 0.5 %.

In total 14 tests on wet samples, 10 in series 1 and 4 in series 2, were carried out at various axial stresses ranging from 67 % to 97 % of the mean wet uniaxial compressive strength. Due to the variation of UCS in each calcarenite block, the accuracy of these percentages is 3 % at a 5 % confidence level. The experiments lasted from 15 seconds to 129 days. Failure occurred along a macroscopic shear plane (see Section 5.3.1).
Fig. 7.4 Preliminary creep tests in the Geulhemmer Groeve. During the final experiments the cores were wrapped in a rubber sleeve and axial strain was measured by LVDT’s, coupled to an amplifier and a Y-t-recorder with pen-plottter.

Since the strain at failure is only a few millistrain the area change equals just a few promille’s. Hence a correction of stress for area change is considered unnecessary.

The experiments on dry samples were carried out in the basement laboratory at Mining Engineering. Temperature and air moisture content were less constant than in the mine: $15 \pm 5 \, ^\circ\text{C}$, and $60 \pm 15 \, \%$ respectively. The samples were oven dried at $95 \, ^\circ\text{C}$ for 24 hours and then immediately packed in plastic foil for storage. Before the test the plastic foil was removed and the samples were packed in a rubber sleeve, to keep moisture out of the specimen. After the experiment the moisture content was measured and proved to be less than $0.07 \, \%$. 
Six experiments could be performed, at uniaxial stresses varying from 61 % to 95 % of the dry uniaxial compressive strength. The test duration was ranging from 15 seconds to 115 days.

7.4.3 Creep tests on prismatic samples

For these experiments the stiff servo-controlled machine of a capacity of 50 kN described earlier was used. The axial displacement was measured by two LVDT's. The creep tests were carried out at stress control and the short-term strength tests at axial displacement control. During the creep tests axial displacement, force and time were recorded every 20 seconds on a floppy disk. The accuracy of the strain measurements was $10^{-2}$ millistrain.

The temperature and air moisture content were more or less the same as in the testing room for the creep experiments on dry samples, i.e 15 ± 5 °C, and 60 ± 15 % respectively. Nine creep experiments were performed at the original moisture content. The four vertical sides of the samples were wrapped in plastic foil when full contact was achieved with the loading platens. The stress levels ranged from 51 to 91 % of the short-term prism strength. The accuracy of these percentages is 6 % at a 5 % confidence level. Loading and unloading occurred at a constant stress rate of about 0.2 MPa/sec. Given the restricted time available for using the compression machine, the tests were stopped after ten hours if the sample had not failed yet. The shortest test lasted 210 seconds.

7.5 EXPERIMENTAL RESULTS

7.5.1 Cylindrical cores

The curves of test cw1-5, performed at a stress of 84 % of the wet UCS, are typical for the creep behaviour of wet calcarenite. The time at which load application was completed, i.e. 30 seconds, was taken as $t=0$. Fig. 7.5a seems to indicate a phase of steady-state or secondary creep (from data point 13 to 19), but closer inspection reveals that the axial creep rate is never really constant. Figs. 7.5 a-d show a phase of decreasing creep rate changing gradually into a phase of increasing creep rate towards failure, which occurs after about $4.8 \times 10^5$ seconds (133 hours), illustrating the subsequent stages of decelerating and accelerating creep. The axial strain rate is at its minimum of about $1 \times 10^9$ sec$^{-1}$ at about $2.7 \times 10^5$ seconds (75 hours). In semi-log plots it can be seen that the axial strain increases linearly with time and in a log-log plot it can be observed that the axial strain rate decreases linearly with time until about 40000 seconds after $t=0$, indicating power law behaviour for the stage of decelerating creep. In a semi-log plot axial strain rate as a function of axial strain also shows more or less linear behaviour for the major part of the decelerating creep stage. No distinct linear $e$-t, $de/dt$-e, or $de/dt$-t relations are to be observed for the accelerating creep phase in these diagrams.
Part II: Laboratory experiments

![Graph a)](image)

![Graph b)](image)
Fig. 7.5 Axial creep strain vs. time in a linear plot for test cw 1-5, showing stages of decelerating and accelerating creep. The numbers refer to data points (a); semi-logarithmic plot of axial creep strain vs. time. The major part of the decelerating creep stage is linear, i.e. $\epsilon = a + b \log t$ (b); double-logarithmic plot of axial strain rate vs. time. The linear decelerating creep stage indicates that a power law applies (c); Semi-logarithmic plot of axial strain rate vs. axial strain (d).
Fig. 7.6 Semi-logarithmic plot of series 1 creep curves for wet calcarenite cores.

Fig. 7.7 Semi-logarithmic plot of series 2 creep curves for wet and dry calcarenite cores.
Fig. 7.8 Semi-logarithmic plot of series 3 creep curves for wet calcarenite prisms of a W/H ratio of 2.

The $\varepsilon_{cr}$-t curves for ten tests on wet samples from block 1 are depicted together in the semi-log graph of Fig. 7.6. It can be noted that the axial strain rate increases with increasing stress.

The creep tests on wet samples from block 2 are delineated in Fig. 7.7, which is of the same scale as Fig. 7.6. The samples show axial strain rates of apparently the same order of magnitude as observed in the tests on the cores from block 1. But, although the UCS for the samples of series 2 exceeds those of series 1, for a given stress the time to failure is about two orders of magnitude less than observed for the first series. For a given percentage of UCS the difference in failure times is even three orders of magnitude. This can be at least partly explained by the higher axial strains at failure for the samples of the first series. Note that the UCS for the samples of the first series is lower than for samples of the second series.

In Fig. 7.7 also the results of 5 creep tests on dry calcarenite samples are presented. It is clear that axial strain rates are considerably lower for dry than for wet samples from the same block and at the same stress. Even at a stress/UCS$_{dry}$ value of 0.95, which corresponds with a stress/UCS$_{wet}$ value of 1.37, no accelerating creep could be detected within test periods of up to 115 days. The results of the sixth test are not included because the specimen failed on initial loading at a stress of 4.95 MPa.
Fig. 7.9 Semi-logarithmic plot of axial strain rate vs. time during the decelerating creep stage for test cw 1-5. Since the graph is not linear, an exponential law does not apply.

7.5.2 Prismatic samples

The results of the creep tests are summarized in the semi-log plot of Fig. 7.8. The square based prisms with a W/H ratio of 2 showed the same general creep behaviour as the cylindrical cores of a W/H ratio of 0.4. A semi-log plot of decelerating creep strain vs. time also resulted in a straight line. Unfortunately tests at relatively lower stresses could not be completed. Hence only three failures could be recorded, for tests 3-4, 3-8 and 3-9. The remaining 6 tests were stopped when creep was still in the decelerating stage.

7.6 DATA ANALYSIS

7.6.1 Cylindrical cores

7.6.1.1 Type of decelerating creep law

From linear de/dt-t relations in double logarithmic plots like Fig. 7.5c it appears that, also for calcarenite, decelerating creep can be described by a power law almost until the axial strain rate starts to increase. By plotting log axial strain rate vs. time for all tests it was verified whether exponential laws might apply. As an example Fig. 7.9, depicting the decelerating part of test 1-5, demonstrates that a linear
dependence does not result in a semi-log plot, contrary to the log-log plot of Fig.
7.5c. Thus an exponential law cannot be applied.

In Figs. 7.5b, 7.6 and 7.7 the axial creep strain was plotted vs. log time resulting
in a straight line for the major part of the decelerating creep stage. Hence it appears
that the decelerating creep strain can be described by a formula of the form of Eq.
7.5 and that the strain-hardening parameter is equal to about -1. Here the following
notation will be used:

\[ \varepsilon = a + b \log t \]  \hspace{1cm} (7.7)

where \( a = \varepsilon_{t=1} \), the total axial strain after one second. The axial creep strain \( \varepsilon_{cr} \) can
be formulated by

\[ \varepsilon_{cr} = \varepsilon_{cr, t=1} + b \log t \]  \hspace{1cm} (7.8)

where \( \varepsilon_{cr, t=1} \) is the axial creep strain at one second. By the approximation \( \varepsilon_{cr, t=1} = 0 \) Eq. 7.8 can be simplified by

\[ \varepsilon_{cr} = b \log t \]  \hspace{1cm} (7.9)

This approximation is justified by the experimental data which show that, also at the
start of a creep test, the axial creep strain accumulated within one second is
negligible. It should also be noted that one second is about the error of the estimation
of the true \( t = 0 \), i.e. the time at which load application has just been completed.
The major advantage of Eq. 7.9 is that the creep process can be characterised by one
single parameter \( b \).

7.6.1.2 Type of accelerating creep law

In Fig. 7.5c it appeared that a power law does not fit the region of accelerating
creep. In the log-log graph no distinct linear relation between log axial strain rate
and log time can be identified. Linear regression of log axial strain rate vs. log time
did not indicate a power law fit. However, by using the variables \( \varepsilon_{t} \) versus \( t_{r} \)
instead of \( \varepsilon \) versus \( t \), the region of accelerating creep can be appropriately delineated
by a power law. Here \( \varepsilon_{t} \) and \( t_{r} \) stand for the axial strain at failure respectively the
time to failure. As an example the log-log plot of Fig. 7.10b clearly shows that for
the wet test cwl1-5 the accelerating region is characterised by a straight line almost
from the point of minimum axial strain rate up to failure.

If we assume that the contribution of accelerating creep is negligible in the
decelerating creep region and vice versa, the decelerating creep region can be
described by Eqs. 7.7 and 7.9 and the accelerating creep region by:

\[ \varepsilon_{f} - \varepsilon = c( t_{f} - t )^{d} \]  \hspace{1cm} (7.10)
Fig. 7.10 Semi-logarithmic plot of axial creep strain vs. time for test cw 1-5 showing the fit by the creep law of Eq. 7.9 for the decelerating stage (a); double-logarithmic plot of axial strain at failure minus axial strain vs. time at failure minus axial strain, depicting the good fit of the creep law of Eq. 7.10 for the accelerating stage (b).
7.6.1.3 Separation of decelerating and accelerating creep

However, in between the regions of pure decelerating and pure accelerating creep, which can be fitted by straight lines, a non-linear transition area exists where the two phases of creep gradually pass into one another. A method is needed to determine at which reading pure decelerating creep ends and where pure accelerating creep starts. Methods to overcome this problem are outlined by Cruden (1987):

- First the linear regression line for decelerating creep is tested for serial correlation in the residuals $u_i$, where

$$
\epsilon_i = a + b \log t_i - u_i
$$

(7.11)

For an adequate fit to the data the residuals must be randomly distributed with a mean of zero. If the value of a residual depends on the value of the next residual, then a systematic variation exists. For example in Fig. 7.10a it can be noticed that for test cw1-5 serial correlation increases significantly if in the analysis data are included beyond reading 11, where the creep curve deflects gradually from a straight line due to an increasing axial strain rate. Serial correlation can be described by the Durbin and Watson (1951) statistic:

$$
DW = \Sigma (u_i - u_{i-1})^2 / \Sigma u_i^2
$$

(7.12)

The authors tabulated two groups of critical values for DW against $n$, the number of observations, at different confidence levels. The 5% confidence level is used here. The two groups of critical values comprise the lower bounds, $dL$, and the upper bounds, $dU$. If the observed DW is less than $dL$, positive serial correlation may be inferred. If DW exceeds $dU$ no serial correlation exists at the significance level concerned. If DW lies between $dL$ and $dU$ the test is inconclusive.

- Secondly the correlation coefficient $R^2$, is calculated as a measure of goodness of fit.

- Finally it is investigated if the calculated value of the correlation coefficient represents a statistically significant relation between $\epsilon$ and $t$. For this purpose the level of significance is determined with the aid of a T-test. The least square sum, $Q^*$, of the obtained regression function ($r$ degrees of freedom) is compared with the least square sum, $Q$, of the function with all coefficients equal to zero except the constant, i.e. the function value for $X$-variable $= 0$ ($k$ degrees of freedom):

$$
T = (Q - Q^*) (n-r) / Q (r-k)
$$

(7.13)

$T$ is characterized by the F-distribution. The significance is equal to the probability that $F$ will exceed $T$. If the significance exceeds a chosen value, the regression function has to be rejected.
Part II: Laboratory experiments

Table 7.2 Decelerating creep regression lines for experiment cw1-5.

<table>
<thead>
<tr>
<th>range</th>
<th>a</th>
<th>b</th>
<th>$R^2$</th>
<th>DW</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-12</td>
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<tr>
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Table 7.3 Accelerating creep regression lines for experiment cw1-5.

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<th>b</th>
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</tbody>
</table>

The decelerating creep law for one creep experiment at a certain stress is then established as follows. First all $e$-$t$ observation pairs of the test are used for the linear regression, which is then subjected to the three criteria mentioned above. The creep law is accepted when $DW$ exceeds $dU$, $R^2$ exceeds 0.90 and the significance is less than 0.05. When the criteria are not satisfied the last observation pair is omitted and the remaining data are fitted again. This process is repeated until the creep law fits all three criteria.

The constitutive law for the accelerating creep region is determined in a similar way, but now the first observation pair is deleted instead of the last reading if the result
Table 7.4 Decelerating creep regression lines for 19 experiments of test series 1 and 2. The power law parameters $a$ and $b$ apply to axial strain in millistrains and time in seconds. For test series 1 and 2 (wet) the UCS refers to UCS$_{wet}$, for the tests of series 2 (dry) to UCS$_{dry}$.

<table>
<thead>
<tr>
<th>Test nr.</th>
<th>Stress (MPa)</th>
<th>Stress /UCS</th>
<th>Data range</th>
<th>$a$</th>
<th>$b$</th>
<th>$R^2$</th>
<th>DW</th>
</tr>
</thead>
<tbody>
<tr>
<td>cw1-1</td>
<td>2.27</td>
<td>0.714</td>
<td>7-16</td>
<td>1.8968</td>
<td>0.0681</td>
<td>0.9976</td>
<td>1.8625</td>
</tr>
<tr>
<td>cw1-10</td>
<td>2.40</td>
<td>0.755</td>
<td>4-11</td>
<td>1.9801</td>
<td>0.0844</td>
<td>0.9966</td>
<td>1.3896</td>
</tr>
<tr>
<td>cw1-8</td>
<td>2.48</td>
<td>0.780</td>
<td>3-12</td>
<td>2.0321</td>
<td>0.1162</td>
<td>0.9984</td>
<td>1.7331</td>
</tr>
<tr>
<td>cw1-9</td>
<td>2.48</td>
<td>0.780</td>
<td>3-8</td>
<td>2.0292</td>
<td>0.1089</td>
<td>0.9942</td>
<td>1.4453</td>
</tr>
<tr>
<td>cw1-5</td>
<td>2.66</td>
<td>0.836</td>
<td>2-9</td>
<td>2.2139</td>
<td>0.1354</td>
<td>0.9782</td>
<td>1.3147</td>
</tr>
<tr>
<td>cw1-6</td>
<td>2.66</td>
<td>0.836</td>
<td>2-14</td>
<td>2.1877</td>
<td>0.1102</td>
<td>0.9942</td>
<td>1.3834</td>
</tr>
<tr>
<td>cw1-2</td>
<td>2.90</td>
<td>0.912</td>
<td>7-13</td>
<td>2.4075</td>
<td>0.1547</td>
<td>0.9915</td>
<td>1.6290</td>
</tr>
<tr>
<td>cw1-4</td>
<td>2.90</td>
<td>0.912</td>
<td>3-12</td>
<td>2.4110</td>
<td>0.1792</td>
<td>0.9988</td>
<td>1.6635</td>
</tr>
<tr>
<td>cw1-3</td>
<td>3.04</td>
<td>0.956</td>
<td>6-12</td>
<td>2.5897</td>
<td>0.1927</td>
<td>0.9873</td>
<td>1.7064</td>
</tr>
<tr>
<td>cw1-7</td>
<td>3.07</td>
<td>0.965</td>
<td>4-7</td>
<td>2.6707</td>
<td>0.2030</td>
<td>0.9994</td>
<td>1.7180</td>
</tr>
</tbody>
</table>

**Series 2, wet**

<table>
<thead>
<tr>
<th>Test nr.</th>
<th>Stress (MPa)</th>
<th>Stress /UCS</th>
<th>Data range</th>
<th>$a$</th>
<th>$b$</th>
<th>$R^2$</th>
<th>DW</th>
</tr>
</thead>
<tbody>
<tr>
<td>cw2-1</td>
<td>2.35</td>
<td>0.671</td>
<td>6-23</td>
<td>1.8628</td>
<td>0.0981</td>
<td>0.9981</td>
<td>1.4557</td>
</tr>
<tr>
<td>cw2-4</td>
<td>2.43</td>
<td>0.694</td>
<td>3-9</td>
<td>1.8993</td>
<td>0.1256</td>
<td>0.9953</td>
<td>1.9286</td>
</tr>
<tr>
<td>cw2-2</td>
<td>2.47</td>
<td>0.706</td>
<td>6-15</td>
<td>1.9835</td>
<td>0.1389</td>
<td>0.9942</td>
<td>2.2659</td>
</tr>
<tr>
<td>cw2-3</td>
<td>2.72</td>
<td>0.777</td>
<td>5-12</td>
<td>2.1418</td>
<td>0.1281</td>
<td>0.9859</td>
<td>1.8960</td>
</tr>
</tbody>
</table>

**Series 2, dry**

<table>
<thead>
<tr>
<th>Test nr.</th>
<th>Stress (MPa)</th>
<th>Stress /UCS</th>
<th>Data range</th>
<th>$a$</th>
<th>$b$</th>
<th>$R^2$</th>
<th>DW</th>
</tr>
</thead>
<tbody>
<tr>
<td>cd2-3</td>
<td>3.09</td>
<td>0.61</td>
<td>4-10</td>
<td>2.1425</td>
<td>0.02708</td>
<td>0.9991</td>
<td>1.5942</td>
</tr>
<tr>
<td>cd2-4</td>
<td>3.49</td>
<td>0.69</td>
<td>4-16</td>
<td>2.3377</td>
<td>0.03346</td>
<td>0.9996</td>
<td>2.5002</td>
</tr>
<tr>
<td>cd2-1</td>
<td>3.86</td>
<td>0.77</td>
<td>4-19</td>
<td>2.6347</td>
<td>0.04117</td>
<td>0.9971</td>
<td>1.3850</td>
</tr>
<tr>
<td>cd2-5</td>
<td>4.26</td>
<td>0.85</td>
<td>5-25</td>
<td>2.8633</td>
<td>0.04772</td>
<td>0.9985</td>
<td>2.2497</td>
</tr>
<tr>
<td>cd2-2</td>
<td>4.80</td>
<td>0.95</td>
<td>3-12</td>
<td>3.2452</td>
<td>0.05016</td>
<td>0.9961</td>
<td>1.6012</td>
</tr>
</tbody>
</table>

of the linear regression does not satisfy the criteria.

The procedure outlined above is extremely time-consuming if carried out by hand. Therefore a computer program was developed which performs the linear regression and calculates DW and $R^2$ for all subsequent series of observation pairs. This program was written as a so called "macro" within the commercially available
Table 7.5 Accelerating creep regression lines for 9 experiments of test series 1 and 2. The power law parameters c and d apply to axial strain in millistrains and time in seconds.

<table>
<thead>
<tr>
<th>Test nr.</th>
<th>Stress (MPa)</th>
<th>Stress/UCS&lt;sub&gt;wd&lt;/sub&gt;</th>
<th>Data range</th>
<th>c</th>
<th>d</th>
<th>R&lt;sup&gt;2&lt;/sup&gt;</th>
<th>DW</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Series 1, wet</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>cw1-8</td>
<td>2.48</td>
<td>0.780</td>
<td>15-24</td>
<td>0.01656</td>
<td>0.2079</td>
<td>0.9877</td>
<td>1.4962</td>
</tr>
<tr>
<td>cw1-5</td>
<td>2.66</td>
<td>0.836</td>
<td>15-28</td>
<td>0.00329</td>
<td>0.3918</td>
<td>0.9972</td>
<td>1.4792</td>
</tr>
<tr>
<td>cw1-2</td>
<td>2.90</td>
<td>0.912</td>
<td>19-26</td>
<td>0.00139</td>
<td>0.5864</td>
<td>0.9922</td>
<td>1.3345</td>
</tr>
<tr>
<td>cw1-4</td>
<td>2.90</td>
<td>0.912</td>
<td>14-23</td>
<td>0.00294</td>
<td>0.4393</td>
<td>0.9952</td>
<td>1.5475</td>
</tr>
<tr>
<td>cw1-3</td>
<td>3.04</td>
<td>0.956</td>
<td>8-14</td>
<td>0.00036</td>
<td>0.8384</td>
<td>0.9892</td>
<td>1.4350</td>
</tr>
<tr>
<td>cw1-7</td>
<td>3.07</td>
<td>0.965</td>
<td>4-11</td>
<td>0.00017</td>
<td>1.0616</td>
<td>0.9987</td>
<td>1.6938</td>
</tr>
<tr>
<td><strong>Series 2, wet</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>cw2-4</td>
<td>2.43</td>
<td>0.694</td>
<td>10-21</td>
<td>0.01570</td>
<td>0.2389</td>
<td>0.9781</td>
<td>1.5470</td>
</tr>
<tr>
<td>cw2-2</td>
<td>2.47</td>
<td>0.706</td>
<td>21-29</td>
<td>0.01290</td>
<td>0.2477</td>
<td>0.9862</td>
<td>1.6415</td>
</tr>
<tr>
<td>cw2-3</td>
<td>2.72</td>
<td>0.777</td>
<td>12-22</td>
<td>0.00581</td>
<td>0.4266</td>
<td>0.9832</td>
<td>1.4359</td>
</tr>
</tbody>
</table>

spreadsheet program of Symphony 4.2. The program can be found in the appendix. To determine the significance the computer program ESM (Simple Statistical Manipulator, in Dutch: Eenvoudige Statistische Manipulator) was used.

7.6.1.4 Assessment of creep law parameters

As an example, the values of DW and R<sup>2</sup> and the parameters of the decelerating creep regression lines for test cw1-5 are summarized in Table 7.2. The data ranges of 2-29 to 2-10 show values of DW less than the upper values dU. For the chosen data range of 2-9 DW exceeds dU and the other two criteria are also satisfied. Regarding the accelerating creep regression lines, the data range of 15-28 could be accepted (Table 7.3). Figs. 7.10 a and b depict graphically the good fit of both regression lines. Note that the units of stress and time for all equations of this chapter are seconds and MPa respectively.

This analysis was repeated for the remaining tests of series 1, 2(wet) and 2(dry) both for decelerating creep and, if ε-t values were available up to failure, also for accelerating creep. The power law coefficients are summarized in Tables 7.4 and 7.5.

Notice that for 2 of the 9 tests which ended in failure empirical descriptions of the subsequent creep regions are based on data ranges which show some overlap. Also Cruden (1987) observed this for one single test and suggested that the physical
Table 7.6 Entire creep curve regression lines for 9 experiments of the first and second test series. The parameters apply to axial strain in millistrains and time in seconds.

<table>
<thead>
<tr>
<th>Test nr.</th>
<th>Stress (MPa)</th>
<th>Stress/UCS</th>
<th>a’</th>
<th>b’</th>
<th>c’</th>
<th>d</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Series 1, wet</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>cw1-8</td>
<td>2.48</td>
<td>0.780</td>
<td>2.4343</td>
<td>0.1282</td>
<td>-1.808 * 10^2</td>
<td>0.2079</td>
<td>0.9979</td>
</tr>
<tr>
<td>cw1-5</td>
<td>2.66</td>
<td>0.836</td>
<td>2.7162</td>
<td>0.1522</td>
<td>-3.228 * 10^3</td>
<td>0.3918</td>
<td>0.9977</td>
</tr>
<tr>
<td>cw1-2</td>
<td>2.90</td>
<td>0.912</td>
<td>2.6260</td>
<td>0.1429</td>
<td>-1.105 * 10^3</td>
<td>0.5864</td>
<td>0.9978</td>
</tr>
<tr>
<td>cw1-4</td>
<td>2.90</td>
<td>0.912</td>
<td>2.7553</td>
<td>0.1685</td>
<td>-2.589 * 10^3</td>
<td>0.4393</td>
<td>0.9998</td>
</tr>
<tr>
<td>cw1-3</td>
<td>3.04</td>
<td>0.956</td>
<td>2.9355</td>
<td>0.1409</td>
<td>-2.053 * 10^4</td>
<td>0.8384</td>
<td>0.9992</td>
</tr>
<tr>
<td>cw1-7</td>
<td>3.07</td>
<td>0.965</td>
<td>3.3126</td>
<td>0.0079</td>
<td>-1.579 * 10^4</td>
<td>1.0616</td>
<td>0.9973</td>
</tr>
<tr>
<td><strong>Series 2, wet</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>cw2-4</td>
<td>2.43</td>
<td>0.694</td>
<td>2.2151</td>
<td>0.1330</td>
<td>-1.843 * 10^2</td>
<td>0.2389</td>
<td>0.9957</td>
</tr>
<tr>
<td>cw2-2</td>
<td>2.47</td>
<td>0.706</td>
<td>2.3637</td>
<td>0.1486</td>
<td>-1.907 * 10^2</td>
<td>0.2477</td>
<td>0.9968</td>
</tr>
<tr>
<td>cw2-3</td>
<td>2.72</td>
<td>0.777</td>
<td>2.3889</td>
<td>0.1339</td>
<td>-4.289 * 10^3</td>
<td>0.4266</td>
<td>0.9980</td>
</tr>
</tbody>
</table>

processes causing decelerating creep and accelerating creep overlap in time. However, for the remaining 5 calcarenite samples no data range overlap can be examined. Probably the same physical processes operated during all 9 tests on calcarenite terminating in failure. Therefore a data range overlap for a certain test does not necessarily indicate an overlap in physical processes of decelerating and accelerating creep. The observed variation in overlap may result from the different ways the chosen statistical methods affect the separate data sets.

7.6.1.5 General law, describing decelerating and accelerating creep

A complete creep law, describing both the decelerating and accelerating creep stages, was established by multiple regression, expressing ε as a function of t^b and (t−t_0)^d. The following form of such a creep law is proposed:

$$\epsilon = a' + b' \log t + c' (t_f - t)^d$$ (7.14)

The definition of the new coefficients a’, b’ and c’ is necessary to fit the whole creep curve, including the transition region between pure decelerating and pure accelerating creep. The parameter a’ does not necessarily represent exclusively the axial strain at t = 1 second but is primarily added to increase the goodness of fit.
Fig. 7.11 The good fit of the general creep law of Eq. 7.15 for experiment cw 1-5 describing both the decelerating and accelerating stages; axial strain vs. time (a) and axial strain rate vs. time (b).
Table 7.7 Decelerating strain as a function of stress and time for the creep experiments of series 1, 2(wet) and 2(dry). The parameters apply to axial strain in millistrains and time in seconds.

<table>
<thead>
<tr>
<th></th>
<th>$\epsilon = f(\sigma, t)$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 1 (wet)</td>
<td>$\epsilon = -0.093 + 0.861 \sigma + 5.096 \times 10^{-3} \sigma^{3.318} \log t$</td>
<td>0.968</td>
</tr>
<tr>
<td>Series 2 (wet)</td>
<td>$\epsilon = 0.194 + 0.715 \sigma + 1.369 \times 10^{-2} \sigma^{2.375} \log t$</td>
<td>0.906</td>
</tr>
<tr>
<td>Series 2 (dry)</td>
<td>$\epsilon = 0.119 + 0.647 \sigma + 5.321 \times 10^{-3} \sigma^{1.466} \log t$</td>
<td>0.996</td>
</tr>
</tbody>
</table>

$$\epsilon = f(\sigma/\text{uc}_{\text{dr}y}, t)$$

<table>
<thead>
<tr>
<th></th>
<th>$\epsilon = -0.093 + 3.909 (\sigma/\text{uc}<em>{\text{dr}y}) + 0.772 (\sigma/\text{uc}</em>{\text{dr}y})^{3.318} \log t$</th>
<th>0.968</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 2 (wet)</td>
<td>$\epsilon = 0.194 + 3.604 (\sigma/\text{uc}<em>{\text{dr}y}) + 0.639 (\sigma/\text{uc}</em>{\text{dr}y})^{2.375} \log t$</td>
<td>0.906</td>
</tr>
<tr>
<td>Series 2 (dry)</td>
<td>$\epsilon = 0.119 + 3.261 (\sigma/\text{uc}<em>{\text{dr}y}) + 0.057 (\sigma/\text{uc}</em>{\text{dr}y})^{1.466} \log t$</td>
<td>0.996</td>
</tr>
</tbody>
</table>

$$\epsilon = f(\sigma/\text{uc}_{\text{we}t}, t)$$

<table>
<thead>
<tr>
<th></th>
<th>$\epsilon = -0.093 + 2.738 (\sigma/\text{uc}<em>{\text{we}t}) + 0.237 (\sigma/\text{uc}</em>{\text{we}t})^{3.318} \log t$</th>
<th>0.968</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 2 (wet)</td>
<td>$\epsilon = 0.194 + 2.503 (\sigma/\text{uc}<em>{\text{we}t}) + 0.269 (\sigma/\text{uc}</em>{\text{we}t})^{2.375} \log t$</td>
<td>0.906</td>
</tr>
</tbody>
</table>

For test cw1-5, for example, the complete creep equation is:

$$\epsilon = 2.716 + 0.152 \log t - 3.228 \times 10^{-3} (t_f - t)^{0.3948}$$

(7.15)

with $t_f = 479700$ seconds and $R^2 = 0.998$. The high goodness of fit is illustrated by
the graphs of Figs. 7.11 a and b. A similar agreement with experimental results was obtained for
the remaining creep tests of the first series and second series which provided data up to failure (Table 7.6).

Thus for a single creep experiment the total axial strain $\epsilon$ for the individual regions
of decelerating and accelerating creep and for the whole creep test are well described
by a function using $t$, $t_f$ and $\epsilon_f$ as independent variables.

7.6.1.6 Incorporation of stress in decelerating creep laws

Next, it was attempted to incorporate the applied stress level into the empirical laws
for both creep regions individually. This is first tried for the decelerating creep stage
of the experiments of series 1. Fig. 7.12 a shows that a is more or less linearly
related to stress: the secant modulus does not vary considerably with stress. Figs.
7.12 b-d show that the parameters b,c and d are related to stress by power laws.
Hence a decelerating creep law for all values of stress applied in the experiments of
the first series must be of the form:

$$\epsilon = a_1 + a_2 \sigma + b_1 \sigma^{b_2} \log t$$

(7.16)
Part II: Laboratory experiments

**a)**

\[ a = -0.284 + 0.940 \times \text{stress} \]
\[ R^2 = 0.98 \]

**b)**

\[ b = 4.902 \times 10^{-3} \times \text{stress}^{3.316} \]
\[ R^2 = 0.93 \]
Fig. 7.12 The relationships between creep law parameters and stress level. Decelerating creep law parameter $a$ is linearly related to stress (a) and parameter $b$ shows a power-law relationship with stress (b); both accelerating creep law parameters $c$ and $d$ can be described as power law functions of stress ($c$ and $d$).
where the first two terms describe the instantaneous axial strain and the last term the axial creep strain. By approximation the first two terms of this equation represent the instantaneous axial strain and the last term the axial creep strain. A multiple regression for all 83 $\epsilon$-t observation pairs of series 1 in Table 7.2 with $\sigma$ and $\sigma^b \log t$ as independent variables yields the following equation with a reasonable measure of fit:

$$\epsilon = -0.093 + 0.861 \sigma + 5.096 \times 10^{-3} \sigma^{3.318} \log t \quad (7.17)$$

$R^2$ is 0.97 and the significance less than $1 \times 10^{-3}$. Similar analyses were performed for the experiments of series 2(wet) and 2(dry). The resulting functions describing decelerating strain with time and stress level as independent parameters are given in Table 7.7. Figs. 7.13 a-c show that the empirical laws of Table 7.7 are generally in good agreement with the experimental data.

The decelerating strain rates as a function of time, derived from the equations listed in Table 7.7, are of the form:

$$\frac{d\epsilon}{dt} = b_1 \sigma^b / t \quad (7.18)$$
Fig. 7.14 Strain rate changes with time for the decelerating creep stage for wet (series 1 and 2) and dry (series 2) calcarenite at three different ratio’s of stress and dry UCS. The curves represent the axial strain rate equations given in Table 7.7.

Fig. 7.15 The reasonable fit of the general law for accelerating creep as a function of both stress and time, according to Eq. 7.20 for series 1.
For three stress/\(\text{UCS}_{\text{dry}}\) ratios the axial strain rate vs. time plots are depicted in Fig. 7.14. The log-log graph shows that:

- at each point of time the axial strain rates for dry calcarenite of series 2(dry) are about one order of magnitude smaller than the axial strain rates for wet calcarenite of series 2(wet) at the same stress/\(\text{UCS}_{\text{dry}}\) level.

- loaded at similar stress/\(\text{UCS}_{\text{dry}}\) the samples of series 2 are deforming about 1/3 order of magnitude faster than the samples of series 1 at each point of time. This means that normalisation of the applied stress by the UCS of the calcarenite sample does not necessarily result in similar axial strains and strain rates for calcarenite samples of different locations. The differences between axial strain rate versus time curves for series 1 (wet), 2(wet) and 2(dry) are comparable if axial strain rates are calculated for given values of stress instead of stress/\(\text{UCS}_{\text{dry}}\).

- the axial strain rates for dry calcarenite are less dependent on the applied stress level than the axial strain rates for wet calcarenite.

### 7.6.1.7 Incorporation of stress in accelerating creep laws

A similar procedure was executed to establish a general equation for accelerating creep, using the power law indicated in Fig. 7.12d:

\[
\epsilon_f - \epsilon = c_1 (\sigma/\text{UCS})^{c_2}(t_f - t)^d \quad \text{with} \quad d = d_1 (\sigma/\text{UCS})^{d_2} \quad (7.19)
\]

or

\[
\epsilon_f - \epsilon = 3.102 \times 10^{-4} (\sigma/\text{UCS}_{\text{wet}})^{-16.7}(t_f - t)^d \quad (7.20)
\]

where \(d = 1.043 (\sigma/\text{UCS}_{\text{wet}})^{6.72}\). Here \(R^2\) is 0.95 and the significance is less than 1 \(\times\) 10\(^4\) for all 57 data points of series 1 in Table 7.3. The good measure of fit is illustrated in Fig. 7.15. The family of straight lines in the log-log graph demonstrate that at a higher stress the point of failure is approached more quickly. For series 2 no reliable general equation could be established for the accelerating creep stage because the too limited range of stresses applied.

Attempts were made to find a formula relating axial strain to time and applied stress for all complete creep curves of a complete series of experiments, describing the axial strain development from the beginning of the test up to failure, but neither for the first series nor for the second series were satisfying results obtained.
Fig. 7.16 Stress vs. time to failure (strength vs. time) for wet calcarenite of series 1 and 2.

Fig. 7.17 Failure time vs. inflexion time for wet samples of series 1 and 2.
Fig. 7.18 Failure time vs. minimum axial strain rate for wet samples of series 1 and 2.

Fig. 7.19 The power-law relationship between time to failure and decelerating creep law parameter b for wet samples of series 1 and 2.
Table 7.8 Time to failure for series 1 and 2 according to Eqs. 7.21 a-c.

<table>
<thead>
<tr>
<th>Stress/UCS</th>
<th>Series 1 (wet)</th>
<th>Series 2 (wet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.90</td>
<td>8.4 hours</td>
<td>6.97 minutes</td>
</tr>
<tr>
<td>0.80</td>
<td>27.4 days</td>
<td>2.48 hours</td>
</tr>
<tr>
<td>0.70</td>
<td>10.5 years</td>
<td>3.33 days</td>
</tr>
<tr>
<td>0.65</td>
<td>163 years</td>
<td>22.9 days</td>
</tr>
<tr>
<td>0.60</td>
<td>3156 years</td>
<td>0.50 year</td>
</tr>
<tr>
<td>0.50</td>
<td>2.68 million years</td>
<td>57.5 years</td>
</tr>
<tr>
<td>0.45</td>
<td>132 million years</td>
<td>889 years</td>
</tr>
<tr>
<td>0.40</td>
<td>10.3 billion years</td>
<td>19010 years</td>
</tr>
</tbody>
</table>

7.6.1.8 The relationship between stress level and time to failure

For the wet samples of series 1 a strong correlation was found between time to failure and stress (Fig. 7.16):

\[ t_f = 2.13 \times 10^{21} \times \sigma^{-37} \]  \hspace{1cm} (7.21a)

or

\[ t_f = 616 \times (\sigma/UCS_{wet})^{-37} \]  \hspace{1cm} (7.21b)

Although only three data points are available, the relation for series 2 appears:

\[ t_f = 27 \times (\sigma/UCS_{wet})^{-26} \]  \hspace{1cm} (7.21c)

The high value of the stress exponent means that an increase in stress of only five percent can reduce the time to failure about one order of magnitude! Thus the long-term stability of wet calcarenite cylindrical cores is extremely sensitive to variations in stress.

It has already been outlined above that at the same stress or stress/UCS the specimens of the second series show faster axial strain rates, at least during the decelerating creep stage. This largely explains why the major part of the samples of series 2 fail in much less time than the samples of series 1, while loaded at similar stresses. The difference in time to failure is about one order of magnitude. Note that the specimens of series 2 are of higher mean UCS (3.5 MPa) than the specimens of
series 1 (3.18 MPa). Yet the series 2 samples creep at a faster rate and fail more quickly than the samples of series 1. Thus for calcarenite a lower short-term strength is not necessarily accompanied by an increased sensitivity to creep deformation. It can also be concluded that for an arbitrary calcarenite sample, and thus also for a calcarenite pillar, the time to failure cannot be predicted just on the basis of mean UCS and applied stress. Thus the time to failure of a whole calcarenite mine cannot be predicted on the basis of creep tests. This problem may come from variations of the microstructure, e.g. porosity, cementation and size, shape and arrangement of grains. At present these variations and their effect on the individual geomechanical parameters are not known.

7.6.1.9 The relationship between time to failure and other creep parameters

For series 1 and 2 the time to failure is also well correlated to the inflexion time, the time at which the axial strain rate has its minimum value, and to the minimum axial strain rate (Figs. 7.17 and 7.18):

\[ t_f = 3.69 \times (t_{\text{inflex}})^{0.949} \]  \hspace{1cm} (7.22)

\[ t_f = 1.22 \times 10^{-4} \times (d\varepsilon/dt)^{-1.06} \]  \hspace{1cm} (7.23)

It is of interest that in both relations the data points of both series fit one and the same regression line. The exponents of inflexion time and minimum axial strain rate are almost equal to one, but the relations cannot be considered linear. Exponents of one would result in deviations of one or more orders of magnitude from actually observed inflexion times and minimum axial strain rates. These data show that the time to failure for an arbitrary calcarenite sample can be assessed when the axial creep strain rate starts to increase.

The relation between \( t_f \) and \( b \), the parameter determining both axial creep strain and creep strain rate at a certain time, is also significant (Fig. 7.19):

\[ t_f = 3.60 \times 10^{-6} \times b^{-12.5} \]  \hspace{1cm} (7.24)

The standard error of \( Y \)-estimation is 0.55 times the order of magnitude. Thus a powerful tool is available to make at least a rough estimate of the time to failure at an early stage of the creep process for any calcarenite sample. The parameter \( b \) can be eventually adjusted after additional measurements in order to achieve a better estimate of the time to failure. At a later stage also the relations of Figs. 7.17 and 7.18 may increase the precision of the estimate.
Fig. 7.20 Long-term strength assessment by the volumetric strain method. The curves refer to the constant strain-rate experiments on wet calcarenite cores, described in Sections 5.2 to 5.4.

Fig. 7.21 The volumetric strain method applied to oven-dried calcarenite cores of the same block as depicted in Fig. 7.20 (see Sections 5.2 to 5.4).
Table 7.9 Decelerating creep regression lines for 9 experiments of test series 3. The parameters a and b apply to axial strain in millistrains and time in seconds.

<table>
<thead>
<tr>
<th>Test Nr.</th>
<th>Stress (MPa)</th>
<th>Stress/pillar strength</th>
<th>a</th>
<th>b</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-7</td>
<td>2.09</td>
<td>0.51</td>
<td>4.5698</td>
<td>0.0912</td>
<td>0.9949</td>
</tr>
<tr>
<td>3-6</td>
<td>2.52</td>
<td>0.62</td>
<td>5.1233</td>
<td>0.1212</td>
<td>0.9951</td>
</tr>
<tr>
<td>3-3</td>
<td>3.07</td>
<td>0.75</td>
<td>5.4888</td>
<td>0.1361</td>
<td>0.9960</td>
</tr>
<tr>
<td>3-5</td>
<td>3.07</td>
<td>0.75</td>
<td>5.5678</td>
<td>0.1212</td>
<td>0.9899</td>
</tr>
<tr>
<td>3-2</td>
<td>3.33</td>
<td>0.81</td>
<td>5.9358</td>
<td>0.1726</td>
<td>0.9966</td>
</tr>
<tr>
<td>3-9</td>
<td>3.41</td>
<td>0.83</td>
<td>6.1536</td>
<td>0.5179</td>
<td>0.9996</td>
</tr>
<tr>
<td>3-8</td>
<td>3.59</td>
<td>0.88</td>
<td>6.2945</td>
<td>0.2583</td>
<td>0.9985</td>
</tr>
<tr>
<td>3-1</td>
<td>3.69</td>
<td>0.90</td>
<td>6.3001</td>
<td>0.2306</td>
<td>0.9970</td>
</tr>
<tr>
<td>3-4</td>
<td>3.73</td>
<td>0.91</td>
<td>6.3850</td>
<td>0.3879</td>
<td>0.9952</td>
</tr>
</tbody>
</table>

7.6.1.10 The assessment of long-term strength

The $\sigma$-t$_f$ data (Fig. 7.16) do not strictly indicate the existence of a long-term strength: on decreasing the stress the time to failure will increase dramatically but no asymptotic value of stress is necessarily reached. According to the power law creep deformation will proceed no matter how low the stress. For the samples of series 1 and 2 times to failure for various $\sigma$/uc$_{swt}$ values, according to Eqs. 7.21, are given in Table 7.8. Note that the specimens of series 2 fail much more quickly. However, data points were collected for series 2, and these only apply to stress levels exceeding 70 % of the UCS. According to the table, if a long term of some thousands of years is considered, long-term strength is 40 to 60 % of the UCS. But it is not certain if the results, acquired in a limited number of experiments of a duration of up to a few months, may be extrapolated in this way. It is not impossible that, if more data points would be acquired, a stress vs. time to failure plot does indicate a true long-term strength.

The time-dependent deformation of the dry specimens of series 2 did not reach the accelerating creep stage and no failures were recorded within the test duration of 115 days, even at stresses of 95 % of the dry UCS.

Since determining the long-term strength directly is time-consuming and complicated by extrapolation, various indirect methods have been proposed. Several options are described by Singh et al. (1977) and Lama & Vutukuri (1978). The volumetric strain method, proposed by Bieniawski (1967), was applied with some success. He distinguished two subsequent stages of microfracture development, namely stable and unstable. As described by Patterson (1978; Chapter 7), during a constant strain-rate test microcracking starts when the stress-strain graph departs from perfectly elastic behaviour. This is, for example, to be observed from dilatancy. According to Bieniawski (1967), unstable microfracturing begins when the specimen starts to dilate (Figs. 7.20 and 7.21). However, this term seems not to be strictly suitable in that microcracks do not uncontrollably propagate without a further increase in stress.
Fig. 7.22 The power-law relationship between decelerating creep law parameter $b$ and stress, according to Table 7.8, for wet calcarenite prisms of $W/H = 2$ (series 3).

Fig. 7.23 Strain rate changes with time for the decelerating creep stage for wet cylindrical cores of $W/H = 0.4$ (series 1 and 2) and prisms of $W/H = 2$ (series 3) at three different ratio's of stress and dry UCS. The curves represent the strain rate equations given by Table 7.7 and Eq. 7.26.
Fig. 7.24 Stress vs. time to failure (strength vs. time) for wet calcarenite cores of series 1 and 2 and wet calcarenite prisms of series 3.

Fig. 7.25 Failure time vs. inflexion time for wet calcarenite cores of series 1 and 2 and wet calcarenite prisms of series 3.
Nevertheless, Bieniawski (1967) and Patterson (1978) found a reasonable agreement between long-term strength estimated from actual creep tests and dilatancy. Since at brittle conditions the calcarenite also deforms by microcracking, in this case by breakage of intergranular cement bonds, this method was also applied here to get an indication of long-term strength.

Figs. 7.20 and 7.21 show the stress vs. volumetric strain curves for respectively the uniaxial compression experiments "Natural" 1 to 4, on cores of natural moisture content, and "Oven dried" 1 to 5, on dry cores (see Table. 5.3). As described in Section 5.2.2, radial strain was measured by a circumferential direct-contact extensometer at mid-height of the sample. Volumetric strain was determined by taking two times the radial strain, minus the axial strain. Due to lateral confinement at the end platens radial strain is not homogeneous over the specimen height, and, as a consequence, the volumetric strain, depicted in Figs. 7.20 and 7.21 is not representative for the samples as a whole. However, this is not a problem because the true uniaxial condition, which exists at sample mid-height, is of interest. By this method, for wet and dry specimens long-term strength values of 67 and 72 % of the wet and dry UCS were estimated respectively. These values are well in excess of the percentages determined above from creep experiments.
7.6.2 Prismatic samples

Creep laws established for the tests on the cylindrical samples also fitted the experiments on the prismatic samples. The parameters a and b of Eq. 7.7 are summarized in Table 7.9. A reasonable power law fit for b vs. stress could be found (Fig. 7.22). The general decelerating creep law, the abnormal high values for tests 4 and 9 omitted, is:

\[ \epsilon = 2.333 + 1.122 \sigma + 1.690 \times 10^{-2} \sigma^{1.615} \log t \]  
(7.25)

with \( R^2 = 0.946 \) and a standard error of \( Y \)-estimation of 0.150. Eq. 7.25 can also written as:

\[ \epsilon = 2.333 + 4.560 \sigma / Sp_{wet} + 1.650 \times 10^{-1} (\sigma / Sp_{wet})^{1.615} \log t \]  
(7.26)

In Fig. 7.23 axial strain rate versus time curves, according to Eq. 7.26 and the equations of Table 7.7, are shown for three different stress/\( \text{ucS}_{\text{wet}} \) levels. It can be seen that the axial strain rates for prismatic samples are of the same order of magnitude as those measured for cylindrical cores.

In Fig. 7.24 it appears that the time to failure at a given stress and, as a result, also the long-term strength, is comparable to those measured on cylindrical samples. However, only in three experiments and within a limited stress range, failure could be detected.

Figs. 7.25 and 7.26 show that the relations between failure time on one hand and inflexion time and minimal axial strain rate on the other hand, established on cylindrical samples, also seem to apply to prismatic specimens. This means that also for pillars, no matter the UCS, the time to failure can be estimated when the axial strain rate starts to increase.

7.7 LONG-TERM STRENGTH OF MINE PILLARS

7.7.1 Delineation of the problem

It was not expected that axial strain rates for prisms of a W/H ratio of two proved to be comparable with those for cylindrical cores, because the confining pressure in the major central part of the prism is known to be higher than in cylindrical cores. This was also shown by the numerical experiments on prisms of \( W/H = 1 \) (Fig. 6.48). Several authors including Lama & Vutukuri (1978) reported that an increasing confining pressure reduces creep rate and the susceptibility to creep in general.

Figs. 7.27 a and b show the distribution of horizontal stress (elastic) for the upper right quadrant of a prism (between machine platens) and a pillar (between calcarenite
Fig. 7.27 The distribution of horizontal stress, normalized by mean vertical stress, for a prism of W/H = 4 (upper right quadrant) in contact with machine platens (a), and for a mine pillar (b).

roof and floor) respectively, both of a W/H ratio of 4. The stress distributions result from numerical experiments which were performed using FLAC in the same way as described in Section 6.13. A comparison with Fig. 6.48 reveals that relatively high horizontal stresses occur in a considerably more extensive part of a prism or pillar for a W/H ratio of four than for a W/H ratio of one. Therefore, it is expected that creep rates decrease and failure times and long-term strength increase with increasing W/H ratio. Such effects were demonstrated for example by Knoll (1973) by experiments on carnalite prisms.

Notice however that the experiments on the calcarenite cores and prisms were not performed with the same sample material. A comparison between cores of series 1 and 2 had already made clear that creep rates and times to failure can differ at least one order of magnitude for calcarenite samples, which were originally taken less than one meter apart in the vertical sense. This effect significantly complicates the comparison of creep behaviour for prisms and cores and the establishment of long-term strength of pillars.

The question arises how to include long-term deformation behaviour into pillar strength formulae and how to assess "life expectation" of pillars. In the previous chapter it was shown that the stress-strain curve and the ratio between peak and
residual strength are considerably affected by W/H ratio. Fig. 7.28 depicts a general stress-strain curve for a prism of a W/H ratio of four. The residual strength at three times the axial strain at failure is 80 % of the peak strength, according to Eqs. 6.6 and 6.7. The long-term strength for cylindrical cores is about 72 % (estimated from volumetric strain) or ranges between 40 and 60 % (estimated from creep tests). The strain path of creep at a constant stress is represented by a horizontal line. If this long-term strength of a cylindrical core would apply to the pillar, then, in case of eventual creep failure, the strain path could be possibly located below the short-term stress-strain curve (Fig. 7.28) and the long-term strength would be lower than the residual strength. At creep failure a lower residual strength than established in short-term experiments would result. This does not appear logical. Therefore the relationship between long-term- and residual strength is further evaluated in the next sections. First, for a better comprehension, a short outline is given of some principles of fracture mechanics.

### 7.7.2 Fracture mechanics and creep rupture of calcarenite

At this stage fracture mechanics and continuum damage models, as reviewed by Costin (1989), are considered. Such models consider that macroscopic failure occurs when the specimen has suffered a certain amount of microstructural damage, i.e. the critical level of damage. It is generally agreed that the dominant mechanism of deformation and failure in brittle rocks under compressive loading is characterized by nucleation, growth and coalescence of microcracks. For the calcarenite microcracking just corresponds to the rupture of the weak intergranular cement bonds. This rock type is also special, in that at increasing confining stress cement bond breakage probably results in pore collapse (Sections 5.3 and 5.5). Cracks extend only if the crack tip stress field, denoted as the stress intensity factor, has attained a minimum value: the critical stress intensity factor. Small-scale sliding and/or extension at grain contacts, as far as the grain-structure allows, results in deformation of the specimen: shortening in the direction of the applied axial stress and expansion in the opposite direction. Accordingly, damage is characterized by a complex function of lengths and orientations of microcracks and their distribution. The total strain is composed of two components: the strain due to microstructural damage proper and the elastic deformation of the cracked elastic matrix. As a complication, the elastic moduli change continuously with an increasing degree of damage.

In a compressive, constant strain-rate experiment, of a duration which precludes important time-dependent deformation, microcracking is a stable process: the stress intensity factor at a crack tip decreases on crack lengthening. Thus crack length only increases at an increase of the applied deviatoric stress, and this process stops after a certain amount of crack growth. However, at a certain degree of damage the crack density is such that interaction between the stress fields at the crack tips develops. When the critical level of damage is attained microcracking becomes unstable: the stress intensity factors at crack tips now increase with crack length. If the applied stress is not relieved the specimen fails, i.e. the microcracks link up resulting in the
Fig. 7.28 The short-term stress-strain curve for a prism of a W/H ratio of 4. Is the long-term strength less than the residual, short-term strength?

Now deformation at low confining stress is further considered. While for experiments of a relatively short duration crack extension only occurs once the critical stress intensity factor has been attained or exceeded, this is not true for long-term experiments. If sufficient time is available cracks can grow and microstructural damage can increase slowly at stress intensity factors below the critical value. This phenomenon is known as subcritical crack growth and its rate increases with the value of the stress intensity factor. The role of water is important, as also observed for calcarenite. During a creep experiment the amount of damage increases due to subcritical crack growth. Due to crack expansion, the stress intensity factors are reduced and the rate of subcritical crack growth as well. This effect is measured as decelerating creep. However, if the applied stress is large enough, the subcritical crack growth brings about the beginning of interaction of microcracks. A gradual acceleration of the creep rate begins finally resulting in failure when the critical level of damage has been reached. The minimum stress level at which a creep experiment culminates in failure can be considered as the long-term strength. At stresses below this critical level a critical crack density is not attained and the creep rate will finally decay to zero.
7.7.3 Evaluation of long-term strength of pillars relative to residual strength

In Chapter 6 it was shown that pillars of W/H ratio's in excess of about 0.6, a ratio which applies to practically all pillars in the calcarenite mines, do not fail along one single macroscopic shear plane as for the tested cylindrical cores. Instead, for such pillars shear fracturing leads to a separation into an hourglass shaped core and parted pillar sides (e.g. Figs. 6.5 and 6.6). The central core is not affected by macroscopic fractures, unless excessive strains are imposed and the pillar is about to collapse. Thus two regions can be distinguished: an external zone where the stress state is such that macroscopic shear fracturing occurs, and an internal zone where the stress state gives rise to homogeneous deformation. In the previous chapter it became clear that reduction of the load carrying capacity of a pillar is brought about by the macroscopic shear fracturing in the external zone. The load carrying capacity of the internal zone is not or hardly affected upon failure. Consecutive post-failure deformation proceeds at more or less the same stress level, denoted as the residual strength (e.g. Fig. 6.4), as long as collapse does not occur, with vertical strains of several hundreds of microstrains. Thus the stress state in the internal zone is such that deformation is homogeneous and without reduction of resistance to load.

Now we consider creep deformation of the pillar. Only in the external zone the stress state is such that subcritical growth of microfractures gives rise to accelerating creep, failure along macroscopic shear fractures and loss of strength. Only in this zone microstructural damage can induce failure at relatively low stresses and only here resistance to load can be considerably reduced by creep. In the internal zone also creep deformation occurs, but here the stress state is different, in that microstructural damage only brings about homogeneous deformation, possibly associated with reduction of pore space. Here subcritical crack growth cannot reduce the resistance to load, as long as the strain does not become too high. Thus residual strength at the short term and at the long term is more or less equal. As a consequence, for pillars of W/H ratio's greater than 0.6 the long-term strength must be equal to or exceed the residual strength, established by short-term experiments. For example, for W/H ratio's of 1, 2 and 4 the long-term strength then becomes at least 42, 60 and 80% of the short-term (peak) strength respectively, which corresponds to 47, 82 and 152% of the UCS.

For pillars of W/H ratio's of less than 0.6 a pillar core or internal zone does not develop. Here failure occurs more or less along one macroscopic shear plane (Fig. 6.10). Once failed, there is hardly any residual strength. This shear fracture geometry is comparable to that of the tested cylindrical cores, and the ratio of long-term- and short-term strength for such pillars must be the same as that for the cores.

7.7.4 Verification of long-term strength of pillars

For the stress vs. time to failure diagram only three data points could be established within a limited range of stress (Fig. 7.24). These points do not allow an estimation of long-term strength. A better method is to apply Eq. 7.25 in order to determine the
Fig. 7.29 Main possibilities of global mine creep curves: no pillar failure (a); one or more pillars fail, due to creep and/or short-term deformation, but no large-scale collapse (b); one or more pillars fail, large-scale collapse (c).

amount of strain in the long term, which is taken as 10000 years. The strain at the inflexion point, at the beginning of accelerating creep, is about 8 millistrain. By putting this strain in Eq. 7.25, the required stress can be calculated. Since a lower stress cannot induce acceleration and creep failure, this value represents an estimation of long-term strength. The long-term strength proved to be 3.7 MPa, which corresponds to 66 % of $\sigma_p$. Indeed, this value exceeds the residual strength of 60 % of $\sigma_p$. For a solid experimental prove of the statement that $\sigma_{lt} \geq \sigma_r$ for pillars of $W/H > 0.6$, much more creep experiments on prisms of various $W/H$ ratio’s are essential.

### 7.8 APPLICATIONS OF RESULTS IN THE MINES

In agreement with the experimental results on calcarenite and the reasoning of Section 7.7 it is decided to incorporate long-term deformation in the calculation of a so-called long-term pillar strength $\sigma_{lt}$ as follows:

\[
\begin{align*}
W/H &< 0.6: \quad \sigma_{lt} = 0.6 \sigma_p \\
W/H &\geq 0.6: \quad \sigma_{lt} = \sigma_r
\end{align*}
\]

(7.27)

So for pillars of $W/H$ ratio’s of less than 0.6 a reduction factor of 40 % should be applied on the value of peak strength. Note that in these cases the residual strength is lower than the long-term strength. For pillars of $W/H$ ratio’s of one and more the residual strength should be taken as long-term strength.
Eq. 7.27 can also be applied to calculate a safety factor \( SF_{lt} \) for an individual pillar (assessment of individual pillar stability) and a total safety factor \( SF_{tot,lt} \) for a whole section of a mine (assessment of large-scale pillar stability):

\[
SF_{lt} = \frac{\sigma_{lt}}{\sigma}
\]  

(7.28)

and

\[
SF_{tot,lt} = \sum \left( \frac{\sigma_{lt} \times A_p}{\sigma_{av} \times A_t} \right) \quad \sum \left( \frac{\sigma_{av} \times A_t}{\sigma_{av} \times A_t} \right)
\]  

(7.29)

Now with respect to validation of pillar strength formulae and tributary area theory in the mines, long-term deformation is also incorporated. Similar to the initial safety factor \( SF_0 \) (Section 6.14.3), a certain general correction factor \( K'_{lt} \) may be necessary:

\[
SF_{lt}' = K'_{lt} \times SF_{lt}
\]  

(7.30)

Then the total safety factor becomes:

\[
SF_{tot,lt}' = K'_{lt} \sum \left( \frac{\sigma_{lt} \times A_p}{\sigma_{av} \times A_t} \right) \quad \sum \left( \frac{\sigma_{av} \times A_t}{\sigma_{av} \times A_t} \right)
\]  

(7.31)

The correction factor \( K'_{lt} \) is expected to be greater than \( K'_{o} \), because loss of strength due to creep deformation is already incorporated in \( \sigma_{lt} \), and thus in \( SF_{lt} \); in principle \( K'_{lt} \) does not correct for creep. \( SF_{lt}' \) is expected to incorporate creep deformation better than \( SF_0' \), because the former is based on \( SF_{lt} \), which considers reduction of strength for each pillar individually according to its shape, while the latter takes creep behaviour into account by one general correction factor which is the same for all pillars. Since long-term strength decreases relative to peak strength at decreasing \( W/H \), the correction factor \( K'_{o} \) for a mine system comprising slender pillars will be relatively low and cannot be applied to another mine which consists of relatively broad and massive pillars. Thus \( K'_{o} \) can only be utilized in other mines if pillars are of comparable geometry. This disadvantage does not exist for \( K'_{lt} \) and therefore it is preferable to use \( SF_{lt} \) and \( K'_{lt} \). The concepts explained above are applied in the following chapters.

It is recommended to measure creep deformation in the mines in order to increase the understanding of creep in calcarenite mine pillars. Particularly the horizontal strain should be measured because this strain increases much more than the vertical (axial) strain when microfracturing has started. Moreover, the measurement of horizontal strain allows to estimate when the volume increase occurs.

Such creep data will make clear in how far load transfer towards other pillars occurs and the other way around. If the creep curve is linear in a semi-logarithmic plot probably no load transfer occurred during the measurements. The decelerating creep parameter \( b \) should be determined in order to estimate the time to failure (without load transfer in the future) using Eq. 7.24. When an acceleration of strain rate is observed the time to failure could be assessed by means of Eqs. 7.22 and 7.23.
7.9. CREEP DEFORMATION OF A WHOLE MINE

If the gallery width and the pillar height is more or less constant, which is generally the case within most mine systems, the ratio between tributary area and pillar area \( A_t/A_p \) increases from large to small pillars, i.e. when \( W/H \) decreases. Accordingly the mean vertical stress on small pillars is relatively high, and, as a consequence, also the creep rate. Thus small pillars tend to deform at a higher rate than large pillars of the same height. At a certain strain, however, such a difference of vertical shortening develops with surrounding, larger pillars that the stress is relieved to a certain extent, depending on the mine stiffness, from the relatively small pillar at the expense of the pillars in its direct vicinity. As a result the creep rate of the small pillar diminishes. Hence, on a pillar scale the mine environment tends to stabilize the creep process. However, this is not true for the mine as a whole. The mean creep deformation of all pillars inside the mine might continue.

On a global scale several possibilities can be recognized. The "creep curve" of the mine as a whole, representing the mean pillar convergence, for each situation is depicted in Fig. 7.29. If for each individual pillar the stress is less than its long-term strength no pillar will ever fail and a large-scale pillar collapse is not possible (Fig. 7.29 a). If a certain pillar fails, as a result of creep and/or short-term failure, its strength is reduced considerably and pillars in the vicinity experience a considerable increase of stress. This might trigger failure of more pillars, corresponding with a domino-effect. Now it depends on general mine stability (see Section 4.3) whether pillar failure, extensive or not, results in a large-scale collapse. In case of a relatively high stiffness and/or high strength of the rock overburden, no collapse is to be expected (Fig. 7.29 b). Otherwise pillar deterioration results in a collapse (Fig. 7.29c). The collapse potential will be studied in more detail in Chapter 11.
PART III

OBSERVATIONS IN THE MINES AND APPLICATION OF RESULTS TO ASSESS MINE STABILITY
CHAPTER 8

THE GEULHEMMER GROEVE

8.1 INTRODUCTION

The Geulhemmer Groeve, situated near the Geul-valley and 6 kilometer downstream of Valkenburg (Fig.2.7), is an extensive mine system of about 10 hectares. It is 700 m long and up to 300 m wide. In the northeastern part a narrow gallery has been created to the Koepelgrot, excavated at a lower stratigraphy level.

The first stability studies were performed in from 1986 to 1988 by Deibel et al. (1987), Vreugdenhil (1987) and Van Steveninck (1988) in order to increase the knowledge of the stability problems of calcarenite mines in general and to test methods of pillar stability calculations. The available data, e.g. pillar dimensions, overburden composition etc., acquired by these authors was one of the reasons to select this mine for a study of pillar stability. Additionally, widespread stress induced pillar fracturing in several degrees occurs, which is not "contaminated" with the effects of a large-scale collapse. Finally, a series of old mine maps was available which indicate the stage of mining at a certain time. Also many reliable dates were written on the pillar walls. This allowed the assessment of the age of the several parts of the mine, which is a necessary parameter to study the factor time regarding pillar deterioration.

The whole mine was subjected to a thorough investigation by Bekendam and Vink in the autumn of 1990 (Vink, 1990; Price & Bekendam, 1991; Bekendam & Price, 1993a and 1993b). For the first time all individual pillar cracks were mapped and block samples for the determination of the UCS were taken at 61 locations. All dates on the pillar walls were mapped. These data are analyzed in the following sections. The characteristics and condition of the mine are dealt with in Sections 8.2 and 8.3 respectively. In 1991 a new type Schmidt hammer was tested and the results are compared with the UCS values (Section 8.4). In Section 8.5 the pillar strength formulae, developed in Chapter 6, are applied. Also an age-map is produced allowing a study of the role of creep deformation. Finally calculated safety factors are validated, according to the principles outlined in Chapters 6 and 7.
Part III: Observations in the mines and application of results to assess mine stability
b)

Fig. 8.1 Northern (a) and southern (b) part of the Geulhemmer Groeve with isopach contour lines and locations of figures.
8.2 CHARACTERISTICS OF THE MINE

8.2.1 Mine geometry

The existing map of the Geulhemmer Groeve was generally correct and accurate to ± 10 cm (Figs. 8.1 a,b). Only a severely damaged area in the northern part had not been mapped precisely. An attempt to assess pillar dimensions in this area during the field work was given up for safety reasons.

The majority of the pillars show a more or less rectangular base, but the overall mine pattern appears irregular due to the geometry of mine system perimeter. The horizontal pillar dimensions generally range from 2 to 15 m. The mean width/length ratio is 0.77. The pillar height normally varies between 2.5 and 3 m. The galleries are usually 3 to 4 m wide. In a small area in the southern part also a deeper level has been mined below the existing mine system. The gallery floors are generally covered by waste debris, which was strongly compacted by the many visitors to the mine.
### Table 8.1 Unit weights of overburden materials for the Geulhemmer Groeve.

<table>
<thead>
<tr>
<th>Formation</th>
<th>Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maas Deposits (gravel, Pleistocene)</td>
<td>18.7</td>
</tr>
<tr>
<td>Formation of Tongeren (sands, Lower Oligocene)</td>
<td>17.3</td>
</tr>
<tr>
<td>Formation of Houthem (limestone, Paleocene) /</td>
<td>17.6</td>
</tr>
<tr>
<td>Limestone of Meerssen (Upper Cretaceous)</td>
<td></td>
</tr>
</tbody>
</table>

Usually the compacted debris actually forming the floor is about 1 meter thick. The roof and floor level measurements of Van Steveninck (1988) are used here.

#### 8.2.2 Geology

The Geulhemmergroeve is excavated in the upper part of the Limestone of Meerssen. The deeper mined level in the southern part corresponds with layers in the middle of the Limestone of Meerssen and the Koepelgrot has been created in the lowermost level of this lithological unit (Felder, 1979).

The total overburden height increases from the entrance in the northwest to a maximum of 38 m in the southeastern part of the mine (Figs. 8.1a,b). The composition of the overburden layers is difficult to assess. No borehole measurements are available of the layers directly above the mine. By means of data from six borings of the Dutch Geological Survey in the direct vicinity (0.1-1.5 km) the overburden composition was estimated. The borehole numbers and locations are indicated by Vink (1991). The stratigraphy is shown in Fig. 8.2.

The average unit weight of the calcarenite mined in the Geulhemmer Groeve was measured. For the unit weights of the overlying layers the values of the nearby situated Curfs-quarry were taken (Deibel et al., 1987). The moisture content of the calcarenite ranges between 7 and 10 %. The unit weights are indicated in Table 8.1.

Joints, mapped in 1987 by Vreugdenhil, are usually widely spaced, i.e. more than 10 m, but locally clusters occur. Their persistence is a some tens of meters and some joints cross the whole width of the mine. Most joints strike NW-SE, but some are running about the direction NE-SW.

Earthpipes, up to a diameter of 3 m at the mining level, occur frequently throughout the whole mine, mostly likely because the irregular limestone-soil interface is situated only a few meters above the mine roof. They were mapped by Vreugdenhil (1987). At several locations the earth has flowed into the mine. In 1988 an earthpipe in the northwestern part of the mine, below the Meldermunsterweg in Geulhem, collapsed, resulting in a circular depression at the surface of about 6 m diameter and five meter
Fig. 8.3 Area at the eastern boundary of the Geulhemmer Groeve with joints and earthpipes. Also the large sinkhole of 1988 and houses in its direct vicinity are indicated.

Fig. 8.4 The sinkhole, formed in 1988 at the Meldermunsterweg above the Geulhemmer Groeve. The circular crater measured more than six meter diameter and five meter depth (Courtesy W. Misere).
Fig. 8.5 Pillar crack intersecting a written date (1886) on the wall. The location is depicted in Fig. 8.1. Since other inscriptions indicate that this part was mined between 1860 and 1880, the pillar damage must have been developed relatively shortly after mining.

depth (Figs. 8.3 and 8.4). It seems that at the same location a sinkhole had also formed in 1947, Afterwards it was filled up with debris. This illustrates that earthpipes which were not supported from below immediately after mining represent a continuous threat at the surface.

Clay bands of a few mm to more than 10 cm thickness are interlayered with the calcarenite roof strata. Often several superposed clay bands occur. They are a major cause of roof falls throughout the whole mine.
Fig. 8.6 Southern part of the Geulhemmer Groeve with pillar damage and age contours.
8.2.3 Mine history

The development of the Geulhemmer Groeve could be traced rather well. Some old maps from the second half of the 19th century and of the beginning of the 20th century were found indicating the expansion of the mine during parts of that time interval (State Supervision of Mine Archives). Provincial archive data include the concession dates for certain parts of the mine. Throughout the whole mine dates have been written on the pillar walls. If dates are not false they indicate a minimal age. In some cases the inscription was intersected by a later formed pillar crack (Fig. 8.5). Except for the oldest, most visited and most northern part the dates generally seem to represent the year of mining. This is stated on account of the following observations:

- on a large scale, wall dates suppose a decreasing age from the entrance in the north towards the south.

- neighbouring pillars show dates of the same period.

- some dates were accompanied by the name of the miner and a text indicating that mining took place in that year (e.g. "In het jaar 1830 is dezen nieuwen weg gemaakt"). These dates are of the same period as dates in the immediate vicinity.

- dates on the walls correspond with data from old maps and archives.

On the basis of wall dates, archive data and old maps, age-contour lines could be drawn. The southern half of the mine could be dated rather accurately (Fig. 8.6), but the age map of the northern half is much less detailed and precise. This is mainly due to the inaccurate character of some of the old maps, the non-existence of maps or archive data from the period before 1850 and the "contamination" of the older parts by date inscriptions long after mining took place. More precise dating could be possible by a more extensive investigation of archives. Also a study throughout the mine will be useful of the mining methods and mining directions, and of style of the inscriptions on the pillar walls. The map of Fig. 8.6 will be used in Section 8.6 in order to study the relation between pillar deterioration and age.

The severely damaged area in the north (Fig. 8.1a) has not been indicated on a map of 1877. But galleries have been drawn with an "open end" towards the presently encountered damaged area. This probably means that this area was already damaged and considered as too unsafe by the mine surveyors or that the area had not yet been exploited. The last possibility seems unlikely because the area considered is situated near the entrance and must have been created in a early stage of the exploitation of the mine. No maps have been found presenting the blocked off area towards the north.

In 1905 12 portraits of the Dutch royal family were sculpted in a gallery wall in the northeastern part of the mine just near the corridor towards the Koepelgrot. In 1971
Fig. 8.7 Pulpit of the underground church in the Geulhemmer Groeve.

Fig. 8.8 Inscription of names of children which were recently baptized in the underground church.
it was realised that this area and its access had become too unstable and the sculptures were moved to a safer location near the entrance. It is unlikely that this time-consuming art work had been carried out inside an already unstable area. Therefore it is concluded that at least some deterioration of this area occurred during the 20th century, i.e. between 1905 and 1971.

8.2.4 Use of the mine

The mine has served several purposes. In 1798 during the French occupation a church was built (Fig. 8.7). This is still used nowadays at Christmas and attracts many visitors. Children are still baptized in this church (Fig. 8.8). Also during the Second World War people hid in the mine. Several feed troughs near the entrance are evidence of cattle housed in the mine. Many parts have been used for mushroom growing. The supported area near the entrance is visited by tourists, especially at Christmas. One part serves as storage of various coal species. Because of the stable atmospheric conditions in this mine measuring tapes for the coal mines were calibrated. Also seismographs are housed in the Geulhemmergroeve. Until the beginning of this century people lived in primitive houses excavated north of the entrance. Recently a concrete supported house has been made inside the mine near the entrance (Fig. 8.1a).

8.3 CONDITION OF THE MINE

Bekendam and Vink observed that widespread pillar fracturing occurs throughout almost the whole mine and particularly in the northern half. Pillar crack maps covering the whole mine can be found in Vink (1990).

In the northern part even a 50 m wide zone exists where all pillars have been failed. However, this area lacks the characteristics of collapse areas encountered in other calcarenite mines, like debris piles at the perimeter and serious pillar shortening. The galleries bounding the area in the north-east are blocked by earth inflow and roof collapses, indicating a continuation of the past mining activities in this direction. The blocked galleries might represent the boundary of a "true" collapse area. Judging by the hour-glass shape most pillars in the area must have been shortened some centimeters, but no significant lowering of the roof could be observed relative to the roof outside the damaged area. The roof in between the damaged pillars is more or less intact. Probably, due to the relatively small horizontal extent of the damaged area, enough load was transferred towards unmined area in the north and the more or less intact parts of the mine to the west and south to prevent its collapse.

It is striking that pillar fracturing decreases substantially towards the south, where mining occurred later. In the southernmost quarter hardly any pillar damage has been observed. This phenomenon will be studied in Section 8.6. In the area near the entrance, which is of touristic importance, the galleries are spanned by a large number of steel beams placed on concrete pillars. Apart from enhancing local roof
Fig. 8.9 Typical roof fall due to clay-layers in the Geulhemmer Groeve. The thickness of the collapsed roof beam is about 40 cm. The pillar height is about 1.5 m. For location see Fig. 8.1b (Courtesy T. Habets).

stability, this support might have played a role in preventing pillar serious deterioration in this area. However, it should also be noted that the overburden weight is less than further eastward in the mine.

As cited above roof falls are to be seen throughout the whole mine due to the presence of clay bands (Fig. 8.9). In the major part of the mine a threat of new roof falls exists. New roof falls occur now and then and are almost impossible to predict. This is illustrated by two photographs of a gallery in the most southeastern part of the mine (Figs. 8.10 and 8.11). The first photograph of 1980 shows the roof, not yet collapsed but already considered as dangerous, while during the fieldwork of 1990 a roof beam of about 15 cm thickness, detached over a length of 10 m, was seen fallen down on the mine floor.

8.4 MEASUREMENT OF UCS VARIATION

8.4.1 Introduction

Former research on the calcarenites has shown that the geotechnical properties like porosity, UCS and deformation moduli vary considerably per stratigraphic unit. Also
Fig. 8.10 Roof in the southern part of the Geulhemmer Groeve, not yet collapsed but considered as dangerous, photographed in 1980 (Courtesy T. Habets).

Fig. 8.11 The same location as depicted in Fig. 8.10, now photographed in 1990. A roof beam of about 15 cm thickness, detached over a length of 10 meter, had collapsed.
Part III: Observations in the mines and application of results to assess mine stability

Geulhemmer Groeve
Northern part

- Passage blocked by:
- Roof collapse
- Earth inflow

UCS contour line (MPa)

* Sample location with average UCS value (MPa)

0 50 M

Access to Koepelgrot
Access to southern part
b)

Fig. 8.12 Northern (a) and southern (b) part of the Geulhemmer Groeve with measured UCS values. UCS was tentatively contoured by Vink (1991), who assumed that the scale of the variability is comparable with that of the 50*50 m measuring grid.
within one unit strong lateral and vertical variations exist (e.g. Grabandt et al., 1983; Lap et al., 1987; Kronieger, 1989). Since pillar strength is linearly dependent on UCS it is essential to know the UCS and its distribution throughout the mine.

Successive surveys showed that the calcarenite rock properties, including UCS, vary considerably in the Geulhemmer mine. In the first survey of 1987 by Van Steveninck only at five locations in the 10 hectares of mined area samples were taken for UCS tests. In 1988 at 24 locations samples were taken and a mine stability analysis was made using these results (Vreugdenhil, 1988). But it was obvious that 24 sample locations was not enough to delineate the UCS-distribution adequately. Therefore in 1990 a more extensive study was undertaken in the same mine.
Sampling, transport, sample preparation and finally UCS-testing is a laborious and costly process. A quick and inexpensive way of determining the UCS would be useful. Therefore the additional goal is to correlate directly measured UCS values with indirect UCS measurements using a Schmidt hammer (type PT).

8.4.2 Direct measurement of UCS variation

In 1990 block samples were taken with an electric chain-saw at 61 locations by Vink and Verwaal. For this purpose the mine was subdivided into squares of 50*50 m. From each square two block samples of 20-30 cm edge length on the average were extracted. To minimize a possible effect of creep on rock strength the samples were taken from more or less unstressed parts, e.g. pillar projections which do not support the roof. The samples, which were extracted generally from about mid-height of the pillars, represented more or less the same stratigraphic level. They were immediately packed in plastic to prevent drying out. From each block sample 4 cores were taken and tested uniaxially at their natural moisture content of about 6 %. The procedure of core preparation and testing was as described in Section 5.2. From here to the end of Section 8.4, the value for one individual test is denoted as ucs, and the mean ucs value for one block, i.e. one location, is referred to as UCS. The variation of UCS is considerable (2 to 4.5 MPa). The sample locations are indicated in Figs. 8.12 a,b. UCS was tentatively contoured at intervals of 0.5 MPa by Vink (1991). Three UCS-values of block samples were not considered for contouring. One sample showed an excessive standard deviation exceeding one MPa. From two samples only one core could be produced.

It is important to note that, when contouring the UCS values, the assumption was made that the scale of the variability is comparable with that of the 50*50 m grid. However, on the basis of general experience this is not taken to be likely. For example, in the Sibber Groeve sample material was excavated at one location of about 1.8 m high, 1.5 m wide and 1 m deep (see Section 5.2.1). While the standard deviation of the UCS of cores from the same 15*50*40 (height, width and depth respectively) cm block sample was generally just about 0.1 MPa, the mean UCS of different block samples varied significantly, from about 2.5 to 4 MPa. If the UCS shows such a considerable variation within this small volume of calcarenite, it does not seem likely that in the Geulhemmer Groeve the UCS can be assumed more or less constant over the full pillar height of 2.5 to 3 m and at a 50 m horizontal scale.

8.4.3 Measurement of UCS variation by means of a Schmidt hammer type PT

The Schmidt hammer type PT was originally developed for building materials of low hardness and strength (UCS of 0.33 to 5 MPa). Contrary to the better known L- and N-type Schmidt hammers the impact on the rock material is not exercised by a linear plunger movement normal to the rock surface. Instead a plunger of a certain weight rotates over 180° around an horizontal axis before it hits the surface perpendicularly (Fig. 8.13). Its impact of 0.883 Nm is of about the same magnitude as applied by
Fig. 8.14 The tentative correlation between UCS and SHV\(_1\), determined in the Geulhemmer Groeve, for categories 1(■), 2 (●), 3(□) and 4 (○) respectively.

Fig. 8.15 The tentative correlation between UCS and SHV\(_f\), determined in the Geulhemmer Groeve, for categories 1(■), 2 (●), 3(□) and 4 (○) respectively.
the L-type hammer (0.735 Nm), but is distributed over a larger area due to its diameter of 40 mm. The mean rebound value is a measure for the UCS at a certain location. The plunger head of the PT-type hammer has a slightly convex shape, while that of the L-type hammer is completely flat. As a consequence, for the PT-type hammer the contact area between plunger and rock surface increases during subsequent blows at the same spot if the rock surface is soft enough to deform and to accommodate to the shape of the plunger head.

Rebound values were collected at each block sample location on a vertical plane, preferably inside the niche resulting from the sample extraction. Otherwise the test was performed at about 15 cm sidewardly from the sample location. Before the measurement an area of 20 by 20 cm was scraped off to a depth of about 5 mm, to remove the relatively harder superficial crust and the soft layer underneath. Tests had shown that Schmidt hammer values did not change any more at increasing depth beyond 5 mm. By means of a carborundum stone the surface was made smooth and planar. On this area Schmidt hammer rebound values were determined at least 5 spots. The measurement spots were separated at least one plunger diameter. At every spot the first rebound value was recorded (shv). The test was repeated several times at the same spot. The rebound value was observed to increase until a constant value was reached, which was recorded as the final rebound value (shv). The plunger penetrated about 1 mm into the calcarenite and left a circular scar approximating its diameter. The readings of one 20 by 20 cm area were averaged resulting in the mean first rebound value SHV, and the mean final rebound value SHV. Spot readings which deviated excessively, i.e. more than 10 units, from the mean value, for example due to the presence of a shell, were excluded. In this case tests were performed at one or more additional spots. Due to abundant shells, at 5 locations in the mine it was not possible to collect 5 rebound values which fulfilled the requirement above. Accordingly, these locations were not considered in the analysis.

8.4.4. Correlation of Schmidt hammer values with direct UCS measurements

In this section an attempt is made to correlate UCS with SHV values for the 54 remaining data points. UCS and Schmidt hammer PT values are more or less linearly correlated for a wide range of UCS-values, at least for concrete (Proceq SA, 1977). To optimize the correlation a selection was made of the SHV values. The range of the sample mean m (=SHV) ± two times the standard deviation corresponds with about 95 % of the data if a normal distribution is assumed. Now it is required that for a given test location 95 % of the shv values can be expected to range from 0.9 m to 1.1 m. Hence, the requirement is 2 s ≤ 0.1 m, or s/m ≤ 0.05. With respect to SHV, and SHV, this condition was not realized for 12 and 5 of the 54 locations respectively. In the same way the UCS measurements were analyzed.
Fig. 8.16 Histogram of W/H ratio's of the pillars in the southern part of the Geulhemmer Groeve.

Now four categories of data points result (for SHV$_1$ and SHV$_f$ each):

1) \((s/m)_{shv} \leq 0.05\) and \((s/m)_{ucs} \leq 0.05\)
2) \((s/m)_{shv} \leq 0.05\) and \((s/m)_{ucs} > 0.05\)
3) \((s/m)_{shv} > 0.05\) and \((s/m)_{ucs} \leq 0.05\)
4) \((s/m)_{shv} > 0.05\) and \((s/m)_{ucs} > 0.05\)

When applying the correlation, the aim is to estimate UCS from field measurements of SHV in any calcarenite mine. Accordingly, at that occasion only s and m of the shv values, and not of ucs, are known. Thus for the establishment of a correlation of any practical significance only the first two categories of data points must be considered. Accordingly, linear regression was applied only to these data points, for SHV$_1$ and SHV$_f$ respectively. The correlations, for each category separately, between UCS and SHV values are depicted in Figs. 8.14 and 8.15.

The correlation between UCS and SHV$_1$ for data points which fulfil \((s/m)_{shv} \leq 0.05\) is:

\[
UCS = -1.46 + 0.073 \times SHV_1, \text{ with } R^2 = 0.52
\]  
(8.1)

At a 95 % confidence level the error of UCS estimation varies from about -0.82 to +0.82 MPa.
The correlation of UCS and SHV$_f$ is:

\[ UCS = -2.56 + 0.060 \times SHV_f, \text{ with } R^2 = 0.48 \]  \hspace{1cm} (8.2)

At a 95% confidence level the error of UCS estimation varies from about -0.90 to +0.90 MPa. The standard deviation of ucs values, directly measured at a given location is 0.21 MPa on the average. Hence at 95% confidence, the ucs value at a given location has a maximum variation of 0.42 MPa from the mean value UCS. When UCS is determined indirectly from SHV values, not only this natural variation exists, but the error due to correlation must be considered additionally. It must be concluded that the error of UCS estimation from SHV values is too large for the determination of pillar strength on the basis of a SHV measurement.

It would be interesting to study also the relation between Schmidt hammer rebound values and E-moduli. It would be also of interest to examine other correlations like UCS-porosity. For example Denis et al. (1986) observed a excellent fit between UCS and porosity for a comparable, weak limestone of a high porosity, mined to the north of Paris.
Fig. 8.18 Age and initial safety factor for pillars of classes 1, 2 and 3 of the southern part of the Geulhemmer Groeve.

8.5 THE CALCULATION OF SAFETY FACTORS AND THEIR VALIDATION

8.5.1 Calculation of safety factors

The southern part of the mine (Fig. 8.6) was chosen to compare the results of pillar stability calculations with the actually observed pillar condition. Here no support measures had been taken at the time of the investigation, as in the northern part. There are relatively few traces of pillar robbing and a reliable age map exists for a subsequent study of the factor time. Only the pillars which do not form part of the mine perimeter are considered. All these pillars, 114 in total, show W/H ratio's of less than four (Fig. 8.16) and are fairly regularly shaped. Hence Eqs. 6.12 and 6.13 can be applied to determine pillar strength. In Section 8.4.2. it was expressed that the UCS contour lines are considered unreliable. Therefore one mean UCS value was applied for the whole southern part of the mine. For the 34 direct UCS measurements the mean value was 2.85 MPa and the standard deviation 0.66 MPa. Pillar stress was calculated by the tributary area method. Observed pillar condition is described by means of the pillar classification of Fig. 6.50. In Fig. 8.17 pillar stresses and pillar peak strengths are depicted for class 1, class 2 and class 3 pillars. Class 4 pillars did not occur in the area.
8.5.2 Assessment of long-term deterioration of pillar stability

In this section pillar damage, represented in pillar classes, is analyzed as an effect of both initial safety factor $SF_0$ and age. In Chapter 7 it was shown that the creep rate increases with stress/strength ratio, which is equivalent to the initial safety factor. If the stress exceeds the long-term strength failure due to creep finally occurs. The time to failure decreases with the value of the initial safety factor. Hence pillar fracturing is more likely at a low initial safety factor and/or a high age.

The pillar age is assessed according to the following rules:

- to pillars, located between two age contour lines and mined in a certain time interval, the mean age of that mining period is given.

- pillars, intersected by an age contour line, get the mean age of both mean ages of the adjacent areas.

- pillars, mined later than 1921, are dated to 1924 and those, mined after 1924 or 1928, are dated to 1930.

Fig. 8.18 shows a diagram of age and initial safety factor for class 1, 2 and 3 pillars\(^1\). To clarify the picture the plot is subdivided for the different pillar classes (Fig. 8.19). Almost all class 1 pillars have initial safety factors exceeding two. Now it can be observed that their age is less than 110 years, apart from four cases (Fig. 8.19a). Class 2 pillars appear to be confined only by an initial safety factor of less than four (Fig. 8.19b). All data-points for class 3 pillars, except one, are characterized by an initial safety factor of less than three and an age of more than 120 years (Fig. 8.19c). Thus, the data points for class 1 and class 3 pillars are located in different fields which do not overlap. They are separated completely by both initial safety factor and age. Accordingly, from this analysis it cannot be confirmed with certainty whether pillar deterioration in this mine is determined by the ratio of short-term strength and tributary area stress or by creep deformation or both. The variables $SF_0$ and age might even be dependent. In that case, ignoring class 2 pillars for simplicity, pillars which are less than 110 years old, would have a $SF_0$ in excess of 2, and pillars older than 110 years would have initial safety factors of less than 3. Another possibility might be that creep deformation ceased within some years after excavation, with some variation. In that case, about 60 years after the last mining activities, the effect of creep does not vary with age.

It is preferable to estimate long-term safety factors. Such factors are determined here according to the guidelines of Section 7.8. The results are depicted in Fig. 8.20. Nearly all class 1 pillars, no matter their age, have a value of $SF_{lt}$ exceeding one.

\(^1\) In Figs. 8.17 to 8.20 data points are depicted without considering the distributions of the parameters along the X- and Y-axis. A maximum likelihood distribution (e.g. two times the standard distribution of each parameter) could be included, but this would have made the graphs unreadable.
Fig. 8.19 Age and initial safety factor for class 1 (a), class 2 (b) and class 3 (c) pillars separately.
Fig. 8.20 Age and long-term safety factor for class 1 (a), class 2 (b) and class 3 (c) pillars separately.
The majority of class 3 pillars have long-term safety factors of less than 1.2. Again, class 2 pillars are more difficult to classify, in that their values of \( SF_n \) are not well confined within a certain range. Regarding the separation of the effects of safety factor \( SF_n \) and age on pillar condition, the same conclusions must be drawn as for \( SF_0 \).

### 8.5.3 Validation of \( SF_0 \)

The following analysis is comparable to that by Salamon & Munro (1967) and by Sheory et al. (1987) of collapsed and stable room and pillar coal mines. As explained in Section 6.14.3, \( SF_0 \) values are to be corrected in such a way that for the 70 fractured pillars the median of the corrected safety factors \( SF_0' \) is equal to one. This condition is fulfilled by a correction factor \( K_0' \) of 0.488. The frequency distribution proved to be log-normal, i.e. the logarithms of the \( SF_0' \) values are normally distributed (Fig. 8.21), with a standard deviation of 0.131. The corrected safety factors for the 44 intact pillars are log-normally distributed as well. Apart from 6 pillars the \( SF_0' \) values for intact pillars are in excess of one.

According to the correction factor of 0.488, the safety factor \( SF_0 \), based on short-term pillar strength formulae, significantly overestimates pillar stability. The possible reasons for this effect were listed in Section 6.14.3. Here the high value of \( K_0' \) can probably mainly be attributed to creep. The majority of pillars shows W/H ratio’s between 1 and 2.5. According to Section 7.8 pillar strength might be reduced 30 to 60 % by creep deformation for this range of pillar shapes. On the average this will almost double the required initial safety factor. Note that the pillar age varies from about 60 to 140 years. Thus in this analysis \( K_0' \) corrects for an "average" period during which pillars were affected by creep.

In addition, a domino effect may be of some importance. Damaged pillars rarely occur separately (Fig. 8.6). Generally they are to be found in clusters, where pillars fracture partly due to an increased load as a result of fracturing of one or more neighbouring pillars. Finally it should be noted that in the southern part of the mine clay seams occur in the direct roof. As outlined in Section 6.14.2, these might have reduced pillar strength.

The spread of values of \( SF_0' \) about the mean of one is considerable. Taking into account the standard deviation of 0.66 MPa about the mean value of 2.85 MPa for the UCS, it is likely that the variation of UCS has contributed significantly to the variation of \( SF_0' \). If we consider \( SF_0' \) as the product of UCS and a residual term, which represents the remaining parameters, the total variance \( \sigma_{SF,0'}^2 \) can be formulated by rewriting Eq. 6.30 as follows:

\[
\left( \frac{\sigma_{SF,0'}}{SF_0'} \right)^2 = \left( \frac{\sigma_{UCS}}{UCS} \right)^2 + \left( \frac{\sigma_{Res}}{Res} \right)^2
\]  

(8.3)
When a normal distribution of SF\(_{0}'\) is taken as a simplification, the left SF\(_{0}'\) term is 0.12, and that of UCS is 0.05. Thus that of the residual term must be 0.07. Accordingly, the relative variance (σ\(_{SF_{0}}\)^2/SF\(_{0}'\))^2 would be reduced from 0.12 to 0.07 at a reduction of UCS variation to zero.

According to the concept of the validation of safety factors, a SF\(_{0}'\) of one corresponds to a probability of 50 % that a given pillar is undamaged. In Table 8.2 probabilities of pillar intactness are listed for the whole range of SF\(_{0}'\) values. The table comprises probabilities which are directly based on the observed frequency distribution of fractured pillars and probabilities according to the above mentioned log-normal frequency distribution of safety factors. The question arises which value of SF\(_{0}'\) can be considered safe. Salamon (1967) proposed an optimum value which lies in the range where 50 % of the intact pillars are concentrated, i.e. in the range between the 25 % and 75 % quartiles. This range R, from 1.130 to 1.525, is indicated in Fig. 8.21. The log-normal distribution of Table 8.2 suggests that 1.5 represents a safe value of SF\(_{0}'\), in that the probability of pillar fracturing is less than 10 %. This value corresponds to a SF\(_{0}\) of 3.0.

### 8.5.4 Validation of SF\(_{h}\)

A similar analysis was performed for the long-term safety factors SF\(_{h}\). The correction factor K\(_{h}'\) proved to be 0.781. This value, not much less than one, indicates that creep behaviour is probably fairly well incorporated. The more or less log-normal frequency distributions for intact and fractured pillars separately are depicted in Fig. 8.22. Apart from 8 cases, the SF\(_{h}'\) values for intact pillars are greater than one. The range R for intact pillars is represented by SF\(_{h}'\) values between 1.07 and 1.70. According to the log-normal distribution (Table 8.3), a SF\(_{h}'\) value of 1.7 should be safe, with a probability of fracturing of less than 10 %. The standard deviation of 0.178 for log SF\(_{h}'\) is greater than that for log SF\(_{0}'\), which is 0.131. Accordingly, for this mine the use of long-term safety factors SF\(_{h}\) in stead of SF\(_{0}\) did certainly not result in a sharper delineation of actual pillar stability.

### 8.5.5 Validation of SF\(_{0}\) and SF\(_{h}\) excluding class 2 pillars

In Sections 8.5.1 and 8.5.2 it became evident that the safety factors SF\(_{0}\) and SF\(_{h}\) of class 2 pillars are hardly confined within a certain range and much more widely distributed than class 1 and 3 pillars. This is probably due to the characterization of class 2 pillars by a wide range of damage. Some class 2 pillars are considerably fractured at three sides, but the majority shows only minor cracks of one or two decimeters long and without visible aperture. For a sharper delineation of actual pillar stability, safety factors are analyzed for exclusively class 1 and 3 pillars.

Now for SF\(_{0}\) the correction factor, denoted K\(_{0}'\), is 0.562 in stead of 0.488, resulting in a safety factor SF\(_{0}'\). The frequency distributions for 21 fractured pillars of class 3 and 44 intact pillars of class 1 are shown in Fig. 8.23. Although the fractured
Fig. 8.21 Frequency distributions of $SF_{0'}$ for fractured and intact pillars, including the range $R$ of safety factors where 50 % of the intact pillars are concentrated.

Fig. 8.22 Frequency distributions of $SF_{l'}$ for fractured and intact pillars, including the range $R$ of safety factors where 50 % of the intact pillars are concentrated.
Fig. 8.23 Frequency distributions of $SF_{0^*}$ for fractured and intact pillars, including the range $R$ of safety factors where 50% of the intact pillars are concentrated.

Fig. 8.24 Frequency distributions of $SF_{It^*}$ for fractured and intact pillars, including the range $R$ of safety factors where 50% of the intact pillars are concentrated.
### Table 8.2 Probabilities of pillar intactness, from directly observed and from calculated log-normal cumulative distributions of safety factors for fractured pillars. The left half refers to the analysis of SF$_0'$. which includes class 2 pillars. The right half involves SF$_0''$, where class 2 pillars are excluded.

<table>
<thead>
<tr>
<th>SF$_0'$</th>
<th>Probability of intact pillar</th>
<th>SF$_0''$</th>
<th>Probability of intact pillar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Observed</td>
<td>Calculated</td>
<td>Observed</td>
</tr>
<tr>
<td>2.3</td>
<td>1.000</td>
<td>0.997</td>
<td>2.1</td>
</tr>
<tr>
<td>2.2</td>
<td>0.986</td>
<td>0.996</td>
<td>2.0</td>
</tr>
<tr>
<td>2.1</td>
<td>0.986</td>
<td>0.993</td>
<td>1.9</td>
</tr>
<tr>
<td>2.0</td>
<td>0.971</td>
<td>0.989</td>
<td>1.8</td>
</tr>
<tr>
<td>1.9</td>
<td>0.943</td>
<td>0.983</td>
<td>1.7</td>
</tr>
<tr>
<td>1.8</td>
<td>0.943</td>
<td>0.974</td>
<td>1.6</td>
</tr>
<tr>
<td>1.7</td>
<td>0.914</td>
<td>0.955</td>
<td>1.5</td>
</tr>
<tr>
<td>1.6</td>
<td>0.886</td>
<td>0.941</td>
<td>1.4</td>
</tr>
<tr>
<td>1.5</td>
<td>0.871</td>
<td>0.910</td>
<td>1.3</td>
</tr>
<tr>
<td>1.4</td>
<td>0.800</td>
<td>0.876</td>
<td>1.2</td>
</tr>
<tr>
<td>1.3</td>
<td>0.743</td>
<td>0.808</td>
<td>1.1</td>
</tr>
<tr>
<td>1.2</td>
<td>0.643</td>
<td>0.726</td>
<td>1.0</td>
</tr>
<tr>
<td>1.1</td>
<td>0.600</td>
<td>0.626</td>
<td>0.9</td>
</tr>
<tr>
<td>1.0</td>
<td>0.486</td>
<td>0.500</td>
<td>0.8</td>
</tr>
<tr>
<td>0.9</td>
<td>0.386</td>
<td>0.363</td>
<td>0.7</td>
</tr>
<tr>
<td>0.8</td>
<td>0.143</td>
<td>0.230</td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>0.043</td>
<td>0.119</td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>0.029</td>
<td>0.045</td>
<td></td>
</tr>
</tbody>
</table>

Pillars, with a standard deviation of log SF$_0''$ of 0.127, are just slightly more densely distributed than in Fig. 8.21 (the standard deviation of SF$_0'$ is 0.131), the distribution functions for fractured and intact pillars show less overlap if SF$_0''$ is considered than if SF$_0'$ is involved. Apart from just one case, the SF$_0''$ values for intact pillars are in excess of one. The range R of SF$_0''$ values for intact pillars lies between 1.305 and 1.760. SF$_0''$ values of 1.5 and more correspond with a probability of class 3 damage of less than 10% (Table 8.2). This agrees with a SF$_0$ of at least 2.70.
Table 8.3 Probabilities of pillar intactness, from directly observed and from calculated log-normal cumulative distributions of safety factors for fractured pillars. The left half refers to the analysis of SF$_k^{*}$, which includes class 2 pillars. The right half involves SF$_k^{**}$, where class 2 pillars are excluded.

<table>
<thead>
<tr>
<th>SF$_k^{*}$</th>
<th>Probability of intact pillar</th>
<th>SF$_k^{**}$</th>
<th>Probability of intact pillar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Observed</td>
<td>Calculated</td>
<td>Observed</td>
</tr>
<tr>
<td>3.0</td>
<td>1.000</td>
<td>0.996</td>
<td>2.5</td>
</tr>
<tr>
<td>2.9</td>
<td>0.986</td>
<td>0.995</td>
<td>2.4</td>
</tr>
<tr>
<td>2.8</td>
<td>0.986</td>
<td>0.994</td>
<td>2.3</td>
</tr>
<tr>
<td>2.7</td>
<td>0.986</td>
<td>0.992</td>
<td>2.2</td>
</tr>
<tr>
<td>2.6</td>
<td>0.971</td>
<td>0.990</td>
<td>2.1</td>
</tr>
<tr>
<td>2.5</td>
<td>0.971</td>
<td>0.988</td>
<td>2.0</td>
</tr>
<tr>
<td>2.4</td>
<td>0.943</td>
<td>0.984</td>
<td>1.9</td>
</tr>
<tr>
<td>2.3</td>
<td>0.943</td>
<td>0.979</td>
<td>1.8</td>
</tr>
<tr>
<td>2.2</td>
<td>0.943</td>
<td>0.973</td>
<td>1.7</td>
</tr>
<tr>
<td>2.1</td>
<td>0.943</td>
<td>0.965</td>
<td>1.6</td>
</tr>
<tr>
<td>2.0</td>
<td>0.929</td>
<td>0.955</td>
<td>1.5</td>
</tr>
<tr>
<td>1.9</td>
<td>0.929</td>
<td>0.942</td>
<td>1.4</td>
</tr>
<tr>
<td>1.8</td>
<td>0.914</td>
<td>0.923</td>
<td>1.3</td>
</tr>
<tr>
<td>1.7</td>
<td>0.871</td>
<td>0.902</td>
<td>1.2</td>
</tr>
<tr>
<td>1.6</td>
<td>0.829</td>
<td>0.875</td>
<td>1.1</td>
</tr>
<tr>
<td>1.5</td>
<td>0.800</td>
<td>0.839</td>
<td>1.0</td>
</tr>
<tr>
<td>1.4</td>
<td>0.757</td>
<td>0.794</td>
<td>0.9</td>
</tr>
<tr>
<td>1.3</td>
<td>0.657</td>
<td>0.739</td>
<td>0.8</td>
</tr>
<tr>
<td>1.2</td>
<td>0.657</td>
<td>0.670</td>
<td>0.7</td>
</tr>
<tr>
<td>1.1</td>
<td>0.629</td>
<td>0.591</td>
<td>0.6</td>
</tr>
<tr>
<td>1.0</td>
<td>0.500</td>
<td>0.500</td>
<td></td>
</tr>
<tr>
<td>0.9</td>
<td>0.400</td>
<td>0.397</td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>0.314</td>
<td>0.295</td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>0.129</td>
<td>0.191</td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>0.057</td>
<td>0.106</td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.029</td>
<td>0.045</td>
<td></td>
</tr>
</tbody>
</table>
Table 8.4 Correction factors, established for the Geulhemmer Groeve, for safety factors $SF_0$ and $SF_\lambda$, and safe values of safety factors, referring to a probability of pillar fracturing of less than 10%. The ' and " symbols refer to analyses with and without class 2 pillars respectively. Note that all corrected safety factors incorporate creep deformation.

<table>
<thead>
<tr>
<th>Correction factor</th>
<th>Corrected safe SF value</th>
<th>Safe SF value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$SF_0' = K_0' \times SF_0$</td>
<td>$K_0' = 0.488$</td>
<td>$SF_0' = 1.5$</td>
</tr>
<tr>
<td>$SF_\lambda = K_\lambda' \times SF_\lambda$</td>
<td>$K_\lambda' = 0.781$</td>
<td>$SF_\lambda' = 1.7$</td>
</tr>
<tr>
<td>$SF_0&quot; = K_0&quot; \times SF_0$</td>
<td>$K_0&quot; = 0.562$</td>
<td>$SF_0&quot; = 1.5$</td>
</tr>
<tr>
<td>$SF_\lambda&quot; = K_\lambda&quot; \times SF_\lambda$</td>
<td>$K_\lambda&quot; = 0.943$</td>
<td>$SF_\lambda&quot; = 1.6$</td>
</tr>
</tbody>
</table>

The same analysis was performed for $SF_\lambda$. Safety factors $SF_\lambda"$ were achieved using a correction factor $K_\lambda"$ of 0.943. The standard deviation of $\log SF_\lambda$ is 0.173, which is comparable to that of $\log SF_\lambda'$, which is 0.178. However, again the distribution functions for fractured and intact pillars show less overlap if class 2 pillars are excluded (Fig. 8.24). Apart from 2 cases, the $SF_\lambda"$ values for intact pillars are in excess of one. The range $R$ for intact pillars is represented by $SF_\lambda"$ values between 1.285 and 2.055. Table 8.3 shows that the probability of class 3 damage is less than 10% for $SF_\lambda"$ values in excess of 1.6, which corresponds with $SF_\lambda"$ values of minimally 1.7.

8.5.6 Conclusions and closing remarks

The results of the analyses of the preceding sections are summarized in Table 8.4. The factors $K$ correct for systematic errors of the calculation method, which apparently resulted in an overestimation of safety factors. Additionally, "safe" values of safety factors are given, which are in excess of one due to random errors of the calculation method. Safe corresponds here to a probability of pillar fracturing of less than 10%.

The same analyses were performed for UCS values of pillars according to the contour map of Fig. 8.12b, instead for one average UCS value. In all cases the spread of safety factors about one was exceeding that for the analysis which used the uniform UCS value of 2.85. This suggests that the UCS contour-lines are indeed not reliable.

It should be noted that the analyses refer to the conditions which are characteristic for the Geulhemmer Groeve. The $K$-values and safe SF-values cannot be applied to any given mine system. In Section 7.8 it was remarked that $K_0'$ cannot be used in other mines if their pillar shapes are different in general, and that it is preferable to use $SF_\lambda$ and $K_\lambda'$. The validation of safety factors will be improved when more mines
are analyzed. The influence of the individual systematic errors on the K-values can be estimated better if the random errors are minimal, and if one or more systematic error sources are more or less absent. In the ideal case for each of the selected mines a different systematic error should be eliminated. To minimalize random errors, the mine systems to be analyzed should satisfy the following conditions:

- the mine consists of pillars of regular shapes.
- the variation of UCS is limited.
- pillars are of more or less the same age.
- the variation of pillar shape is limited.

Some systematic errors can be more or less eliminated if for one or more mines:

- the thicknesses and unit weights of the overburden units are well known.
- the mine is horizontally extensive, to eliminate effects of arching towards the abutments.

The analysis can be further improved when not a number of individual pillars forming part of one mine system are considered, but a number of intact and collapsed/completely damaged mines or parts of mines, which each comprise pillars of the same shape and UCS. In that case load redistribution between pillars of different shapes and different creep rates according to shape do not complicate the results. Such an analysis was performed by Salamon & Munro (1967). They found safety factors for collapsed cases which were considerably more densely distributed around a value of one than observed in the Geulhemmer Groeve.
CHAPTER 9

THE HEIDEGROEVE

9.1 INTRODUCTION

The Heidegroeve is of particular interest because here the most recent large-scale pillar collapse occurred, in June 1988. An account of the collapse, based on the work of Price (1990) and co-workers, is presented in Section 3.5.8. Since then mine surveys have been continued regularly until today to monitor the condition of the still accessible parts. The pillar condition and collapse geometry was recorded in more detail in the spring of 1992. This chapter gives more details about the characteristics of the mine, including the collapse area, and deals with new field data. This makes up the basis of an attempt to assess pillar stability and the factors which gave rise to the collapse.

9.2 CHARACTERISTICS OF THE MINE

9.2.1 Mine geometry

The Heidegroeve is situated on the southwestern side of Valkenburg (Fig. 2.7). The entrance, recently walled up because of safety reasons, is located at the Plenkertstraat. Now the mine can only be entered through the adjacent mine system known as the "Katakomben". The mine, maximally 100 m wide, extends, under a rising topography, some 300 m to the southeast of the entrance passing beneath the wooded area, known as the Polferbos (Fig. 9.1).

The map of the mine, made by the Germans during the Second World War, proved to be accurate to ± 10 cm. The majority of the pillars is more or less rectangular and their horizontal dimensions commonly vary from 4 to 12 m. In most areas of the mine the pillar height is about 2 m, but in the central part the miners chose to extract also about 2.6 m of the overlying strata. In this way approximately 4.6 m high pillars were created (Figs. 2.8 and 9.1). Locally only the higher level has been mined and in some places the hardground in between the two levels was not excavated resulting in two storeys separated by the hardground. The galleries are usually 3.5 to 4 m wide. The floor level, slowly rising towards the southeast, had
Fig. 9.1 The Heidegroeve with locations of figures and UCS-determination and contours of mine floor and ground surface.
been measured by the Germans. The galleries are not filled with calcarenite waste material.

9.2.2 Geology

According to Felder (1979) the Heidegroeve has been excavated in the lower level of the Limestone of Meerssen. This is in accordance with borehole observations (RGD, Rijks Geologische Dienst) 600 m to the southeast of the entrance of the Heidegroeve. At about 68 m NAP\(^1\) the first flintlayers were found, which must represent the Limestone of Emael. The Horizont of Laumont will then be situated at about 70 m NAP. In the Heidegroeve the Maastrichtian rocks dip towards the northwest, approximately 8 m over a distance of 300 m. This means that the Horizont of Laumont is situated at about 53 m NAP at the entrance of the Heidegroeve. The thickness of the Limestone of Nekum is about 12 m near

\(^1\) NAP means "Nieuw Amsterdams Peil", the Dutch reference sea level.
Table 9.1 Unit weights of overburden materials for the Heidegroeve.

<table>
<thead>
<tr>
<th>Formation</th>
<th>Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maas Deposits (gravel, Pleistocene)</td>
<td>18.7</td>
</tr>
<tr>
<td>Formation of Tongeren (sands, Lower Oligocene)</td>
<td>17.3</td>
</tr>
<tr>
<td>Limestone of Meerssen (Upper Cretaceous)</td>
<td>17.6</td>
</tr>
</tbody>
</table>

Valkenburg (Felder, 1979) and hence the Horizont of Caster, the base of the Limestone of Meerssen, will be situated at about 70 m NAP. This height corresponds well with that of the mine floor at the entrance of the Heidegroeve.

Observations from boreholes and surface outcrops (RGD) indicate that the erosional upper surface of the Maastrichtian Limestone does not show much variation in height above sea level on a 100 meter scale to the southwest of Valkenburg. The top of the Maastrichtian Limestone, i.e. the Limestone of Meerssen, is estimated to be situated at 100.5 m NAP.

The geological map of Southern Limburg (RGD) and surface outcrops demonstrate that Pleistocene Maas deposits only cover the southernmost part of the Heidegroeve. Here the base of these sediments is lying at about 125 m NAP. The Maas deposits are capped by up to one meter of loess. The latter deposits have not been involved in the calculations as a separate overburden unit because of their relatively insignificant thickness. In stead it was assumed that the Maas deposits continue up to the topographic surface.

The stratigraphy of the Heidegroeve area is outlined in Fig. 9.2. The overburden consists of Limestone of Meerssen, Tongeren Sands and in the southernmost part also of Maas deposits. Their unit weights are shown in Table 9.1. The total overburden height increases from 4 m near the entrance to 51 m in the southernmost extremity of the mine.

Joints rarely occur and are mainly confined to the NNE part of the mine near the access to the katakomben (Fig. 9.3). Their strike is predominantly W-E to WNW-ESE, i.e. more or less parallel to the slope of the topography. The dip ranges from subvertical to 60° towards the north. Earth pipes have not been found.

9.2.3 History and use of the mine

The history of the Heidegroeve has been outlined by Habets (1988). In the beginning of the nineteenth century mining started with the exploitation of the mine system later known as the "Katakomben" (the catacombs), mainly for the purpose of
Fig. 9.3 The Heidegroeve with natural discontinuities and pillar cracks.
creating a depot for gunpowder fabricated in the factory at the other side of the Plenkertstraat. After the closure of the gunpowder factory mining continued, despite the moderate building stone quality of the calcarenite. The mine workings later known as the Heidegroeve were initially excavated from the entrance galleries of the present Katakomben. Round 1900 a part of the mine was separated to create an imitation of the Roman catacombs which still serve as a tourist attraction. A new entrance was made to the west of the existing one (Fig. 9.1) to continue with the exploitation of the Heidegroeve. During the 1920’s mining activities decreased significantly because the overburden stress in the southern part was such that the steel cutting saws became trapped in the saw cut, which closed on the saw. After the end of the calcarenite exploitation in the 1930’s the mine was hired for mushroom growing.

In 1943 the Germans made a survey of the mine. In 1944 they converted the mine into a bomb proof factory for the production of radio components for aeroplanes (Silvertant, 1992). Many pillar corners were rounded off but reinforced with brickwork. Some galleries were raised and pillar walls were covered by a layer of water-glass, which unfortunately obscured nearly all previous inscriptions. A comparison between the mine plan produced in 1943, prior to their designing the underground factory, and a modern plan shows that additional openings were made by them, reducing pillar size and safety (Fig. 9.4). Even one whole pillar was excavated (pillar 89) and only replaced by brick walls at its former outline. The factory was just operating when in September 1944 the town of Valkenburg was liberated by the Allies.

After a short period of use by the American Army the mine was used for mushroom growing for some years, interrupted by a brief attempt at breeding worms for fishing bait. At least up to the seventies the mine appeared to be in good condition and had the reputation of considerable stability. But then pillar cracking started. It is difficult to trace exactly when deterioration set in. During the summer of 1987 a significant further deterioration of the Heidegroeve was observed and reported by the State Inspectorate of Mines. Particularly the pillars adjacent to the future collapse area were damaged. Photographs of Silvertant, taken throughout the year 1987, do not show significant pillar damage in the area to collapse. In the summer of 1987 the mine was closed and abandoned. As reported above, one year later the southern part of the mine collapsed. In 1989 the entrance of the Heidegroeve was walled up. The only way to enter the mine for inspection is through a narrow gap from the Katakomben (Fig. 9.1).

9.3 THE PRESENT CONDITION OF THE MINE

9.3.1. Individual pillars

An extensive investigation was carried out in April 1992. Every individual pillar crack and surface spall was mapped (Fig. 9.3). In between the collapsed area in the southeast and the intact northwestern part of the mine a transition zone of about 50
Fig. 9.4 The Heidegroeve with the additional excavations made by the Germans during the Second World War.
m width exists, in which pillar damage decreases gradually towards the entrance. Particularly in the 4.6 m high workings severe pillar deterioration has been encountered. Several pillars could be classified as failed. Often the brick cladding has parted from the pillar walls and fallen on the floor.

On 13 april 1992 an earthquake occurred of magnitude 5.7 (Richter scale; Fig. 3.15) near Roermond about 40 km to the north. During inspections in several calcarenite mines in May 1992, the Heidegroeve proved to be the only one which was affected, albeit to a small extent. Locally the separation between a pillar core and an already formed slab had increased up to about 20 cm and sometimes a slab had fallen down on the mine floor. In agreement with general mining practice no additional pillar cracking had developed. At two localities the pile of rock fragments at the boundary of the collapse area had changed and increased in height. As a consequence the entrance to the collapse area is now blocked off completely.

9.3.2 Characterization of the collapsed area

In may 1993 the underground area of collapse was examined into more detail. Fig. 9.5 shows a cross-section of the boundary of the collapse, as observed near pillar 52 (for location see Fig. 9.1). This situation is also characteristic for the rest of collapse
boundary. Approaching from the more or less intact area a pile was encountered existing of calcarenite rock debris (Fig. 3.20). The slope, i.e. the angle of repose, is 40° to 50°. The rock fragments are generally angular in shape and up to several meters in size. The rock debris block the passage into the area of collapse. But during the years immediately after the collapse this area had appeared accessible. It was the 1992 earthquake which gave rise to a settlement of the rock fragments and the access was completely blocked off since then.

The collapse is clearly bounded by a steep fault, which is not of tectonic origin. The collapse induced fault is dipping at an angle of 70° to 80° towards the "intact" part of the mine. The direction of dip is always parallel to the perimeter of the collapse. Standing on top of the debris pile, the fault could be observed up to more than 10
m above the original roof level, till it was obscured due to small undulations of the fault and lack of illumination. The fault orientation seems to be unchanged towards higher levels. It may assumed that the fault maintains the observed orientation up to the rock-soil interface. The fault surfaces appeared fresh and there was not a single evidence that existing joint surfaces had been utilized for the fault movement. As a matter of fact, no joints are to be observed in this part of the mine. No infill of soil was encountered, probably due to the considerable calcarenite overburden thickness of about 20 to 25 m. The fault showed an opening of up to some decimeters.

In most cases an arch was formed by the collapse of the roof at the basis of the fault. The height of the arch was up to 8 m. The collapse at the side of the hanging wall\(^2\) was generally higher than at the side of the foot wall. Often roof fragments of several meters in size fell down. According to the position of shell layers, these blocks moved over a distance of some meters relative to the dome surface at the "intact" side (Fig. 9.6). Due to the chaotic situation this offset often appears as the vertical movement of the main roof rock mass over the area of collapse, but a closer inspection reveals that this is not true.

Apart from the roof falls at the basis of the fault, both hanging wall and footwall were unbroken and massive. Therefore the complete rock overburden is considered to have collapsed as one continuous rock mass.

The actual downward displacement of the caved calcarenite overburden could not be assessed directly because the footwall was obscured by rock debris. However, the

\(^2\) Hanging wall and footwall are the rock masses immediately above and below the fault respectively.
downward displacement can be estimated indirectly. The vertical offset of the normal faults at the surface attained values of 80 cm (Fig. 3.17). Although these are accompanied by antithetic faults with a smaller vertical offset, it can be assumed that surface subsidence over the underground collapse is generally several decimeters. This amount of ground movement is considered as a minimal estimation of the displacement at the mine level. The maximum downward displacement of the calcarenite overburden can be derived from the extraction ratio, which is about 60 %. For an original gallery height of 2 m inside the area of collapse (unpublished photographs of Silvertant) the displacement is maximally 1.2 m. Considering also a bulking effect (see also Section 10.3.2) and the lateral pillar support of rock debris, the downward displacement will certainly not have exceeded one meter. An impression of the transition into the area of collapse is presented in Fig. 9.7.

At one locality at the collapse boundary (Fig. 9.1) not only the steep fault in the overburden was to be seen, but also a fault of similar dip, but sloping towards the collapse, in the floor (Fig. 9.8). The fault could be distinguished over a horizontal distance of several meters and down to a depth of more than five meters below the mine floor. The fault appeared fresh and showed a constant slope, like the faults extending into the roof. It is possible that such a fault, obscured by rock debris, is present at more locations of the collapse perimeter.
Fig. 9.9 Ground movements over the Heidegroove. Solid arrows indicate directions of ground movement. At the upper side the shape of the subsidence and horizontal strain profiles are depicted.
Fig. 9.10 Stress and initial strength for pillars of class 1 to 4 for the complete Heidegroeve just prior to the collapse. Class 3 to 4 is attributed to the pillars in the collapsed area.

Fig. 9.11 Stress and long-term strength for pillars of class 1 to 4 for the complete Heidegroeve just prior to the collapse. Class 3 to 4 is attributed to the pillars in the collapsed area.
9.4 ANALYSIS OF GROUND MOVEMENTS ASSOCIATED WITH THE COLLAPSE

9.4.1 Movements within the calcarenite

The observations in the mine indicate that the caved calcarenite rock mass was not broken up but remained more or less continuous. The rock mass is bounded by faults which continue up to the base of the overlying soil units, maintaining about the same orientation as observed at the mined level (Fig. 9.9).

Mohr-Coulomb theory predicts eventual shear fracturing at an angle of $45^\circ - \phi/2$, which is about $30^\circ$ for the calcarenite, with the arching stress. A fault, as observed here over the Heidegroeve, finally developed as a result of shear stress in combination with an important tensile stress due to overburden weight.

The fault was generated at the mining level, where stress concentrations were maximal. Thence fast upward propagation occurred, promoted by a decrease of contact area with the hanging wall and the weight of the caving rock mass. The angle of the fault plane with the horizontal of about $70^\circ$ is here referred to as the cave angle. Often also the terms angle of break (Whittaker & Reddish, 1990) or Bruchwinkel (Fenk, 1976) are used. But, since these expressions also form part of the terminology of trough subsidence (see below), the term cave angle is preferred.

The fault observed in the floor, of unknown extent, could be explained by the existence of comparable stress concentrations at the lower edges of the collapsed mine system. A comparable geometry of faulting in both roof and floor was described by Knoll & Kuhnt (1991) in their analysis of rock bursts. But such fault movement would be against the action of gravity, according to the direction of shear stress, and is not to be expected at this shallow depth. Therefore the fault in the floor must be ascribed to unique conditions, e.g. an underlying mine opening. This phenomenon is studied in more detail in Section 9.7.

9.4.2 Movements within the soil formations

Due to the rock movement a depression was suddenly formed at the base of the soil units. The soils, obviously of a much lower stiffness and cohesion, reacted in a different way as the calcarenite. The movement within the soil mass can be described more appropriately as flow, as opposed to the brittle response of the underlying rock. A classical subsidence trough tended to be formed, limited by an angle of draw of about $45^\circ$. Here the supra-critical case applies. This means that the width of the underground opening in relation to its depth was such that the full potential subsidence developed (e.g. Kratzsch, 1983; Whittaker & Reddish, 1989).

Such a subsidence area is bounded by a zone of horizontal extension and a zone of shortening, separated approximately by the perimeter of the depression at the base of the soil. The location of the maximum horizontal extensional strain is given by the angle of break (Fig. 9.9), which is expected to be about $70^\circ$. The values of the
Fig. 9.12 The Heidegroove with initial pillar safety factors.

Limit of underground collapse

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Entrance from Katakomben

Collapse area
angles of draw and break are estimated on the basis of subsidence observed over coal mines in South Limburg and over salt solution cavities in Twenthe (Bekendam, 1996). However, the normal subsidence trough was interrupted by cracks at the zone of maximal tension (Figs. 3.18 and 9.9). This tension was relatively high because the depression at the base of the soil was considerable and suddenly formed. As often observed in various subsidence cases (e.g. Kratsch; 1983, p. 175), an antithetic fault developed in the direction of the centre of the subsidence area. In between the faults a trench was formed. The existence of such an antithetic fault is attributed by the curvature of the main fault which induced a loss of lateral support within the hanging wall. It is not known if the main fault continues down to the calcarenite. Since a height difference of one or more decimeters existed between both sides of the fracture zone, downward and sideward soil movement was certainly concentrated along the faults, and subsidence in the zone outside the angle of break was strongly reduced. The horizontal shortening of the fence (Fig. 3.19) can be explained by its location within the zone of compression.

Here the area of surface subsidence is indicated by the angle of draw, applied at the base of the soil mass. In a more conservative approach the line of draw starts at the mining level (Fig. 9.9). For the calcarenite an angle of draw of 35° is used.

9.5 ESTIMATION OF UCS

Only at one location and with some difficulty a sample could be taken. The UCS was 2.7 MPa. Indirect measurements by Schmidt hammer on five locations (Fig. 9.1) throughout the mine, performed according to the method presented in the previous chapter, showed little variation in UCS. The final SHV values ranged from 86.3 to 92.5, corresponding to UCS values of 2.62 and 3.0 Mpa respectively, according to Eq. 8.2, with an average of 2.76 MPa. It was decided to use an UCS value of 2.75 Mpa for the whole mine.

9.6 PILLAR STABILITY ACCORDING TO STRENGTH FORMULAE

For all pillars, including those in the collapsed area, both initial and long-term strength were determined by means of Eqs. 6.12 and 6.13 respectively. Pillar stress was calculated by the tributary area method. The modest size reductions at some pillars by the Germans were incorporated. The calculated safety factors were compared with the pillar condition just before the collapse. According to the reports of the State Supervision of Mines the damage to the pillars, visible at present, in the not collapsed part of the mine had already developed, at least for the major part, before the collapse. All pillars inside the collapse area are tentatively considered to be fractured (class 3 to 4). It seems that otherwise a collapse would not have occurred here.

Pillar stresses and initial pillar strength values are depicted in Fig. 9.10. Similarly as the pillars in the Geulhemmer Groeve, class 1 pillars generally show initial safety
Fig. 9.13 Stress and initial strength for pillars of class 1 to 4 for the Heidegroeve, except the collapsed area, just prior to the collapse.

Fig. 9.14 Stress and long-term strength for pillars of class 1 to 4 for the Heidegroeve, except the collapsed area, just prior to the collapse.
factors exceeding two. Most class 2 pillars have safety factors between one and three. Two pillars near the northern perimeter of the mine, just outside the collapse area, show too low safety factors for class 1. Probably damage was prevented due to load transfer towards the abutments in their direct vicinity and the absence of nearby heavily damaged pillars. But the more severely damaged class 3 and 4 pillars are not characterized by low values of SF₀. On the contrary, most of these have safety factors exceeding two, i.e. within the same range as class 1 pillars.

Regarding long-term pillar strength (Fig. 9.11) the class 1 pillars usually show values of SF₀ of more than one, as in the Geulhemmer Groeve. But also here the anomaly is to be observed, in that the majority of class 3 and 4 pillars have long-term safety factors within the same range as intact pillars.

In order to find an explanation for the deviating safety factors of the damaged pillars, the initial safety factors SF₀ for each pillar are depicted on the mine map (Fig. 9.12). Remarkably, the pillars with the lowest SF₀ are located outside the present collapse area. With the exception of one pillar at the southeastern extremity of the mine, all pillars of an initial safety factor of less than 1.5 are located outside the collapse area. Apart from one case, all these pillars are concentrated in the two storey workings which are still standing. Nearly all pillars inside the collapse area show safety factors of more than two. Apparently the relationship between safety factor and pillar class is abnormal at least for the pillars inside the collapse area.

Therefore an additional analysis was performed, at this occasion only for the pillars still standing, while the class 3 pillars inside the collapse area are excluded. Pillar stress is depicted versus initial and long-term pillar strength respectively in Figs. 9.13 and 9.14. Now the graphs appear "normal" and comparable to the results of the Geulhemmer Groeve and other mines: all class 3 pillars have initial safety factors of less than three, and most of those less than two, while the majority of class 1 pillars is characterized by SF₀ values of more than two. Nearly all class 3 pillars have long-term safety factors of less than two, and a considerable number of those of less than one. Most class 1 pillars have long-term safety factors exceeding two. Thus in Figs. 9.13 and 9.14 safety factors for heavily damaged and intact pillars are now concentrated in separate ranges, according to expectation.

Because of the direct vicinity of the collapse and the particular circumstances of this event, to be explained in the next section, it was decided not to validate safety factors.

9.7 EXPLANATION FOR THE COLLAPSE

The results of the previous section indicate that a relatively strong part of the mine deteriorated and collapsed, while the weakest part has also been fractured but is still standing. Considering pillar strength the collapse could be expected to have initiated in the two storey workings. Here pillar strength and safety factors were minimal. Additionally, the post-peak pillar load-convergence slopes are relatively steep,
Fig. 9.15 Perimeters of the Heidegroeve and the northwestern part of the Gemeentegrot, as known from existing mine plans. Also the collapse area of the Heidegroeve and the approximate boundary of the Gemeentegrot collapse are indicated. The unexpected collapse of the Heidegroeve can be explained by an extension of unstable mine workings of the Gemeentegrot, mined at a lower level, beneath the Heidegroeve.

according to their low width/height ratio's. The total mine span is maximal at this part and mine stiffness is minimal accordingly. Hence, here the relationship between pillar stiffness and mine stiffness is the most favourable for a large-scale collapse. It must be concluded that an additional, significant factor must have been involved.

The only explanation which agrees well with the observations in the mine is that the collapse of the Heidegroeve was triggered by the collapse of another mine system at a lower level. Ample evidence exists that the presence of such a mine system is possible. It is known that at the perimeter of the nearby Gemeentegrot to the east several unmapped mine workings exist which are collapsed for an important part
Fig. 9.16 Increased pressure in the upper working due to abutment pressure of the lower working (a) and redistribution of pressure due to deflection (b).

(Fig. 9.15). For example, such mines systems were detected in 1995 during the site investigation for the erection of a new casino. Less than 50 m east of the Heidegroeve collapse, a series of steep depressions, known as the "Heksenkeuken" (kitchen of witches), are to be found at the surface. These were probably formed as a result of the collapse of an ancient mine system underneath. From the open-air theatre, located in one of these depressions, several galleries have been observed which are blocked off by debris or come to a dead end. Also in the area between the Heksenkeuken and the Katakomben there are signs of calccrenite mining, both underground and in open quarries (Silvertant, 1991). Even a connection with the nearby Gemeentegrot is reputed to have existed. The Gemeentegrot was excavated in the lower level of the Limestone of Nekum. This is five meters below the lower level of the Limestone of Meerssen, in which the Heidegroeve and the Katakomben were created (Fig. 9.2).

The presence of a second mine which extends to the present limit of collapse explains the unexpected occurrence of a collapse in a part of the Heidegroeve which was relatively stable and not significantly deteriorated just prior to the event. It also offers an explanation as to why the collapse was preceded by severe pillar deterioration particularly at its perimeter. It is generally known (e.g. Kratsch, 1983; p. 130) that the pressure in the upper working will increase due to the abutment
pressure of the lower working (Fig. 9.16a). Additionally, the strata between both workings will subside more strongly over the centre of the failing and collapsing lower working that at its edges. This effect creates an extra increase of pressure at the pillars of the upper working above the abutment of the lower mine system (Fig. 9.16b). A series of borings from the surface could indicate the presence of such a second collapsed mine working.

If a lower collapsed mine system indeed exists the downward displacement of the overburden over the Heidegroeve may be greater than estimated in Section 9.3.2. The observed fault in the floor of the Heidegroeve would certainly extend down to the lower system, but does not allow much displacement due to its geometry. In that case the rock mass between both levels moved downward by deflection or along additional faults. Such a deflection might have created the observed fault opening. The orientation of the fault suggests that it originated as a shear fault at the edge of the upper working. If the fault had grown from the lower working its direction of dip should have been the opposite, like the fault in the roof of the Heidegroeve.

It can be concluded that the possibility of additional, inaccessible mine systems at a lower level creates an extra hazard, because their deterioration cannot be seen. Accordingly, their collapse may give rise to an unexpected collapse of the upper working, which was previously considered stable.
CHAPTER 10

THE FALLENBERG

10.1 INTRODUCTION

The Fallenberg suffered two large-scale collapses, in 1705 and 1920, which destroyed more than half of the mine system. Brief accounts of these collapses are presented in Sections 3.5.1 and 3.5.5. This mine was selected to study the still accessible collapsed area of 1920 and to perform a pillar stability analysis of the still intact part.

The Fallenberg is one of the calcarenite mine workings excavated in the Cannerberg, to the south of Maastricht (Fig. 2.7). The total mined area is about 16 hectares, but as a result of the collapses 10 hectares were ruined, leaving about 4 hectares in the south, about 2 hectares in the north and about 0.3 hectare in the east near the original entrance (Fig. 3.8). This original entrance, located at the Fallenbergweg, has been sealed. In the southeast the Fallenberg adjoins the mine system of the Boschberg, which covers about 8 hectares. The Boschberg was used as a NATO command post from 1954 to 1992. In the relatively intact southwestern part of the Fallenberg several paintings, drawings and sculptures, made by Jesuits, are to be seen. This part of about 4 hectares is better known as the Jezuïetenberg (Fig. 10.1). The present entrance to the Jezuïetenberg dates from 1954. The entrance galleries are situated near the boundary between the Fallenberg and the Boschberg and are separated from the main part of the Boschberg by artificial walls.

In the winter and spring of 1990 a fieldwork was performed by Bekendam and Orlic. The latter used the acquired data for his MSc-thesis (Orlic, 1990). The whole Jesuitenberg was studied, from the limits of the collapsed area in the north to the boundary with the Boschberg. From 1991 to 1993 some excursions were undertaken by the author to study the area of collapse and its boundary. The temperature and air moisture content at various locations inside the Jezuïetenberg, measured by Bacquerra (unpublished data) from April 1991 to February 1992, proved to be 9-11°C and more than 90% respectively.
Fig. 10.1 The Jezuïetenberg with locations of UCS determination and isopach contour lines.
10.2 CHARACTERISTICS OF THE MINE

10.2.1 Mine geometry

The mine has been mapped at the end of the 19th century by a Jesuit, who performed this task with dedication but lacked professional surveying skills. The resulting map proved not to come up to the requirements of accuracy. Pillar outlines were measured more precisely during the fieldwork. The corrected map is accurate to about 0.5 m (Fig. 10.1). This accuracy is still insufficient according the requirements stated above. Yet, because the horizontal dimensions of the pillars are relatively large, it was decided that the map could be used for a stability analysis. At the end of 1996 a new, more precise map was completed by Stevenhagen, which will be used in the near future to increase the accuracy of stability assessments.

The mine system of the Jeuzuïtenberg shows a more or less radial pattern. Most pillars are almost rectangular in shape with horizontal dimensions typically ranging from 6 to 12 m and an average width/length ratio of about 0.75. Their present height commonly varies from 2.5 to 3.6 m with an average value of 2.9 m, but has been much larger initially. This can be seen in the entrance galleries of the mine where pillar heights reach almost 6 m. These differences in pillar heights result from varying thicknesses of waste material dumped on the mine floor. However, this
material was well compacted in the course of time, and the pillar height presently
to be observed is used for calculations. The present width/height ratio of the pillars
usually varies from 2 to 4. The extraction ratio is about 49 %.

10.2.2 Geology

On the basis of literature (Felder, 1981) and observations in the mine it is concluded
that the Jezuïetenberg has been excavated in the upper level of the Limestone of
Nekum (Fig. 10.2). The roof is formed by the hardground of the Caster Horizont.
In deeper parts of the mine the flint layers just below the Kanne horizont are to be
seen.

Within the Jezuïetenberg the bedding is dipping about 1 degree towards the NNW.
The overburden height is fairly constant, ranging from 36 m near the entrance to
more than 41 m near the collapsed area in the north (Fig. 10.1). During the
construction of the ventilation shaft for the Boschberg just to the south of the
Jezuïtenberg the top of the calcarenite was observed to be situated 10 m above the
mine roof level (the Caster horizont), i.e. at about 78 m NAP (personal
communication, W. Van den Bosch). At boring 281 of the Dutch Geological Survey,
situated at about 250 m to the west of the northern limit of the 1920 collapse, the top
of the calcarenite was encountered at 75 m NAP. Accordingly, it is assumed that the
erosional contact between the calcarenites and the Tongeren Sands is more or less
horizontal. Taking the dip towards the NNW of the calcarenites into account, the
calcarenite roof thickness will vary from about 7 to 10 m in the southern part to
about 11 to 14 m in the northern part of the Jezuïtenberg. Sparse surface outcrops
and borehole data indicate about 5 m of Tongeren Sands, covered by sands and
gravels of the Pleistocene Maas deposits and about 5 m of loess (Fig. 10.2).

The unit weights are assumed to be more or less the same as near the Heidegroeve
and the Geulhemmer Groeve. Since the values for the soil units hardly differ and
since their relative thicknesses are not well known, a unit weight of 18 kN/m³ is
applied. As usual, a value of 17.6 kN/m³ is taken for the calcarenite.

In this mine system two types of joints can be distinguished (Fig. 10.3):

- The first type is represented by plane, regular, relatively persistent
discontinuities which could be traced sometimes for more than 100 m. These
discontinuities have the same appearance as the joints generally encountered
in the calcarenite mines. Two joint sets are to be distinguished: one striking
SSW-NNE to SW-NE and another striking E-W to ESE-WNW. All joints are
dipping more than 70 degrees. Joint spacing is generally more than 10 m.
They are without infill.
Fig. 10.3 Natural discontinuities, earthpipes, roof fractures and roof falls, mapped only for the not collapsed part of the mine.
Fig. 10.4 The Jezuïetenberg with mining directions.
Fig. 10.5 Jesuits posing at a sphinx sculpture which was destroyed later during the collapse of 1920.

Fig. 10.6 In 1993, during a visit to the collapsed area, the remains of the sphinx sculpture were encountered amongst the debris fragments.
The second type consists of irregular, low-persistent discontinuities, which can not be traced for more than a few meters. They are more abundant than the first joint type. Only the most prominent discontinuities of this kind are mapped. These discontinuities often weaken pillars and roofs. The orientation of the irregular joints is generally ESE-WNW to ENE-WSW. Some are infilled with clay, particularly in the northern part.

Earthpipes are found in two clusters in the northern part of the mine (Fig. 10.3). Their diameter can reach up to 30 cm. These earthpipe clusters are probably associated with solution holes of a larger scale in the calcarenite above.

10.2.3 History and use of the mine

According to Schreiner (1960) mining in the hill of the Cannerberg began in the Middle Ages. However, it is not known during which century the first galleries of the Fallenberg were excavated. Part of the mining history was revealed by studying the geometry of the saw cuts which allowed to reconstruct the mining directions (Fig. 10.4). The mining front at the time of the 1705 collapse (Section 3.5.1) is situated at the eastern perimeter of the present Jezuïtenberg. Mining directions observed near the access galleries more to the east show that part of the present Boschberg (Fig. 3.8) was already mined from the area to collapse. Up to 1705 mining was undertaken from an entrance located between the "old entrance" and the "new entrance" indicated in Fig. 3.8.

After the collapse the mine, originally known as the Jufferenberg or St.Lambrichtsberg (de Bruijn, 1903) was named the Fallenberg. The original entrance was destroyed during the collapse and a new entrance was created more to the north (the "old entrance" in Fig. 3.8). This entrance, now sealed, still exists. The mining directions clearly show that the present Jezuïtenberg was mined from the north, where this entrance enabled a continuation of the exploitation. According to Schreiner (1960) mining continued from this entrance until about 1880. Annual reports of the State Supervision of Mines also indicate that the mining activities ceased at that time. In the Jezuïtenberg most dates on the pillar walls are from the 19th century. The oldest dates in this part of the mine are from the second half of the 18th century. Inside the 1920 collapse area dates of the first half of the 18th century were observed. In 1920 one of the largest collapses ever recorded for the calcarenite mines occurred (Section 3.5.5, Fig. 3.8). After this 7 hectare collapse the mine was closed during a year.

On the basis of the accounts above it can be concluded that the Fallenberg, with the exception of the oldest collapse area, was mined after 1705 from the northernmost entrance, which is now welded up. The southernmost still intact part of the Fallenberg, presently known as the Jezuïtenberg, was probably excavated from the second half of the 18th century till about 1880. The area which collapsed in 1920 was created earlier during the 18th century according to the mining directions and dates on the pillar walls.
Fig. 10.7 Pillar damage, mapped only for the not collapsed part of the mine.
Since 1880 Jesuits from a nearby seminary visited the abandoned mine system and created many sculptures, paintings and drawings. Because of their fine art works the mine got its present name. Because of vandalism by other visitors the entrance ("old entrance" in Fig. 3.8) was closed and a "look" in connection with the Boschberg served since 1904 as entry of the Fallenberg, only accessible for the Jesuits. In 1954 this access was shifted 15 m and is known as the "Heksenpoort" since then ("new entrance" in Fig. 3.8). Also after a sort of iconoclasm in 1906 by youths, who got access by destroying the closed old entrance, the Jesuits continued with their artistic hobby.

The collapse of 1920 destroyed the major part of the art works created before that time (Figs. 10.5 and 10.6). During a year the mine was closed, but in 1921 the Jesuits were allowed to carry on with their art works in the southern part at a safe distance from the collapse area. Fortunately many impressive art works were created since then. They can be considered as by far the best collection to be found in the calcarenite mines. Nowadays the safe parts of the Jezuïetenberg can be visited by tourists by appointment.

10.3 THE PRESENT CONDITION OF THE MINE

10.3.1 Individual pillars

Fig. 10.7 shows that severe pillar deterioration is concentrated along the collapse boundaries. Within 30 to 40 m distance from the limit of the 1920 collapse, pillars are heavily fractured and often show an hourglass geometry. Here pillars are categorized as class 3 or 4. Outside this transition zone pillar damage gradually decreases towards the south. Less and less pillar cracks occur, first independently of natural discontinuities but finally only along already present joints. In the southernmost 100 m hardly any pillar crack was observed.

In this context it should be noted that the overburden thickness increases less than 3 m from south to north and that pillar and gallery geometry and extraction ratio are more or less uniform throughout the Jezuïetenberg.

The pillars in the above mentioned transition zone were seriously damaged but did not collapse. Strains were apparently sufficient to generate fractures, which separate pillar sides from the central core, with hour-glass geometry as a result. But these strains were insufficient to induce some displacement or rotation of significance of the pillar sides into the galleries, as observed inside collapse areas. Considering the stress-strain experiments on calcarenite prisms (Chapter 6), the vertical strain of the transition zone pillars probably does not exceed about 30 millistrain. This corresponds with about 10 cm shortening for the pillars of about 3 m height.

1 A "look" is local dialect for a break through between two mine systems.
Fig. 10.8 General impression of the transition from the intact part of the Jezuïetenberg towards the 1920 collapse.
Fig. 10.9 Roof arch at the southern boundary of the 1920 collapse. The top of the debris ridge beneath is situated one meter above the original roof level.

In the galleries directly in front of this slope many roof falls have occurred (Fig. 10.3). Roof layers of up to 0.5 m thickness fell down and the present immediate roof still appears unstable.

In the last 40 years guides of the mine observed that in the immediate vicinity of the collapse area some roof beams and pillar slabs, which had already parted from the pillar core, came off now and then. This must be considered as a mere result of the action of gravity on already instable rock fragments. There is no evidence that additional cracking or other pillar damage has developed since the collapse, but this can certainly not be ruled out.
Fig. 10.10 Collapsed pillar about 20 m inside the collapse area. Here spalling extends up to about one meter inside the pillar. Pillar slabs often rotated and fell down on the gallery floor. At this location the gallery height is still close to the original three meter. No roof falls occurred.

Fig. 10.11 Ruined pillars inside the 1920 collapse, about 100 m from the southern boundary. Pillar spalling extends to more than one meter inside the pillar. Pillar slabs rotated into the gallery before being trapped by the quickly subsiding overburden. Here the open space between debris and roof is 1.5 to 2 m.
10.3.2 Characterization of the 1920 collapsed area

The southern boundary of the 1920 collapse with the Jezuïetenberg can generally be characterized as depicted in Fig. 10.8. Beyond the transition zone of severely damaged pillars a debris pile was encountered, analogous to the Heidegroeve (Section 9.3.2). Unlike the Heidegroeve, the passage into the collapse is not blocked off.

The top of the debris pile, which has developed like a ridge parallel to the collapse boundary, reaches up to 1 to 2 m above the original roof level. Over the debris a V-shaped arch at an angle of 60° to 80°, again parallel to the limit of collapse, has been formed up to 5 m above the original roof level (Fig. 10.9). Unlike the Heidegroeve, the collapse is not bounded by a fault. Along the whole boundary it can be clearly observed that, apart from disruptions of the immediate roof, the roof layers of the main rock overburden are continuous.

Inside the collapse area the pillars were obviously significantly shortened. Massive slabs from the pillar sides had collapsed on the gallery floor or had just rotated some degrees, generally about the lower edge of the slab, and then remained in position (Fig. 10.10). Often slabs, rotated to some extent or not, became trapped under the quickly subsiding overburden (Fig. 10.11). By considering the original saw and chisel pattern on the failed pillar walls, it could be assessed that the generally thin upper edge of such slabs must have been crushed by the roof.

Fig. 10.12 shows how the angle of rotation gives an estimation of the amount of vertical roof displacement or pillar shortening. Without crushing, the ratio of final (H₁) and initial pillar height (H₀) is simply given by the cosine of the angle of rotation \( \alpha \). Accordingly, the amount of pillar shortening is given by:

\[
\Delta H_{rot} = H_0 \left( 1 - \cos \alpha \right)
\]  

(10.1)

If an amount of vertical crushing \( \Delta H_{crush} \) occurs, the total pillar shortening must be considered as the sum of a rotation and a crushing component:

\[
\Delta H_{tot} = H_1 \left( 1 - \cos \alpha \right) + \Delta H_{crush}
\]  

(10.2)

These equations show that, if no crushing is taken into account, the angle \( \alpha \) gives a minimum value of pillar shortening.

The observed rotation angles of up to 20° indicate minimum roof displacements of up to 18 cm according to Eq. 10.1. The amount of crushing, as far as could be estimated, increases from several cm near the margin to several dm in the centre of the collapse area. An increase of pillar shortening towards the centre of the collapse is also indicated by slabbing of pillars further into the pillar core. At some tens of meters from the limit of collapse not only "primary" slabs (e.g. Fig. 6.5), according
Fig. 10.12 Geometry of pillar slab rotation in relation to vertical roof displacement.

to an angle of about 30° with the vertical, developed but also parts of the pillar core beyond were affected by spalling (Fig. 10.11). In spite of the often considerable amount of roof and pillar debris filling up the galleries, a pillar shortening of about 1.5 m in the central part of the collapse could be estimated. This shortening decreases to a few decimeters near the margin. More precise measurements were not performed because of obvious safety reasons.

Roof beams, up to 0.5 m thick, had often collapsed, resulting in more or less rectangular roof elevations. Particularly at gallery intersections more extensive roof collapses had occurred here and there, with U-shaped roof arches as a result. Roof falls were frequently partly bounded by joints. In the area of collapse about one third of the roof remained more or less intact.

Particularly towards the centre pillar slabs are pushed several decimeters into the gallery (Fig. 10.13). The resulting increase of pillar width could have been one of the mechanisms which arrested the collapse. Moreover, the galleries were filled with roof and pillar debris, which occupy more volume than intact rock, according to the bulking effect. The ratio of the volumes of broken and intact rock is expressed by the bulking factor B:

\[ B = \frac{V_{\text{broken}}}{V_{\text{intact}}} \]  

(10.3)

On the basis of literature values (Blyth & De Freitas, 1984; Meier, 1988) the bulking factor for calcarenite is estimated at 1.5 to 1.9. The bulking effect must have been a significant factor in reducing the open space inside the galleries to often about 1.5 m (Fig. 10.13). Locally the debris formed a pile which left less than 0.5 m space
Fig. 10.13 Galleries with 0.5 to 1.5 m of open space in the centre of the collapse. Roof and pillar debris almost reaches the original roof level.

below the ceiling. The confining effect of the debris, supporting the pillar walls, is considered here as the primary mechanism which arrested the collapse (see also Section 10.4.1.1).

Also the intact area to the north of the 1920 collapse (Fig. 3.8) was visited. The majority of the pillars was not or just slightly fractured. The northern boundary of the collapse and the transition zone beyond can be characterized similarly as the southern collapse limit.

10.3.3 Characterization of the 1705 collapse area

The 1705 collapse is bounded by the northern part of the Boschberg, which presently serves as the access mine system to the Jezuïetenberg, the eastern part of the Jezuïetenberg, the south-eastern margin of the 1920 collapse and part of the intact area near the old entrance (Fig. 3.8). Along this complete perimeter access into the 1705 collapse was blocked, generally by soil flowed into the galleries and to a minor extent by calcarenite debris. Approaching from the intact mine system of the Boschberg (about five connections) and the Jezuïetenberg (only one connection) no transition zone of severely fractured pillars was encountered, contrary to the 1920 collapse area. The presence of dominantly soil indicates that the calcarenite roof is thin. Also significant earthpipes and/or joints might account for the earth inflow.
Fig. 10.14 Schematic load-displacement diagram for a collapsing pillar.

Additionally, the continuity of the calcarenite roof as a whole must obviously have been disrupted. It can be concluded that the collapse boundary is comparable to the situation as depicted in Fig. 9.7 for the Heidegroeve, although in that mine faulting of the mine floor occurred and no earth inflow.

10.4 ANALYSIS OF THE COLLAPSES

10.4.1 The 1920 collapse

10.4.1.1 Mechanisms of collapse termination

The collapse was stopped by three mechanisms:

1. The confining effect of the roof and pillar debris supports the walls of the collapsed pillars. Accordingly, the effective pillar height is reduced which results in a more favourable pillar shape factor and hence a greater pillar strength. Due to the bulking effect roof falls contribute positively to this mechanism.

2. Pillar slabs move into the galleries and the effective width of the pillar, although intensively fractured, increases. As a consequence, the pillar shape factor increases and pillar strength is raised. Another result is that the average vertical pillar stress decreases.

3. Apart from roof falls at its base, the calcarenite roof mass as a whole remains continuous, even across the collapse margins. Hence a certain roof stiffness, i.e. resistance against further convergence, is preserved.
An additional mechanism, not observed in the Fallenberg, might be:

4. Direct support of the gallery roof by pillar and roof debris, possibly assisted by the bulking effect.

The effect of the pillar strengthening mechanisms 1) and 2) can be depicted in a load-displacement diagram (Fig. 10.14). For the 1920 collapse area the load-bearing capacities of the pillars apparently increased after some decimeters of vertical shortening, i.e. some hundreds of millistrains, close to the original values for intact pillars. Without mechanism 3) the load bearing capacities after the collapse are probably even greater than those when the pillars were still unfractured, because also the additional dynamic component of loading during the collapse must be considered. Thus, pillar collapse is a self-arresting event, even without mechanism 3). Theoretically, the collapse cannot proceed further anyway when the galleries are completely filled up with debris. This ultimate roof convergence can be easily estimated by taking into account extraction ratio, original pillar height and bulking factor. Such an estimation may serve to assess the maximum subsidence at the surface.

10.4.1.2 Insufficient large-scale pressure arching

In Section 4.3.1 and Fig. 4.12 it has been outlined that over an area of failing pillars a pressure arch develops inside the overburden. A large-scale collapse becomes inevitable when the failing pillars cannot support the weight of the material within the arch. However, over the mine systems in the Cannerberg (Fig. 2.7), i.e. all excavations depicted in Fig. 3.8, the thickness of the calcarenite roof is generally just 10 to 15 m (Fig. 10.15). The width of the collapsed area ranges from 150 to 200 m, which cannot be spanned by a pressure arch within the rock overburden. The calcarenite is covered by about 30 m of soil, but this material is of a relatively low stiffness (10-50 MPa) compared with calcarenite (500-1500 MPa) and not capable of transferring stress for this situation. Even if the complete overburden consisted of calcarenite, the arching capacities of the about 40 m thick rock mass would be insufficient to span a mine system of more than 150 m width. It can be concluded that, if the pillars are weakened by fracturing and if the total mine span and the thickness and composition of the overburden are considered, the large-scale collapse potential for the mine studied here is significant.

10.4.1.3 Deformation behaviour of a thin rock overburden

Unlike the Heidegroeve, the collapse is not accompanied by faulting at its margin. It is attempted to explain this by considering the elastic deflection of the calcarenite overburden. The vertical displacement by elastic deflection of the calcarenite roof without any pillar support can be roughly approximated by extending the two dimensional roofbeam theory of Obert & Duvall (1967) for additional distributed loading of the beam by a thickness of soil. The convergence s in the centre of the
span can be considered as the sum of the convergence resulting from the weight of the calcarenite beam \( s_1 \) and the convergence induced by the weight of the soil overburden \( s_2 \). To get an impression of the effect of the various parameters, the convergence in the centre of the span is given for the situation without pillar support:

\[
    s = s_1 + s_2 = \frac{\gamma_1 w^4}{32E_1 h_1^2} + \frac{\gamma_2 h_2 w^4}{32E_1 h_1^3}
\]  

(10.4)

where

- \( \gamma_1 \) = unit weight of calcarenite
- \( \gamma_2 \) = unit weight of soil
- \( h_1 \) = thickness of calcarenite
- \( h_2 \) = thickness of soil
- \( w \) = span
- \( E_1 \) = E-modulus of calcarenite

The components of convergence \( s_1 \) and \( s_2 \) increase with \( w^4 \), and with \( h_1^{-2} \) and \( h_1^{-3} \) respectively. It can be imagined that this elastic downward deflection of the mine roof is one or more orders of magnitude greater for the collapse area of the Fallenberg, with a span of about 150 m and thicknesses of calcarenite and soil of about 10 and 30 m respectively, than for that of the Heidegroeve, where the span is about 70 m and the typical thickness of both calcarenite and soil is about 22.5 m. If support by the collapsing pillars is ignored and if elastic flexure is not interrupted by faulting, \( s_1 \) and \( s_2 \) for the Heidegroeve and the Fallenberg collapse areas would be in the ratio of 1:370 and 1:1100 respectively. It should be noted that this is a rough estimation, because beam theory does not strictly apply to such a small span/thickness ratio as for the Heidegroeve. Due to its relatively large span and small thickness the calcarenite roof of the Fallenberg is less stiff than that of the Heidegroeve.
In the Fallenberg the roof deflection of up to about 1.5 m in the centre of the collapse area could develop without roof faulting in shear at the collapse margin (Fig. 10.15). This could now be explained as follows. Due to the relatively low stiffness of the calcarenite roof arching stresses, and, as a consequence, also shear stresses at the collapse margin, were low during the collapse. No shear faulting of the roof occurred during the collapse. The collapse was arrested, mainly by the mechanisms 1) and 2) outlined in Section 10.4.1.1, before the calcarenite roof had attained its maximum elastic deflection. After the collapse the load carrying capacity of pillars and debris in between was such that arching stresses in the calcarenite roof remained low. During the Heidegroeve collapse shear stresses at the collapse margin were probably much higher, due to the relatively high roof stiffness. The calcarenite roof attained its maximum elastic deflection before post-collapse pillar strengthening, with considerable arching stresses towards the abutments as a result. Probably at the beginning of the collapse, the arching stresses exceeded the shear strength of the rock, faulting at the collapse margin occurred, and pillars collapsed further till the load carrying capacity inside the collapse area had recovered.

In the Fallenberg nevertheless, tensile stresses at the collapse margin at the lower side of the calcarenite roof mass, where the curvature of the roof was at its maximum, must have been high enough for the development of tensile fracture zones. These are represented by the roof arches described in the previous sections. Again, tensile rupture of the complete calcarenite roof was inhibited due to the residual support of the pillars.

10.4.1.4 Deterioration of pillars near the limit of collapse

Since the roof rock mass, not disrupted by faulting at the collapse margin, maintained more or less its lateral continuity, load transfer from the collapse towards the area in its direct vicinity was still possible to a limited extent. This had possibly a deteriorating effect on the pillars just outside the limit of collapse. Initially, before the collapse, these pillars were already loaded in excess of their tributary area load due to small-scale arching from the fractured pillars just inside the area about to collapse. During the collapse itself the pillars inside the collapse area were crushed and could not resist the load of the quickly subsiding overburden. As a consequence, during the few seconds of the collapse the pillars just outside the collapsing area were loaded a bit more severely. This could explain the serious fracturing of the pillars within 30 to 40 ms from the limit of collapse, i.e. in the transition zone mentioned in Section 10.3.1 and visible in Figs. 10.7 and 10.14.

Also after the collapse the pillars of this transition zone might have been suffering an extra pressure from the most nearby destructed pillars beyond the collapse limit, although the calcarenite roof mass is considerably reduced in thickness at the collapse margin due to the arch formation. Here a similar mechanism as depicted in Fig 9.15b could apply.
Fig. 10.16 Stress and initial strength for pillars of class 1 to 4 for the complete Jezuïetenberg, to the south of the 1920 collapse.

Fig. 10.17 Stress and long-term strength for pillars of class 1 to 4 for the complete Jezuïetenberg.
Whether pressure from the collapsed zone towards its vicinity is being transferred or not, this is certainly true for load transfer from the damaged transition zone towards the intact part of the Ježuïetenberg in the south. Due to the domino-effect the pillar damage inside the transition zone might extend over the rest of the mine system of the Ježuïetenberg in the course of time by creep deformation. No evidence has been collected of additional pillar fracturing in the Ježuïetenberg after the collapse. Therefore, a measuring program will be carried out in the near future to find out whether creep deformation occurs of (still) more or less intact pillars at various distances from the transition zone.

10.4.1.5 Location of the collapse

It is somewhat difficult to explain the location of the collapse within the mine. Since the only available mine plan is very inaccurate for the collapsed part, it is impossible to assess if pillar dimensions were an important factor. The following factors could apply:

- the total east-west mine span of the collapsed part is maximally about 225 m, which is about 50 m in excess of that of the intact Ježuïetenberg.

- since mining was undertaken from the north, the collapsed part is (older than the intact area and hence less affected by creep deformation.

- while the level of the topographic surface is more or less the same from south to north, i.e. about 105 to 107 m NAP, the calcarenite rocks are dipping about 2 m per 100 m towards the north inside the Ježuïetenberg. If this dip is extrapolated towards the north, the overburden height for the collapsed area (42-48 m) exceeds that for the Ježuïetenberg (mainly 38-42 m). This might be the most serious factor.

10.4.2 The 1705 collapse

This collapse is not bounded by a 40 m wide transition zone of class 3 and 4 pillars. The pillars within the 40 m limit can generally be characterized as class 1 or 2 (Fig. 10.7). Near the entrance galleries of the Boschberg (Fig. 3.8) the pillars bounding the collapse are more or less undamaged.

This can be explained by the rupture of the complete rock overburden at the collapse boundary by faulting. Contrary to the 1920 collapse, during and after the collapse transfer of excess load from the collapsed to the intact part was more or less inhibited by the separation of the adjacent roof rock masses. Thus the stability of the pillars of the intact part is not negatively affected by additional loads during and after the collapse, when the calcarenite roof rocks fail by faulting at the collapse margin. This principle will also be illustrated in the next chapter.
Fig. 10.18 The relationship between initial safety factor, pillar class and distance from the 1920 collapse.

Fig. 10.19 The relationship between long-term safety factor, pillar class and distance from the 1920 collapse.
Contrary to the 1920 collapse, convergence in the collapsing area of 1705 was not reduced towards the boundary by a roof plate of some stiffness, connected to the intact area. Accordingly, it can be imagined that such a gradient in convergence does not exist here. Pillar shortening would be more or less the same throughout the collapse in case of a constant overburden height. Here the overburden height decreases towards the east and pillar shortening can be expected to decline accordingly into that direction.

10.5 PILLAR STABILITY ANALYSIS ACCORDING TO STRENGTH FORMULAE

10.5.1 Individual pillars

Samples for the determination of the unconfined compressive strength were taken at five locations (Fig. 10.1). The mean values for locations 1 to 5 were 1.76, 1.48, 1.56, 1.59 and 0.75 MPa respectively. The low value for the last sample, which is due to the presence of abundant shells, is not considered representative and omitted. The mean value from the remaining four tested cores is 1.60 MPa with a standard deviation of 0.2 MPa.

For all pillars of the Jeuzietenberg, just to the south of the 1920 collapse, both initial and long-term strength were determined according to the principles of Sections 6.14.2 and 7.8. Since most pillars are regularly shaped, strength equations 6.6 and 6.7, using W/H as the shape parameter, were applied. A few pillars with an irregular outline were subdivided (Section 6.14.2). The subdivision of Orlic (1990) was used. Some pillars show a W/H ratio exceeding four and strength was assessed by means of Eqs. 6.14 and 6.15. Pillar stress was calculated by the tributary area method. Regarding pillar classification, the influence of joints on pillar fracturing are considered according to Section 6.14.1.2. The results are depicted in Figs. 10.16 and 10.17.

Practically all pillars have initial safety factors exceeding 1. It is striking that neither for initial nor for long-term strength a correlation exists between safety factor and pillar class. Contrary to the southern part of the Geulhemmer Groeve for example (Section 8.5.1), for each pillar class the distribution of safety factors appears to be more or less random. Also Orlic (1990) did not manage to establish a satisfactory correlation. But Fig. 10.7 reveals that pillar damage is concentrated near the 1920 collapse margin. Therefore safety factors were depicted in relation to the distance from the collapse boundary (Figs. 10.18 and 10.19). The correlation between this distance and pillar class is evident. Class 4 pillars only occur at a distance of less than 10 m from the collapse boundary, while class 3 pillars are exclusively to be observed within 80 m and class 2 pillars within 140 m from the collapse. Neither initial nor long-term safety factors are correlated to the distance from the collapse limit. It can be concluded that the deterioration of the pillars of the Jeuzietenberg is a result of the 1920 collapse. This is in accordance with the theory outlined in Section 10.4.1.4, which says that the pillars directly outside the perimeter are
additionally loaded just before and during the collapse, and possibly also afterwards to a less extent.

At more than 140 m distance from the collapse boundary, only intact pillars occur, with SF₀ and SFₖ values generally in excess of 1.2 and 0.8 respectively. However, outside the area of influence no fractured pillars occur, making an adequate field validation of safety factors impossible.

### 10.5.2 Large-scale pillar stability and general mine stability

For the total safety factors for the whole Jezuïetenberg, i.e. the part of the mine to the south of the 1920 collapse, the following values were calculated:

\[
\begin{align*}
\text{SF}_{\text{tot,0}} &= 2.01 \text{ (see Eq. 6.28)} \\
\text{SF}_{\text{tot,lt}} &= 1.50 \text{ (see Eq. 7.29)}
\end{align*}
\]

Until now validation of safety factors is only performed in the Geulhemmer Groeve. Using the correction factors, established for that mine (Table 8.4), the following safety factors result as a provisional estimation:

\[
\begin{align*}
\text{SF}_{\text{tot,lt}',0} &= 1.17 \\
\text{SF}_{\text{tot,lt}'',0} &= 1.41
\end{align*}
\]

As explained in Section 7.8, K₀-values should not be applied to other mines. These values appear on the small side, but a comparison between Figs. 8.17 and 9.13 on the one hand and Fig. 10.16 on the other hand reveals that intact pillars of the Geulhemmer Groeve and Heidegroeve generally have SF₀ values of more than 2, while those of the Jezuïetenberg are predominantly characterized by SF₀ values in excess of 1.2. This difference suggests that the correction factors K₀', and K₀'' for the Jezuïetenberg are significantly greater, and closer to one, than for the other two mines. As a consequence, SF₀' and SF₀'' are probably greater than the values determined above. Similarly, the values of SFₖ' and SFₖ'' are likely underestimated as well. Unfortunately, as outlined in the previous section, this cannot be confirmed here on the basis of an analysis of fractured pillars because of the proven influence of the collapse.

The assessment of the overall stability of the Jezuïetenberg can be significantly improved when more experience has been gained of the validation of safety factors. Also the UCS should be measured at more locations in the Jezuïetenberg to increase the accuracy of the safety factor values.

Whether pillars in the Jezuïetenberg are still deforming, and, if so, at which rate, will be measured in the near future at several locations of the mine system.

Whether the Jezuïetenberg will collapse if pillar deterioration would proceed, possibly resulting in a critical value of the total safety factor, is difficult to predict.
with the presently available information. But this will become more clear in the next chapter, which concerns general mine stability. Anyhow the situation of the Jezuïetenberg seems slightly more favourable than that of the collapsed part of the Fallenberg directly to the north. This part of the mine is comparable to the Jezuïetenberg, but the mean overburden thickness of the latter is about 12 % less. Additionally, the average mine span of the Jezuïetenberg is about 22 % less. The pillar dimensions of the collapsed area are not well known.
CHAPTER 11

OTHER INSTABLE OR COLLAPSED MINES; SYNOPSIS OF GENERAL MINE STABILITY AND COLLAPSE POTENTIAL

11.1 INTRODUCTION

In the two preceding chapters only three collapse areas are analyzed. Even for so few cases the characteristics of the collapses diverge significantly. So far, two major types of collapse areas can be distinguished. A collapse bounded by faults, which offset the complete calcarenite overburden, is denoted type A (e.g. the Heidegroeve collapse). When the calcarenite roof mass is not disrupted by continuous faults at the collapse margin, but subsided merely by downward deflection, type B applies (e.g. the 1920 collapse of the Fallenberg). Here a number of additional collapsed mines are shortly described here, in order to get a more general impression.

It was stressed in Chapters 3 and 4 that for a large-scale collapse two conditions must be satisfied:

- the large-scale pillar stability, expressed by the total safety factor, is insufficient, i.e. the load carrying capacity of all pillars together is too low in relation to the total overburden load.
- the general mine stability is insufficient, i.e. the arching capacity of the overburden is inadequate to prevent a large-scale collapse of a deteriorated mine section.

While the first condition was dealt with in Chapters 6 to 10, this chapter concerns also the second one. Some mined areas exclusively comprise failed pillars but did not collapse. Apparently the total load carrying capacity of all pillars together, i.e. the total safety factor, was insufficient, but upon widespread pillar failure enough load was transferred towards the mine abutments. By evaluating a few of such cases an attempt will be made to analyze under which circumstances the arching capacity of the overburden is sufficient to protect a mine consisting of deteriorated pillars against complete destruction.
Fig. 11.1 The 1845 collapse area in the Gemeentegrot. The hatched areas mark the boundary of the collapse where it was inspected in November 1995. The picture in the lower right corner comprises a profile parallel to the limit of collapse. The remaining drawings are orientated more or less perpendicular to the boundary, with the area of collapse to the right.
Part III: Observations in the mines and application of results to assess mine stability
Fig. 11.2 The locations of the photographs are indicated in Fig. 11.1. a) Access to the collapse blocked by debris. b) At least 0.5 m of pillar shortening inside the collapse. Note that opposite pillar walls have almost met as a result of intense deformation. The open space is now less than 1.5 m high. c) Photograph taken from the debris pile, about 4 m above the original roof level. In the lower right corner a 2 m high pillar is visible. The top of the roof arch is about 5 m above the original roof level. The fault, bounding the collapse, is dipping towards the right, where the intact area is situated.
11.2 THE COLLAPSES IN THE GEMEENTEGROT

In Sections 3.5.3 and 3.5.4 short accounts are given of collapses in 1845 and 1886 respectively. A third collapse area is located in the northwestern part of the mine system (Fig. 3.10). In November 1995 the boundaries of the 1845 and the northern collapse were inspected. Some years earlier the limit of the 1886 collapse had been briefly visited.

The observations at the 1845 collapse are summarized in Figs. 11.1 and 11.2. The collapse area is everywhere bounded by a normal fault, which is dipping towards the intact area of the mine at an angle of 50° to 80°. Locally the fault could be observed to continue for more than 10 m above the original roof level, before it was obscured. Often a roof arch of several meters high has been formed at the boundary. Access to the collapse area was generally blocked by debris. Only at the most western limit clay streaks on the roof were observed. These indicate that the fault reaches the top of the rock overburden. It can be concluded that the collapse is of type A.

The northern collapse of unknown age showed basically the same characteristics as observed at that of 1845. Again the collapse is limited by faults, and can be designated type A accordingly. Here the roof arch at the boundary is often more than 15 m high and bounded by steep, 80° degree walls. The debris pile is more than 10 m high. It does not give access to the collapse area.

The 1886 collapse was only briefly studied, but at its margin faults in the roof formation could be observed. Hence the collapse is of type A.

11.3 THE COLLAPSE AREA OF THE MUIZENBERG

This collapse, indicated in Fig. 3.11 is shortly dealt with in Section 3.5.6. The area was studied in September 1995. From the nearby Cannerberg the collapse is bounded by a more than 5 m high debris pile. No major faulting in the roof was observed here. The pillars of the Cannerberg mine system in the direct neighbourhood are intact. Inside the collapse the open space is generally varying from a few decimeter to 2 m. Massive roof falls at gallery intersections are the rule. The gallery, depicted in Fig. 11.3, is more or less closed by two opposite pillar wall segments, which moved and rotated into the open space of originally 3 to 4 m width. Due to the absence of faults at the margins this collapse is characterized as type B, like the nearby 1920 Fallenberg collapse, about 600 m to the north (Fig. 3.8).

11.4 THE COLLAPSE AREA OF THE ROOSBURG

The boundary of this collapse (see also Section 3.5.7), which was visited in September 1995, is characterized by steep faults dipping towards the intact area. Clay streaks indicate that the fault reaches the rock-soil interface. The collapse area is completely blocked by debris (Fig. 3.13). The pillars in the direct vicinity of the
collapse are more or less undamaged. Hence, with regard to pillar deformation, the transition towards the collapse is abrupt. The pillars in the intact area are about 4 m high and often relatively small in horizontal direction due to pillar robbing. The dimensions inside the collapse are not known. Clearly the collapse is of type A.

11.5 THE AREA OF FAILED PILLARS IN GROEVE DE SCHENK

This mine, also visited in September 1995 and indicated in Fig. 2.7, consists of two parts of respectively 1.1 and 1.2 ha. The northern part, near the entrance, is more or less intact while the southern part, interconnected by one gallery, has seriously deteriorated. Here all pillars can be characterized as class 3 or class 4 (failed). Pillar shortening is maximal in the centre of the about 70 m wide mine system, where dominantly failed pillars with hour-glass geometry occur. Near the abutments pillars are mainly class 3. Nevertheless, the area has not collapsed. The overburden has only deflected slightly downward over the mine. The maximum convergence occurred in the centre and is roughly estimated to be 10 cm at most. Dates written on the walls from 1910 to 1920 clearly circumvent the fractures. Hence the deterioration must be dated to earlier times. It can be concluded that here the arching capacity of the roof mass is apparently enough to prevent widespread pillar collapse.
Fig. 11.4 The area studied within the southeastern part of the Gemeentegrot with natural discontinuities and pillar cracks.
11.6 THE AREA OF FAILED PILLARS IN THE GROOTBERG

The excavation of the Grootberg forms the southeastern part of the large Avergat mine workings, situated in Belgium (Fig. 2.7). This mine was studied for a stability analysis (Bekendam & Price, 1992; Bekendam, 1993; 1995). The Grootberg measures about 50 by 120 m and nearly all pillars have failed, due to pillar robbing. This small mine is bounded by unmined rock in the south and at the other sides by pillars of several tens of meters width, which serve as abutment pillars. Despite the bad state of the pillars, the mine did not collapse.

11.7 THE AREA OF FAILED PILLARS IN THE SOUTHEASTERN PART OF THE GEMEENTEGROT

This part of the mine, which is more or less isolated from the rest of the Gemeentegrot (Fig. 3.10) by unmined areas, measures about 140 by 70 m. It was investigated together with W. Dirks and Prof. D.G. Price in July 1989. The results of the stability analysis can be found elsewhere (Bekendam & Price, 1989; Dirks, 1990; Bekendam & Dirks, 1990). During this fieldwork pillar dimensions were measured to about 0.20 m accuracy. In 1992 pillar cracks were mapped according to the newly developed mapping system.

The pillars are irregularly shaped, mainly due to pillar robbing. Some pillars were mined in such an extent that pillar width hardly exceeds one meter. Their height is about 2 m. Only in the western part of the area studied the galleries are deepened and pillar height reaches a height of 3.5 m. Fig. 11.4 shows that nearly all pillars are seriously fractured and be characterized as class 3 or 4. Yet the mine system did not (yet) collapse.

Most written dates on the walls are from the first half of the 19th century. It is assumed that in this period this part of the mine was excavated. Some inscriptions, including a few from the beginning of the 20th century, are offset by fractures. Hence pillar deterioration was going on well after the creation of the mine.

11.8 TWO TYPES OF COLLAPSES

In total 8 collapse areas were investigated underground. As noticed previously, two types of collapse can be recognized (Fig. 11.5) The type A and type B collapses were analyzed more extensively in Sections 9.4.1 and 10.4.1 respectively:

Type A. Downward movement of the rock overburden occurs along collapse-induced faults at the collapse limit. The faults are inclined towards the relatively intact area at a cave angle of generally 60° to 80°. Pillar shortening inside the collapse area can be expected to be more or less similar from the limit towards the centre. When the fault develops during the collapse, the calcarenite roof mass is intersected over its whole thickness and load transfer
from the collapsing area towards the pillars just outside the collapse ceases. However, from that moment onwards these pillars are additionally loaded by the calcarenite and soil above the fault. The amount of additional loading can be derived geometrically, and depends on the cave angle and the thicknesses of calcarenite and soil. The collapse stops due to mechanisms 1, 2 and/or 4, recognized in Section 10.4.1.1.

Type B. Downward movement of the rock overburden occurs by mere deflection, without disruption over its full height at the margin. Pillar shortening inside the collapse area increases from the limit towards the centre. Since the calcarenite roof mass maintains more or less its lateral continuity at the collapse margin, load transfer from the collapse towards the area in its direct vicinity is still possible, albeit much less than before and during the collapse because the load carrying capacity in the destroyed area was restored for the major part. Hence also in this case pillars just outside the collapse may still be additionally loaded. The collapse may be stopped by all four of the mechanisms listed in Section 10.4.1.1.
Table 11.1 Compilation of the main characteristics of the studied failed and/or collapsed areas. The mean values are listed here. Most areas considered are more or less rectangular. The mine span recorded here corresponds with the smallest of both horizontal dimensions. The number of pillars refers to the number contained within the mine span.

<table>
<thead>
<tr>
<th>Mine area</th>
<th>Area of pillar deterioration (ha)</th>
<th>Mine span w (m)</th>
<th>Overburden height h (m)</th>
<th>Rock overburden height h_{ro} (m)</th>
<th>w/h_{ro}</th>
<th>Pillar width/height ratio W/H</th>
<th>Number of pillars in cross-section</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>No collapse</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NE Geulhemmer Groeve</td>
<td>0.4</td>
<td>40</td>
<td>30</td>
<td>5</td>
<td>6.0</td>
<td>1.25</td>
<td>5</td>
</tr>
<tr>
<td>S Groeve De Schenk</td>
<td>1.2</td>
<td>70</td>
<td>40</td>
<td>20</td>
<td>3.5</td>
<td>1.2</td>
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<td>35</td>
<td>15</td>
<td>4.7</td>
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<tr>
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<td>50</td>
<td>28</td>
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<td></td>
<td></td>
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<tr>
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<td>0.4</td>
<td>70</td>
<td>45</td>
<td>23</td>
<td>3.0</td>
<td>3.6</td>
<td>5</td>
</tr>
<tr>
<td>Fallenberg 1705</td>
<td>3.0</td>
<td>100</td>
<td>20</td>
<td>10</td>
<td>10.0</td>
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<td>6</td>
</tr>
<tr>
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<td>1.5</td>
<td>30-80</td>
<td>25</td>
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<td></td>
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<td>45</td>
<td>15</td>
<td>13.3</td>
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<td>35</td>
<td>15</td>
<td>12</td>
<td>1</td>
<td>11</td>
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Table 11.1 comprises a compilation of the main characteristics of the studied collapse areas. The majority, i.e. 6 cases is of type A, while only two can be classified as type B. Note that both collapses with by far the greatest ratio of span S and calcarenite overburden thickness h_{calc} are of type B, which is explained in Section 10.4.1.2.

11.9 ANALYSIS OF GENERAL MINE STABILITY AND COLLAPSE POTENTIAL

Table 11.1 also contains 4 cases of extensive pillar failure without collapse as a consequence. Three of these cases were outlined in the preceding sections, while the instable area of the Geulhemmer Groeve was described in Section 8.3 and Fig. 8.1a. This section concerns general mine stability, i.e. the arching capacity of the overburden to prevent large-scale collapse of areas of extensive pillar failure.
Fig. 11.6 Rock overburden thickness vs. mine span. A line, based on the studied cases so far, is drawn as an approximate limit between collapsed and not collapsed areas. Note that both type B collapses show by far the greatest mine spans. According to this graph, the Heidegroeve should not have been collapsed if no workings underneath were present.

11.9.1 General mine stability in terms of mine span and rock overburden height

Rock overburden height and mine span are depicted in Fig. 11.6. The northern collapse area of the Gemeentegrot and that of 1845 are not included because their horizontal extent is not known or too irregular respectively. The collapsed and not collapsed areas studied so far can be separated by a line, which is considered as an approximation of the maximum pressure arch for a certain rock overburden height. Note that both type B collapses show by far the greatest mine spans. All collapsed mines have spans in excess of 80 m. According to this graph, the Heidegroeve should not have been collapsed if no workings underneath were present. The data point of the present Jezuitenberg plots near those of the Fallenberg and Muizenberg, which indicates that, if widespread pillar failure would occur, the Jezuitenberg will not remain stable but collapses according to type B.

This graph may serve as a rough approximation as how to delineate general mine stability. Differences in total overburden height, strength and E-modulus of the rock roof formations, pillar layout (i.e. extraction ratio) and pillar stiffness are not incorporated.
Fig. 11.7 Critical strata stiffness for an extraction ratio of 50% according to Ozbay (1989; see also Fig. 4.16). A normalized value of about 0.05 separates collapsed and not collapsed mines. It appears that the Heidegroeve, indicated by a cross, should not have collapsed without the influence of workings underneath.

11.9.2 General mine stability in terms of pillar and strata stiffnesses

The cases dealt with in Fig. 11.6 are also depicted in Fig. 11.7. Here the results of Ozbay (1989; see also Section 4.3.3.6) are extrapolated for w/h_{calc} ratio's exceeding 8. The chosen extraction ratio of 50% approximates that of most calcarenite mines. The critical strata stiffness seems a suitable parameter to verify the possibility of a sudden collapse, because the pillars of all mines considered are failed. Contrary to the analysis of the preceding section, the pillar layout (extraction ratio, number of pillars in cross-section) and the E-modulus of the roof strata are now incorporated as well. The normalized critical strata stiffness proved to be relatively low for all cases. Except for the Heidegroeve which is indicated by a cross, the data points for the collapsed mines are situated below the value of 0.05, while those for the not collapsed mines are situated slightly above. As in Fig. 11.6, it appears that for the Heidegroeve the conditions were more favourable than for the other collapsed mines.

Although the individual influences of width and height on post-peak stiffness of calcarenite pillars could not be well established (Figs. 6.35-6.38), an attempt was made to analyze collapse potential more quantitatively by comparing critical strata stiffness \( \lambda_c \) with post-peak pillar stiffness \( \lambda \). Even when taking the most favourable values of \( \lambda \), for all mines, including those which did not collapse, the critical strata stiffness proved to be greater, i.e. less negative, than post-peak pillar stiffness. This suggests that a violent collapse is well possible for all ten of the studied cases.

It should also be noted that strata stiffness according to Fig. 11.7 only applies if the roof overburden remains intact, i.e. for type B collapses. If roof faulting occurs, as during type A collapses, the strata stiffness becomes practically zero, and large-scale collapse is inevitably violent. It appears that stiffness criteria do not represent an adequate tool to assess the general mine stability stability of the calcarenite mines.
11.9.3 Synopsis of large-scale collapse

Now the conditions for a large-scale collapse can be described as follows (Fig. 11.8):

1. A large-scale collapse of a given mined area is possible when most pillars of that area have failed, i.e. most pillars are deformed to such an extent that the slope of their load-displacement curves has become negative. Due to fracturing pillar strength has decreased considerably.

2. Now it depends on the arching capacity of the overburden if the reduced total load carrying capacity of the pillars is still sufficient to support the load of the overlying calcarenite and soil. If not enough load is transferred towards the abutments and/or relatively strong pillars beyond the area considered, a collapse becomes immanent. The arching capacity is determined for an important part by the ratio of mine span and rock overburden height (Fig. 11.6).

3. During the deterioration of pillars preceding the collapse deflection of the overburden over the mined area had already steadily increased. But now roof
Fig. 11.9 Schematic illustration of arching stress, i.e. load transfer towards the collapse boundary, before, during and after the collapse.

deflection accelerates. The stiffness criteria determine whether the collapse is violent or not. For example, when the critical strata stiffness is less, i.e. more negative, than post-peak pillar stiffness, violent, uncontrolled collapse is not possible. The analysis for ten calcarenite mines suggests that, if collapse occurs, it is generally violent (Fig. 11.7). This is confirmed by experience. All ten large-scale collapses, described in this thesis, were violent and uncontrolled.

4. During the uncontrolled and violent collapse the overburden is less supported from below. As a consequence, the arching stress towards the perimeter of the collapse increases for some seconds (Fig. 11.9). It depends on the stiffness and strength of the overburden rock mass whether the arching stress gives rise to faulting at the collapse margin (type A) or not (type B). Fig. 11.6 indicates that the ratio of mine span and rock overburden height determines the type of collapse for an important part.
CHAPTER 12

CONCLUSIONS AND RECOMMENDATIONS

12.1 APPLICATION OF THE SYSTEM TO ASSESS PILLAR STABILITY AND LARGE-SCALE COLLAPSE POTENTIAL

In Fig. 12.1 a scheme is presented which summarizes how to assess the possibility of a large-scale collapse on the long term of a given mine or mine section. It should be noted that local instability problems like roof instability and earth inflow from organ pipes are not considered here. The greater part of this scheme concerns the assessment of individual safety factors and the total safety factor, the latter expressing large-scale pillar stability, i.e. the capability of all pillars together to support the overburden load. If SF_{tot,lt}' is less than a certain safe value, somewhere between 1 and 1.5, this first criterion of mine safety is not fulfilled. The second criterion of mine safety is to be found at the bottom of the scheme. This criterion considers general mine stability and is based on the arching capacity of the overburden over a deteriorated mine section. A large-scale collapse is only to be expected if neither of the criteria are fulfilled, i.e. when both large-scale pillar stability and general mine stability are insufficient.

When applying this system of large-scale collapse potential assessment, experience is essential, particularly when choosing values of K_n' and a safe SF_{tot,lt}'. The criterion of general mine stability (Fig. 11.6) should be applied carefully. Additionally, the analysis should always be combined with a thorough visual inspection of pillar damage.

Safety of a calcarenite mine with regard to large-scale collapse is important when:

- a) touristic or non-touristic exploitation is planned or already exists in the mine or in a section of the mine.

- b) houses or other surface structures are planned or already situated above the mine and its direct vicinity.

- c) areas outside the mine near the entrances are inhabited or otherwise frequently visited (in view of the air blast which accompanies a collapse).
Fig. 12.1 Scheme depicting the assessment of pillar stability and large-scale collapse potential.
- d) the mine contains art works, inscriptions etc. of great social-cultural value.

When mine safety is considered important for one of these reasons, the system, summarized in Fig. 12.1, is a relatively quick and inexpensive method to assess if much more costly and time-consuming (one to several years) monitoring of strain by means of measuring equipment inside the mine is necessary. Since the criterion of general mine stability (Fig. 11.6) represents a rough approximation, it is advised to undertake the necessary actions (see below) whatever this criterion is fulfilled or not. Three possible outcomes of the analysis, combined with a visual inspection of pillar damage, can be distinguished:

1. The total safety factor exceeds the safe value and no serious pillar damage is detected: neither installation of a measuring system, nor support measures are necessary, but visual inspection must be continued on a regular basis, e.g. once a year.

2. The total safety factor is more or less equal to or even less than the safe value and some serious pillar damage is detected, but this damage cannot be visually observed to increase rapidly: installation of a measuring system and continuous monitoring of pillar strain at various locations in the mine, combined with visual inspection, is necessary. If strain continuously increases at such a rate, that this may give rise to a large-scale collapse in the future, support must be designed and applied. While designing the support measures, the system of pillar stability calculation (Fig. 12.1) serves to determine how much support is necessary, and how to achieve a maximum increase of load carrying capacity of the pillars, and thus of safety, at minimal costs.

3. The total safety factor is less than the safe value and there is a visually observable rapidly increasing pillar deterioration: a large-scale collapse is imminent. The mine has to be abandoned immediately and must be closed off. This also applies to the area outside the mine near the entrance. Support measures, to protect surface structures, can only be taken from outside the mine, i.e. by injection of grout into the mine from the surface through a series of boreholes. However, without access to the interior of the mine the application of support is difficult, if not impossible, to control.

12.2 GENERAL PRINCIPLES OF SUPPORT MEASURES

If possible, support should be applied from inside the mine. Walls are to be constructed for (temporal) support of the filling material. Pillars can be enlarged or galleries can even be completely filled up. The support can reinforce pillars in two ways. In the first place, due to the lateral support the shape factor of the pillars $N_{shape}$ increases and therefore also their peak strength $\sigma_p$, residual strength $\sigma$, and long-term strength $\sigma_k$. Secondly, in the ideal case also the roof is directly supported, in order to reduce pillar stress $\sigma$, but this is generally difficult to achieve. Although, if this can be accomplished, the support structure should have a stiffness preferably
comparable to that of the calcarenite pillars. Obviously, the support measures should result in a value of $SF_{tot,lt}$ which is safe. Additionally, the support should be designed in such a way, that maximum increase of load carrying capacity is obtained using a minimum amount of material: i.e. the locations of support should be selected in such a way that the pillar shape factors increase as much as possible. Such an economic approach can be realized by the application of the system of pillar stability assessment developed here.

12.3 RECOMMENDATIONS TOWARDS IMPROVEMENTS OF THE SYSTEM

In general the system of pillar stability and collapse potential assessment could be improved by:

- reducing the systematic and random errors, outlined in Section 6.14.

- a better estimation of these errors, i.e. quantification of individual systematic errors and standard deviations of individual random errors.

To achieve this the following investigations are recommended:

- an extensive measuring program to assess variation of UCS inside individual pillars (i.e. at a cm- to m-scale), and UCS variation from pillar to pillar over a whole mine (i.e. at a m- to 100 m-scale). Such a measuring program becomes more feasible if a method of indirect UCS determination can be developed, which is not only quick and inexpensive, but also precise and accurate.

- more creep tests on model pillars of various width/height ratio’s, in order to determine long-term strength, in relation to pillar shape, more accurately.

- performing short- and long-term compression tests on model pillars, resembling an actual mine configuration, instead of experiments on prismatic samples.

- making one or more boreholes directly above the mine studied, in order to assess the relative thicknesses of the individual overburden layers more accurately.

- numerical experiments to assess load redistributions between pillars of different shape, damage class and position relative to the mine abutments, and to refine the criterion of general mine stability. Such experiments by the author are in progress.

- validation of calculated safety factors in more calcarenite mines of varying outline and age, to assess the influence of the various error sources on $K$-values.
These investigations should result in a better accuracy of input parameters and their relationships. Additionally, the error distribution of the input parameters should be described more accurately, allowing, by preference using Monte Carlo simulation methods, a more accurate delineation of the distribution of the output parameters, i.e. $SF_{it}$ and $SF_{tot,lt}$. The aim is that these safety factors can be determined more accurately, and that safety of a mine can also be formulated in terms of probabilities.
APPENDIX

MACRO FOR DECELERATING AND ACCELERATING CREEP REGRESSION LINES

Within the spreadsheet program of Symphony 4.2 a macro was written to separate and describe the regions of decelerating and accelerating creep, as outlined in Section 7.6.1.3.

{BreakOn}
{Let D1, log: string} ~
{Let D2, time} ~
{GOTO} A4 ~
{MENU} rncReadings ~ {.DOWN 49} ~
{GOTO} D57 ~
@Count(Readings) ~
{MENU} rv ~ D58 ~
{GOTO} A64 ~
+D58+1 ~
{GOTO} D4 ~
{MENU} f1 ~ {.RIGHT 4}{UP}{DOWN $D$58} ~
@Log({LEFT 2}) ~
{Let E1, mili: string} ~
{Let E2, strain} ~
{RIGHT}
{MENU} c, {LEFT} ~ {DOWN}. {LEFT}{UP 2}{DOWN $D$58} ~
{Let F1, u_i} ~
{Let G1, (u_i-u_i-1)^2: string} ~
{Let H2, u_i^2: string} ~
{Let J1, range} ~
{Let K1, a} ~
{Let L1, b} ~
{Let M1, R^2: string} ~
{Let N1, DW} ~
{OnError AB32} ~
{SERVICES}aaSTAT.APP ~ q
{If $D$58 > 3}{branch AB36} ~
{MENU} rndReadings ~
{Branch AB71} ~

write column names

go to first entry
make range of involved readings
go to D57
put value of @count(Readings) into D58

initial value of A64 = D58+1
to first log (time) entry
format u, values
calculate log(time)

call stat.app
conditional loop
delete range Readings
Appendix: Macro for decelerating and accelerating creep regression lines

{GOTO} A65 ~
+ A64-1 ~
{MENU} rv ~ A64 ~
{GOTO} D4 ~
{MENU} rrrx. {UP} {DOWN} $A$64 ~
y{RIGHT}. {UP} {DOWN} $A$64 ~
oA54 ~ g
{RIGHT} 2
+ $D$55 + $C$61 * {LEFT} 2 - {LEFT} ~
{RIGHT}
{LEFT} ~ {LEFT} {UP} * 2 ~
{RIGHT}
+ {LEFT} 2 * 2 ~
{LEFT} 2
{MENU} c. {RIGHT} 2 ~ {DOWN} . {RIGHT} 2
{UP} 2 {DOWN} $D$58 ~
{RIGHT}
{DOWN} $D$58
@sum({UP}. {DOWN} 2 {UP} $D$58) ~
{RIGHT}
@sum({UP}. {DOWN} 2 {UP} $D$58) ~
{GOTO} J4 ~
{DOWN} @Count(Readings)
{UP} $D$58
+ $D$58 {RIGHT}
+ $S$61 {RIGHT}
+ $D$55 {RIGHT}
+ $D$57 {RIGHT}
+ {LEFT} 7 {PgUp} {DOWN} 3 {DOWN} $D$58 /
{LEFT} 6 {PgUp} {DOWN} 3 {DOWN} $D$58 ~
{MENU} rv {LEFT} 4 ~ {UP} {LEFT} 4 ~
{MENU} e {LEFT} 4 ~
{Blank} F4...H55 ~
{Branch} AB32 ~

{Let} D2, (t-t_f): string ~
{Beep} 4 {GOTO} {?} ~
{MENU} rncReadings . {HOME} {DOWN} 3 ~
{GOTO} D57 ~
@Count(Readings) ~
{MENU} rv ~ D58 ~
{GOTO} A64 ~
+ D58 + 1 ~
{GOTO} A4 ~
{GetNumber} "Enter t failure: ", B2 ~
{GetNumber} "Enter strain at failure: ", C2 ~
{GOTO} D4 ~
{Blank} D4..E55 ~
@Log($B$52-{LEFT} 2) ~
{Let} E2, e_f-e ~
{GOTO} E4 ~
@Log($C$52-{LEFT} 2) ~

subtract 1 from A64

go to first log(time) entry
linear regression

calculate u_i values

go to F4

calculate sums

calculate "range" to "DW"
rang values
for "range" to "DW"

{Let} D2, (t-t_f): string ~
{Beep} 4 {GOTO} {?} ~
{MENU} rncReadings . {HOME} {DOWN} 3 ~
{GOTO} D57 ~
@Count(Readings) ~
{MENU} rv ~ D58 ~
{GOTO} A64 ~
+ D58 + 1 ~
{GOTO} A4 ~
{GetNumber} "Enter t failure: ", B2 ~
{GetNumber} "Enter strain at failure: ", C2 ~
{GOTO} D4 ~
{Blank} D4..E55 ~
@Log($B$52-{LEFT} 2) ~
{Let} E2, e_f-e ~
{GOTO} E4 ~
@Log($C$52-{LEFT} 2) ~
{MENU} c. {LEFT} ~ {DOWN}. {LEFT}{UP 2}{DOWN $D$S58} ~
{MENU} c1..N1 ~ O1..S1 ~
{If $D$S58 > 3}{Branch AB94} ~
{MENU} rndReadings ~
{Quit} ~
delete range Readings end of program

{GOTO} A65 ~
+ A64-1 ~
{MENU} rv ~ A64 ~
{GOTO} D4 ~
{DOWN @Count(Readings)-$A$S64} ~
{MENU} rrx. {UP}{DOWN $A$S64} ~
{UP}{DOWN $A$S64} ~
y{RIGHT}. {UP}{DOWN $A$S64} ~
oA54 ~ g
{RIGHT 2} ~
+$D$S55 + $C$S61*{LEFT 2} - {LEFT} ~
{RIGHT} ~
({LEFT} -{LEFT}{UP})^2 ~
{RIGHT} ~
+ {LEFT 2}^2 ~
{LEFT 2} ~
{MENU} c.{RIGHT 2} ~ {DOWN}.{RIGHT 2}
{UP 2}{DOWN $D$S58} ~
{RIGHT}{DOWN $D$S58}
@sum({UP}. {DOWN 2}{UP $D$S58}) ~
{RIGHT} ~
@sum({UP}. {DOWN 2}{UP $D$S58}) ~
{UP $D$S58}{RIGHT 7} ~
+$D$S58{RIGHT} ~
+$D$S55{RIGHT} ~
+$C$S61{RIGHT} ~
+$D$S57{RIGHT} ~
+{LEFT 12}{DOWN $D$S58} / {LEFT 11}{DOWN $D$S58} ~
{MENU} rv {LEFT 4} ~ {UP}{LEFT 4} ~
{MENU} e{LEFT 4} ~
{Blank F4..H55} ~
{Branch AB90} ~
subtract 1 from A64
to first log(time) entry linear regression
calculate $u_i$ values
go to F4
calculate sums
to range-DW table calculate "range" to "DW"
range values for "range" to "DW"
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**LIST OF ABBREVIATIONS AND SYMBOLS**

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<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<td>A, ( A_p )</td>
<td>Pillar/prism area</td>
<td>m²</td>
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<tr>
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<td>Area of pillar/prism core</td>
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<td>( A_s )</td>
<td>Area of pillar/prism sides</td>
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<tr>
<td>( p_c )</td>
<td>Load on pillar/prism core</td>
<td>N</td>
</tr>
<tr>
<td>( p_s )</td>
<td>Load on pillar/prism sides</td>
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<tr>
<td>R</td>
<td>Correlation coefficient</td>
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</tr>
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<td>Description</td>
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<td>--------</td>
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<tr>
<td>$S_{tot}$</td>
<td>Total load carrying capacity</td>
<td>N</td>
</tr>
<tr>
<td>s</td>
<td>Convergence</td>
<td>m</td>
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<td>$s_r$</td>
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<tr>
<td>SF</td>
<td>Safety Factor</td>
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<td>$SF_{lt}$</td>
<td>Long-term safety factor</td>
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</tr>
<tr>
<td>$SF_{lt}', SF_{lt}''$</td>
<td>Corrected long-term safety factor</td>
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<td>$SHV_i$</td>
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</tr>
<tr>
<td>t</td>
<td>Time</td>
<td>s</td>
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<td>$t_f$</td>
<td>Time at failure</td>
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<tr>
<td>$W_e$</td>
<td>Effective pillar/prism width</td>
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</tr>
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<td>w</td>
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<td>Definition</td>
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<td>Rotation angle of pillar slab</td>
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<td>$\nu$</td>
<td>Poissons' ratio</td>
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<td>Cube compressive strength</td>
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<td>$\sigma_{ov}$</td>
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<td>Residual pillar strength</td>
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<td>$\sigma_1$</td>
<td>Major principle stress</td>
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</tr>
<tr>
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<td>Friction angle</td>
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SAMENVATTING

PIALAARSTABILITEIT EN GROOTSCHALIGE INSTORTING VAN VERLATEN "ROOM AND PILLAR" KALKSTEENMIJNEN IN ZUID-LIMBURG

A. PILAARSTABILITEIT EN GROOTSCHALIGE INSTORTING

In het zuidelijk deel van de Nederlandse provincie Limburg en het aangrenzende deel van Belgisch Limburg zijn uitgebreide "room and pillar" gangenstelsels aangelegd in zachte, tijdens het Boven-Krijt (Maastrichtien) afgezette kalkstenen (calcareniet). De ondergrondse kalksteenwinning is nu vrijwel geheel is beëindigd, en deze mijnen, in de volksmond bekend als "mergelgrotten" en wetstechnisch aangeduid als "mergelgrooves", zijn nu vooral van economisch belang als toeristische attractie. In deze mijnen zijn verscheidene stabiliteitsproblemen aan de orde. Terwijl instabiliteit van het dak en het binnenstromen van aarde uit orgelpijpen slechts lokale gevolgen hebben, kan, tengevolge van een domino-effect, instabiliteit van pilaren uiteindelijk leiden tot de vernietiging van een heel gangenstelsel. Uit verslagen van de meest belangrijke grootschalige instortingen blijkt dat verscheidene hectares gemijnd gebied binnen enkele seconden kunnen worden verwoest. Meestal verlopen er tientallen jaren, of zelfs meer dan honderd jaar, vanaf het einde van de mijnbouwaktiviteiten totdat een grootschalige instorting plaatsvindt. Lange-termijnantasting van de pilaren tengevolge van kruip blijkt hierbij een belangrijke rol te spelen.

Grootschalige instortingen zijn niet alleen fataal voor mensen die zich ophouden binnen het instortende gebied zelf. De krachtige drukgolf, die altijd optreedt bij een instorting, heeft in het verleden ook dodelijke slachtoffers gemaakt in andere delen van de mijn en zelfs buiten, dichtbij de ingang. Daarnaast worden aan het oppervlak breuken en instortingskraters gevormd, die een bedreiging vormen voor huizen en andere bouwwerken.

Dit onderzoek richt zich op de stabiliteit van pilaren vanwege de grootschalige consequenties daarvan. Om de mogelijkheid van een grootschalige instorting vast te stellen, moeten twee vragen worden beantwoord:

344
- grootschalige pilaarstabiliteit: is de capaciteit van alle pilaren samen voldoende om de gesteente- en grondlagen boven het gehele gangenstelsel te ondersteunen, en blijven op de lange termijn de pilaren over het algemeen dus intact en onaangetast door scheurvorming?

- algemene mijnstabiliteit: als de grootschalige pilaarstabiliteit onvoldoende is, dus als het bezwijken van pilaren op uitgebreide schaal plaatsvindt, zijn de daklagen dan sterk en stijf genoeg om desalniettemin een grootschalige instorting te voorkomen.

Dit proefschrift beoogt het inzicht te vergroten in alle aspecten die van belang zijn voor pilaarstabiliteit en grootschalige instorting van de kalksteenmijnen. Het doel hiervan is empirische en analytische methodes en richtlijnen te ontwikkelen die van nut zijn bij het beantwoorden van beide hierboven gestelde vragen voor elke willekeurige kalksteenmijn. Verwacht wordt dat verscheidene uitkomsten van het onderzoek ook relevant zullen zijn voor "room and pillar" mijnen in het algemeen.

B. INDIVIDUELE PILAARSTABILITEIT

De stabiliteit van een afzonderlijke pilaar wordt doorgaans uitgedrukt in een veiligheidsfactor (SF), die gelijk is aan de verhouding van pilaarsterkte en pilaarspanning. Een toename van SF boven de waarde één betekent een steeds grotere pilaarstabiliteit.

De spanning op een pilaar wordt analytisch berekend door middel van de zogenaamde "tributary area" methode. Hierbij wordt het gewicht beschouwd van de gesteente- en grondlagen boven de pilaar en een deel van de gangen die de pilaar begrenzen. De werkelijke pilaarspanning kan echter enigszins afwijken van deze waarde, als gevolg van de complexe mechanische wisselwerking tussen iedere afzonderlijke pilaar en zijn omgeving. Zo zal de kracht op een pilaar die relatief veel "meegeeft" gedeeltelijk worden overgedragen op pilaren met een grotere stijfheid en vooral op het ongemijnde gebied rond het gangenstelsel. De mate van deze herverdeling van krachten neemt toe met de stijfheid van de daklagen. De werkelijke kracht op afzonderlijke pilaren, die variëren in vorm en deformatie-gedrag, kan met analytische of empirische methodes niet precies worden bepaald binnen een willekeurige, drie-dimensionale mijnconfiguratie. Desondanks geeft de "tributary area" methode een redelijk nauwkeurige schatting, die over het algemeen aan de conservatieve kant is.

De sterke van een pilaar, dat wil zeggen de maximale spanning die een pilaar kan verdragen zonder barstvorming, wordt hier geformuleerd als een produkt van de uniaxiale compressieve sterkte (UCS), bepaald op een gesteente-kern op laboratorium schaal, en factoren die deze waarde aanpassen aan de grootte en vorm van de pilaar in de mijn.
De UCS van de calcareniet varieert doorgaans van 1,5 tot 4 MPa. Deze lage waardes kunnen worden toegeschreven aan de hoge porositeit van gemiddeld 45 % en de zwakke cementatie van de korrels onderling. Deze waardes hebben betrekking op gesteente-monsters met het natuurlijke vochtgehalte van 7 tot 8 %. Niet alleen voor uniaxiale belaste cilindrische kernen, maar ook voor pilaren in de mijн is de spanningstoestand zodanig, dat het bezwijkgedrag bros is. Er bestaan sterke aanwijzingen dat het broos bezwijken van de calcareniet gecontroleerd wordt door de vorming en groei van intergranulaire micro-scheurjtjes, hetgeen voor dit type gesteente neerkomt op het verbreken van de zwakke cement-verbindingen tussen de korrels. Bij het bezwijken onder compressie groeien de micro-scheurjtjes aan elkaar tot een macroscopisch schuifvlak.

Het effect van grootte en vorm op de sterkte werd vastgesteld door middel van uniaxiale compressie-experimenten op calcareniet prisma's in het laboratorium. Er werd geen schaal-effect vastgesteld. Verder kan, afgezien van geïsoleerde diaklazen, de calcareniet op de schaal van een mijn worden beschouwd als een min of meer continue gesteente. Daarom wordt voorgesteld een schaal-factor van één te gebruiken. Tijdens de compressie-experimenten, die werden uitgevoerd met een constante vervormingsnepheid, konden drie fases worden onderscheiden (Fig. 1). Tijdens fase I nam de spanning toe en er vormden zich geen macroscopische barsten. Vervolgens werd de pieksterkte bereikt en verschenen bij de hoeken de eerste axiale scheurjtjes. Gedurende fase II groeiden de barsten in vertikale richting en nam de sterkte van het prisma af. Een verandering in de helling van de spanning-vervormings curve markeerde het begin van fase III. Het spanningsniveau dat dan werd bereikt werd gedefinieerd als de residuaire sterkte. Men nam verder voornamelijk het verwijden van al gevormde barsten waar. Aan het einde van het experiment bleek het prisma te zijn opgesplitst in vier losstaande zijkanten rond een zandloper-vormige kern (Fig. 2). Ook werd de relatie duidelijk tussen de van de buitenkant zichtbare barsten en de deformatie-structuren binnen in het prisma.

De pieksterkte bleek lineair toe te nemen met de breedte/hoogte verhouding (W/H). Dit ging ook op voor de residuaire sterkte. Echter, voor pilaren met een W/H van meer dan vier werden piek- en residuaire sterkte het best beschreven door een logaritmische functie van W/H. Een analytisch model werd opgesteld dat, overeenkomstig de experimentele uitkomsten, liet zien hoe bij toenemende W/H de residuaire sterkte de pieksterkte benadert. Voor pilaren met een onregelmatiger horizontale omtrek werden formules ontwikkeld die de sterkte beschrijven als functie van oppervlak, omtrek en hoogte, in plaats van breedte en hoogte. Numerieke experimenten lieten zien dat de elastische spanningsverdeling binnen in het laboratorium geteste prisma's waarschijnlijk iets gunstiger is dan binnen werkelijke pilaren in een mijn. Daarom zouden de hier afgeleide relaties de sterkte van pilaren enigszins kunnen overschatten. Op basis van de experimenten werd verder een classificatie van pilaar-schade voorgesteld, die bestaat uit vier niveau's.

Tot hier toe was de bepaling van pilaarsterkte gebaseerd op laboratorium-experimenten van korte duur. Vervolgens werd het lange-termijn-, of kruipgedrag, van cilindrische- en prismatische calcarenietmondsters bestudeerd. Zoals de meeste
gesteentes onder brosse condities, vertoonde de calcareniet, bij belasting onder een constante spanning, na instantane vervorming een fase van afnemende kruipsnelheid. Deze fase ging, wanneer de opgelegde spanning groter was dan een bepaalde minimum waarde, de lange-termijnsterkte, via een buigpunt over in een fase van toenemende kruipsnelheid. Deze laatste fase leidde uiteindelijk tot het bezwijken van het monster.

Zowel de af- als toenemende kruipsnelheid bleek gerelateerd te zijn aan tijd en opgelegde spanning, verheven tot een bepaalde macht. De aanwezigheid van water deed de kruipsnelheid forse toenemen. Een duidelijke relatie werd gevonden tussen bezwijk tijd en opgelegde spanning. Een kleine toename van de spanning kon de bezwijk tijd enkele orders van grootte doen afnemen. Ook kon worden aangetoond dat de parameters voor de fase van afnemende kruipsnelheid gecorreleerd zijn aan de bezwijk tijd. Dit betekent dat enkele metingen tijdens het begin van het kruipproces al de bezwijk tijd kunnen aangeven. Het is nog niet mogelijk om op basis van de nu verzamelde experimentele gegevens voor calcareniet pilaren, ongeacht hun vorm, de lange-termijnsterkte vast te stellen. De kruip-experimenten leverden wel een schatting op voor pilaren met een W/H ratio die kleiner is dan 0.6, namelijk 60 % van de (korte-termijn) pieksterkte. Voor grotere W/H ratio's kon worden berekend dat een conservatieve schatting van de lange-termijnsterkte wordt verkregen door de residuaire sterkte te hanteren.

C. GROOTSCHALIGE PILAARSTABILITEIT EN VALIDATIE VAN BEREKENDE VEILIGHEIDSFACTOREN

Om over een eenvoudig criterium voor grootschalige pilaarstabiliteit te beschikken, werd de totale veiligheidsfactor SF\textsubscript{tot} gedefinieerd. Verder werden alle fouten bij het bepalen van veiligheidsfactoren geëvalueerd. Hierbij kwamen zowel toevallige fouten, bijvoorbeeld ten gevolge van onbekende variaties in UCS, als systematische fouten, bijvoorbeeld ten gevolge van het domino-effect, aan de orde. Deze fouten maakten een correctie van de berekenende veiligheidsfactoren noodzakelijk. Dit was mogelijk door de berekenende veiligheidsfactoren te vergelijken met de waargenomen graad van pilaarschade. Zowel de individuele- als de totale veiligheidsfactoren konden worden aangepast voor systematische fouten door middel van een correctiefactor K. De toevallige fouten veroorzaakten dan nog een zekere spreiding van gecorrigeerde veiligheidsfactoren voor afzonderlijke pilaren. Maar bij het bepalen van de totale veiligheidsfactor bleken de toevallige fouten aanzienlijk af te nemen bij een toenemend aantal pilaren, en dus kon de grootschalige pilaarstabiliteit met grotere precisie worden bepaald. Verder zijn de beste resultaten te verwachten wanneer lange-termijnpilaarsterktes worden gebruikt.

D. ONDERZOEK IN DE GEULHEMMER GROEVE

In de Geulhemmer Groeve bleek de variatie van de UCS op een 50 meter-schaal aanzienlijk te zijn. Er werd een ruwe correlatie gevonden tussen UCS en Schmidt-
hammer waardes (type PT), in een poging een tijd- en kostenbesparende methode te ontwikkelen om in-situ de UCS te bepalen. Helaas bleek voor de bepaling van pilaarsterkte de correlatie niet nauwkeurig genoeg te zijn.

In dit gangenstelsel kon voor de verschillende delen afzonderlijk de ouderdom worden vastgesteld. Hierdoor zou het mogelijk kunnen zijn om de waargenomen pilaarschade te relateren aan ouderdom en berekende veiligheidsfactor. Klasse 1 (intact)- en klasse 3 (barsten van boven naar beneden) pilaren bleken van elkaar te worden gescheiden door zowel een initiële, korte-termijnveiligheidsfactor van 2 als een ouderdom van ongeveer 110 jaar. Dientengevolge kon niet met zekerheid worden vastgesteld of de mate van pilaarverslechtering in dit gangenstelsel afhankt van de verhouding tussen pilaarsterkte en "tributary area"-spanning of van de hoeveelheid kruipeervorming of van beide. Desondanks lijkt de laatste mogelijkheid het meest waarschijnlijk. Wat betreft lange-termijnsterkte, bleken klasse 1 en klasse 3 pilaren duidelijk van elkaar te worden gescheiden door een lange-termijnveiligheidsfactor van één.

De initiële veiligheidsfactoren werden gevalideerd, en kruipe werd daarbij automatisch verdeesteerd, door vermenigvuldiging met een zodanige correctiefactor dat deze voor gebarsten pilaren verdeeld kwamen te liggen rond een waarde van één. Voor de grote meerderheid van de intacte pilaren was de gecorrigeerde veiligheidsfactor groter dan één. De lange-termijnveiligheidsfactoren hoefden nauwelijks gecorrigeerd te worden: hun correctiefactor bedroeg ongeveer één.

E. ONDERZOEK IN DE HEIDEGROEVE

In deze mijn vond de meest recente instorting plaats (1987, 0.4 ha). Bij een onderzoek ondergronds bleek dat tijdens de instorting breuken waren gevormd aan de rand van het instortingsgebied, waardoor het gehele kalksteenadak naar beneden is gekomen. Deze breuken helen in de richting van het niet-ingestorte deel en doorsnijden het complete kalksteenadak. Een dergelijke instorting wordt hier gedefinieerd als type A (Fig. 3). Afhankelijk van de "cave angle" neemt, na de breukbeweging, de spanning op het aan de instorting grenzende deel van het gangenstelsel enigszins toe. De omtrek van het verzakkingsgebied aan het oppervlak, die wordt gevormd door afschuivings-breuken, valt ongeveer samen met die van de ondergrondse instorting.

De instorting en de locatie waar deze plaatsvond binnen de mijn kunnen onmogelijk worden verklaard door de individuele veiligheidsfactoren van de pilaren. Verder zijn er breuken waargenomen die zich uitstrekken in de vloer van het gangenstelsel, en is bekend dat er dichtbij de Heidegroeve gangenstelsels zijn aangelegd op een lager niveau. Al met al bestaan er sterke aanwijzingen dat de instorting van de Heidegroeve, die enkele jaren eerder door niemand was verwacht, is veroorzaakt door de instorting van een onderliggend gangenstelsel.
F. ONDERZOEK IN DE FALLENBERG

De instorting van 1920 in de Fallenberg (7 ha) is anders van karakter. Hier boog het kalksteendak aanzienlijk naar beneden door zonder dat dit aan de rand van de instorting over de gehele dikte doorbrak. Dit soort instorting wordt gedefinieerd als type B (Fig. 3). Omdat de continuïteit van het kalksteendak min of meer bleef gehandhaafd, moet er vóór, na en vooral tijdens de instorting spanning zijn overgedragen van het instortingsgebied naar zijn directe omgeving. Hierdoor zijn ook pilaren buiten het instortingsgebied verslechterd.

Door onderzoek in het instortingsgebied werd duidelijk dat de instorting tot een einde kwam voornamelijk ten gevolge van de zijdelingse steun van de pilaarwanden door de van pilaren en dak afkomstige gesteentefragmenten. Een analyse van veiligheidsfactoren liet zien dat de pilaarschade binnen het aangrenzende gangenstelsel, dat bekend is als de Jezuïetenberg, uitsluitend is veroorzaakt door de instorting.

G. ALGEMENE MIJNSTABILITEIT EN PRACTISCHE TOEPASSINGEN VAN HET ONDERZOEK

Om inzicht te krijgen in de algemene mijnstabiliteit, werden de dikte van het gehele kalksteendak en de breedte van de mijn in een grafiek tegen elkaar uitgezet voor zes ingestorte gebieden en voor vier gebieden die ondanks het bezwijken van de pilaren voor instorting gespaard waren gebleven (Fig. 4). De punten in de grafiek kunnen duidelijk van elkaar worden gescheiden door een lijn, die bij benadering de maximale spanningsboog weergeeft voor een gegeven dikte van het kalksteendak. De grafiek ondersteunt ook de overtuiging dat de instorting van de Heidegroeve te wijten is aan speciale omstandigheden, zoals de aanwezigheid van een onderliggend instabiel gangenstelsel. De meeste instortingen zijn van het type A. Beide type B instortingen (9 en 10, Fig. 4) worden gekenmerkt door verreweg de grootste overspanning. Het beschrijven van de mogelijkheid van een grootschalige instorting in termen van pilaar- en dakstijfheid bleek voor deze kalksteenmijnen geen geschikte methode te zijn.

Op basis van de resultaten van dit proefschrift wordt een systeem voorgesteld ter bepaling van de mogelijkheid van een teokomstige grootschalige instorting. De toepassing van dit systeem moet altijd worden gecombineerd met een visuele inspectie van pilaarschade. Afhankelijk van de uitkomst, worden bepaalde maatregelen aanbevolen. Deze kunnen variëren van slechts regelmatige visuele inspectie tot het onmiddellijk afsluiten van het gangenstelsel. In tussenliggende gevallen wordt aanbevolen meetapparatuur te installeren om de ontwikkeling van pilaarvervorming kwantitatief te registreren. Op basis van deze metingen kan het noodzakelijk blijken om ondergrondse verstevigingen aan te brengen. Het systeem kan van nut zijn omdat het een relatief snelle en goedkope methode is om vast te stellen of de veel duurdere en veel tijd in beslag nemende (één tot verscheidene jaren durende) registratie van vervorming door middel van meetapparatuur noodzakelijk
is. Verder kunnen de resultaten van dit proefschrift worden gebruikt om de meest geschikte meetopstelling te selecteren, ter registratie van pilaarvorming, om de meetresultaten te interpreteren, en om een economisch haalbaar ondersteuningsplan te ontwerpen. Het systeem kan vooral worden verbeterd door een betere kennis van de UCS-verdeling en van het kruipgedrag van verschillend gevormde pilaren.

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Roland F. Bekendam
**Fig. 1** Stress-strain diagram and crack development for a compression test on a calcarenite prism of a width/height ratio of one.

**Fig. 2** General fracture geometry of failed prisms of width/height ratio’s of one and more.
Fig. 3 De twee belangrijkste soorten grootschalige instortingen in de kalksteenmijnen.  
*Fig. 3 The two main types of large-scale collapse in the limestone mines.*

![Diagram of pillar stability and collapse types](image)

Fig. 4 Dikte van het kalksteendak versus breedte van het gangenstelsel. Op grond van de tot nu toe bestudeerde gevallen kan een lijn worden getrokken, die bij benadering de maximale spanningsboog weergeeft voor een gegeven dikte van het kalksteendak.  
*Fig. 4 Rock overburden thickness vs. mine span. A line, based on the studied cases so far, is drawn as an approximate limit between collapsed and not collapsed areas.*
SUMMARY

PILLAR STABILITY AND LARGE-SCALE COLLAPSE OF ABANDONED ROOM AND PILLAR LIMESTONE MINES IN SOUTH-LIMBURG, THE NETHERLANDS

A. PILLAR STABILITY AND LARGE-SCALE COLLAPSE

Extensive room and pillar mines have been excavated in weak calcarenites of Maastrichtian age (Upper-Cretaceous) in the southern part of the province of Limburg, The Netherlands, and in adjacent parts of Belgium. In these abandoned mines, which now mainly serve as an economically important tourist attraction, stability problems are not uncommon. While instability of the roof and earth inflow from organ pipes are of mere local importance, instability of pillars may eventually give rise to the destruction of a whole mine, due to a domino-effect. Accounts of the major large-scale collapses in the past revealed that during such an event several hectares of mined area may be ruined within a few seconds. A time span of tens of years or even more than hundred year usually exists between the end of the mining activities and the large-scale collapse. In this regard long-term pillar deterioration due to creep proves to play an important role.

Large-scale collapses are not only fatal to people inside the collapsing area. The strong air blast, always generated by a collapse, also claimed casualties elsewhere in the mine and even outside, near the mine entrance. Additionally, faulting and sinkhole formation arise at the surface, threatening houses and other surface structures.

Accordingly, this research concerns the stability of pillars. To assess whether a large-scale collapse of a mine is possible, two questions impose:

- large-scale pillar stability: is the capability of all pillars together sufficient to support the total overburden load, and, accordingly, will the pillars generally remain intact and free of stress-induced fractures on the long-term?

- general mine stability: if large-scale pillar stability is insufficient, i.e. if widespread pillar failure occurs, are the roof strata strong and stiff enough to prevent a large-scale collapse?
The aim of this work is to increase the understanding of all aspects related to pillar stability and large-scale collapse of the calcarenite mines. This should result in empirical and analytical methods and guide-lines, which are useful in answering both questions posed above for any given calcarenite mine. Various results may be relevant to room and pillar mines in general.

B. INDIVIDUAL PILLAR STABILITY

The stability of an individual pillar is usually expressed by a safety factor (SF). This property is equal to the ratio of pillar strength and pillar stress. An increase of SF above one corresponds with an increase of pillar stability.

Pillar stress is calculated analytically, by the tributary area method. This considers the overburden load directly over the pillar and the portion of the galleries at its perimeter. However, actual pillar stress may deviate from this value to some extent due to the complex mechanical interaction of each singular pillar and its mine environment. For example, the load on relatively "soft" pillars tends to be partially relieved and transferred towards stiffer pillars and particularly to the mine abutments. The extent of this reallocation of pillar load increases with the stiffness of the roof strata. The actual loads on individual pillars of varying shape and load-deformation behaviour, forming part of an arbitrary, three-dimensional mine layout, cannot be determined exactly by analytical or empirical methods. Nevertheless, the tributary area method gives a reasonable estimation which is, on the average, on the conservative side.

Pillar strength, i.e. the ultimate stress level a pillar can sustain without fracturing, is expressed here as a product of the uniaxial compressive strength (UCS), measured on a laboratory scale, and factors which adapt this value to the size and shape respectively of the actual mine pillar.

The UCS of the calcarenite, determined on cylindrical specimens, generally ranges from 1.5 to 4 MPa. These low values can be ascribed to the high porosity of 45% on the average and the weak intergranular cementation. The values apply to samples at the natural moisture content of 7 to 8%. Not only for uniaxially loaded cylindrical cores, but also for mine pillars the stress state is such, that failure is brittle. Evidence exists that brittle failure of calcarenite is controlled by intergranular microcracking, which for this rock type corresponds to breakage of the weak intergranular cement bonds. At failure under compression microcracks link up to form a macroscopic plane of shear rupture.

The size and shape effect were assessed by uniaxial compression experiments on calcarenite prisms in the laboratory. Measurements did not indicate a size effect. Apart from some isolated joints, the calcarenite is a more or less continuous rock on the mine scale. Accordingly a shape factor of one is used. For the compression experiments, performed at a constant strain rate, three phases could be recognized (Fig. 1). During phase I the stress increased and no macroscopic fracturing occurred.
Then peak strength was attained and the first axial cracks appeared, starting from the prism corners. During phase II cracks grew in a vertical direction and the prism strength decreased. A change in stress-strain slope marked the onset of phase III. The subsequent stress level was denoted as residual strength. Mainly widening of existing cracks was to be observed. At the end of the test an hour-glass shaped core and separated prism sides (Fig. 2) could be observed. Now the relationship between visible fracturing at the exterior and deformation structures inside the prism became apparent.

Peak strength is linearly related to prism width/height ratio (W/H). This proved to apply to residual strength as well. For prisms of a W/H ratio exceeding four, peak- and residual strength must be described by a logarithmic function of this ratio. An analytical model was developed which, in agreement with the experimental data, showed how residual strength approaches peak strength at increasing W/H ratio. For prisms of a more irregular horizontal perimeter relationships were developed which delineate strength as a function of prism area, circumference and height, instead of width and height. Numerical experiments showed that the elastic stress distribution may be slightly more favourable for the laboratory setup than for the actual mine situation. Accordingly, the derived shape functions might overestimate pillar strength to some extent. On the basis of the relation between fracture development and stress-strain diagrams, a classification of pillar damage was proposed, comprising four stages.

So far, the assessment of pillar strength was based on laboratory experiments of short duration. Subsequently, the creep behaviour of cylindrical and prismatic calcarenite samples were studied. Like most rocks under brittle conditions the calcarenite, loaded at a constant stress, showed instantaneous deformation followed by a stage of decelerating creep. This stage passed into a stage of accelerating creep through an inflexion point if the applied stress exceeded a certain minimum level or "long term strength". The accelerating creep phase ended in shear failure of the sample.

Both decelerating and accelerating creep rate proved to be related to stress and time by power laws. The presence of water increased creep rates considerably. The time to failure was evidently related to the applied stress level. A small increase in stress could reduce the time to failure by one or more orders of magnitude. It could also be demonstrated that the decelerating creep parameters established during the first creep strain increments are well correlated to the time to failure. This means that a few measurements in the beginning of the creep process may indicate the time to failure. Not enough experimental data were available to directly establish the long-term strength of a calcarenite pillars of arbitrary W/H ratio. It was estimated that, for pillars of W/H of less than 0.6, 60 % of the peak strength should be taken as the long-term strength. For higher W/H ratio's it was reasoned that long-term strength can be conservatively estimated by choosing residual strength.
C. LARGE-SCALE PILLAR STABILITY AND VALIDATION OF CALCULATED SAFETY FACTORS

The total safety factor $SF_{\text{tot}}$ was formulated, as a simple criterion to express large-scale pillar stability. All random errors, e.g. due to unknown UCS variations, and systematic errors, e.g. due to the domino effect, of the safety factor calculations were evaluated. Due to these errors validation of calculated safety factors was necessary. A method of such a validation was proposed, which is based on a comparison of calculated safety factors of individual pillars with the degree of observed pillar damage. An adjustment of both individual and total safety factors for systematic errors could be made by applying a correction factor $K$. A certain spread in corrected safety factors for individual pillars was brought about by a number of random errors. But, when determining a total safety factor, the random errors proved to be reduced considerably at an increasing number of pillars, accordingly with a greater precision of the assessment of large-scale pillar stability as a result. The validation is expected to give the best outcome when long-term strength values are utilized.

D. FIELD STUDIES IN THE GEULHEMMER GROEVE

In the Geulhemmer Groeve the variation of UCS on a 50 m scale proved to be considerable. A correlation was established between UCS and Schmidt-hammer type PT values, in an attempt to develop a quick and inexpensive method of in-situ UCS determination. Unfortunately, the error of UCS estimation from SHV values proved to be too large to determine pillar strength with sufficient precision.

In this mine the age of the various parts could be assessed. This could be an opportunity to relate actual damage to the age and calculated safety factor for each individual pillar. Class 1 (intact) and class 3 (cracks from top to bottom) pillars proved to be divided by both an initial (short-term) safety factor of two and an age of 110 years. Accordingly, it could not be confirmed with certainty whether the degree of pillar deterioration in this mine depends on the ratio of short-term strength and tributary area stress or on the amount of creep deformation or both. Nevertheless, the last option seems to be most likely. Applying the calculated long-term strength, class 1 and class 3 pillars are fairly well separated by a long-term safety factor of one.

Initial safety factors were validated, and creep was necessarily incorporated, by the multiplication by such a correction factor, that those for fractured pillars became distributed about one. For the majority of intact pillars corrected safety factors were in excess of one. For long-term safety factors hardly any correction was necessary: their correction factor proved to be close to one.
E. FIELD STUDIES IN THE HEIDEGROEVE

In this mine the most recent collapse occurred (1987, 0.4 ha). A field study revealed that downward movement of the rock overburden had occurred along collapse-induced faults at the collapse limit. These faults are dipping towards the area still standing and intersect the whole calcarenite roof mass. Such a collapse is denoted here type A (Fig. 3). According to the cave angle the load on the adjacent mine section increases somewhat subsequent to faulting. The perimeter of the subsidence area at the surface, formed by normal faults, corresponds more or less with the collapse area underground.

The collapse and its location inside the mine could not possibly be explained by the individual pillar safety factors. Additionally, faults were observed which extend into the floor, and mines, excavated at a lower mining level, are known to exist in the immediate vicinity. There is strong evidence to believe that the collapse of the Heidegroeve, which was totally unexpected some years before, was brought about by the collapse of an excavation beneath.

F. FIELD STUDIES IN THE FALLENBERG

The 1920 collapse of the Fallenberg (7 ha) is of a different character. Here the calcarenite overburden moved downward by mere deflection, without disruption over its full height at the collapse margin. This category of collapse is defined type B (Fig. 3). Since the lateral continuity of the calcarenite roof mass was more or less maintained, load transfer from the collapse towards its direct vicinity must have occurred, before, after and particularly during the event. This gave rise to an increase of pillar deterioration outside the collapse area.

Visits inside the collapse revealed that the collapse was stopped mainly as a result of the confining effect of pillar and roof debris, which support the pillar walls. An analysis of safety factors showed that pillar damage inside the adjacent, still standing part of the mine, which is known as the Jezuiëntenberg, is caused exclusively by the collapse.

G. GENERAL MINE STABILITY AND PRACTICAL APPLICATIONS OF THE RESEARCH

In order to study general mine stability, mine span and rock overburden height were depicted for six collapsed areas and four areas of failed pillars which did not collapse (Fig. 4). The data points can be clearly separated by an approximate line, which represents the maximum span of a pressure arch for a given rock overburden height. This graph also supports the belief that the collapse of the Heidegroeve was brought about by special circumstances, as the presence of unstable workings underneath. Most collapses are of type A. Both type B collapses (9 and 10, Fig. 4) show by far
the greatest mine span. The assessment of collapse potential in terms of pillar and strata stiffnesses did not prove to be an adequate method for the calcarenite mines.

On the basis of the results of this thesis a system is proposed to assess the possibility of a future large-scale collapse. The application of the system should always be accompanied by a visual inspection of pillar damage. Depending on the outcome of the method, distinct measures are advised. Such measures can vary from only continuing regular visual inspections to immediate closure of the mine. In intermediate cases measuring equipment should be installed to monitor the development of pillar strain quantitatively. Such measurements may indicate the necessity to apply underground support. The system may be useful because it represents a relatively quick and inexpensive method to assess if the much more costly and time-consuming (one to several years) monitoring of strain by means of measuring equipment is necessary. Additionally, the results of this thesis can be used to design and interpretate pillar strain measurements and to design economically feasible support measures. The system of large-scale collapse potential assessment can be improved particularly by a better knowledge of the distribution of UCS and of the creep behaviour of pillars of various shapes.

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