INTERPRETATION AND MODELLING: GEOTECHNICAL AND THERMAL, TO SUPPORT THE DEVELOPMENT OF AN INTEGRATED SITE DESCRIPTIVE MODEL

Client: NUCLEAR DECOMMISSIONING AUTHORITY (NDA)

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Interpretation and Modelling: Geotechnical and Thermal, to Support the Development of an Integrated Site Descriptive Model

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EXECUTIVE SUMMARY

The Radioactive Waste Management Directorate (RWMD) of the Nuclear Decommissioning Authority (NDA) is undertaking work to develop an understanding of approaches to carrying out a site characterisation programme. RWMD proposes to develop and present the information derived from site characterisation activities for a geological disposal facility in the form of a single integrated Site Descriptive Model (SDM), which will describe the geometry, properties of the bedrock and water, and the associated interacting processes and mechanisms, which will be used to address the information requirements of all end users. The SDM will be divided into parts comprising clearly defined disciplines:

- Geology;
- Hydrogeology;
- Hydrochemistry;
- Geotechnical;
- Radionuclide Transport Properties;
- Thermal Properties; and
- Biosphere.

Work is being undertaken to provide descriptions of the techniques available for the modelling and interpretation of site characterisation data in each of these disciplines. This report provides a review of approaches that have been adopted to geotechnical and thermal data interpretation and modelling.

The approaches to geotechnical and thermal data interpretation and modelling at a number of sites have been reviewed. These case studies have been presented in terms of the three generic host rock types being considered by RWMD, which are:

- Higher-strength rocks: spent fuel disposal in Swedish granite (SKB), spent fuel disposal in Finnish granite (Posiva), intermediate-level waste disposal in higher-strength rock in the UK (Nirex);
- Lower-strength sedimentary rocks: spent fuel and high-level waste disposal in clay in Switzerland (Nagra); and
- Evaporites: transuranic radioactive waste disposal in salt in the US (US DOE).

Key findings of the reviews are as follows:

- Care should be taken in deciding the simulation scale and the simulation volume given the different scales of measurements and rock volumes of importance to rock mechanics and thermal modelling. That is, it is important to understand the upscaling factor, especially concerning rock strength.
- For geotechnical site understanding, a strong link to the geological model which provides the geometric framework for expanding point values into volumes is important.
- Understanding the rock stress regime at a site may be complicated if the site has a complex deformation history.
- Statistical and geostatistical approaches are useful for determining the geotechnical and thermal properties at a site and understanding the scale effects.
- In lower-strength sedimentary rocks, interpretation and modelling is likely to focus on the development of models that describe rock behaviour by using micro-mechanical models, short and medium-term deformation models and long-term deformation models (creep) as well as linear and bi-linear rock mechanical models.
- High-resolution 3-D seismic reflection surveys can be used to characterise and interpret geotechnical and thermal parameters, especially in a homogeneous formation.
- Hydro-mechanical and thermo-mechanical couplings should be taken into account in the interpretation of data from lower-strength sedimentary rocks.
- Laboratory analysis, constitutive model formulation, and numerical modelling are likely to be required in the development of thermal/structural codes to understand the creep deformation of evaporites. The models are likely to evolve significantly as information from underground excavations becomes available.
Derivation of parameter values and statistical analysis can generally be achieved using commercially-available software, such as Excel, GSLIB, ISATIS and MATLAB. However, programme-specific algorithms and routines are used. Therefore, while the software environment can be purchased, it will become more programme-specific and less transferable over time.

A common suite of commercial codes for geomechanical analysis and modelling is provided by Itasca (an international engineering company), including FLAC, PFC and DEC in 2-D and 3-D. The PFC codes concern micromechanical analysis of geomaterials and particulate systems, while the FLAC and DEC codes are used to simulate the large-scale behaviour of structures under loads. The distinction between FLAC and DEC is continuum versus discrete element modelling. The former is more generally applied in the lower-strength rock environments, but all of the codes have been applied across all of the rock types.

Site-specific proprietary software has been used for site-specific models, which have been developed for complex geomechanical simulations. The models tend to rely on observations in underground excavations for parameterisation and/or verification and, as such, may be developed to a large degree later in a site investigation programme. However, by modelling such disturbances, it is possible to improve understanding of large-scale geomechanical behaviour at a site.

A review of interpretation and modelling of rock mechanical and thermal data in other sectors (civil engineering, oil and gas and geothermal) identified a range of different data interpretation approaches and tools, but these methods generally focus on understanding specific characterisation pertinent to the engineering or resource sector to which they are being applied. For example, geomechanical and hydromechanical models such as Poly3D, Visage and ANSYS are used in the oil and gas industry to predict changes in the stress state during reservoir production. These specialist tools have limited applicability to GDF site investigation and SDM development, but they may be applicable to later stages of GDF excavation, assessment and monitoring.

Use of the modelling tools for interpreting geotechnical data from surface-based site investigations requires a significant degree of experience and there may be a lack of such experience specifically within the nuclear decommissioning industry in the UK. The necessary experience does exist in other industries, such as the oil and gas industry, mining and civil engineering, and should be readily transferable, but there are concerns regarding the depth and availability of such resources when required for GDF site investigation in the UK. A large number of Earth scientists is being produced in emerging economies, but there is not expected to be any surplus in global supply.

Broadly, the skills required to interpret thermal and geotechnical data are similar for the range of GDF host rock types being considered by RWMD. However, the development of SDMs for evaporite and lower-strength sedimentary rocks specifically requires an understanding of the elasto-plastic behaviour of rocks and specific skills in the development of rock creep models. For example, the salt creep model developed as part of the transuranic radioactive waste disposal programme in the US involved a highly skilled research team and there is a limited capacity for such modelling worldwide.

The overall approach to site characterisation will depend on the host rock selected and the extent to which the proposed site has undergone previous investigation and characterisation. It is anticipated that data will be obtained from seismic surveys, borehole investigations, down-hole logging and rock sampling for laboratory tests. A strategic approach to interpreting and modelling geotechnical and thermal data has been set out that describes the different techniques to be used to derive each parameter. Many of the techniques to derive rock mechanics and thermal properties involve the use of standard, well-established data interpretation methods, although such techniques have limits on applicability that depend on the assumptions underpinning the models. As data become available, large-scale numerical modelling would be undertaken to derive geotechnical and thermal parameters on a large scale that respects geological structure, such as deformation zones, and accounts for coupling between different between processes, uncertainty and spatial variability.
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1 INTRODUCTION

1.1 BACKGROUND

The Radioactive Waste Management Directorate (RWMD) of the Nuclear Decommissioning Authority (NDA) is responsible for implementing the geological disposal of higher-activity radioactive waste. Higher-activity radioactive wastes include high-level waste (HLW), intermediate-level waste (ILW) and the small fraction of low-level waste (LLW) that cannot be managed under the UK Government policy for near-surface disposal of solid LLW (Defra et al. 2008). In addition to these wastes, spent fuel, separated plutonium and separated uranium may need to be managed through geological disposal. These wastes and other materials are included in a Baseline Inventory of materials potentially requiring geological disposal (Defra et al. 2008).

Geological disposal facility (GDF) implementation will involve identification, study and investigation of a candidate site or sites as part of the Managing Radioactive Waste Safely (MRWS) programme (Defra et al. 2008). The MRWS programme is being undertaken in stages, which will include a programme of surface-based investigations at candidate sites to identify a preferred site for underground investigations. The surface-based investigations would be required to acquire and interpret information on the geological, hydrochemical, hydrogeological, geotechnical, thermal, and environmental characteristics of the site(s). The information acquired would be used as an input to the development of the GDF safety case, for engineering design of the disposal facility, and to demonstrate confidence to the key stakeholders that the potential disposal facility site is adequately understood.

As part of the on-going GDF implementation programme, RWMD has been undertaking work to develop an understanding of approaches to carrying out a site characterisation programme (NDA 2011). The objectives of this Site Characterisation Project are to:

- Develop approaches to the design and implementation of needs-driven, information-led investigations (surface-based and underground investigations, etc.) to meet the needs of information users within RWMD, Regulators and other key stakeholders;
- Undertake sufficient preparatory work such that, when required, the surface-based site investigations can be implemented in a timely and efficient manner; and
- Ensure that the necessary arrangements are in place so that resources are available to support the subsequent implementation of site characterisation.

RWMD proposes to develop and present the information derived from site characterisation activities for a geological disposal facility in the form of a single integrated Site Descriptive Model (SDM). This is a model describing the geometry, properties of the bedrock and water, and the associated interacting processes and mechanisms, which will be used to address the information requirements of all end users. Such an approach will ensure that:

- The understanding of the different aspects of the geosphere such as the geology, hydrogeology and hydrochemistry is developed in a consistent manner; and
- The different end users base their design and assessments on the same understanding and evidence base.

There are several elements to the preparation of a Site Descriptive Model, including:

- Definition of the volume of ground that needs to be included in the model;
- Subdivision of the model into geometric units so as to permit the description of spatial variability in a meaningful manner; and
- Assignment of parameters (values and/or statistical distributions) to the defined geometric units.

RWMD anticipates that the SDM will be divided into parts comprising clearly defined disciplines which may form either chapters or discipline-based models such as (NDA 2011, §3.5.1.3):

- Geology;
- Hydrogeology;
- Hydrochemistry;
- Geotechnical;
Radionuclide Transport Properties; 
Thermal Properties; and
Biosphere.

The SDM will describe the current situation at the site and where relevant the historical development of conditions at the site to support the conceptual understanding. It will not include prediction of the future evolution of the conditions at the site which will be included within the safety assessment work.

RWMD has identified the need for work to further understand approaches to interpreting and modelling site characterisation data that would be needed to support the development of an SDM. RWMD has therefore commissioned work to provide descriptions of the state-of-the-art processes and techniques available for the modelling and interpretation of site characterisation data in each of the disciplines identified above.

This report provides a review of approaches to geotechnical and thermal data interpretation and modelling. Separate reports have been prepared for the other disciplines.

1.2 OBJECTIVES
The objectives of this report are to provide RWMD with an account of:

- The processes and tools that have been used in international radioactive waste management programmes for processing, interpreting and modelling the geotechnical and thermal data acquired through site characterisation;
- Any additional processes and tools that are used in other sectors for processing, interpreting and modelling geotechnical and thermal data acquired through site characterisation that may be applicable to a UK-based waste management site characterisation programme;
- The availability of resources (tools and specialist practitioners) for undertaking such a programme of processing, modelling and interpretation in the UK;
- The extent to which the required resources may vary in response to variations in the geological environment at the site(s) in the UK that are being characterised; and
- The identification of any gaps in available resources to support a UK-based site characterisation programme.

1.3 SCOPE AND APPROACH
To meet the project objectives, descriptions have been provided of the various techniques that have been adopted by radioactive waste management organisations in the UK and overseas to translate acquired data into understanding to support site descriptions as part of their site characterisation programmes. The review covers site characterisation work undertaken in support of the development of disposal facilities in the different types of geological environment being considered by RWMD. That is, programmes to develop GDFs in the following three types of host rock have been considered:

- Higher-strength rocks, which would typically comprise crystalline igneous and metamorphic rocks or geologically older sedimentary rocks where any fluid movement is predominantly through discontinuities (e.g. fractures);
- Lower-strength sedimentary rocks, which would typically comprise geologically younger sedimentary rocks where any fluid movement is predominantly through the rock matrix (although fluid movement may not be confined to the rock matrix); and
- Evaporites, which would typically comprise anhydrite (anhydrous calcium sulphate), halite (rock salt) or other evaporites that result from the evaporation of water from water bodies containing dissolved salts.

The GDF programmes studied are as follows:

- Spent fuel disposal in Swedish granite (SKB);
- Spent fuel disposal in Finnish granite (Posiva);
- ILW disposal in higher-strength rock in the UK (UK Nirex Ltd);
- Spent fuel and HLW disposal in clay in Switzerland (Nagra); and
Transuranic radioactive waste disposal in salt in the US (US DOE).

In addition, a review of data interpretation and modelling activities that have been undertaken in support of site characterisation programmes in other sectors has been undertaken. The review covered the following sectors:

- Civil engineering;
- Oil and gas exploration and exploitation; and
- Geothermal energy exploration and exploitation.

The review includes descriptions of the tools, including processing and modelling software, visualisation tools and other techniques that have been used in data interpretation and modelling.

Note that the reviews are limited to consideration of methods used to interpret and model data acquired through surface-based site investigations. Methods for interpreting and modelling data obtained from underground rock laboratories or other types of excavation are outside the scope of the review.

Based on the outputs of the review, recommendations are made for processes that could be followed as part of a strategic approach to interpreting and modelling of data arising from a site characterisation programme in the UK.

1.4 REPORT STRUCTURE

Section 2 sets out the general objectives for geotechnical and thermal data interpretation and modelling as an introduction to the reviews of site characterisation work undertaken in radioactive waste management programmes. Subsequent sections present details of the case studies for higher-strength rock (Section 3), lower-strength rocks (Section 4) and evaporites (Section 5). Assessment of data on soils and near-surface conditions is discussed in Section 6. Section 7 presents a review of geotechnical and thermal data interpretation and modelling methods used in other sectors. Section 8 discusses the modelling tools used in geotechnical and thermal data interpretation. Section 9 presents a strategic approach to geotechnical and thermal data interpretation and modelling for site characterisation based on the review findings and Section 10 provides a summary of the project.
2 OBJECTIVES OF DATA INTERPRETATION AND MODELLING

2.1 INTRODUCTION

NDA (2011, §4.1) describes the information requirements of a GDF site characterisation programme. The key output of this programme will be a presentation of the evidence to support the safety case, engineering design and environmental assessments. RWMD is developing a single integrated SDM describing the geometry, properties of the bedrock and water, and the associated interacting processes and mechanisms that will be used to address the information requirements of all end users. As discussed in Section 1.1, the SDM will be divided into several discipline-based models, including a geotechnical model and a thermal properties model, which are the focus of this report. The requirements of these models are discussed in the following sub-sections. The case studies presented in Sections 3, 4 and 5 describe the methods by which geotechnical and thermal data acquired through surface-based site investigations are interpreted to construct components of SDMs.

2.2 GEOTECHNICAL MODEL REQUIREMENTS

The NDA (2011, §4.6), state that the purpose of the geotechnical model component of the SDM is to describe the geomechanical properties of:

- The host rock formation for engineering design of the underground works (e.g. tunnels and vaults);
- The cover sequence for design of the accesses to the host rock formation; and
- Near-surface soils and rocks to facilitate design of the surface infrastructure.

More specifically, the information on geotechnical characteristics will be used to:

- Determine whether the selected site is large enough to accommodate the GDF;
- Determine and assess the distribution of in situ rock stresses at the site;
- Determine the mechanical properties of the intact rocks;
- Determine the characteristics and mechanical properties of discontinuities;
- Determine the characteristics of the rock mass, including the effects of construction activities;
- Identify and characterise particular geological features;
- Determine the distribution and engineering properties of soils at the site; and
- Characterise the materials that could be used for construction purposes and how they behave under GDF conditions.

The geotechnical model also provides input to the safety assessment through consideration of glacial loading, the impact of the excavation-disturbed zone, potential for spalling and the significance of construction-related groundwater flow paths.

The information required for the geotechnical model is listed in Table 2.1, although note that this report is concerned with information on the response of fractures to stress and strain changes: information relating to fracture orientations, distributions, etc. has been provided in the report on interpretation and modelling of geological data in support of the development of the SDM (Shaw et al. 2011).
Table 2.1: Information required for the geotechnical component of the SDM.

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<thead>
<tr>
<th>Properties</th>
<th>Parameters</th>
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<tbody>
<tr>
<td>Rock properties</td>
<td>Density</td>
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<tr>
<td></td>
<td>Primary wave (P-wave) and secondary wave (S-wave) velocities at different frequencies</td>
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<td></td>
<td>Grain compressibilities</td>
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<td>Rock deformation properties</td>
<td>Modulus of deformation (static and dynamic), including Young’s modulus</td>
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<td></td>
<td>Shear modulus</td>
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<td>Poisson’s ratio</td>
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<td>Creep properties (appropriate constitutive model)</td>
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<td>Strain rate dependence</td>
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<td>Rock strength</td>
<td>Compressive strength</td>
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<td>Tensile strength</td>
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<td></td>
<td>Shear strength</td>
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<td></td>
<td>Triaxial strength parameters</td>
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<td>Distribution of variation (usually not normally distributed)</td>
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<td>In situ stress</td>
<td>Magnitudes, and orientations of principal stresses, spatial distribution</td>
</tr>
<tr>
<td>Fracture properties</td>
<td>Toughness</td>
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<td></td>
<td>Normal and shear stiffness</td>
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<td>Cohesion</td>
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<td>Dilation</td>
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<td>Length distribution and continuity</td>
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<td>Aperture variations</td>
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<td>Fracture infilling material</td>
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<td>Frequency and clustering</td>
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<td>Orientation and distribution</td>
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<td>Scale</td>
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<tr>
<td>Soil properties</td>
<td>Particle size distribution</td>
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<td>Density</td>
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<td></td>
<td>Strength</td>
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<td></td>
<td>Consolidation properties</td>
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2.3 THERMAL PROPERTIES

The thermal component of the SDM will be important when significant heat generating waste is expected to be present in the GDF. Information on thermal properties would be required for various purposes (NDA 2011, §4.8) including to:

- Provide input to hydrogeological modelling;
- Assess temperatures in underground workings for engineering design; and
- Provide input to thermal modelling of near-field conditions.

The information required for the thermal model is listed in Table 2.2.

Table 2.2: Information required for the thermal component of the SDM.

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<td>Thermal conductivity</td>
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<td></td>
<td>Specific heat capacity</td>
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<td></td>
<td>Temperature distribution, including geothermal gradient</td>
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<td></td>
<td>Thermal expansion coefficient</td>
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<td></td>
<td>Natural heat generation</td>
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3 GDF CASE STUDIES: HIGHER-STRENGTH ROCKS

3.1 FORSMARK, SWEDEN

3.1.1 BACKGROUND

The Swedish Nuclear Fuel and Waste Management Company (SKB) is responsible for the disposal of radioactive waste from Swedish nuclear power plants. SKB has carried out site investigations for a facility for the geological disposal of spent fuel in the municipalities of Östhammar (Forsmark area) and Oskarshamn (Laxemar area). In June 2009, the Forsmark site was selected by SKB as the site for the proposed disposal facility. In March 2011, SKB submitted applications to the Swedish Radiation Safety Authority (SSM) and the Environmental Court for a permit to construct and operate the facility. The location chosen is adjacent to the Forsmark nuclear power plant and SFR, an existing facility for disposal of LLW/ILW (Figure 3.1). The disposal concept involves placing spent fuel assemblies in cast iron inserts within copper canisters and emplacing the canisters in a series of bentonite-lined vertical deposition holes in the host rock (SKB 2011).

The location at Forsmark was chosen primarily because of the favourable geological conditions. The dominant rock type is metamorphic medium grained granite to granodiorite and is relatively homogeneous; two fault zones adjacent to the site have shielded a volume of rock from damage related to tectonic movements. The shielding has led to the creation of a zone of nearly intact rock at disposal facility depth (approximately 500 m) with few fractures and low hydraulic conductivity. The same mechanisms have, however, also created an unusually high stress state that increases the risk of spalling of rock and other types of stress-induced rock damage (Martin 2005). The primary geotechnical challenges during construction of the facility will, therefore, be to control the risk of stress-related damage that could have adverse effects on the stability and the long term evolution of the hydraulic conductivity at the site. The rock is expected to be largely self-supporting with very little need for rock support. The hydraulic conductivity of the rock mass is also expected to be relatively low, due to the low fracture intensity in the rock mass (SKB 2011).

Figure 3.1. The visualised lay-out of the proposed disposal facility at Forsmark. The Forsmark Nuclear Power Plant is in the upper left corner of the figure and the offices and access roads for the SFR LLW/ILW disposal facility are visible at the top of the figure (image from www.skb.se).
To support the permit application, SKB has developed a SDM for the Forsmark site. This model is reported as SDM-Site Forsmark (SKB 2008a). SKB’s development of the SDM for Forsmark has followed an iterative process and is structured as shown in Figure 3.2 (SKB 2002; SKB 2004; SKB 2005; SKB 2006). As is illustrated in Figure 3.2, the different discipline descriptions in the SDM are interrelated with several feedback loops and with geology providing the essential geometrical framework.

SKB has characterised the rock domain at the Forsmark site at different scales, from the regional scale characterised by deformation zones that persist for more than 10 km, to the local fracture scale characterised by discontinuities that are less than 10 m in length (Table 3.1) (Stephens et al. 2007). Fracturing at different scales is an important consideration in the geotechnical interpretations. The structures that are classified as deterministic are those that can be traced at the surface and on the walls of excavations. The geometry and layout of the disposal facility is optimised to avoid these large structures. The greatest uncertainty lies in the fractures and fracture sets that have geometric features that are smaller than the excavated regions. The extent of fractures that remain undetected can only be estimated statistically. Therefore, estimates of geotechnical or thermal properties of the rock mass have an associated statistical uncertainty, which depends on the properties of different fracture sets.

Figure 3.2. The information flow from site investigations to site description (SKB 2001; SKB 2008a).
Table 3.1. Terminology and general description of the brittle structures in the Swedish bedrock (Stephens et al. 2007 after Andersson et al. 2000).

<table>
<thead>
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<th>Terminology</th>
<th>Length</th>
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<td>Regional deformation zone</td>
<td>&gt; 10 km</td>
<td>&gt; 100 m</td>
<td>Deterministic</td>
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<td>Local major deformation zone</td>
<td>1 km – 10 km</td>
<td>5 m – 100 m</td>
<td>Deterministic (with scale-dependent description of uncertainty)</td>
</tr>
<tr>
<td>Local minor deformation zone</td>
<td>10 m – 1 km</td>
<td>0.1 m – 5 m</td>
<td>Statistical (if possible, deterministic)</td>
</tr>
<tr>
<td>Fracture</td>
<td>&lt; 10 m</td>
<td>&lt; 0.1 m</td>
<td>Statistical</td>
</tr>
</tbody>
</table>

There is significant spatial variability in the fracture pattern observed in different parts of the candidate volume at the Forsmark site, so the SDM has been divided into sub-volumes in order to more accurately handle that variability. The result is an SDM with several fracture domains corresponding to slight changes in both fracture intensity and rock quality. From SKB’s perspective, the scales of the domains correspond to different scales of the stability analyses. The smallest scale for which SKB considers stability issues is on the size of the individual deposition tunnels. There are consecutively larger scales for modelling of stability corresponding to the increase in size of the fracture domains. Any mechanical events smaller than tunnel scale are not considered harmful for the stability of the excavations, but they could affect the suitability of individual deposition holes.

The modelling scales applied in SKBs site characterisation programme are illustrated in Figure 3.3 (SKB 2010a). The SDM based on surface investigations is supported by models at the regional and local scale (Figure 3.3a). Generally, the local model covers the volume within which the disposal facility is expected to be placed, including accesses and the immediate surroundings. Hence, the local model needs to be detailed enough for the needs of repository engineering and safety assessment. The boundaries of these models are rectangular to aid integration with, and presentation through, SKB’s rock visualisation system (RVS, Curtis et al. 2007). As excavation and construction proceeds, different numerical models at each scale can be used to predict stability issues that may arise and to model the stress changes occurring at each level. For example, large-scale stress changes due to the excavation of the main tunnels can be used as input to the models for the deposition tunnels. This allows for a continuous optimisation of the layouts and the design of the excavation during construction.

Figure 3.3. Illustration of modelling scales: a) regional and local modelling scale; b) local scale and repository scale; c) deposition area scale; and d) tunnel scale (SKB 2010a).
3.1.2 BACKGROUND ON GEOTECHNICAL AND THERMAL INVESTIGATIONS

SKB has had a siting process in progress for the deep disposal facility since 1992. A step-by-step site selection process started with consideration of the entire country and first involved elimination of counties with unsuitable bedrock. This eliminated most of the Scandes and the limestone-dominated counties in southern Sweden. The bedrock in the rest of the country is dominated by a crystalline basement and, although the variation is large, suitable rock volumes could be found in most other counties in Sweden. Therefore, the selection of candidate sites for the site investigation programme was primarily based on non-geological factors, such as community support and the feasibility. At the end of the selection process, two major candidates were selected for site investigations: Laxemar/Simpevarp and Forsmark (SKB 2001).

SKB has a target model for the rock properties (Andersson et al. 2000) and the purpose of each step of the site investigations was to narrow down the uncertainties of each relevant parameter so that a pass/fail decision could be made on whether to continue with the next step of the site investigations, discard the site or to deem the site as suitable for a facility (Andersson et al. 2004).

Data acquisition comprised three main categories of investigations:

- Geoscientific and ecological investigations of the surface system. These included geophysical surveys and the compilation of bedrock and Quaternary cover geological maps.
- Borehole investigations, comprising:
  - Drilling of long, so-called telescopic boreholes (the upper c. 100 metres are percussion drilled whereas the remainder are core drilled), conventionally-drilled cored boreholes, percussion boreholes and shallow boreholes through Quaternary deposits.
  - Measurements carried out during drilling, investigations of drill cores and drill cuttings during and after drilling, and down-hole logging. One example of these investigations is the geological mapping of boreholes using the Boremap system.
  - Sampling of intact rock material for several types of laboratory investigations.
- Monitoring of geoscientific parameters and ecological objects. Monitoring expanded successively during the site investigation period.

The detailed strategy regarding, for example, location of new boreholes, selection of borehole sections for groundwater sampling, and layout design for ground geophysical measurements, was, to a large extent, established according to an iterative process, where previous results guided subsequent decisions. This process involved a close integration between the site investigation and site modelling teams. Important decisions regarding investigation strategy have always been substantiated in so-called decision documents, which are registered in SKB’s internal documentation database “SKBdoc” and as appendices in model reports (SKB 2008a).

Table 3.2 lists the site investigation measurements for rock mechanics and the associated measurement techniques and standards. Table 3.3 provides the same information for measurements related to thermal properties and characteristics. Borehole data in support of the Forsmark SDM come from 25 core-drilled boreholes at 12 drill sites, with a total borehole length of c. 17,800 m, 38 percussion-drilled boreholes, with a total borehole length of c. 6,500 m, and more than 100 monitoring wells in the Quaternary cover, so-called soil wells (SKB 2011). By drilling inclined and deviated boreholes it was possible to sample fractures. Table 3.4 illustrates the borehole measurements used to develop the SDM for rock mechanics and Table 3.5 summarises the borehole measurements supporting the thermal SDM.
Table 3.2. Site investigation measurements relating to rock mechanics and the corresponding standards and SKB measurement protocols (SKB 2001b). Q and RMR = rock mass Quality and Rock Mass Rating rock classification systems.

<table>
<thead>
<tr>
<th>Method</th>
<th>Parameter</th>
<th>Comment (reference)</th>
</tr>
</thead>
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<tr>
<td><strong>Investigation of rock stresses</strong></td>
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</tr>
<tr>
<td>Rock stress measurements</td>
<td>Rock stress results</td>
<td>Method description /Amadei and Stephansson, 1997/.</td>
</tr>
<tr>
<td>• overcoring</td>
<td>• size and direction; 3D method</td>
<td>Method description /Stephansson, 1983/.</td>
</tr>
<tr>
<td>• hydraulic fracturing</td>
<td>• size and direction; 2D method</td>
<td>Method description /Ljunggren and Raillard, 1987/.</td>
</tr>
<tr>
<td>• hydraulic tests on pre-existing fractures (HTPF)</td>
<td>• size and direction; 3D method</td>
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</tr>
<tr>
<td>• overcoring on cutout surfaces</td>
<td>• direction, 2D method</td>
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</tr>
<tr>
<td>• borehole breakouts, measurement with caliper</td>
<td>• direction of principal stresses in plane perpendicular to borehole</td>
<td>Method description /Dart and Zoback, 1987/.</td>
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<td>• mapping of core discing</td>
<td>• high rock stresses in borehole wall in relation to rock strength</td>
<td>Mapping instructions.</td>
</tr>
<tr>
<td>• focal plane analysis from seismic monitoring, local network</td>
<td>• stress field (direction)</td>
<td>Method description /Engelder, 1993/.</td>
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<tr>
<td><strong>Investigation of mechanical properties</strong></td>
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</tr>
<tr>
<td>Rock mechanical laboratory tests</td>
<td>Mechanical properties</td>
<td>Standard /SRM, 1999/.</td>
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<tr>
<td>• uniaxial compression tests</td>
<td>• strength, Young’s modulus and Poisson’s ratio</td>
<td>Standard /SRM, 1977/.</td>
</tr>
<tr>
<td>• determination of P-wave velocity</td>
<td>• P-wave velocity</td>
<td>Standard /SRM, 1983/.</td>
</tr>
<tr>
<td>• triaxial compression tests</td>
<td>• strength</td>
<td>Standard /SRM, doc No 8 1977/.</td>
</tr>
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<td>• Brazilian test</td>
<td>• tensile strength</td>
<td>Standard /SRM, doc No 1 1974/.</td>
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<td>• normal loading tests on fractures</td>
<td>• tensile strength, normal stiffness</td>
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<tr>
<td>• shear tests on fractures</td>
<td>• shear strength, shear stiffness</td>
<td>Standard /SRM, doc No 1 1974/.</td>
</tr>
<tr>
<td>• core mapping</td>
<td>• RMR, C-value</td>
<td>Mapping description /Bennawski, 1975; Grimstad and Barton, 1993/.</td>
</tr>
<tr>
<td>Other laboratory tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>intact rock</td>
<td>Other properties</td>
<td>Standard /ASTM D4535-85/ or /ASTM D535-92/.</td>
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<tr>
<td>• determination of thermal expansion</td>
<td>• coefficient of thermal expansion</td>
<td>Standard /DIN 52102-RE VA/.</td>
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<td>• density determination</td>
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<td>• x-ray diffraction</td>
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<td>Processing of geophysical data</td>
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<td>Method description (no reference).</td>
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<tr>
<td>seismic measurements and sonic logging</td>
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</table>
Table 3.3. Site investigation measurements relating to thermal properties and the corresponding standards and SKB measurement protocols (SKB 2001b). TPS = Transient Plane Source (laboratory method for measurements of thermal conductivity and thermal diffusivity of rock samples).

<table>
<thead>
<tr>
<th>Method</th>
<th>Parameter</th>
<th>Comment (reference)</th>
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<tr>
<td><strong>Investigation of thermal properties of the rock</strong></td>
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<td><strong>Field methods</strong></td>
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<tr>
<td><strong>Laboratory methods</strong></td>
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<td>Thermal properties</td>
<td>Method description, TPS /Gustafsson, 1991/.</td>
</tr>
<tr>
<td>• determination of heat capacity</td>
<td>heat capacity</td>
<td>Method description, TPS /Gustafsson, 1991/.</td>
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<td>• determination of density</td>
<td>density</td>
<td>Standard DIN 52102-RE VA.</td>
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<td>• determination of porosity</td>
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<td>Standard DIN 52103-A.</td>
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<td>• determination of chemical and mineralogical composition</td>
<td>chemical and mineralogical composition</td>
<td>Method description for ICP, SEM and EDS.</td>
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Table 3.4. Borehole tests used for the rock mechanics model in the Forsmark SDM (Glamheden et al. 2007). The left-hand column lists the borehole identifier.

<table>
<thead>
<tr>
<th>Borehole Identifier</th>
<th>Laboratory testing</th>
<th>Rock mass charaterisation</th>
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<th>Hydraulic methods</th>
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1 Only part of the borehole intersecting the rock volume of interest was tested.
### Table 3.5. Borehole tests used for the thermal model in the Forsmark SDM (Back et al. 2007).

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<td><strong>Data from core-drilled boreholes</strong></td>
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<td>Anisotropy in thermal properties for KFM04A and KFM05B</td>
<td>P-06-205</td>
<td>Estimation of anisotropy in thermal properties.</td>
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<td>Inter-laboratory comparison of TPS measurements for KFM01A</td>
<td>P-07-194</td>
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<td>KFM04A</td>
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Rock properties

The high quality of the rock expected at the Forsmark site has not necessitated any particular method development in terms of rock testing and SKB has utilised standard laboratory methods for determination of the mechanical properties of rock samples. The rock at the Forsmark site displays a slightly higher tensile strength and lower fracture frequency than would be considered typical of Swedish bedrock. This, in conjunction with a high stress state, indicates that the primary failure mechanism that may cause concern for the safety and stability is spalling. Spalling has been observed nearby during construction of the recirculation tunnel of Forsmark 3 (Carlsson and Christiansson 2007).

Fracture properties

The large number of cores retrieved from the target rock volume provided material for a many laboratory tests of the properties of both intact rock and natural rock fractures. The strength and deformability of the natural rock fractures were determined in two ways (Lanaro and Fredriksson 2005):

- By means of tilt tests where shearing is induced by sliding due to the self-weight of the upper block when the fracture is progressively tilted;
- By means of direct shear tests where shearing is induced by actuators that apply a load perpendicular and parallel to the fracture plane.

For the Forsmark site investigation, tilt tests were performed on 142 samples retrieved from four boreholes. The results did not indicate any trend with depth as well as an unsurprisingly high scatter.

Direct shear tests were performed on samples from the same boreholes used for the tilt tests and by two different laboratories. These tests also measure normal and shear stiffness for the fractures, but not at the same levels of stress as would be encountered in situ. The direct shear tests yielded a higher mean friction angle compared to the tilt tests, but also with a considerable scatter. The normal stiffness of the fractures varied considerably between the two laboratories. As the fracture stiffness is non-linear and highly dependent on the normal stress, the variation is probably due to different measuring techniques and the inherent sensitivity of the secant stiffness measure. The measured values are also highly scattered which contributes to the difficulty in analysing these properties.

In situ stress

Regional stresses were determined based on an understanding of plate tectonics and from the World Stress Map Project (Glamheden et al. 2007 §6.1): a general WNW-ESE compression is expected in Scandinavia. The in situ stress state has been measured at the Forsmark site using hydraulic fracturing (HF), overcoring and hydraulic tests on pre-existing fractures (HTPF) as well as various indirect observations, including borehole breakout studies, core discing and microcrack porosity studies on laboratory samples (SKB 2008a, §7.2.4). These tests and observations all indicate a general orientation of the major principal stress in the range of N130–150° in the region around Forsmark (Glamheden et al. 2007 §6.1).

Thermal properties

Laboratory measurements of the thermal conductivity and thermal diffusivity of water saturated rock samples were made using the TPS (Transient Plane Source) method (SKB 2008a, §6.2.1). The TPS method involves making measurements on core samples. The scale of the measurements is limited by the diameter of the core samples retrieved and the method allows for measurement of thermal anisotropic conditions.

Heat capacity has been determined indirectly from thermal conductivity and diffusivity measurements using the TPS method, and directly using the calorimetric method (SKB 2008a, §6.2.4).

Fluid temperature has been measured in most cored boreholes at Forsmark (SKB 2008a, §6.2.5). Uncertainties were high and as a result only data from "approved boreholes" were used based on consideration of errors associated with the logging probe and time between drilling and logging.

The thermal expansion coefficient was measured on core samples in the laboratory.

3.1.3 GEOTECHNICAL DATA INTERPRETATION AND MODELLING

The process for developing the SDM for rock mechanics is illustrated in Figure 3.4 (Glamheden et al. 2007). The basis for the modelling is the geometrical division of the geological model into rock domains, fracture domains and
deformation zones (Table 3.1; Stephens et al. 2007). As the available site-specific data only cover a small portion of the investigated site, the mechanical properties have been evaluated on the fundamental assumption that the rock volume within each geological domain has similar mechanical characteristics throughout.

The two important components of the rock mechanics model for the SDM are the geometrical and mechanical characterisation model and the model of the in situ stress state. Laboratory testing was used to establish the primary data on the intact rock and discrete fractures for the development of the mechanical model. These primary data were then used in combination with the geological model as input to two modelling approaches, one empirical and one theoretical, to estimate the mechanical properties of the rock mass at the deposition drift and hole scale. The empirical approach estimates the rock mass properties based on classification indexes and empirical relationships routinely used in rock engineering, whereas the theoretical approach utilises numerical models developed specifically for the SDM (Olofsson and Fredriksson 2005). The in situ stress model was developed from direct and indirect measurements, and numerical modelling that respected the geometry and structure in the geological models.

![Flowchart for the rock mechanics site descriptive model (Glamheden et al. 2007 after Andersson et al. 2002).](image)

**Rock and fracture properties**

Spatial variability of the fracture mechanical properties was evaluated by analysing the statistical spread in the measured data and by comparing the results from different stages and modelling steps in the development of the Forsmark SDM (Glamheden et al. 2007 §4.4). A small statistical spread and insignificant changes in derived parameter values between modelling steps indicated homogeneous conditions. The analysis also showed that there was little difference in the mechanical properties of different fracture sets. The parameters evaluated from the tilt tests and direct shear tests were presented as a function of elevation. By visual inspection of the results, Glamheden et al. (2007, Section §4.4) concluded that it is difficult to see any clear trends in the parameters with elevation. Mean,
minimum and maximum values and standard deviations were derived for the peak friction angle, peak cohesion coefficient, residual friction angle, residual cohesion coefficient, normal stiffness, shear stiffness and dilatancy angle for each fracture set. Uncertainties on the mean were quantified according to the central limit theory for a 95% confidence interval.

The methodology used in the empirical approach to estimate rock mass properties is described in Röshoff et al. (2002). The strength parameters of the rock mass are estimated using the linear Mohr-Coulomb (M-C) and non-linear Hoek-Brown (H-B) failure criteria, which are applicable to homogeneous and isotropic rock masses. The theoretical approach is based on numerical simulations with the use of the 3DEC software. The methodology is built upon three different models: the discrete fracture network (DFN) model which is used to simulate the fracture network in the rock mass, the 3DEC mechanical model which is used to calculate the rock mass mechanical properties, and a statistical model for estimation of combined variability. The modelling procedure is described in detail in Olofsson and Fredriksson (2005). The input parameters for the modelling are based on the geological DFN model and the determined properties of intact rock and fractures. The rock mass deformation and strength properties estimated from the empirical and theoretical approaches are in reasonable agreement for fracture domains.

The results of the theoretical and empirical approaches were harmonised to derive a description of the rock mass. In the harmonization process, the rock mass properties for fracture domains were assigned by making the following assumptions (Glamheden et al. 2007 §5.3):

- The minimum and maximum values were assigned based on the smallest and the largest value estimated by both approaches. The standard deviation and uncertainty of the mean were selected from the largest values presented by both approaches.
- The rock mass deformation modulus and Poisson’s ratio were determined by averaging the mean values estimated by the two approaches.
- The uniaxial compressive strength based on the empirical Hoek-Brown failure criterion approach was assumed.
- The apparent cohesion and friction angle based on Mohr-Coulomb failure criterion were assumed to be directly comparable. Averaging was applied to the mean values estimated by the two approaches.
- The rock mass tensile strength was based on the empirical approach, because no value was evaluated from the theoretical approach.

The harmonised values from the two methods are considered to be representative of the rock mass outside deformation zones. Harmonisation of the properties of the deformation zones was not performed due to a judgment that such a harmonisation was inappropriate (Glamheden et al. 2007). As regards the deformation zones, it is recommended that users who apply the evaluated results should consider which one of the two approaches is the most appropriate for the case being analysed.

**In situ stress**

The primary purpose of the numerical models of in situ stress state at Forsmark was to assess the impact of the major deformation zones on the stress magnitudes within the target volume at repository depth and to evaluate the local stress spatial variability. A series of three dimensional 3DEC numerical model simulations was carried out (Glamheden et al. 2007). In the regional 3DEC numerical model, no stress ratios were specified, gravity was imposed and model boundaries were displaced (compressed) in one direction, while the other boundary remained fixed. The compression was stopped once the maximum horizontal stress in the model approached measured values. Since most of the measurements were recorded above 300 m depth, there was no match with repository depth and there was no attempt to match the stress ratios. The results indicate that the stress field in the target volume is relatively homogeneous. The steeply dipping deformation zones cause only very limited perturbation of the stress field in close proximity to the deformation zones, while a gently dipping deformation zone perturbs all of the stresses in the rock mass above the zone.

By applying the mean principal stresses at repository depth to a 3DEC model containing discrete fractures based on a DFN model, the stress variability due to discrete fractures at the scale of a 5 m diameter deposition tunnel was examined. The analyses indicate that the major principal stress obtained from overcoring measurements could be expected to vary spatially by ±5 MPa in magnitude and ±9 degrees in orientation for the rock mass conditions expected at Forsmark. The variability in the stress magnitudes and orientations from the simulations are considered as an upper bound for the Forsmark rock mass at repository depth (Glamheden et al. 2007).
The rock mechanical model can have a number of different realisations representing the uncertainty of the model. The span of the different models represents three different types of variability: conceptual uncertainty, data uncertainty and spatial variability. The spatial variability is not an uncertainty as such, but it is a main cause of data uncertainty as it influences the variation of the measurements. The conceptual uncertainty is governed mainly by the uncertainty of the geometric description of the fracture zones and is mainly a matter of interpretation of the geophysical measurements.

The uncertainty in the mechanical properties has been expressed by means of a range of variation in the evaluated mean values and a standard deviation for a 95% confidence interval. Minimum and maximum truncation values based on the observed minimum and maximum for the tested population have also been evaluated (Glamheden et al. 2007). For the dominant rock type, the uncertainty in the mean values is small for the mechanical properties of both the intact rock and the rock mass. This fact indicates homogeneous conditions in the target volume. Microcracking due to release of stresses during core drilling can disturb laboratory samples. This causes an increase in the uncertainty of the compressive and tensile strength measurements with depth. Changes in testing procedure for measuring the normal stiffness in the direct shear tests may have also caused uncertainty in the earlier tests, with the uncertainty being substantially reduced for later results.

The uncertainty in the in situ state of stress has been quantified by technical auditing of measurement results, statistical data analysis and numerical modelling (Martin 2007, Glamheden et al. 2007). The orientation of in situ stresses and the magnitude of the vertical stress component are the aspects of the stress model that are judged to have highest confidence. The basis for the confidence in the orientation is the conformity in the results between measuring methods and indirect observations at different scales and its agreement with regional seismic studies. The magnitude of the uncertainty in the minimum and maximum horizontal stresses has been possible to constrain because “classical” borehole breakouts have not been observed at depth. While uncertainty in the horizontal stress magnitudes remains using this approach, the stress model provides an upper bound estimate. The results from the HF measurements were omitted since it is suspected that the minimum horizontal stress is incorrect and corresponds to the vertical stress. Moreover, these results do not agree with the general state of stress determined for the Fennoscandian shield. Neglecting the HF data is considered to result in a conservative estimation of the in situ state of stress.

A systematic assessment of confidence in the model, including treatment of uncertainties and evaluation of alternative interpretations, has been carried out (SKB 2008b). Generally, it is judged that key aspects of the Forsmark SDM, i.e. information needed for facility design and safety assessment, have a high level of confidence. The overall reason for this confidence is the relative wealth of data from the target volume and the consistency between independent data from different disciplines. Some aspects of the SDM have lower confidence. This is handled by providing wide uncertainty ranges, bounding estimates, producing alternative models or defining additional lines of action to be completed either in the immediate future or during underground investigations.

3.1.4 THERMAL DATA INTERPRETATION AND MODELLING

Thermal conductivity

The thermal properties of basic minerals can also be used to derive the thermal properties of the rock. A homogenous rock matrix is likely to have thermal properties very close to the average of its constituent minerals. SKB has developed an indirect approach to estimating thermal properties based on the observation that certain types of igneous rock consist of a limited number of common minerals and that the thermal properties of the rock are a function of the thermal properties of its constituent minerals. As the density of the rock varies with mineral content in a correlated fashion – although the correlation may be weak – the density provides an indirect way of estimating the thermal conductivity (Figure 3.5) (SKB 2008a, §6.2.2; Back et al. 2007). The density can be evaluated using a standard log measurement in a borehole.

SKB’s overall strategy for determining the thermal properties for a site is described in Back and Sundberg (2007) (Figure 3.6). The methodology for thermal site descriptive modelling is based on stochastic simulation of the lithology and of the associated thermal conductivity. When merged, the main result is a set of equally probable realisations of thermal conductivity. Of special interest is the lower tail of the thermal conductivity distribution. The thermal conductivity will play a key role in determining the layout of the deposition holes as the maximum surface temperature is a function of the heat transport capacity of the host rock. As the canister spacing has a major influence on the total cost of the final repository there is a clear financial incentive in limiting the uncertainty in the thermal conductivity measurements. SKB has estimated that the uncertainty in thermal conductivity has to be kept below 5% in order to be able to build and operate a disposal facility in line with the parameters used (Hökmark and Fälth 2003). However, the anisotropy and inhomogeneity of the rock mass has put a limit to the usefulness of the current methods as the
practically achievable accuracy is still above 10 - 15%. SKB is therefore participating in an ISRM-organised research venture to develop usable methods with sufficient accuracy (SKB 2010b).

The methodology for thermal site descriptive modelling is applied separately for each rock domain (Figure 3.6). At the Forsmark site, the chosen rock volume has been modelled as two subdomains with different thermal properties. Both domains contain inclusions of amphibolite which has the lowest thermal conductivity of the rock types found. The mapping of major amphibolite inclusions will be a key issue during the excavation of the repository area in order to avoid having deposition tunnels insulated by regions of amphibolite. The frequency of large amphibolite bodies has, however, probably been overestimated during the initial stages of thermal modelling for the Forsmark repository (Sundberg et al. 2008).

The simulation scale (1 in Figure 3.6) determines how lithological data (2) should be prepared and if a change of support (scale) (5) is required for the thermal data (4). The lithological data acquired from boreholes and mapping of the rock surface need to be reclassified into Thermal Rock Classes (TRCs) (3). The main reason is to simplify the simulations; only a limited number of categorical classes can be handled in the simulations. The lithological data are used to construct models of the spatial statistical structure of each TRC in 3D using Markov chain analysis (7). The modelling consists of calculating transition probabilities followed by expert adjustments based on geological interpretations (Carle and Fogg 1997). The result is a set of transition probability models that are used in the stochastic simulation of TRCs (8). The stochastic simulation is performed with the software T-PROGS. The intermediate result of this first stochastic simulation is a number of realisations of the geology, each one equally probable.

Figure 3.5. Thermal conductivity (TPS measurements) versus density for granite, granodiorite and tonalite (101051) and amphibolite (102017) at Forsmark. The “Model” curve shows the regression used to derive thermal conductivity from density logging within the density interval 2,625–2,850 kg/m³ for the Forsmark SDM (Back et al. 2007).
Based on the thermal data, a spatial statistical thermal model is constructed for each TRC (9). The model describes the statistics and the spatial correlation structure of thermal conductivity for a TRC. Development of the model is performed in three steps:

1. Qualitative trend analysis. There may be large spatial trends of thermal conductivity for some rock types. Qualitative trend analyses is performed to reveal trends that can be significant for the thermal modelling.

2. Fitting a distribution model to the histogram. A distribution model is fitted to the histogram of each TRC. This is performed by smoothing the histogram with the smoothing algorithm in the geostatistical software GSLIB (Deutsch and Journel 1998). A very important aspect to consider is how to model the tails of the histogram where there are no data. The following principles are suggested for setting lower and upper limits of thermal conductivity in the distribution models for each TRC:
   a. The distribution model should cover the range of the data.
   b. Since the number of data points is limited it can be assumed that values outside the range of the data exist.
   c. Where possible, and where justified, a theoretical lower limit (minimum value) can be approximated from assumptions regarding the mineral compositions of “extreme” cases. By “extreme” it is meant mineral compositions which produce the lowest thermal conductivities. The distribution model of a TRC should reflect all rock types belonging to that TRC.
3. Variogram modelling (structural analysis). Variogram analysis is performed in order to construct a variogram model which characterises the essential features of a TRC’s spatial variability/dependence. Thermal conductivity data for most rock types are too few to produce any reliable variograms. For this reason, density loggings are employed. It is assumed that density and thermal conductivity exhibit similar, although not identical, correlation structure. Experimental variograms are created for the main rock type in each TRC. The variogram modelling is an important step in the thermal modelling because it dictates how the variance is reduced when the scale increases. This is important for the tails of the thermal conductivity distribution at different scales. The variogram model is associated with modelling uncertainty.

The statistical thermal models are used in the simulation of thermal conductivity based on Sequential Gaussian Simulation (SGS) (10), resulting in a number of equally probable realisations of thermal conductivity. In the next step, the realisations of TRCs (geology) and thermal conductivity are merged together (11), i.e. each realisation of geology is filled with simulated thermal conductivity values. The result (13) is a set of realisations of thermal conductivity that considers both the difference in thermal conductivity between different TRCs and the variability within each TRC. If the result is desired in a scale different from the simulation scale, an upscaling can be performed (12). Upscaling is undertaken using the Self Consistent Approximation (SCA) approach (see Back and Sundberg 2007). The result can then be presented in a number of ways, for example as 3D illustrations, histograms and statistical parameters. The thermal realisations can also be used as input to design of a repository and for mathematical modelling of temperatures in and around a repository.

Regarding data uncertainty, uncertainties in the orientation of the boreholes and in the orientation of geological objects in the boreholes are judged to have little or no effect on the results of thermal modelling. TPS measurements of thermal properties on isotropic samples are considered to be reliable. If the samples show anisotropic thermal behaviour, which to some degree is the normal case in the Forsmark area (foliated or lineated samples), the uncertainty is larger, especially in the determination of thermal diffusivity and consequently also the determined heat capacity. However, the heat capacity has also been determined directly by a calorimetric method and these measurements are considered to be more reliable. The mean values determined for the thermal anisotropic factor are thought to be quite reliable but the spatial variability has large uncertainties. Only "approved" boreholes have been used for more recent direct temperature measurements. Therefore, uncertainties in temperature data are much smaller than for earlier data.

There are several uncertainties associated with the different steps of the stochastic modelling, of thermal conductivity (Back et al. 2007, Sundberg et al. 2008). Five uncertainties are believed to be most important for the results at rock domain level: the simulation scale, the simulation volume, the spatial statistical structure of TRCs (lithology), the spatial statistical thermal models, and the simulation technique:

- **Simulation scale** - evaluation indicates that using 1-m simulations to represent subordinate rock types gives a too conservative estimate of lower percentiles due to discretisation errors. However, it is assumed that this uncertainty is more or less eliminated at the 5-m scale. In order to describe the size distribution of subordinate rock types in an accurate way, it is considered necessary to perform simulations at a number of different scales.

- **Simulation volume** - there are two situations when the limited simulation volumes could be a problem: (1) when the lithological simulation volume is so small that the statistics of the corresponding rock volume deviate from the true domain statistics, and (2) when the correlation lengths of thermal properties are similar to, or longer than, the length of the simulation volume. Neither of these uncertainties is believed to have had any major impact on the thermal modelling results at the canister scale.

- **Spatial statistical structure of TRCs (lithology)** - there are several uncertainties associated with the developed models of the proportions and the spatial statistical structure of the TRCs. Most of these are coupled to the lack of knowledge concerning detailed geological information. Uncertainties are largest for rock types with low proportions and heterogeneous rock domains. Geological heterogeneities have been dealt with by dividing the domains into subdomains, reducing the uncertainty significantly. The uncertainty concerning the variability in proportions is believed to be small. Based on confidence intervals for TRC proportions at borehole scale, this uncertainty has only a minor effect on the lower thermal conductivity tail (the 1-percentile may vary by about 1%). The proportions of each TRC in the lithological simulations deviate somewhat from the proportions of different rock types estimated as part of the modelling carried out by the geological team, primarily because borehole occurrences shorter than 5 cm were excluded from borehole data used in the lithological simulations and the two estimations are based on slightly different sets of boreholes. The effect of these discrepancies on the thermal model at the 5-m scale is considered to be small.
Spatial statistical thermal models - limited data for some TRCs result in uncertain spatial statistical thermal models. When data are few and show large variability, the shape of a histogram cannot only be based on measurements. In addition, the lower limit of thermal conductivity of a TRC is usually not known and must be determined based on expert opinion. The variograms require even more data. It has been assumed that thermal conductivity exhibits a similar correlation structure to density. This is a reasonable assumption that makes possible the creation of variograms, but the associated uncertainty is not known. In spite of the uncertainties, the spatial statistical thermal conductivity models are believed to be more reliable than in previous versions of the thermal site descriptive modelling.

Simulation technique - this uncertainty is closely related to the simulation scale and the simulation volume. Comparing the results of simulations against the input models shows that the outputs resemble the inputs relatively well. Therefore, this uncertainty is believed to have only a minor influence on the results.

The thermal models are judged to represent the modelled rock domains and their variability in an appropriate way. The main reasons for this confidence is that the spatial statistical thermal models for most thermal rock classes are based on a satisfactory amount of data (SKB 2008a). The highest confidence is placed in the results for the domain with the higher degree of homogeneity in geology and thermal properties. Larger uncertainties associated with the output of the geological simulations for the other domain imply that the overall distribution of thermal conductivity is somewhat more uncertain for this domain.

Heat capacity

Mean and standard deviation values for heat capacity were calculated for different rock types from measured data. Calorimetric measurements were used in modelling because the TPS values indicated some bias. A relationship between heat capacity from direct measurements and thermal conductivity established in the thermal modelling and described by a second order regression equation was used to calculate heat capacity SKB (2008a, §6.5.1).

Temperature

The geothermal gradient was calculated by evaluating the mean in situ temperatures measured in eight boreholes at different depths.

Thermal expansion coefficient

Mean and standard deviation values of the thermal expansion coefficient were calculated for different rock types. Measurements made at the Åspö underground laboratory, and as part of the site investigations in Forsmark and Oskarshamn, showed no significant variation due to rock type (e.g. Sundberg and Ländell 2002, Sundberg et al. 2005).

3.1.5 DATA INTERPRETATION TOOLS

SKB stores all information regarding site characterisation in its SICADA database (SKB’s site characterisation database). The database includes an extensive array of observations, including all the data from drilling and other site investigations.

In addition to SICADA, SKB has developed a CAD-like visualisation system called RVS (SKB’s Rock Visualisation System) (Curtis et al. 2007). The latest release (4.0) is based on the microstation CAD software and the Microsoft Access database system. This software is custom made to work directly with parts of SICADA in order to facilitate the interpretation of the data stored in that database. The software can handle visualisation of both 2D and 3D data and has a set of tools for visualisation of rock domains, such as intrusions and lenses. Although SKB has worked on the software for about a decade, the software has not been widely adopted outside SKB and it is unclear whether the software is still in active development.

The estimates of the rock mass properties based on numerical simulations use the 3DEC software (Itasca 2003). The methodology is built upon three different models: the DFN model which is used to simulate the fracture network in the rock mass; the 3DEC mechanical model which is used to calculate the rock mass mechanical properties; and a statistical model for estimation of combined variability.
Modelling of the thermal conductivity for a TRC uses the geostatistical software GSLIB (Deutsch and Journel 1998). The modelling of the structure uses the T-PROGS (Transition PRObability GeoStatistics; Figure 3.7) (Carle 1999, GMS 2006) software for the transition probability analysis and the stochastic simulations. This software utilises a transition probability-based geostatistical approach to:

- Model spatial variability of categorical data, e.g. rock classes, by 3D Markov Chains;
- Set up indicator co-kriging equations for predicting rock categories at positions where observations have not been made; and
- Formulate the objective function for simulated annealing for finding the global maximum of the predicted model, i.e. for finding the optimal spatial configuration given the selected input parameters.

### 3.1.6 LESSONS LEARNT

The conformity in the results between indirect observations and direct stress measurements at different scales forms the basis for high confidence in the in situ stress orientation. An upper boundary of the magnitude of the maximum horizontal stress has been developed from the observed lack of stress-induced damage in drill cores and boreholes down to a depth of 1,000 m. The magnitude of the vertical stress component was measured using hydraulic fracturing and it was found to be equivalent to the calculated weight of the overburden. Modelling studies found that steeply dipping deformation zones cause only very limited perturbation of the stress field in close proximity to the deformation zones, while a gently dipping deformation zone perturbs the stresses in the overlying rock mass.

The Forsmark site exhibits homogeneous conditions and uncertainties in the mechanical and fracture properties have been expressed as variations in the evaluated mean values and a standard deviation for a 95% confidence interval.

Thermal conductivity plays a key role in determining the layout of deposition holes and it is thus important to limit uncertainty in thermal conductivity for thermal modelling. The lower tail of the thermal conductivity distribution is of particular importance. The shape of the tail, at various scales, is mainly determined by how the rock types are modelled.

The thermal modelling for Forsmark indicates that great care should be taken when the simulation scale and the simulation volume is decided. Measurements are performed in the cm-dm scale, subordinate rocks with importance for the thermal modelling occur down to the dm-m scale, correlation lengths range from dm to 50 m or longer, and the domain properties may require simulation volumes of 50 to 100 m or more. If the simulation scale is too large, discretisation errors may occur regarding subordinate rock types. On the other hand, if the simulation scale is too small, the simulation volume is restricted by the capacity of the computer. This may lead to other problems, e.g. difficulty of reproducing the variogram models during simulations, underestimation of the spatial variability in the lithology, and a too large variance reduction during upscaling.
3.2 OLKILUOTO, FINLAND

3.2.1 BACKGROUND

Posiva Oy is responsible for the final disposal of spent nuclear fuel from nuclear power stations in Finland. Posiva’s disposal plans for spent nuclear fuel are based on the Swedish KBS-3 concept with a series of disposal galleries situated at level of -420 meters below the sea level (Figure 3.8). The basis for current planning is a total accumulation of approximately 5500 tonnes of spent fuel, which will lead to around 2800 canisters to be disposed of in Finnish crystalline bedrock.

The Finnish bedrock typically has a block tectonic geological setting and the objective of geological investigations in all site characterisation phases is to identify suitable rock blocks or volumes to provide sufficiently good conditions for safe disposal. Precambrian metamorphosed crystalline bedrock in Olkiluoto is composed of various types of migmatitic gneisses and granites (Posiva 2005, 2007). Crystalline rocks in Olkiluoto characteristically have very low total porosity and are intersected by fractures and fracture zones (Posiva uses the term Brittle Deformation Zone) (Anttila et al. 1999).

The siting programme for Finnish nuclear fuel waste disposal began in 1979. The programme leading to the start of underground investigations is summarised in Figure 3.9. The work progressed from regional studies, to identification of investigation areas (1983 – 1986 desk studies), to selection of five sites for preliminary characterisation (1987 - 1992) and to detailed characterisation of four sites (1993 - 2000) (Posiva 2003, §2.1). These investigation and research phases ended in 2001 when the Finnish Parliament ratified the Government’s favourable Decision in Principle (DiP) on Posiva’s application to locate the repository at Olkiluoto in western Finland. In the DiP, it was stated that further investigations would be needed to show that the operation of the facility will be safe, both in the short- and long-term. These investigations started the site confirmation phase (Posiva 2003, §1.1).

Figure 3.8. Posiva’s disposal facilities planned to Olkiluoto site (source: www.posiva.fi).
SITE IDENTIFICATION SURVEY (DESK STUDIES)  
(1983-1986)
- Also Lavio test hole for borehole investigation methodology development

PRELIMINARY SITE INVESTIGATIONS at five sites (1987-1992)
- Geological mappings, drilling of deep boreholes KR1-KR6 and shallow boreholes
- Geophysical studies (airborne, ground survey, borehole logging)
- Network of multi-level piezometers
- Installation of multi-packer systems into deep boreholes KR1-KR5
- Monitoring of groundwater/hydraulic heads in shallow and deep boreholes (start)
- Sampling of groundwater and rain water from the surrounding area, from wells, piezometers, deep boreholes KR1-KR5
- Rock stress measurements in borehole KR1 at depth level of 470-900 m
- Rock mechanics laboratory tests from deep boreholes KR1-KR6, KR5
- Rock mechanics field tests (point load) from deep boreholes
- Thermal property laboratory tests from boreholes KR2, KR3 and KR5

DETAILED SITE INVESTIGATIONS/ PHASE I (1993-1996) and PHASE II (1997-2000) at four sites
- Regional geological studies (lineament interpretation, gravimetric survey, mapping)
- Geophysical studies (ground survey, borehole logging, acoustic-seismic study of the seabed)
- Groundwater sampling from deep boreholes including pressured waters sampling
- Measurements of hydraulic conductivity of deep boreholes, long-term pumping tests
- Installation of shallow groundwater observation tubes
- Ecological studies related to EIA, EIA/nature survey
- Rock stress measurements in boreholes KR2, KR4 and KR10 at depth level of 300-800 m
- Extensive rock mechanics laboratory tests from deep borehole KR10 and few from KR2, KR4
- Rock mechanics field tests (point load) from deep boreholes
- Monitoring of the deformation of bedrock with GPS network (start)

SITE CONFIRMATION (PRE-ONKALO) PHASE, Olkiluoto site (2001-2004)
- Regional geological studies (mapping)
- Geophysical studies (ground survey, borehole logging)
- Water sampling from the surrounding area and rain water, from groundwater tubes, shallow boreholes and deep boreholes including pressured groundwater sampling
- Measurements of hydraulic conductivity of deep boreholes
- Vegetation and forest inventories, Ground frost measurements
- Extension of groundwater monitoring network (observation tubes), hydraulic conductivity measurements in shallow boreholes
- Rock stress measurements in borehole KR24 at depth level of 290-390 m, Kaiser Effect study in borehole KR14
- Rock mechanics anisotropic laboratory testing from deep boreholes KR12 and KR14
- Rock mechanics field tests (point load) from deep boreholes
- Microseismic monitoring network (start)

Figure 3.9. Different surface-based site characterisation phases before the ONKALO. The geotechnical issues (i.e. rock mechanics and thermal issues) are highlighted as red (modified from Posiva 2003).

Posiva has launched a three-stage underground rock characterisation programme at Olkiluoto. In the first stage, the baseline conditions of the Olkiluoto site were established, an improved description of the potential target rock volume provided, and the basis set for the choice of the access locations of an underground facility. In the second stage of the underground characterisation programme, Posiva started in 2004 to construct an underground characterisation facility (not an underground research laboratory) called the ONKALO. Methods and equipment were further developed and tested for the investigations at greater depth. In the third stage, the actual characterisation of the target rock volumes was commenced. The findings of the ONKALO and other investigations will provide the knowledge needed for an application, supported by a Preliminary Safety Assessment, to construct a final repository for spent nuclear fuel at the site. Posiva is planning to apply for a construction licence by the end of 2012 and an operation licence in 2018. The target is to begin disposal operations in around 2020.
During the preliminary and detailed investigation phases, there was no systematic scheme to establish SDMs for any of the sites. Conceptual site models were established but in separate reports. At the desk study stage, each site had a conceptual model (prediction) starting from their geology and the corresponding rock mass properties. During the preliminary and detailed investigation phases, these conceptual models were checked and updated based on the investigation results.

Preliminary phase bedrock models were developed for all sites. Models were based on lithological and tectonic interpretations and descriptions, structural framework (fracture zones and fracturing) and hydraulically-determined features. Version 1 for Olkiluoto was published at the end of the preliminary investigation phase in 1992 (Saksa et al. 1993), Version 2 during supplementary site investigations (Saksa et al. 1996), and Version 3 at the end of the surface-based characterisation programme (Saksa and Lindh 1999). Compilation of the first SDM was undertaken by Anttila et al. (1999). This report summarised the geological (including also rock mechanics and thermal properties), geochemical and hydrogeological characteristics of the site based on the large amount of data and material gathered since 1987. The report presented the updated bedrock models (lithological and structural), indicated which parts of the rock mass were believed to show most promise for locating a repository and discussed the general principles associated with the design and construction of the repository. Quaternary overburden cover was also described, but not modelled in 3-D explicitly.

The work by Anttila et al. (1999) was followed by a rock mass classification report by Äikäs et al. (2000) aimed at determining suitable bedrock volumes for the repository construction based on the empirical rock classification Q-system. The work naturally required a geological (bedrock) model in order to estimate the geotechnical conditions based on data from ten deep boreholes. The geological model comprised intact rock volumes or blocks bounded by intersecting fracture zones as discontinuities considered to be significant with regard to rock engineering or hydrogeology. As an example, a visualisation of the 3-D block structure is shown in Figure 3.10. Geotechnically, the model consisted of mechanical and thermal rock properties, rock quality (Q-value) and the in situ stress at a depth level of 300-700 m. The report by Äikäs et al. (2000) also included information on some drilling parameters of the main rock types like DRI (Drilling Rate Index), CAI (Cerchar Abrasion Index) and Vickers hardness.

Figure 3.10. Block model of Olkiluoto from the rock engineering classification study - R refers to fracture zone (Äikäs et al. 2000).
The classification in Äikäs et al. (2000) has been developed further for the Olkiluoto site by McEwen (2002) and Hagros et al. (2003 and 2005) by means of a Host Rock Classification system (HRC). The HRC has been further developed to its current scheme called Rock Suitability Criteria (RSC) (Hellä et al. 2009).

A systematic procedure to establish an integrated SDM started in the early 2000s, when the underground rock characterisation programme was launched and the baseline of the Olkiluoto site was established. The baseline report summarises the conditions as they are revealed by the studies and investigations made from the surface before the construction of the first underground access i.e. the ONKALO (Posiva 2003). A comprehensive database included information both from surface and a total of more than 20 cored deep boreholes. The procedure to establish the SDM and deal with uncertainties (conceptual uncertainty, data uncertainty, spatial variability and confidence) utilised very much the strategy work developed by SKB (Andersson et al. 2002). Posiva also followed SKB’s strategy to develop a thermal SDM (Sundberg 2003).

Since the baseline report, Olkiluoto Site Descriptions have been produced approximately every second year by the Olkiluoto Modelling Task Force (OMTF) and the results have been published as Olkiluoto Site Description reports (Posiva 2005, 2007, 2009, 2011). The rock mechanics data are integrated with other disciplines (Figure 3.11). All Site Description Reports include the disciplines; surface conditions (e.g. soil), geology, rock mechanics (also thermal issues), hydrogeology and hydrogeochemistry. The latest reports (Posiva 2009, 2011) also include transport properties.

![Figure 3.11. Posiva’s Site Descriptive Model scheme.](image-url)
3.2.2 BACKGROUND ON GEOTECHNICAL AND THERMAL INVESTIGATIONS

Table 3.6 illustrates the geotechnical (mechanical and thermal) parameters that have been measured as part of surface-based investigations in Posiva’s siting and site investigation programme.

**Table 3.6.** Surface-based measured rock and soil geotechnical parameters (marked as x) in Posiva’s site investigation programme.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Preliminary investigations</th>
<th>Detailed investigations</th>
<th>Site confirmation phase</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Intact rock</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>Measured during rock mechanics and thermal lab tests. Gamma-gamma density from geophysical borehole loggings.</td>
</tr>
<tr>
<td>Young’s modulus (lab. test, field test)</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Shear, bulk modulus</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Calculated from Young’s modulus and Poisson ratio.</td>
</tr>
<tr>
<td>Fracture toughness</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Measured later (after 2004).</td>
</tr>
<tr>
<td>Creep properties</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>The creep effect was estimated (Eloranta et al. 1992). Creep was not considered important for hard crystalline rock.</td>
</tr>
<tr>
<td>Compressive peak strength (lab. and field test)</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Compressive crack damage strength (lab. test)</td>
<td>-</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Compressive crack initiation strength (lab. test)</td>
<td>-</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>-</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Shear strength</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Not considered an important parameter to understand Olkiluoto rock behavior.</td>
</tr>
<tr>
<td>P-wave velocity</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>Measured from core samples.</td>
</tr>
<tr>
<td><strong>Thermal</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thermal conductivity</td>
<td>x</td>
<td>x</td>
<td>-</td>
<td>Also calculated from mineral composition. Posiva has also developed a borehole tool to measure conductivity and diffusivity in situ (Kukkonen et al. 2005).</td>
</tr>
<tr>
<td>Thermal diffusivity</td>
<td>-</td>
<td>x</td>
<td>-</td>
<td>Also calculated from measured conductivity, specific heat capacity and density.</td>
</tr>
<tr>
<td>Specific heat capacity</td>
<td>-</td>
<td>x</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Expansion coefficient</td>
<td>x</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td><strong>Fractures</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fracture stiffness</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Due to the difficulty in realistically estimating the large-scale mechanical properties of fractures</td>
</tr>
<tr>
<td>Fracture cohesion</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Fracture friction</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Parameter</td>
<td>Preliminary investigations</td>
<td>Detailed investigations</td>
<td>Site confirmation phase</td>
<td>Note</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------------------</td>
<td>--------------------------</td>
<td>-------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Fracture dilation</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>from small-scale laboratory samples, emphasis was placed on understanding geometrical features of fractures and describing them geologically, such as fillings, roughness and alteration. However, some lab tests (cores from ONKALO) have been performed.</td>
</tr>
</tbody>
</table>

**Fracture/brittle deformation zones**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Preliminary investigations</th>
<th>Detailed investigations</th>
<th>Site confirmation phase</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fracture zone, strength, deformation</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Calculated/estimated from Q.</td>
</tr>
</tbody>
</table>

**Rock mass**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Preliminary investigations</th>
<th>Detailed investigations</th>
<th>Site confirmation phase</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>In situ stress</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Assumed to be between crack initiation and crack damage strength.</td>
</tr>
<tr>
<td>Deformation</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Calculated from Q.</td>
</tr>
<tr>
<td>P- and S-wave velocity</td>
<td>-</td>
<td>X</td>
<td>x</td>
<td>Measured from borehole loggings and VSP-soundings.</td>
</tr>
<tr>
<td>Rock quality, Q</td>
<td>-</td>
<td>X</td>
<td>x</td>
<td>GSI (Geological Strength Index) calculated from Q.</td>
</tr>
</tbody>
</table>

**Ambient temperature, geothermal gradient**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Preliminary investigations</th>
<th>Detailed investigations</th>
<th>Site confirmation phase</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ambient temperature, geothermal gradient</td>
<td>x</td>
<td>X</td>
<td>x</td>
<td>Data from flow logs and normal downhole geophysical logs.</td>
</tr>
</tbody>
</table>

**Soil properties**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Preliminary investigations</th>
<th>Detailed investigations</th>
<th>Site confirmation phase</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>-</td>
<td>X</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Grain size distribution</td>
<td>-</td>
<td>X</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Strength</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Consolidation properties</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Soil properties are not relevant due to relatively thin layer (2-4 m) above the bedrock.</td>
</tr>
</tbody>
</table>

Posiva’s site characterisation programme has also drawn on data available in the literature and experience from investigations in geological environments of similar rock type, tectonic settings and depth. Experience from underground research laboratories (URLs) has also been collected, particularly from the Canadian and Swedish URLs (Juvonen 2002).

The following are important aspects of the site characterisation programme:

- Sampling has been focused on particular rock units;
- Spatial variability has also been considered, with sampling in various boreholes and at various depths;
- Basic statistics are produced routinely (number of samples, means, distributions, medians, quartiles, confidence limits, geostatistics).

**Rock properties**

During rock mechanics laboratory testing, density, porosity and saturation were determined on core samples according to the ISRM suggested methods (ISRM 1986, 2007). The properties were calculated from specimen’s wet and dry mass and the volume (Hakala and Heikkilä 1997b). During the laboratory testing, density was determined by the Archimedean principle of weighing specimens in water and air (Kukkonen 2000). Geophysical gamma-gamma radiometric densities were measured as one parameter in the geophysical borehole loggings. The density values were additionally calibrated with petrophysical analyses (e.g. see Lahti et al. 2001).

Rock strength and deformation property tests have been performed according to the ISRM Suggested Methods (ISRM 1986, 2007). The first tests during the preliminary investigations consisted of uniaxial compressive tests to get the uniaxial compressive strength (UCS), Young’s modulus and Poisson ratio (Matikainen and Simonen 1992). As new
findings arose from the Canadian URL concerning the brittle rock strength concept (Martin 1994), Posiva started to develop the testing procedure (Hakala and Heikkilä 1997a). In the detailed investigation phase, the laboratory tests consisted of uniaxial compressive test, triaxial compressive tests, tensile tests and damage-controlled uniaxial and triaxial cyclic loading tests. Acoustic emission monitoring was also used in the compressive loading tests to detect initiation of micro cracking in the core sample (Hakala and Heikkilä 1997b). As a result UCS (or peak strength), peak strength as a function of the confinement, crack damage strength, crack initiation strength, tensile strength, Young’s modulus and Poison ratio were determined. Also, information was developed on how accumulated damage affects the strength values.

Because Olkiluoto migmatitic gneiss is anisotropic (foliated), the testing system was further developed in the site confirmation phase by using additional strain guages to better understand strength and strain anisotropy both in compression tests and tensile tests (Hakala et al. 2005). As a result, information was developed on how the strength values and deformation properties change with the foliation angle.

The UCS has also been estimated in the field using point load tests performed with a portable test unit termed the Rock Tester. At Olkiluoto, the point load test was performed systematically on rock cores during the site investigation core-drillings. The test was conducted for rock cores at about 30 metre intervals. Tests were made according to ISRM method (ISRM 1986, 2007). The test gives the point load index (Is) which then can be converted to the UCS. Description of the test procedure is found in Pohjanperä et al. (2005).

Other supporting rock property parameters, like the acoustic P-wave velocity, have been measured from core specimens in petrophysics study (Hakala and Heikkilä 1997b, Öhman et al. 2009b) and in acoustic logging in the boreholes (Öhman et al. 2009a). Intact rock strength is defined by laboratory tests, and the rock mass (volume of rock excluding the brittle deformation zones quality) is based on the empirical Q-classification system and associated Geological Strength Index (GSI) values (Posiva 2005, 2007, 2009, 2011).

Fracture properties

As indicated in Table 3.6, Posiva has focused on understanding geometrical features of fractures (orientation, length) and describe them geologically in terms of properties such as fillings, roughness and alteration. The fracture database has been used to try to estimate fracture properties (see Rautakorpi et al. 2003, §3.1 and Appendix 1). Stereoplots were developed to infer major fracturing sets and their types. Stereoplots and fracture continuities are essential input parameters for rock (key) block analysis. Only a few laboratory shear tests have been performed (Posiva 2009). The empirical Q-classification system has been used recently to map joint indexes (roughness number, alteration number) to estimate fracture properties (Posiva 2007, 2009, 2011) but this is not reliable from borehole cores (short trace lengths). The method works better from tunnel mapping.

Emphasis has also been placed on understanding the geometrical features of fracture zones (orientation, length, size) and again describing them geologically (fillings, roughness, alteration). No in situ tests on fracture zones have been performed. Again, the empirical Q-classification system and the associated GSI-value have been used to estimate fracture zone properties (Johansson et al. 2002, Posiva 2005, 2007, 2009, 2011). The methods used to estimate the properties of brittle deformation zones at Olkiluoto are presented in Hudson et al. (2008).

During the preliminary and detailed investigation phases, an older Finnish classification system called the Finnish Engineering Geological Rock Classification (Korhonen et al. 1974) was used to classify fracture zones based on the fracturing classes RI – RIV (ranges from RI=single joint type zone to RIV=fractured zone with clay filling) (Saksa et al. 1998, Åikäs et al. 2000). The classes had some relation to mechanical properties, although this relationship was not well established.

Fracture toughness tests have been performed on 12 core samples from the ONKALO (Siren 2012). The Mode I fracture toughness was determined using two different methods to account for two different fracturing directions. The methods are the Chevron Bend (CB) test as proposed in the ISRM Suggested Method and a method based on the Brazilian Disk (BD) experiment. The Mode II fracture toughness was determined using the Punch-Through Shear with Confining Pressure experiment on the remaining pieces from the CB testing.

In situ stress

The in situ stress state at various depths have been primarily measured from surface with two conventional methods: hydraulic fracturing (Klassen and Lejon 1990, Ljunggren and Klassen 1996, Ask et al. 2011) and 3D-overcoring with Borre probe (Ljunggren and Klassen 1996, Sjöberg 2003). The stresses were measured in deep boreholes at depths of 290 – 900 m during the site investigations. A summary of Olkiluoto stress state measurement results acquired by
2002 is presented in Malmlund and Johansson (2002), which stated that only 40% of the stress measurements in deep boreholes at Olkiluoto were successful.

An attempt was also made to study the possibility of estimating the in situ stress on cores using the Kaiser Effect. Tests were at depth level of 80 m and 500 m but the results were not encouraging (Lehtonen 2005).

Some indirect methods have been used to see possible indications of the high stresses in the boreholes; core discing observations (first ten deep boreholes at each site, except four boreholes at Hästholmen, Sacklen 1999) and borehole breakouts (Ask and Ask 2012). Core discing mapping has been done systematically since Sacklen (1999) as part of the geological mapping from all boreholes.

**Soil properties**

Grain size distribution and water content of 9 soil samples have been determined in laboratory tests (Hagros 1999). Mineralogical analyses with the XRD method, 31 geotechnical analyses including sedigraph (grain size) analysis, determination of organic matter (humus content), specific area and particle density grain size were conducted and 25 soil samples were chemically analysed (Lintinen et al. 2003). The sampling locations and description of the methods are described in Lintinen et al. (2003). Soil properties are not important for Posiva’s studies for underground construction.

**Thermal properties**

Thermal conductivity has been determined in laboratory tests (Kukkonen and Lindberg 1995, 1998, Kukkonen 2000, Kukkonen et al. 2011a) in boreholes using the TERO production logging tool developed for Posiva by the Geological Survey of Finland (Kukkonen et al. 2007, 2011c), and calculated based on theoretical calculations from mineral composition (Kukkonen and Lindberg 1995, 1998). The TERO device has been constructed currently for 76 and 56 mm diameter core drilled holes and is based on measuring conduction of heat from a 1640 mm long hollow aluminium cylinder. A foil-like heating resistor is placed on the inner surface of the hollow cylinder and a maximum heating power of 49 W is applied. A total of 28 thermistors are placed in four lines along the inner surface of the aluminium cylinder. Convection in the borehole is prevented with soft silicon rubber packers. A measurement heating time of 6 hours is followed by a cooling time of 12 hours.

In the laboratory tests, thermal conductivity has been measured with the steady-state divided bar method using an apparatus built at the Geological Survey of Finland (see e.g. Kukkonen 2000). In total, around 400 samples have been taken at the repository depths (Kukkonen et al. 2011a). Inaccuracies of thermal conductivity values are considered to be smaller than 5%.

The inverse temperature gradient method has also been used for testing purposes to estimate thermal conductivity in boreholes (Kukkonen et al. 2011b). The method requires good accuracy for temperature reading.

Thermal diffusivity has been measured with the TERO probe at the same time as thermal conductivity measurements (Kukkonen et al. 2011c). Thermal diffusivities have been also measured in the laboratory using a commercial instrument, ISOMET 104 (Kukkonen and Lindberg 1998). In addition, thermal diffusivities have been calculated from the laboratory values of conductivity, specific heat capacity and density (Kukkonen and Lindberg 1998, Kukkonen 2000, Kukkonen et al. 2011a).

Specific heat capacity has been measured on the same samples as used for conductivity measurements using a calorimetric method (Kukkonen and Lindberg 1998, Kukkonen 2000, Kukkonen et al. 2011a).

Borehole temperature data have been obtained from difference flow logs and normal downhole geophysical logs (Posiva 2011). Special temperature measurement campaigns have been also carried in some boreholes specifically to estimate thermal conductivities (Kukkonen et al. 2011b).

Thermal expansion properties have been measured on core samples with a Huggenberger Tensotast (Kjorholt 1992) and with an extensometer (DEMEC inv no 102266) (Åkesson 2012).

**3.2.3 GEOTECHNICAL DATA INTERPRETATION AND MODELLING**

**Rock and fracture properties**

It was realised in the early site investigation phase, based on initial results and experiences from other deep underground facilities (mainly mining) in Finland, that possible rock damage, spalling or failures due to stress/strength
ratio conditions may become evident in a deep disposal facility. Block failures were not likely at depth. Emphasis was then placed on understanding rock strength and the laboratory testing procedure was developed, as described above (Hakala and Heikkilä 1997a). The interpretation of the stress-strain loading curves was based mainly on the findings at the Canadian URL to determine the key strength parameters: peak strength, crack damage strength and crack initiation strength (Martin 1994, Hakala and Johansson 1994). Initially, it was important to understand the onset of damage (crack initiation strength) in rock samples. Damage controlled tests (cyclic loading) were then also conducted.

Determination of the key strength parameters is shown in Figure 3.12 and explained in Hakala and Heikkilä (1997a,b). The crack damage strength is defined as the reversal of the volumetric strain curve. At this point the total volume of the specimen turns from compaction to dilation. The crack initiation strength is defined as a stress level where the crack volumetric strain deviates from zero. In addition, acoustic emission (AE) measurements from cumulative AE counts are used to define the crack damage strength and the crack initiation strength. Primarily the initiation of microcracking is interpreted as the crack initiation strength (Figure 3.13).

The crack initiation strength as statistically interpreted was used as a rock damage indicator in the disposal facility in the first rock mechanics analyses (Johansson and Rautakorpi 2000), although it was understood that rock mass strength (spalling strength) is probably higher than the crack initiation strength. Determination of the rock mass strength can draw on information on the laboratory peak strength (i.e. the in situ spalling strength is of the order of 50 – 60 % of the uniaxial compressive strength) but in reality it requires in situ testing underground and is not possible from surface based investigations.

![Figure 3.12](image-url)  
Interpretation of the key strength parameters from a stress-strain curve determined by laboratory testing of a core sample, OL-KR10 400.01 m (Posiva 2008).
Figure 3.13. Interpretation of critical stress states from AE results (Hakala and Heikkilä 1997b).

As the main rock type in Olkiluoto is foliated, rock testing was developed further to perform anisotropic testing using additional strain gauges aligned according to the foliation (Hakala et al. 2005). The procedure to determine the elastic parameter is described by Hakala et al. (2005). Rock strength data interpretation was basically similar (strain values and AE counts) to that used in Hakala and Heikkilä (1997b). The main information obtained from such testing is how foliation affects the rock strength and deformation properties (Figure 3.14). The latter is especially important if an anisotropic solution is used in the interpretation of the rock stress measurements. The results exhibited deformation anisotropy of about 1.4 (ratio of modulus parallel to foliation/modulus perpendicular to foliation), which could be significant when evaluating the rock stress measurement data (Hakala and Sjöberg, 2006).

Field tests using point load tests on cores from all boreholes and laboratory data show a coefficient factor of 20 is feasible to determine the UCS of Olkiluoto rocks (Pohjanperä et al. 2005). ISRM suggests a factor of 20 - 25 should be used but states also that, especially for anisotropic rock, the ratio can vary between 15 and 50 (ISRM 1986, 2007). The rock strength has not shown any depth dependency (Posiva 2006).

A numerical code PFC3D (Particle Flow Code) developed by Itasca was tested to simulate the laboratory tests and further understand the effect of foliation on the rock properties (Hakala 1998, Wanne 2002). Based on simulation results, it was found that a particle mechanical model can represent many of the features observed in the laboratory tests (Hakala 1998). Numerical simulations were compared to laboratory observations and the schistosity or foliation was studied by generating an anisotropic particle structure of matrix particles and oriented band particles (Wanne 2002). Numerical responses and the laboratory tests were found to be quite similar (Figure 3.15).
Figure 3.14. Observed effect of anisotropy on critical strength (above) and deformation values (below) (Hakala et al. 2005).

Figure 3.15. Numerical laboratory tests simulations with PFC3D compared with laboratory tests a) laboratory test in blue (Hakala 1998) and b) laboratory tests in yellow shaded area (Wanne 2002).
Posiva has developed a Rock Mechanics Model (RMM) which is a description of significant features and parameters at Olkiluoto related to rock mechanics (Remes et al. 2009, Mönkkönen et al. 2012). The main objective was to develop a tool to predict rock quality, rock properties and the potential for stress failure which can be used for the design of the ONKALO and disposal facilities. The current version (version 2.0) of the RMM has sub-models of intact rock strength, \textit{in situ} stress, stress failure, thermal properties, rock mass quality, and the properties (quality) of the brittle deformation zones (Mönkkönen et al. 2012). Geostatistical analyses are used to determine interpolation or extrapolation distances for the RMM input data.

Figure 3.16 shows an example of the RMM for rock quality (GSI-values). Discrete planar coloured features are deterministic brittle zones. The underlying data in Version 2 of the RMM are from all surface based investigation boreholes and from the ONKALO access tunnel mappings. The basic block size is 10x10x10 m and the sub-block size is 2.5x2.5x2.5 m. GSI-values were estimated using an inverse distance weighting as the interpolating method (Mönkkönen et al. 2012).

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure3.16.png}
\caption{GSI-values of the Olkiluoto site (Version 2.0, Mönkkönen et al. 2012). ONKALO facilities are not included in the model, although shown here for visualisation purposes.}
\end{figure}

\textit{In situ} stress

In overcoring stress measurements, the \textit{in situ} stresses have been calculated conventionally from the measured strains and with knowledge of the elastic properties of rock. The complete three-dimensional stress tensor is determined from a single measurement. The rock has been assumed to exhibit a CHILE condition, i.e. Continuous, Homogeneous, Isotropic and Linear Elastic. The methodology for data evaluation and interpretation is found in Sjöberg (2003). Where the rocks show anisotropic behaviour, as at Olkiluoto, the anisotropy has been taken into account in the interpretation of the overcoring results (Hakala and Sjöberg, 2006). In such cases, the rock is assumed to exhibit a DIANE condition (Discontinuous Inhomogeneous, Anisotropic and Non-Elastic). The work by Hakala and Sjöberg (2006) included the development of the original CSIRA code by Amadei (1983) and modifications of the probes to determine elastic constants (Transversely Isotropic media). An example of the results is shown in Figure 3.17. The difference between the assumptions of CHILE and DIANE is, however, small.

As problems were encountered in the overcoring stress measurements, work was undertaken to improve the quality of interpretation of overcoring results (Hakala 2006). A computer program was developed to simulate the transient strains and stresses during overcoring in any \textit{in situ} stress and coring load conditions. The solution was based on
superposition of elastic stresses. Measured strains can be compared to calculations to check if the measured transient behaviour is accordant with the interpreted in situ state of stress. If not, the in situ state of stress can be calculated based on transient or final strain values. The transient stresses can be compared to the strength envelope of intact rock and thereby estimate core damage potential. The analysed case studies showed clearly the benefit of having an objective method to study the reliability of stress measurement data. The study also developed a comprehensive list of factors to be considered when performing in situ stress measurements, and procedures to interpret measured data in order to improve the quality of interpretation.

In hydraulic fracturing, in situ stresses have been calculated conventionally. The estimates were based on second break-down pressure, where tensile strength is estimated as the difference between the breakdown pressure and the re-opening pressure (Klasson and Leijon 1990, Ljunggren and Klasson 1992). Owing to problems getting vertical hydrofractures, methods such as sleeve fracturing and hydraulic testing on pre-existing fractures were also used in Olkiluoto in the recent hydraulic fracturing measurements (Ask et al. 2011). Also, en echelon theory was used in the recent measurements by Ask et al. (2011). This theory considers whether the observed induced fractures can be explained by tensile or shear failure; thus it is a tool that may provide additional information to help constrain the stress field. Some difficulties remain with hydraulic fracturing; one relates to the borehole inclination, which is not optimal for hydraulic stress measurements (the hole should be vertical, corresponding to the lithostatic vertical stress component); the other relates to the Olkiluoto stress regime, which is most likely a thrust environment, meaning that all the stress magnitude results from the hydraulic fracturing are questionable.

The stress state at the Olkiluoto site is expected to be locally perturbed by the presence of major brittle deformation zones (faults), therefore, large-scale stress modelling work has been conducted (Figure 3.18) (Valli et al. 2011). The modelling results have been compared with the stress measurement data with the objectives of increasing the understanding of the rock stress regime at the site and understanding the influence of deformation structures on the stress field locally.

![Diagram of principal stress orientations](image-url)

**Figure 3.17.** Principal stress orientations assuming either isotropic or transversely isotropic solution. Example from Olkiluoto borehole OL-KR24, 310.12 m (Hakala and Sjöberg 2006).
A supplementary methodology to interpret the in situ stress state in the case of extensive core discing has also been developed (Hakala, 1999). It is based on modelling core discing observations as a post-state analysis to gain understanding of in situ stress conditions. Interpretation involves the study of stresses induced by coring or overcoring and fracture growth using FLAC3D. The interpretation concluded that the spacing, shape and initiation point of discing are stress state-dependent. If direct stress measurements cannot be used, core discing and ring discing can be used to estimate the stresses. Interpretation assumptions and conceptual modelling involved assuming CHILE conditions, the borehole is aligned with the in situ stress, and the discing is caused by tension (Hakala 1999).

3.2.4 THERMAL DATA INTERPRETATION AND MODELLING

Thermal data (laboratory, in situ) have been analysed statistically according to main rock types: veined gneiss, tonalitic-granodioritic-granitic gneiss, diatexitic gneiss, mica gneiss and pegmatitic granite. The scatter of values, standard deviations of conductivity and the histograms of main rock types which overlap, depict the geological variability in the migmatitic rock formation as a whole. Thermal conductivity is anisotropic in Olkiluoto rocks and the lowest conductivities are measured perpendicular to foliation and the highest values along foliation with an average anisotropy factor of 1.25 ± 0.25. The average thermal properties have been corrected for the elevated temperature up to 60 °C (Kukkonen et al. 2011a).

Kukkonen et al. (2011a) suggest that one potential method to estimate thermal properties in larger scale is to utilise correlation between thermal conductivity and density (other correlations were also studied) as used by SKB for the major rock types measured. Density is easy and quick to measure in the laboratory and in boreholes, which would make it an efficient estimator. However, the coefficient of correlation was relatively low when comparing thermal conductivity and density for Olkiluoto rock types.

The in situ measurements with TERO devices refer to a much bigger volume (0.1 – 1.2 m³) than laboratory measurements. TERO data interpretation has the benefit of both the heating and cooling measurement. TERO data interpretation is based on either a numerical inversion method (Kukkonen et al. 2005, 2007) or a simpler analytical solution (Korpisalo et al. 2012). The analytical solution is based on the solution of an infinite line model and/or an approximate solution of the hollow tube model (Blackwell's large time solution, Blackwell 1954). Numerical inversion uses a time-dependent heat conduction model. In the interpretation the water layer, acting as resistance and a heat capacitor, is taken into account. The parameter determination is based on fitting measured temperature data with the forward modelling of heat transfer from a heated cylinder with a finite length and conductivity (2-D cylinder symmetric finite element model). Thermal conductivity and diffusivity are calculated. The inversion applies a damped Gauss-Newton method that locally approximates the given nonlinear problem with the linear least squares problem.
Challenges in the measurement instrument and data interpretation are water circulation leakages, small movements of the probe and instrumentation related noise (heat) (Kukkonen et al. 2005, 2007). In an earlier study, theoretical considerations with simulated data suggested that inversion results from the cooling part are less affected by various sources of bias. Results of the TERO device are in agreement with sample values. For example, the probe is sensitive particularly for conductivity in the radial direction from the borehole, and detected higher values of conductivity in the anisotropic gneiss compared to the sample (Kukkonen et al. 2011). Mineral banding runs typically across the boreholes.

When considering thermal properties within the disposal facility volume, upscaling is needed to treat the effect of parameter variations when the volume changes from small drill core sample sizes to bigger volumes, i.e. to about 10 m scale. The spatial correlation of thermal properties has been investigated with geostatistical methods, using variogram analysis and conditional simulations with kriging, as well as an estimation of the histogram properties at various scales (Mönkkönen et al. 2012, Posiva 2012). The experimental variogram is based on all measured values, without any classification of the data set according to rock types. Different rock types, however, show differences in their average and standard deviation values, and differences may also exist between their spatial correlation structures, but the present data set is too small to provide any reliable information regarding the possible difference between rock types. It might be possible to improve the variogram estimation using other variables, which would correlate with the thermal conductivity (and other thermal properties). An example of the thermal model is shown in Figure 3.19.

Sundberg et al. (2003) compared different measurement methods used by Posiva and SKB for thermal conductivity and specific heat of rock samples. They propose that for future investigations a set of thermal conductivity standard materials should be selected for calibration. The material should have thermal properties in the range of typical rocks, be fine-grained and suitable for making samples of different shapes and sizes.

Figure 3.19. Thermal properties at repository depth (350 – 500 m) based on geostatistical modelling (Mönkkönen et al. 2012).

3.2.5 DATA INTERPRETATION TOOLS

The rock mechanics model by Remes et al. (2009) and Mönkkönen et al. (2011) has been developed using the Gemcom Surpac® software and all data are stored in an Access database. Surpac supplies an interface for the database and it is used as an interpretation and visualization tool. Surpac is a commercial code but macros have been developed during this work. ISATIS software by Geovariances® was applied in the geostatistical analysis of thermal properties.

In the stress modelling work by Valli et al. (2011), a three-dimensional numerical code 3DEC based on the distinct element method was used (Itasca 2012a). The distinct element method uses an explicit time-marching scheme to solve the equations of motion directly. FLAC3D software was used to study core discing observations to interpret the
rock stress state (Itasca 2012b). PFC3D software was used to study the foliation effect on the rock properties (Itasca 2012c). Itasca codes are commercially available.

In the interpretation of the overcoring stress measurements, the original CSIRA code by Amadei (1983) was compiled as a Windows™ console application (Hakala and Sjöberg 2006). In order to make the use of the CSIRA code more convenient, a Microsoft Excel™ interface was developed for pre- and post-processing. The Excel interface was designed especially for both the Borre probe and the CSIRO-HI cell so that a minimum amount of input data (for a normally presented format) is required.

A graphical interface system was designed and constructed for the TERO borehole tool. Graphical Interface (TGI) is a versatile analysis and interpretation toolbox, which is developed in a MATLAB® environment. In addition, the TGI system requires the latest version of FEMLAB. The implementation of the analytical solution is also done with MATLAB® software. It is understood that the CSIRA code and TERO tool and interpretation techniques could be made available to RWMD site characterisation activities.

3.2.6 LESSONS LEARNT

Although general information on other research studies from similar geological conditions can been utilised, it is important to conduct site-specific investigations to understand the geotechnical properties of the site. The rock mechanics and thermal description of the Olkiluoto site has been satisfactory based on data from surface investigations. The underground facility has further confirmed the situation and allowed larger scale in situ testing, for example, to measure rock stresses. Understanding the rock stress regime at a site may be complicated if the site has a complicated deformation history and understanding the near-surface rock stress regime is always challenging.

It is important to understand upscaling factors, especially concerning rock strength and thermal properties. Establishing a strong link between the geotechnical model and the geological model provides the geometric framework to expand point values into volumes and is the main upscaling method for rock mechanics and thermal parameters. Statistical and geostatistical approaches have been shown to be very promising for predicting the geotechnical properties at a site and to understand the scale effects.

The links between different disciplines must also be considered. For example, rock and thermal properties have strong interactions with geology. This is because the lithological, brittle deformation and ductile models have been utilised in the development of the rock mechanics sub-models (the stress model, the thermal model and the rock mechanics model). Moreover, there are links with the hydrogeological and hydrogeochemical models, because these disciplines have used the same supporting geological information.

It is also important to evaluate and understand uncertainties relating to geological information and rock mechanics measurements and data. During Posiva’s programme, as the investigations have proceeded and more is learnt about the rock mass and its properties, the uncertainties have gradually been reduced. Uncertainties have reduced as the number of boreholes and associated investigations have increased from a few up to tens of boreholes. Consequently details in the models have increased. However, the reduction in uncertainties as the Posiva programme has developed has not been verified.
3.3 SELLAFIELD, UK

3.3.1 BACKGROUND

In the 1990s, Nirex was responsible for providing and managing facilities for the safe disposal of ILW and certain LLW, and in 1991, Nirex chose an area near Sellafield, west Cumbria, as the focus of its investigations into the development of a geological disposal facility. The Sellafield area lies over the transition zone between the western margin of the Lake District Massif, of Lower Palaeozoic metamorphic and igneous rocks, and the East Irish Sea Basin of younger sedimentary rocks.

By early 1997, 29 deep boreholes had been drilled in the Sellafield area as part of Nirex’s investigation programme, to provide information that allowed a three-dimensional (3D) picture of the geotechnical characteristics to be developed. The information from these surface-based boreholes, together with information from a variety of geophysical surveys, enabled Nirex to define what was termed the Potential Repository Zone (PRZ), an area underlain by a volume of rock which had been identified as having the potential for the development of a deep repository (see Figure 3.20, Figure 3.21 and Figure 3.22).

The potential repository host rock at Sellafield comprises basement rocks of the Ordovician Borrowdale Volcanic Group (BVG). These volcanic rocks form a sequence dominated by welded ignimbrite sheets which have a proven thickness in excess of 1 km and which, in the PRZ, are well-fractured and strong (Michie, 1996). The top surface of the BVG is at a depth of approximately 520 m in the central part of the PRZ (Figure 3.20). The Brockram, a 70 m thick sedimentary breccia of Permian age, rests unconformably on the BVG and is itself overlain and overlapped by early Triassic sandstones of the Sherwood Sandstone Group (SSG). The SSG consists of three formations, the lowest of which is the St Bees Sandstone Formation overlain in turn by the Calder and Ormskirk Formations (Barnes et al., 1994). The basal 80 m of the St Bees Sandstone Formation, the North Head Member, is differentiated from the overlying sandstone by an increase in the frequency of thin beds of claystone and siltstone, interbedded with sandstone.

All the boreholes within the PRZ provide evidence for faults and significant populations of relatively steep (>55°) faults are distributed irregularly throughout the PRZ. Displacements of the order of 5 to 25 m were estimated for many of these faults; the evidence for such structures is contained in reports such as Nirex (1996b).

Structurally, the PRZ lies between three major fault zones, the Lake District Boundary Fault Zone, the Seascale-Gosforth Fault Zone and the Fleming Hall Fault Zone (some of which are shown in Figure 3.22). The Seascale-Gosforth Fault Zone comprises a suite of east-north-east trending faults extending from Seascale with an increasing throw, reaching a cumulative displacement of several hundred metres down to the south and linking with the faults of the Lake District Boundary Fault Zone. South of the area the throw across the Lake District Boundary Fault Zone is in excess of 1.5 km and together, the Seascale-Gosforth and Lake District Boundary Fault Zones define a boundary of an onshore extension of the East Irish Sea basin. North of the Seascale-Gosforth Fault Zone the influence of the Lake District Boundary Fault Zone is much reduced and the Seascale-Gosforth Fault Zone transfers a large part of the displacement on the Lake District Boundary Fault Zone to the Fleming Hall Fault Zone. Locally the Fleming Hall Fault Zone forms the edge of the preserved Carboniferous rocks and marks changes in both sedimentary facies and thickness of the Permian and early Triassic rocks.

It is important to recognise that the work on geotechnical and thermal data interpretation and modelling was carried out before the SDM concept had been developed by SKB, and so was not designed to provide data that would be used in the development of an integrated model of the site. In particular, the geotechnical investigations were used mainly to provide indications of the stability of underground openings and for disposal facility design scoping studies (e.g. Rawlings et al. (1996), Davies et al. (undated)) and also for studies of the EDZ. As Davies et al. (undated) state: “much of the geotechnical interpretation and modelling work carried out has been directed towards gaining an understanding of the nature and extent of the EDZ, both in terms of its potential impact on the post-closure safety of a repository and on variations in the EDZ which could have an influence on selecting a preferred location and design for the repository vaults.” Some of this information and modelling is likely to be included in a future SDM developed for a site, in relation to the general stability of underground openings at depth. Much of it, however, would seem more likely to be presented separately in other reports, for example those associated with understanding the EDZ and also in relation to the design of the repository, as is currently the situation in Posiva’s programme at Olkiluoto.
Figure 3.20.  Deep boreholes drilled by Nirex at Sellafield.  For cross section see Figure 3.21.

Figure 3.21.  Simplified cross section through the Sellafield site.  For line of section see Figure 3.20.
3.3.2 BACKGROUND ON GEOTECHNICAL AND THERMAL INVESTIGATIONS

The primary objective of the geotechnical studies was to establish a database of information on the major rock units within the study area. The geotechnical database included descriptions of the geology, spatial properties of the discontinuities, rock material properties, in situ stresses and rock mass properties.

The rocks contain a significant number of discontinuities and it was readily apparent that a description of the rock needed to consider both the properties of the rock material and the discontinuities. Geotechnical information was obtained from a range of data acquisition activities including:

- Geotechnical logging of rock cores to determine rock mass characteristics.
Laboratory testing of rock cores to derive rock material parameters.

Correlation of static rock properties with dynamic rock mass properties derived from a range of geophysical investigations.

Collation of data to produce geotechnical datasheets for use in numerical modelling studies.

Measurements of thermal properties of core material.

Measurements of in situ stresses.

Thermal logging of boreholes to produce temperature logs.

An extensive programme of core characterisation was undertaken by the British Geological Survey. The programme consisted of undertaking a wide range of laboratory tests on intact core to obtain geotechnical, geophysical, hydrogeological and thermal properties for use in specialist studies. The testing programme was designed to provide a full range of geotechnical index data for correlation purposes and engineering data which, when combined with the index test results and other data, provided the input parameters for numerical modelling and design. A summary of the data obtained from Sellafield with direct relevance to geotechnical studies (strength and deformability, density, porosity and P wave velocity) is given in Nirex (1997b). These results were obtained from eleven boreholes.

The following strength and deformability parameters were measured:

- Unconfined compressive strength – determined from tests carried out on saturated 38 mm and 95 mm diameter samples.
- Indirect tensile strength – assessed using the Brazilian disc test.
- Triaxial strength – determined using the Hoek-Franklin triaxial test. The complete failure envelope was obtained using two data analysis techniques, the Mohr-Coulomb (linear) and the Hoek-Brown (non-linear) methods.
- Elastic parameters – derived from tests on single strain-gauged machine specimens under triaxial conditions to determine Young’s modulus and Poisson’s ratio.

Rock cores obtained from the deep boreholes were logged to derive measurements of rock quality, using both the Q system and the Engineering Rock Mass Classification, to derive values of Rock Mass Rating (RMR).

Simple index testing (roughness profile, tilt testing and Schmidt hammer tests) was also undertaken on selected joints identified in the rock cores to provide joint shear strength parameters (JRC, JCS and \( \phi_r \)). These data were used to predict the shear strength, coupled closure-flow, and shear-dilation-flow behaviour via the Barton-Bandis (BB) model used in the discontinuum UDEC-BB numerical modelling code. A total of 337 BVG fractures were index tested and these tests, together with the Q logging and orientation studies, were used to characterise the most geotechnically-significant fractures related to cavern stability. In addition, Direct Shear Tests were carried out on 18 fractures from the St. Bees Sandstone, two fractures from the Brockram, and 33 fractures from the BVG.

In addition to Direct Shear Tests, three Coupled Shear Flow Tests were undertaken on selected fractures from the St. Bees Sandstones, and four from the BVG. The objective of the Coupled Shear Flow Tests was to subject a representative rock joint sample to realistic levels of effective normal stress, while at the same time measuring its change in hydraulic conductivity and degree of closure with shear.

Thermal properties of the core were determined for each borehole (e.g., Entwistle et al. 1994; Jones and Gunn, 1995a, b). In some boreholes only the thermal conductivity was determined on a relatively small number of samples, in particular from the BVG, but also from some of the other geotechnical intervals (also referred to as lithostratigraphical units). In other boreholes other thermal properties were also determined (e.g., thermal expansion and specific heat capacity). These measurements were made on core from boreholes intersecting the PRZ, in particular the three Rock Characterisation Facility (RCF) boreholes seen in Figure 3.20.

Four methods were used to obtain the stress data, as summarised in Nirex (1997a): HFSM (Hydrofracture stress measurement), borehole breakout analysis, density log analysis and OCSM (Overcoring stress measurement).
3.3.3 GEOTECHNICAL DATA INTERPRETATION AND MODELLING

The majority of the geotechnical interpretation and modelling work carried out by Nirex was directed towards gaining an understanding of the rock mass with specific reference to the stability of underground openings, e.g. Rawlings et al. (1996). The areas of specific interest were:

- Interpretation of fracture data to produce statistically-corrected data sets.
- Distribution of rock mass properties, in terms of Q and RMR, in 3D and the analysis of the spatial variability of these properties.
- What were termed ‘gross geotechnical predictions’ related to the proposal to construct the RCF (Rock Characterisation Facility within the PRZ).
- In situ stress – azimuths and orientations of $\sigma_V$, $\sigma_{H_{\text{max}}}$ and $\sigma_{H_{\text{min}}}$ and their changes with rock type, location and depth.
- Precedent practice – the use of precedent experience of cavern construction was considered as one method of identifying feasible cavern geometrical envelopes for a range of rock mass quality over a range of cavern depths. The results of this study enabled the maximum spans for underground openings to be determined – these calculations were based on analysis of existing underground openings, together with the Q values determined for the BVG and for fault zones in the BVG.
- Repository design scoping studies (e.g. Nirex, 1995), specifically:
  - The nature and extent of the excavation disturbed zone (EDZ), both in terms of its potential impact on the post-closure safety of a repository and on variations in the EDZ which could have an influence on selecting a preferred location and design for the repository vaults. Modelling was carried out to examine further the likely magnitude and extent of the EDZ using two techniques (with specific reference to the issues of cavern orientation, cavern depth, and cavern spacing):
    - the distinct element code UDEC-BB to scope out the dependence on depth and orientation of the size of the EDZ around caverns (Nirex, 1995); and
    - a Rock Engineering Systems approach to scope out the coupled relationships to be considered for design purposes (Nirex, 1997c)
  - Cavern orientation.
  - Cavern depth.
  - Spacing between vaults.

Rock and Fracture Properties

Nirex planned initially to develop ‘Data Summary Sheets’ which, in combination with a parallel process of developing models, would be used to make predictions (initially for the RCF shafts), which in turn would be used to define the measurements, and to compare these predictions against the measurements that needed to be taken (Gibb, 1997). The Data Summary Sheets contained geotechnical parameters and characteristics for each of the identified major geotechnical units. These data fall into three types: (i) what might be termed ‘basic data’ (e.g., lithological description of the core), (ii) data which required some standard procedure of data manipulation to be performed (e.g., calculation of Q or RMR), and (iii) data where more complex interpretation, statistical analysis, etc. had to be performed (e.g., 3D block diagram of geotechnical-significant fractures). Data summary sheets were produced for the following geotechnical units:

- Undifferentiated St Bees Sandstone.
- North Head Member.
- Brockram.
- Faulted Longlands Farm (BVG).
- Altered Longlands Farm (BVG).
Bulk Longlands Farm (BVG).

The data for each unit comprised the following datasets:

- A description of the lithology.
- A representative 3D block diagram of the geotechnically-significant fractures.
- Material properties (from the core and from geophysical wireline logs).
- Spatial properties of fractures, based upon data abstracted from boreholes.
- Rock mass properties expressed as Q and RMR.
- Rock mass strength parameters.
- Rock mass deformability parameters.
- Rock mass hydraulic parameters.
- Fracture mechanical parameters (BVG only).
- In situ stress data.

In some cases alternative models would have been tested, such that the predictions and measurements made would have been designed to discriminate between different models. In the case of the geotechnical gross predictions, the models, and hence the predictions of gross rock mass response, were based on two alternative approaches: i) assuming the rock mass behaved as an elastic continuum, and ii) assuming the rock mass behaved as a discontinuum. In all cases, protocols were to be established in advance of any measurements being made in the RCF that would establish the extent to which it was expected that the measurements would match the predictions.

The plan was:

- For each lithostratigraphical unit (of which there were six in relation to the RCF – see above), determine parameters and rock mass constitutive models to describe the rock mass behaviour. A particular requirement of this study was that a wide range of parameters should be selected to allow subsequent geotechnical modelling to be carried out using a range of analytical and numerical techniques.
- For each lithostratigraphical unit, analyse the interpreted datasets from the key references to provide parameter values. Generally the ranges and appropriate means of parameter values were determined, although in some instances, the interpreted dataset size was too limited for this to be meaningfully carried out.
- Rock Mass Characterisation Data Summary Sheets were to be prepared for each lithostratigraphical unit, containing parameter values and sources for all data. These would contain the rock mass properties and the characteristics of the fractures.

Additional data processing was required to compile a database of geotechnically significant fractures (termed by discontinuities Nirex, as some were not strictly-speaking fractures, but could for example, be mineral-filled veins that could, in theory, open – this was the termed the Geotechnical Dataset), based upon the fracture logs and the BGS mineralisation logging. The detailed procedures followed to compile this dataset are described in Gibb (1997). Following compilation of the base dataset, discontinuity sets were selected and analysis was carried out to determine spatial characteristics. After preliminary analysis of orientation characteristics, it was decided to treat all discontinuity types as a single dataset, rather than to carry out analysis of individual types. Discontinuity set numbers were assigned, and orientation and spacing parameters calculated.

As part of this work a conceptual model for radioactive waste disposal in the Sellafield area was developed (REC, 1995). The model is based on the Fully-Coupled Model, which is part of the Rock Engineering Systems (RES) methodology. The approach was to consider the disposal process as a system and to specify the associated objectives. Then the variables defining the state of the system were identified (the state variables), together with all the pair-wise interactions between the variables. A coupling algorithm was invoked, so that all chains of mechanisms linking the state variables could be taken into account. This resulted in a Global Interaction Matrix (GIM) which enabled the consequences of any changes in the state variables to be established, i.e. the system outputs could be determined from the system inputs. The work was in several parts, with the work by REC concentrating on geotechnical aspects of the disposal system.
REC (1997a) presents the ‘gross discontinuum’ geotechnical predictions for Phase 1A of the RCF. The intention was that predictions made before shaft excavation could be compared with the observed rock response during and after excavation. The term ‘gross discontinuum’ referred to the fact that the pre-existing fractures in the rock mass could be taken into account in the studies and that the predictions should concentrate on the gross trends which would be exhibited by each of five geotechnical units: St Bees Sandstone, North Head Member, Faulted Longlands Farm, Altered Longlands Farm and Bulk Longlands Farm.

The predictions were in the three subject areas of: fracture frequency, potential rock block instability and shaft wall displacements. It was found that the local values of rock stress were not as predictable for a discontinuum and hence not as useful for discrimination purposes, because of their dependence on the idiosyncrasies of the local fracture geometry and the associated difficulties of obtaining reliable site values.

REC (1997b) presents studies that were carried out to predict displacements that would occur during shaft excavation in Phase 1A of the RCF construction. The studies were carried out using analytical and numerical methods on the assumption that the rocks likely to be penetrated by the shaft would behave as a homogenous, radially isotropic and linearly-elastic material.

It is unclear to what extent the results from any of this type of work would be included in any future SDM, as all of this work is associated either with the stability of underground openings or with investigations regarding the EDZ. An SDM, as defined by SKB and as used by the NDA, does not contain such information – it is contained within separate reports; although it is likely that reference would be made in an SDM to the possible extent and form of an EDZ.

Analysis of spatial variability

An area of particular interest was the development of an understanding of the spatial variability of the rock mass. In relation to the geotechnical activities, this was carried out with the specific objective of identifying suitable locations for the repository vaults at depth. A more important reason for this work was to develop a better understanding of the physical factors which influenced groundwater flow throughout the site. A pilot study was initiated to demonstrate the feasibility of predicting rock mass quality using geophysical data derived by combining borehole-derived measurements with seismic attribute data available from a trial 3D reflection seismic survey (Nirex 1997d).

The basic strategy employed by Nirex for this part of the investigations was to:

- Determine rock mass and fluid properties from borehole wireline logs and cores.
- Develop single, relative indices for rock quality and hydrogeological properties from the wireline log data; use geostatistical methods to investigate the downhole spatial variability of rock mass and fluid properties; derive borehole acoustic impedance from the wireline log data; relate borehole acoustic impedance to rock quality and hydrogeological indices using fuzzy logic methods.
- Correlate directly between borehole acoustic impedance and measures of rock quality, porosity and permeability.
- Invert the trial 3D seismic reflection data to produce a 3D seismic acoustic impedance dataset.
- Use geostatistical methods to determine the spatial variability of seismic acoustic impedance in three dimensions, and then apply that variability to the borehole-derived acoustic impedance dataset to provide an insight into the three-dimensional spatial variability of rock mass properties.

The first step involved the identification of rock mass properties of importance, and a comparison of the methods used to determine those properties, i.e. laboratory tests on core samples and in situ measurements from wireline logs.

Wireline logs of neutron porosity, bulk density, compressional wave velocity and shallow resistivity were used to develop a Wireline Property Log, which was combined with other parameters (the Stoneley Wave Reflection Coefficient and borehole records of Flow Zones and Potential Flowing Features) to produce rock quality and hydrogeological indices. The purpose of the indices was to enable data of interest for rock quality and hydrogeological assessments to be summarised and combined, in order to indicate the principal characteristics of the rock mass within a given volume.

Acoustic impedance was considered to be a key parameter in the search for a method of extrapolating rock mass properties from boreholes into the 3D rock mass, since it can be derived from wireline logs to give borehole acoustic impedance, and from the inversion of seismic data to give seismic acoustic impedance. Establishing a relationship...
between borehole acoustic impedance and the rock quality index, the hydrogeological index, and properties such as porosity and permeability, would thus enable these properties to be extrapolated into three dimensions by way of the seismic acoustic impedance volume.

Two approaches to the problem of relating rock quality and hydrogeological properties to acoustic impedance were investigated. The first approach involved the correlation of borehole acoustic impedance to the rock quality and hydrogeological indices, using fuzzy logic methods, and the second the direct correlations between borehole acoustic impedance and the standard geotechnical parameters Q and RMR, and between acoustic impedance, porosity and permeability. The two approaches are complementary, each allowing relationships between acoustic impedance and rock quality and hydrogeological properties to be established, and thus enabled those properties to be extrapolated throughout the 3D rock volume.

The 3D distribution of acoustic impedance was determined from the data obtained by the trial 3D seismic survey in the PRZ. Variography was used to determine the spatial variability of acoustic impedance derived from the seismic data set, and geostatistical simulation was undertaken to test whether realisations of 3D acoustic impedance could be generated for the PRZ from wireline log data. The simulation took the spatial variability of the seismic data and applied it to the wireline data, so that wire line-derived values could be extended away from boreholes in three dimensions.

Finally, the empirical relationships between acoustic impedance and RMR, porosity and permeability were combined with the 3D distribution of acoustic impedance to produce 3D images of the distribution of RMR, porosity and permeability in the volume of rock around the RCF (Figure 3.23).

Nirex emphasised that the studies undertaken were preliminary in nature and were aimed at investigating methodologies for incorporating geophysical data into estimations of rock mass and fluid properties, at scales determined by the resolution of the geophysical measurements. It was concluded that the work demonstrated the feasibility of determining rock mass properties, throughout a volume of rock, relevant to geotechnical and hydrogeological aspects of an underground repository. There were, however, some reservations associated with the limited data of the trial 3D seismic survey and the fact that certain aspects of the analytical approach required further consideration, for example the correlation between acoustic impedance and the rock quality and hydrogeological indices.

**In situ stress**

An integrated assessment of the in situ stress field at Sellafield was carried out and is summarised in Nirex (1997a) and described in considerably more detail in Nirex (1996a). The work was split into two major components: (i) the evaluation and presentation of the orientations of the maximum principal stress, \( \sigma_{Hmax} \) and (ii) the analysis and estimation of the magnitudes of the three principal stresses. The results were compared with two other indicators of the local stress field, firstly, the fault plane solutions of seismic events in the vicinity of the site and, secondly, the direction of the major faults in the proposed RCF area. The comparison demonstrated that all of the results were consistent with each other.

All the methods for determining the in situ stress assume that the data were obtained from homogeneous and isotropic rocks whose deformational responses are linearly elastic. Also, the HFSM and breakout data processing theories assume that the principal stresses are oriented vertically and horizontally.

The in situ stress orientations and magnitudes were estimated using independent methods. The primary orientation results were estimated from processing 72 sets of induced hydraulic fracturing and breakout images obtained from ten Sellafield boreholes by geophysical logging. The results were compared with those derived from overcoring stress measurements in one of the boreholes and with seismic fault plane solutions. The trends of the in situ principal stress magnitudes with depth were determined or computed from in situ density and hydrofracture stress measurement data from five boreholes and from the overcoring stress measurement results.

Data were obtained from 77 borehole zones and were processed to estimate the in situ principal stress magnitudes and orientations at Sellafield. All the boreholes are approximately vertical. The overcoring data were restricted to the depth range of 150 to 250 m in a single borehole but, otherwise, the coverage was comprehensive and consistent in all the boreholes. Corrected density log data, used for estimating the \( \sigma_v \) magnitudes, were obtained from five of the HFSM boreholes.
Figure 3.23. Distribution of RMR in 3D in the volume of rock around the three RCF boreholes. Interpolation algorithms allow extrapolation away from the boreholes. RMR provides an indication of the maximum size and depth of underground openings, related to stand-up time etc.

Breakout processing was, even then, widely accepted as a method of defining the ($\sigma_{\text{max}}$) azimuth and excellent agreement with other techniques had been reported in a variety of geological environments (e.g. Zoback, 1992). $\sigma_{\text{min}}$ magnitudes were estimated mainly from analysis of pressure versus flow rate data obtained from the slow refracture operations. Where the slow refracture results were not readily amenable to unambiguous interpretation, because of operational difficulties during flow testing, the shut-in pressure versus time data were used to estimate the $\sigma_{\text{min}}$ magnitudes. In the latter case, it was found that the shut-in pressure versus logarithm of the Homer time plot provided consistent results. The application of these alternative techniques ensured that suspect data were not used in the analysis.
A total of 29 HFSM tests were carried out - 28 of these provided useful data, from which $\sigma_{\text{min}}$ and $\sigma_{\text{max}}$ were obtained. The $\sigma_V$ magnitudes were assumed to be equivalent to the overburden pressures and were calculated by integrating the density logs of all five HFSM boreholes.

The orientation data were analysed statistically to seek reliable and valid correlations and an overall mean azimuth of all the data was derived (Figure 3.24). Mean values were also derived by borehole, by depth range and by lithology. They were then tested against fault plane data, local structural data and the overcoring results. Also, the mean values and the variances were compared statistically. Rose diagrams, showing the mean orientations and 95% confidence limits of the data were shown on borehole vertical sections, along with faults and lithologies. The data for each borehole and each formation were tested for Gaussian normality using the Shapiro-Wilks test. The dataset was also checked for correlations between azimuth and depth in the borehole.

The data were then compared. The HFSM, OCSM and breakout processing were all independent operations. The 29 HFSM tests were carried out in five boreholes over a depth range of approximately 660 to 1840 m, with 28 of these providing useful results. Investigations were carried out to investigate the effects of scale on the different stress measurement methods.

An analysis of the orientation data related to fault plane solutions for selected events was carried out to compare these results with those from HFSM. Fault plane solutions have a relatively large error and the spread of results is in excess of 30°, and it was concluded that the seismic data supported the results of the direct measurements.

The work to define the azimuth of the \textit{in situ} maximum principal stress at Sellafield involved processing and orienting data from many different sources. In particular, the various ultrasonic imaging (pre-UBI (Ultrasonic Borehole Imager)) measurements had to be checked and aligned to the orientation of the FMI logs. The close agreement and consistency of the results showed that this process was successful and did not introduce any systematic errors.

**Figure 3.24.** Depth trends of $\sigma_{\text{Hmax}}$ azimuth by measurement type and lithology (from Nirex, 1997a).
As regards the stress magnitudes, consistency of the derived stress magnitude was checked by comparing stress estimates for hydraulic fracturing and borehole breakout at specific locations, where these features were found close to each other. The checks also confirmed the need for further evaluation of the rock strength parameters obtained from laboratory testing on cores during these investigations.

The pore pressure data were interpolated from a collation from Westbay Monitoring Systems, backed up with reliable environmental monitoring analysis results. The computed $\sigma_V$ values at the HFSM locations were analysed using regression analysis.

Magnitudes of $\sigma_{H_{\text{max}}}$ were computed, either directly from the induced strains during overcoring or were calculated from HFSM results and pore pressure data. Two major sources of uncertainty influence the values of $\sigma_{H_{\text{max}}}$. Firstly, the value of $\sigma_{H_{\text{min}}}$ is scaled by a factor of three and therefore contributes three times the intrinsic variability of the $\sigma_{H_{\text{min}}}$ value. Secondly, the value of effective tensile strength of the formation is always uncertain. These effects were discussed extensively in Nirex (1997a) and, in particular, in Nirex (1996a).

The work demonstrated that the horizontal principal stress magnitudes could be estimated by linear regression equations, if the depth below ground level was known. Also, that the magnitude of $\sigma_V$ could be estimated, either by using the equation provided or by integrating density logs of boreholes in the area. All of these methods were used to estimate the stress magnitudes throughout the PRZ with minimal ambiguity.

Confidence in the derived trends was gained from comparison with the OCSM results, which gave in situ stress results in terms of the complete stress tensor. The OCSM results appeared to conform closely with magnitude-depth relationships derived from the HFSM and corrected density log data, and with orientations determined from HFSM and breakout analysis. This allowed Nirex to infer that the necessary assumptions made for HFSM and breakout analysis were indeed valid and that it was reasonable to predict stresses elsewhere in the PRZ using the derived trends. Also, more rigorous estimates of $\sigma_V$ from each borehole were determined using computed lithological thicknesses and mean pressure gradients obtained from the density logs of boreholes in the area of the proposed RCF. These revealed a spread of results which were in excellent agreement and well within the 95% confidence limits of the values obtained, thus confirming that the derived equation for $\sigma_V$ could be used for other areas of the Sellafield site.

### 3.3.4 THERMAL DATA INTERPRETATION AND MODELLING

Thermal conductivity measurements were made using a standard procedure from the ASTM (American Society for Testing and Materials), developed in 1987. In contrast to the other geotechnical measurements, there is no discussion of any of the thermal measurements, no analysis of their spatial variation, etc., apart from a single number for each type of thermal measurement indicating the potential error.

Thermal expansion was determined using a British Standard dilatometer technique used for engineering ceramics. Only a relatively small number of measurements were made on core from a single borehole; for example in boreholes RCF3 there were six measurements on the BVG.

Specific heat capacity was measured using the ASTM standard test for determining the specific heat capacity of thermal insulation. Again, only a relatively small number of measurements were made on core from a single borehole; for example in boreholes RCF3 there were six measurements on the BVG.

### 3.3.5 DATA INTERPRETATION TOOLS

The methods for converting the raw geotechnical data into indices that can be used to describe the geotechnical characteristics of the rock mass (i.e. the Q system and RMR) have been used for many years and are a standard procedure. Modifications are occasionally made to the RMR and Q systems, but the way these indices are determined has essentially remained the same, the main difference these days being that the values are determined more easily using standard software.

In addition, data from simple index testing on rock cores and fracture surfaces, as discussed above, were used to predict the shear strength, coupled closure-flow, and shear-dilation-flow behaviour via the Barton-Bandis (BB) model used in the discontinuum UDEC-BB numerical modelling code. Geotechnical software is now employed to make calculation of the relevant rock behaviour indices easier than it probably was in the 1990s - there have been many advances in the development of the UDEC software, which is still used for modelling in 2D, but which has now been generally superseded by other codes which permit the reliable 3D modelling of rock masses, and which are routinely used in radioactive waste disposal programmes around the world.
There is little discussion in the documents about the accuracy, usefulness, etc. of the data derived from wireline logging tools, except with regard to the determination of fracture orientations, as described above. These wireline tools are continuously improving and it is likely that it will be possible to obtain more useful data that could be used for geotechnical purposes in any future investigation than was the case for the investigations at Sellafield in the 1990s.

The methods now used to interpret in situ stress measurements are similar to those used in the 1990s, although there is a better understanding of the effects of rock properties, such as rock mass anisotropy and the presence of large fracture zones, on the stress values obtained. 3D modelling of such effects is now carried out, using codes such as 3DEC, and there is a better understanding of the concept of the Representative Elementary Volume (REV) when applied to the interpretation of stress measurements.

3.3.6 LESSONS LEARNT

It is unclear what lessons might have been learnt from the Sellafield investigations, as none of the reports that discuss and summarise the geotechnical programme carried out discuss such matters. There is also very little discussion in Nirex (1997a) regarding the overall conclusions of the in situ stress measurement programme. There is only discussion of some specific problems associated with some of the stress measurements.

Although some specific codes were written for analysing some of the data (e.g., by BGS for some of the rock core characterisation programme), these were only simple codes and are unlikely to be of any use in any future programme, as more suitable geotechnical software is now easily available that did not exist in the 1990s.
4 GDF CASE STUDIES: LOWER STRENGTH SEDIMENTARY ROCKS

4.1 OPALINUS CLAY PROJECT, SWITZERLAND

4.1.1 BACKGROUND

NAGRA (the National Cooperative for the Disposal of Radioactive Waste) was set up in 1972 by nuclear power plant operators and other organisations that use radioactive materials in Switzerland to carry out research into the geological disposal of radioactive wastes. Site investigation and characterisation has involved regional and site-scale studies aimed at identifying disposal sites for L/ILW, spent fuel and vitrified HLW from spent fuel reprocessing. NAGRA’s research includes consideration of spent fuel, HLW and long-lived ILW disposal in the same underground facility. Key phases in NAGRA’s programme to develop a geological disposal facility (GDF) for spent fuel and HLW are depicted in Figure 4.1 (NAGRA, 2002a).

Since the early 1980s, NAGRA’s research on potential host rocks has been concentrated in northern Switzerland. Areas exhibiting conditions favourable for disposal (geologically simple setting, low tectonic deformation, low tectonic activity, adequate thicknesses and low hydraulic permeability) were identified in the Central Plateau and Jura Mountains. Surface-based investigations were initiated in northern Switzerland in 1981, with the first phase involving project Gewähr, which explored crystalline basement rocks under the sedimentary cover rock sequence. These investigations continued in the form of the site-specific project Kristallin-I until 1994. Site characterisation had been extended after 1988 to include sedimentary rocks in northern Switzerland. After a desk-based study to compare and evaluate potential disposal facility settings, the Opalinus Clay and the Lower Freshwater Molasse were identified as the most promising sedimentary rock options (NAGRA, 1994, §1.2) and, since 1994, the Opalinus Clay in the Zürcher Weinland area has been the main focus of NAGRA’s site investigations (NAGRA, 2002b).

Figure 4.1. Key phases in NAGRA’s GDF research programme (NAGRA, 2002a).
The Opalinus Clay formation is characterised as an over-consolidated shale formation (the overburden thickness varies, but is estimated to have been at least 1000 m in the past) and deposited around 180 Ma (Aalenien). Opalinus Clay consists mainly of incompetent, silty and sandy shales, with mainly clay minerals and micas, some quartz, and calcite and inclusions of siderite, pyrite and organic carbon (NAGRA, 2002c, §3.2 and §5.3).

The Opalinus Clay is considered suitable as a host rock, because:

- It is lithologically homogeneous and of sufficient thickness (100 to 120 m) for a disposal facility.
- It is located at a depth of at least 450 m below ground surface in the investigation area, and is located at depths of 650 to 850 m in the 22 km² 'first priority' sub-area for the potential disposal facility.
- Favourable rock properties have not been altered by geological events and the rock is not an economically viable natural resource.
- It has a simple layer topography and slight variability of facies and has only slightly disturbed subhorizontal bedding.
- The predictability of the formation properties is high due to a good understanding of the geological evolution of the investigation area.
- It has crystalline basement rock directly beneath the Mesozoic sedimentary rocks.
- It allows flexibility in the placement of underground facilities due to the near-constant thickness and its large lateral extent. The slight tilt of the rock (between 3 and 6 degrees towards the south-east) makes it possible to select the depth for the repository.

As part of its Opalinus Clay research programme, NAGRA is involved in an international research project that is running experiments at the underground rock laboratory at Mont Terri, which is situated in the Opalinus Clay formation. The Mont Terri Rock Laboratory is accessible through the safety gallery of the Mont Terri motorway tunnel. In the area where the laboratory is located, the overburden varies between 230 and 320 m and the rock strata dip takes a steeper angle of approximately 45° to the southeast. Regarding the experiments undertaken at this underground research laboratory, only the data interpretation and modelling techniques that can be applied in a surface based site characterisation programme have been considered in this project case study.

Data on the Opalinus Clay are also available from drilling programmes undertaken for resource exploration in the area, and NAGRA has used these data in its site characterisation work. NAGRA also keeps a watching brief on rock characterisation work carried out in other projects (e.g., civil engineering projects), particularly in Switzerland (NAGRA, 1994, Section 3.6). Review of the literature and direct contact with the geologists involved provides NAGRA with valuable generic information (e.g., rock-mechanics data).

NAGRA has published the findings of its investigations of the Opalinus Clay in a series of project reports, many of which are written in German. The main source of geoscientific information for the Opalinus Clay is the geoscientific synthesis report from Project Opalinus Clay (NAGRA, 2002c), which is written in German. Information on geodynamics and geotechnical and thermal properties is presented in Sections 3.5, 3.8 – 3.9, 4.4 to 4.5, 5.6 to 5.7 and 7.2 of the geoscientific synthesis report, and this material provides the main source of information for the project case study; supporting documents referred to in NAGRA (2002c) are generally written in German and have not been consulted.

4.1.2 BACKGROUND ON GEOTECHNICAL AND THERMAL INVESTIGATIONS

The surface based investigations carried out as part of Project Gewähr and the Kristallin-I project included (Thury et al. 1994):

- Drilling seven deep boreholes between 1982 and 1989 to depths of between 1300 and 2500 m, covering a total of 6400 m in overlying sediments and 5900 m in the crystalline rock.
- Undertaking 400 km of 2-D seismic reflection surveys and 230 km of seismic refraction surveys during 1991 and 1992.
- Regional neotectonic studies, including stress measurements.
- An extensive borehole measurement and logging programme that included geotechnical and thermal investigations.
Several of the deep boreholes penetrated the Opalinus Clay layer, and the clay cores were studied as part of NAGRA’s investigation programme. Table 4.1 indicates the geological setting of the boreholes in the crystalline basement study area (Mazurek, 1998). Table 4.1 includes deep boreholes that have been drilled as part of resource exploration programmes in the area, for which information on the Opalinus Clay is available. NAGRA has used these data in its site characterisation work.

NAGRA’s site investigation programme focused on gaining an understanding of hydrogeological conditions, supported by geological and geophysical studies. Geotechnical measurements were made, but these were not the main focus of the borehole investigations, as is evident from the listing of the main technical reports from the drilling programme shown in Table 4.2 (Thury et al. 1994). However, conventional geomechanical methods, such as uniaxial and triaxial pressure tests, tensile tests and creep tests, were performed to provide geotechnical inputs to planning for shaft and tunnel construction. In additional, the direction of the maximum horizontal rock stress was determined on the basis of oriented borehole diameter measurements (borehole wall asymmetry and breakouts) using a technique that was under development at that time.

The geophysical logging results (Table 4.2) include borehole stress measurements and temperature (gradient) logging. Note that the ambient temperature logging was different to the temperature measurements made as part of the fluid logging undertaken after the drilling was completed (Thury et al. 1994). The latter was undertaken as part of the logging to locate borehole-induced short-circuiting and to map in- and out-flow points (i.e. measurements were made of electrical conductivity and temperature along the borehole under pumping conditions to locate groundwater inflow with high precision (± 1 m)).

The 2-D seismic survey undertaken in 1991 to 1992 as part of Project Gewähr supported the selection of the Opalinus Clay for investigation as a potential host rock. This survey was complemented by data sets compiled from oil and gas industry investigations, resulting in almost complete coverage of the eastern Swiss Molasse basin and the eastern Tabular Jura. The analysis and interpretation of these data sets provided the basis for NAGRA’s structural models of the area.

The second phase of surface-based site investigations and the characterisation programme for the Opalinus Clay (undertaken from 1994) included:

- Regional geological and tectonic analysis.
- A 3-D geophysical seismic reflection survey on an area of approximately 50 km$^2$ (during 1997).
- Drilling of the 1007 m Benken borehole and associated measurements and rock sample analyses (1998 to 1999).
- Experiments as part of the international research programme in the Mont Terri rock laboratory.
- Regionally based comparative studies of the Opalinus Clay as well as comparisons with clay formations investigated in other countries.

The 3-D seismic survey in the Zürcher Weinland provided structural data for a potential siting region of about 50 km$^2$ (NAGRA, 2009). A map of the potential siting area is shown in Figure 4.2 and a geological cross-section of the region is illustrated in Figure 4.3 (NAGRA, 2002a).

Considerable effort was also put into analysis of geothermal characteristics at the regional scale. Study of data from 59 boreholes in northern Switzerland at depths in the range of 160 – 5448 m produced temperature variation and heat flow maps calculated from the temperature gradients and thermal conductivities of the rock sequences in conjunction with structural and tectonic geology data from northern Switzerland and southernmost Germany and integrating hydrogeological data and knowledge. Two anomalous high heat flow areas (up to 170 mW/m$^2$) were observed, some 30 – 40 km to WSW and SW from the Benken borehole site. They were interpreted to be associated with the Permo-Carboniferous trough, where ascending deep groundwater flow is bounded by low permeability zones and driven up by recharge along the northern edge of the Alps and the southern Black Forest.

NAGRA has also participated in geothermal well investigations to assess potential geothermal energy development from the Tertiary upper sedimentary rock layers (for example, in Blaser et al. 1994). Geothermal energy, as well as thermal and mineral water resources, was also evaluated as possibly forming a part of the natural resource inventory of the proposed siting area, but it was concluded that the site does not have potential in these aspects.
Table 4.1. Summary of the boreholes that penetrated the crystalline basement in the study area (Mazurek, 1998). The boreholes at Böttstein, Weiach, Riniken, Schafisheim, Kaisten, Leuggern and Siblingen were commissioned by NAGRA.

<table>
<thead>
<tr>
<th>borehole (abbreviation)</th>
<th>coordinates</th>
<th>drilled for</th>
<th>year</th>
<th>top crystalline level m</th>
<th>crystalline rock types</th>
<th>sediments above top crystalline level, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Böttstein (BOE)</td>
<td>659341/266556</td>
<td>radwaste</td>
<td>1982/83</td>
<td>315</td>
<td>biotite granite</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Herden (HER)</td>
<td>711308/274597</td>
<td>oil/gas</td>
<td>1981/82</td>
<td>2127</td>
<td>biotite orthogneiss</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Kaisten (KAI)</td>
<td>644641/265824</td>
<td>radwaste</td>
<td>1984</td>
<td>297</td>
<td>mainly metapelite gneiss</td>
<td>Early Permian (Rötliengen)</td>
</tr>
<tr>
<td>Koblenz (KOB)</td>
<td>661759/273936</td>
<td>salt</td>
<td>1954</td>
<td>157</td>
<td>biotite gneiss</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Kreuzlingen 1 (KRE)</td>
<td>729201/276169</td>
<td>oil/gas</td>
<td>1962</td>
<td>2534</td>
<td>cordierite-biotite melagranite</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Leuggern (LEU)</td>
<td>657664/271208</td>
<td>radwaste</td>
<td>1984/85</td>
<td>223</td>
<td>mainly metapelite gneiss above biotite granite</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Lindau 1 (LIN)</td>
<td>692815/255098</td>
<td>oil/gas</td>
<td>1964</td>
<td>2305</td>
<td>cordierite-biotite melagranite</td>
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<tr>
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<td>632708/231789</td>
<td>oil/gas</td>
<td>1963</td>
<td>1824</td>
<td>biotite gneiss</td>
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</tr>
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<td>623993/266209</td>
<td>coal</td>
<td>1875</td>
<td>367</td>
<td>banded series</td>
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</tr>
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<td>thermal water</td>
<td>1983</td>
<td>372</td>
<td>banded series above granite</td>
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<td>662460/272834</td>
<td>salt</td>
<td>1969/70</td>
<td>319</td>
<td>biotite gneiss</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Schaffisheim (SHA)</td>
<td>653620/248760</td>
<td>radwaste</td>
<td>1983/84</td>
<td>1490</td>
<td>granite, syenite, monzonite, diorite granite</td>
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</tr>
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<td>salt</td>
<td>1823/24</td>
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</tr>
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<td>radwaste</td>
<td>1988/89</td>
<td>349</td>
<td>cordierite-gneiss, andalusite-cordierite-bearing twomica granite</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
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<td>1983</td>
<td>2020</td>
<td>biotite gneiss</td>
<td>Late Carboniferous (Stephanian)</td>
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<td>Wintersingen (WIN)</td>
<td>629106/261606</td>
<td>coal</td>
<td>1939</td>
<td>421</td>
<td>andalusite-cordierite-bearing twomica granite</td>
<td>Early Permian (Rötliengen)</td>
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<tr>
<td>Zurzach Z1 (ZUR)</td>
<td>663952/271229</td>
<td>salt</td>
<td>1913/14</td>
<td>415</td>
<td>andalusite-cordierite-bearing twomica granite</td>
<td>Early Triassic (Buntsandstein)</td>
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<td>Zurzach T1 (ZUR)</td>
<td>663972/271224</td>
<td>thermal water</td>
<td>1955</td>
<td>415</td>
<td>andalusite-cordierite-bearing twomica granite</td>
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<td>thermal water</td>
<td>1965</td>
<td>425</td>
<td>andalusite-cordierite-bearing twomica granite</td>
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<tr>
<td>Zurzach Z3 (ZUR)</td>
<td>663742/271482</td>
<td>thermal water</td>
<td>1979/80</td>
<td>402</td>
<td>andalusite-cordierite-bearing twomica granite</td>
<td>Early Triassic (Buntsandstein)</td>
</tr>
<tr>
<td>Zuzan 1 (ZUZ)</td>
<td>635030/263400</td>
<td>coal</td>
<td>1939/40</td>
<td>256</td>
<td>andalusite-cordierite-bearing twomica granite</td>
<td>Early Permian (Rötliengen)</td>
</tr>
<tr>
<td>Zuzan 2 (ZUZ)</td>
<td>635410/263470</td>
<td>coal</td>
<td>1940</td>
<td>243</td>
<td>andalusite-cordierite-bearing twomica granite</td>
<td>Early Permian (Rötliengen)</td>
</tr>
</tbody>
</table>
Table 4.2. Reporting structure for NAGRA’s deep drilling programme in crystalline rock site characterisation (Thury et al. 1994).

<table>
<thead>
<tr>
<th>Nagra NTB</th>
<th>Böttstein</th>
<th>Weiach</th>
<th>Riniken</th>
<th>Schafisheim</th>
<th>Kaisten</th>
<th>Langgurn</th>
<th>Siblingen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geophysical Logging</td>
<td></td>
<td></td>
<td>WEGER et al. 1986</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Fluid Logging</td>
<td></td>
<td></td>
<td>NAGRA 1980a</td>
<td></td>
<td></td>
<td></td>
<td>KELLEY et al. 1991</td>
</tr>
<tr>
<td>Gas in Drilling-fluid</td>
<td></td>
<td></td>
<td>HINZE et al. 1986</td>
<td></td>
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</tr>
<tr>
<td>Long-term Monitoring</td>
<td></td>
<td></td>
<td>SCHNEIDER &amp; SCHLANKE 1986</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Multipacker</td>
<td>PICKENS et al. 1985</td>
<td>BELANGER et al. 1990</td>
<td>MoNEISH et al. 1990</td>
<td></td>
<td>BELANGER et al. 1990</td>
<td></td>
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<tr>
<td>Water Sampling</td>
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<td></td>
<td>WITWER 1986</td>
<td></td>
<td></td>
<td></td>
<td>BLASER &amp; SCHOLTIS 1991</td>
</tr>
<tr>
<td>Hydrochemical Data Set</td>
<td>PEARSON 1985</td>
<td></td>
<td>PEARSON et al. 1986</td>
<td></td>
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<td></td>
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<tr>
<td>Isotope Hydrogeology</td>
<td></td>
<td></td>
<td>PEARSON et al. 1991</td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>
Figure 4.2. The 3-D seismic reflection survey area and the potential repository and ‘first priority’ areas (NAGRA, 2002a).

Figure 4.3. NW-SE geological cross-section along the potential siting area in the Opalinus Clay (NAGRA, 2002a).
The Opalinus Clay is a homogeneous claystone formation that was deposited uniformly over large areas of Northern Switzerland. As a result, parameters determined at other locations (e.g., in the Mont Terri Rock Laboratory) are transferrable to the investigation area in the Zürcher Weinland. However, analyses of geotechnical behaviour required that the differences in local rock-independent boundary conditions (e.g., the different host rock overburden and stress field) had to be taken into account (NAGRA, 2002c). The geometrical boundaries of the tectonically quiet Opalinus Clay layer in the Zürcher Weinland area were determined accurately from the 3-D seismic campaign (NAGRA 2002a).

The geotechnical and thermal parameters and measurement techniques used in the investigation programme are summarised in Table 4.3 (NAGRA, 2002c).

<table>
<thead>
<tr>
<th>Parameter category</th>
<th>Specific type</th>
<th>Parameter(s)</th>
<th>Method of measurement (reference)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geotechnical</td>
<td>Rock petrophysics</td>
<td>Bulk, matrix and grain densities</td>
<td>From core samples in laboratory, bulk density from logging, literature data.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Porosity, water content</td>
<td>Derived from various density measurements of the core samples.</td>
</tr>
<tr>
<td>Rock deformation properties</td>
<td>Young’s modulus (E-modulus), Shear modulus, Poisson’s ratio and anisotropy</td>
<td>Laboratory testing of core samples and borehole in situ densities, laboratory P- and S-wave velocities, borehole dilatometer.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Swelling, amount and pressure, anisotropy</td>
<td>Odometer testing, literature data, Mont Terri rock laboratory.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Creep properties</td>
<td>Core sample tests at confining stresses 0.6 and 10 MPa.</td>
</tr>
<tr>
<td>Rock strength†</td>
<td>Compressive strength</td>
<td></td>
<td>Uniaxial and triaxial tests of core samples with varying porosities and water content, borehole dilatometer testing.</td>
</tr>
<tr>
<td></td>
<td>Tensile strength</td>
<td>Laboratory Brazilian test, along and cross-stratification.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Shear strength</td>
<td>Triaxial testing.</td>
<td></td>
</tr>
<tr>
<td>In situ stress</td>
<td>Magnitude and direction of principal stresses</td>
<td>Hydraulic fracturing, borehole breakage analysis, regional seismicity studies.</td>
<td></td>
</tr>
<tr>
<td>Fracture properties</td>
<td>Toughness</td>
<td>No fracturing in natural conditions; induced fracturing and changes have been analysed.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Normal and shear stiffness</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Cohesion</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Dilation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil properties</td>
<td>Particle size distribution</td>
<td>Not explicitly reported; stated that taken from existing databases.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Density</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Consolidation properties</td>
<td>Discussed in connection with micro-mechanical modelling.</td>
<td></td>
</tr>
<tr>
<td>Thermal</td>
<td>Rock petrophysics</td>
<td>Thermal conductivity</td>
<td>From core samples in the laboratory in saturated and dry conditions.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Specific heat capacity</td>
<td>Sample measurements, calculation based on principle of two mixed substances.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Thermal expansion coefficient</td>
<td>Axial deformation of the core samples (heating-cooling cycle).</td>
</tr>
<tr>
<td></td>
<td>Formation</td>
<td>Ambient temperature and thermal gradient</td>
<td>Continuous logging and discrete recordings in long-term monitored sections, continuous gradient and per rock unit calculated.</td>
</tr>
</tbody>
</table>

†Core samples were preserved in a pressure vessel to prevent disturbances.
4.1.3 GEOTECHNICAL DATA INTERPRETATION AND MODELLING

**In situ stress**

The hydraulic fracturing method was used in the Benken borehole to measure the magnitudes of the principal horizontal stresses. Results were obtained from 13 measurement levels in a depth range of 150 to 800 m. The largest horizontal stress direction $170^\circ \pm 10^\circ$ deviates slightly from that measured in other boreholes in northern Switzerland and from the general stress field orientation in Central Europe, but is consistent with the regional seismicity and earthquake studies. Seismicity analysis is in turn connected to the regional tectonic analysis carried out. The interpretation of principal stresses took into account the influence of pore water pressure and applied elasto-plastic or poro-elastic approaches. The calculated ratio between principal stress components is about 1.5, which is slightly lower than is given by standard interpretation (about 2.0) when pore pressures are incorporated in the analysis. Pore water pressures of 5 MPa have been reported (NAGRA 2002c, Section 5.7.3), but no further details are available on the data processing.

The fracturing induced during drilling and borehole wall breakouts were mapped and interpreted, which gave reliable estimates of the stress direction. Borehole wall breakouts were analysed using an analytic cylindrical geometry model (the Kirch equations for 1-D state). Two bounding conditions were assumed, covering situations in which the borehole was stable when silica rinse was used and in which strong breakout occurred when the borehole was water flushed for hydraulic testing. Compressive strengths were available for normal and relieved stress conditions, such that stress distributions with known minimum horizontal principal stress could be estimated. Under normal conditions, the water content of the rock was about 4.5 wt% and under relieved stress and increased pore space conditions the water content was about 6 wt%. The maximum to minimum stress ratio was interpreted to be between 1.1 and 1.6 and the maximum stress component at a depth of 650 m was estimated to be about 20 MPa. The minimum horizontal and vertical principal stress components in the host rock at greater depths were estimated to be approximately equal in magnitude. The stress field was considered to be compatible with the regional tectonic setting.

**Rock properties**

The geomechanical properties of the clay are determined largely by its microstructure and the degree of compaction and the water content are of great significance. For example, there is a strong dependence of rock strength on water content. The measurement and interpretation of rock porosities and densities is important in determining the water content.

The modulus of deformation and Poisson’s ratios were calculated by NAGRA (2002c) from ultrasonic testing of Benken borehole core samples (density, P- and S-wave values). The laboratory analysis measured properties perpendicular, parallel and at an angle of 45° to bedding to evaluate anisotropic behaviour. Measurements were made on 15 to 27 core samples (depending on orientation to bedding) and mean values and standard deviations were evaluated (NAGRA 2002c, Section 5.7.3). In situ values were calculated from geophysical well logging curves and by using borehole dilatometer sondes. A total of 153 sampling locations were analysed using indirect geophysical measurements.

The laboratory and *in situ* test conditions differed in that laboratory samples could indicate possible damage and increased porosity due to pressure release. The data interpretation also considered the scale-dependence of the analysis (core samples on the centimetre scale were used in the laboratory whereas borehole measurements were on the scale of metres) and the different test frequencies applied (1 MHz in the laboratory and 5 to 15 kHz in the borehole). Laboratory and *in situ* measurements were compared and differences assessed. No significant differences were observed between the laboratory and *in situ* values. Measurements of the modulus of deformation were also compared with measurements made with a dilatometer test in the Mont Terri laboratory with good agreement (normally they differ and the dynamic value is larger than the static value).

Data interpretations and analyses also covered:

- Volumetric deformation as a function of pressure in porous media.
- Normal stress induced porosity changes, loading and load release reversibility (compaction and decompaction).
- Isostatic stress conditions for normally consolidated and overconsolidated states.
- Anisotropy with respect to stratification.
The following tests on clay rock strength were reported:

- Uniaxial compressive strength against water content (wt%).
- Axial deformation against axial stress in triaxial testing, with varying water content.
- Axial deformation against volumetric deformation, with varying water content.
- Comparison of uniaxial strength values extrapolated from triaxial test and from uniaxial tests, with varying water content.
- The effects of sample orientation (anisotropy) with varying water content.

The interpretation of rock strength utilised triaxial and uniaxial tests and extrapolation of uniaxial strengths from triaxial testing. In addition, a deformation model for the clay was applied assuming elastic, strain hardening and strain softening regimes. Water content was found to be the main factor affecting the strength of the Opalinus Clay, as indicated in Figure 4.4, and it was recognised that the absolute water content rather than the degree of water saturation determines the rock strength via various contact and capillary processes.

Although special pressurised core vessels were used in drilling and during geological inspection, microcracking occurred, and the measured strengths of rock samples are conservative. Loading rate and cycle lengths were also studied, and a slight but significant influence was observed. Higher strengths were measured at faster loading rates due to lower micro crack lengths (10% difference in strength in the range $10^{-8}$ s$^{-1}$ to $10^{-6}$ s$^{-1}$). Water saturated samples show a stronger dependence of rock strength on loading rate.

![Figure 4.4](image.png)

**Figure 4.4.** A) Young’s modulus and B) maximum strength as a function of water content, measured parallel to the clay bedding (NAGRA, 2002c, Section 5.7.3). Blue symbols show data from Benken and red symbols show data from Mont Terri. In A), the shaded areas depict values from dilatometer measurements, filled symbols indicate samples with in situ water and open symbols indicate dried or saturated samples. For the tests shown in B) a confining pressure of 10 MPa was applied.
One challenge has been that in situ experiments do not monitor the pore pressure of the saturated samples. If the desaturation is limited to the core borders, the strength was interpreted as being underestimated, and if the sample is saturated unevenly, it can lead to overestimation of the strength.

Swelling properties were interpreted from laboratory measurements and in situ observations. Anisotropy was analysed and both the amount of swelling and the swelling pressures were higher perpendicular to stratification than parallel to it. The swelling pressure data interpretation also considered differences between in situ and laboratory measurements, dependence on water saturation, water chemistry, clay (smectite) content, layering and orientation. Swelling pressures were measured to be 0.3 to 1 MPa in situ, but up to 2 MPa in laboratory tests. An average swelling pressure of 1.4 MPa was reported (NAGRA, 2002c).

Thermal coupling and temperature-induced mechanical changes have been reported briefly in terms of two influential factors:

- Volumetric expansion of pore water and rock due to their different thermal expansion coefficients combined with pore water pressure increase in undrained conditions,
- Volumetric shrinking by drying and associated suction.

It was interpreted that, in general, drainage may lead to an increase in strength and lowering of the creep rate, while expansion can lead to an increase in creep rate.

**Evolution of the clay**

Rock deformation occurs over different time scales, from the short term to the very long term (i.e. rock creep). Because long-term deformation behaviour is important, but observations over the long-term are not available, the general understanding of deformation processes and concepts and the implementation of micro-mechanical rock models are important (NAGRA, 2002c, Section 5.7). The process understanding builds on analyses of clay mineralogy, the role of water and deformation processes. The influence of the various deformation processes on rock strength was analysed by NAGRA and water content was found to be an important factor.

The micro-mechanical numerical modelling to evaluate the long-term behaviour of clay was undertaken using PFC2D. The modelling involved evaluation of mudstone consolidation from clay particles and evaluation of anisotropy evolution, followed by evaluation of stress-dependent deformation behaviour. Due to the complexity of the calculations, interactions between water and clay particles were neglected. Both elastic and elasto-plastic cases were analysed in detail. The elastic case did not allow breakage of the particle contacts but the elasto-plastic case did. The model obeys the typical non-linear character of claystone to evaluate maximum strength.

The modelling of clay deformation also recognised localisation effects. That is, the model included the influence of the variability of clay particle contact parameters on macroscopic strength and deformation and the dependence of deformation on the number of the particles present (particle contacts per unit area). No further details of the PFC2D modelling are provided by NAGRA (2002c). Modelling results were considered to provide good explanations of field and laboratory testing observations.

NAGRA (2002c) provided physical explanations for the micro-mechanical modelling and sample testing. Displacement of pore water plays an important role in rock deformation, and interaction of the clay mineral particles was considered to define the deformation curve (primary transient creep) and elasto-plastic responses. Formation of microcracks was interpreted to have a minor role in deformation.

The Opalinus Clay does not contain fractures in undisturbed in situ conditions and fault zones do not represent preferential flow-paths because efficient self-healing occurs. Laboratory rock mechanics experiments led to the identification of time-dependent deformation in the micro-range, disintegration, swelling and stress changes as processes relevant to self-healing. Self-healing processes were also observed during in situ experiments at the Mont Terri laboratory and evidence for self-healing is provided by the absence of mineral veins and alterations in the clay.

The modelling included interpretation of fracture development in clay using an analysis based on general Mohr-Coulomb theory and distinguishing the behaviour of the less competent clay materials. It was concluded that clays exhibit dilatant shear fractures and small openings at low normal stresses and at high normal stresses compressive shear fractures having insignificant opening.
Data interpretation and model development for engineering feasibility assessment

Although the engineering feasibility related geotechnical calculations are beyond the scope of this study, the derivation and interpretation of parameter values for these calculations is of interest. Two numerical continuum modelling schemes were applied: a modified Mohr-Coulomb model to study the stability of underground structures and development of brittle fractures in an excavation disturbed zone and a modified Cam-Clay model to study deformation during compaction and decompaction phases (NAGRA, 2002c, Section 5.7.4). The models took account of rock fracturing and rock strength anisotropy in addition to typical parameters for over-consolidated clay rock, such as strain hardening and softening, dilatancy, formation of shear bands, strength-deformation dependence on water content, swelling capacity and creep.

The linear Mohr-Coulomb model included hydro-mechanical coupling and the modelling was extended using a more complex bi-linear Mohr-Coulomb model with multiple coupling. The FLAC code was used with in-house developments to include anisotropic Darcy’s law, transversely isotropic bi-linear strain or softening with a tensional stress limit to cover elasto-plastic behaviour and creep laws. The hydro-mechanical coupling was included through data exchange between program modules after sequential calculation steps at very small time intervals. It was assumed that the bulk compressional modulus of the clay particles is much larger than modulus of the rock (i.e. compression acts on the pore water component).

The model was calibrated using hydraulic and geomechanical field data and laboratory test values and differences were assessed. The calibration involved principal stress measurements and uni- and triaxial laboratory values from Benken borehole cores and yielded values for Young’s modulus, Poisson’s ratio, tensional strength, peak cohesion, friction angle and residual cohesion and friction angle. An example of the model calibration results is provided in Figure 4.5. The model was developed further to include a special solidification approach with respect to cohesion and friction angle parameters. The model successfully reproduced the stress-deformation results measured by triaxial testing. The model could, in principle, reproduce the permanent material solidification and deformation caused by loading and unloading cycles.

![Figure 4.5. Model calibration with laboratory data (Benken) for the strength of the isotropic rock matrix (linear and bi-linear models) (NAGRA, 2002c, Section 5.7.4).](image)

Long-term creep parameters and modelling

Development of a long-term creep and geomechanical model has been a major component of geotechnical interpretation (NAGRA, 2002c, Sections 5.7.4 and 5.7.5). A rheological creep model has been developed involving primary, secondary and tertiary creep (load and peak strength determine if tertiary creep occurs). NAGRA used a model developed by Salzer et al. (1998) that took account of hardening and relaxation (recovery). The model assumed a threshold value of 7.5 MPa to ensure that the rock mass is in equilibrium under in situ conditions. The
model was calibrated against long-term creep measurements at Mont Terri and model parameters, and in particular the secondary creep rate, were evaluated.

The modified Cam-Clay model provides a relatively simple conceptual model of the elasto-plastic deformation behaviour of a mudstone in both normally consolidated and over-consolidated states. Model parameters were derived from consolidation experiments at Mont Terri, and from the geomechanical and permeability tests in the Benken borehole. The parameters interpreted were:

- The over-consolidation ratio (OCR).
- The friction parameter when OCR = 1.
- The friction parameter when OCR > 1.
- The negative slope VCL (virgin consolidation line).
- The negative slope RRL (rebound-reconsolidation line).
- Apparent cohesion.
- Maximum effective stress.

The maximum effective stress was estimated from the evolution and burial history of the Opalinus Clay, and the maximum horizontal stresses and pore water pressures. Values of maximum effective stress deduced ranged between 10 to 25 MPa (subject to uncertainties). The uncertainties were reflected in the determination of the OCR value.

**Longer-term creep testing**

A set of laboratory investigations was undertaken to determine the creep rate under various conditions and for different stages of creep. Pressures of up to 15 MPa were applied and the steady creep rate at 10 MPa was estimated to be about $10^{-8}$ to $10^{-9}$ %s$^{-1}$. The values were compared to Mont Terri laboratory tunnel convergence measurements and to measurements at Site de Meuse and Haute Marne (French rock laboratories). Creep was also measured in long-term isostatic permeameter testing using small core samples and in a cross-bedding configuration. Volumetric deformation was recorded over injection and recovery pressure periods (re-consolidation). Using volumetric deformation values, the compression modulus $C_m$ can be evaluated for drained conditions including the effects of residual dilatancy using the following formula:

$$C_m = NS \left( \frac{1+e}{\kappa} \right)$$

where

- $NS$ is the average effective normal stress (MPa),
- $e$ is the void ratio, which is related to rock porosity $\phi$ such that $e = \phi(1 - \phi)$,
- $\kappa$ is the negative slope from the load-reload curve (measured).

$C_m$ is about 2.1 GPa when $NS$ is 14 MPa and $e$ is 0.12 (with the porosity equal to 0.11) and $\kappa$ is 0.0075 (NAGRA, 2002c, Section 5.7.3).

NAGRA (2002c) also noted that the bulk compression modulus can be calculated from the storage coefficient derived from hydraulic tests. Both methods were applied using data from the Benken borehole investigations to allow comparison.

**4.1.4 THERMAL DATA INTERPRETATION AND MODELLING**

Ambient temperature was based on continuous logging data. Temperature logging was done two-months after drilling to exclude as much drilling induced disturbance as possible. It was compared to monitoring recordings made after in packed-off sections. Thermal gradient was calculated and assessed against lithological factors – rock anisotropy, layering and gradual variations in layers, porosity and quartz content which influence strongly the thermal conductivity.

Heat capacity, thermal conductivity and thermal expansion coefficients were determined from sample measurements – first two from the Benken borehole and the last from Mont Terri rock laboratory samples. Thermal conductivity was interpreted based on a few samples in saturated and dry conditions, parallel and perpendicular to clay bedding and controlling the quartz content. Thermal conductivity was 3.22 ±0.11 W/m·K parallel to the bedding in saturated conditions and 2.77 ±0.12 in dried samples. Values perpendicular to the bedding were 1.70 ±0.16 and 1.57 ±0.04,
respectively. Quartz content in clay is about 20 – 25 % but gets lower down to 10 – 15 % in the lowermost part of Opalinus Clay layer and lowering thermal conductivity values by 30 – 40 %. Thermal expansion coefficient was calculated based on axial length deformation of the samples in heating-cooling cycle.

No specific techniques or methods related to measurements or their interpretation were reported. During the thermal evolution analysis for the repository, the initial temperatures of Opalinus clay and over- and underlying formations were re-calculated by using a simple heat conduction model (Johnson et al. 2002). In addition, thermal conductivities and heat capacities are given. Results showing the in situ temperature gradient data with associated geological factors and back-calculated temperatures are combined to presentation shown in Figure 4.6.

It looks evident in the light of the reported material that NAGRA has collected a very extensive subsurface data set of thermal characteristics and properties per lithologies, having data both from the regional geothermal analysis and from their sedimentary and crystalline site investigations in northern Switzerland.

Figure 4.6. A) Derived temperature gradient log from the Benken borehole with associated geology and B) back-calculated temperature profile with the heat conduction model. Reproduced from A) NAGRA (2002a, chapter 4.6.1) and B) Johnson et al. (2002, Fig. 7).

4.1.5 DATA INTERPRETATION TOOLS

NAGRA used the modified Cam-Clay model, which is a critical state soil mechanics model that describes the behaviour of saturated remoulded soils in compression tests. The model assumes isotropic, elasto-plastic conditions and deformation of the clay as a continuum. The modified Cam-Clay model takes long-term creep into account in a form that avoids numerical stability issues associated with incremental plastic strain. The modified Mohr-Coulomb model in the FLAC code was also used to interpret mechanical data.

The models were used to interpret and analyse elastic and plastic deformation regimes and limits on deformation behaviour. The void ratio and the magnitudes of the stress components (deviatoric and effective normal stresses) influence the limiting state. Increasing void ratio (i.e., increasing porosity) reduces the magnitudes of the stress components required to reach the critical state condition.

Micro-mechanical modelling to simulate the formation of the Opalinus Clay composition and structure was undertaken using the PFC2D code. PFC2D (available from Itasca International Inc.) is a general purpose micromechanical discontinuum analysis code for modelling geomaterial particles and particulate systems in two dimensions. Particle
distributions and geometries can be set and bonding forces determined, and calculations can be calibrated against laboratory experiment data. NAGRA (2002c) does not provide a detailed description of the code's usage, and the justification for using a model of 2-D particles to represent clay particles, but the 2-D code allows a larger number of particles to be considered than a 3-D code and, as such, the 2-D code is likely to model clay behaviour more reliably than a 3-D code.

All of the methods and codes used by NAGRA are used worldwide in geotechnical analysis. No special data interpretation tools have been reported for thermal rock characterisation.

4.1.6 LESSONS LEARNT

NAGRA's interpretation and modelling has focused on the development of models that describe in detail clay and mudrock behaviour and create a solid understanding of the processes involved. Models applied include:

- Micro-mechanical rock models.
- Short and medium-term deformation models.
- A very long-term deformation model (creep).
- Rock mechanical linear and bi-linear Mohr-Coulomb models.

Because most of the models are small-scale models, their validity for site characterisation has been assessed by comparing results to observations from larger-scale experiments in the Mont Terri laboratory.

The review materials studied do not describe in detail the development of the geotechnical or thermal site models. Geological and tectonic regional models have been central to site model development. Engineering and design assessments have applied the conceptual geological site model and added geotechnical and thermal parameters for the Opalinus Clay host rock and the overlying and underlying formations for geomechanical and thermal analyses for the planned disposal facility. No explicit geometrical and parametrical site geomechanical model has been established or described. Thermal parameters have included the ambient temperature, thermal conductivity and heat capacity (Johnson et al. 2002). In light of the comprehensive analyses conducted, it appears relatively straightforward to adapt rock parameters to tunnel and site-scale modelling.

The overall geological setting, results from the high-resolution 3-D seismic reflection survey and formation homogeneity have enabled the characterisation and interpretation of geotechnical and thermal parameters for the Opalinus Clay primarily using data from one deep borehole and from the Mont Terri research laboratory site, although data from other boreholes have been used. The local overburden and stress field conditions were considered in the interpretations.

The geosynthesis (NAGRA, 2002c) does not include evaluation of geomechanical or thermal parameters for the sedimentary rock sequence overlying the Opalinus Clay, although NAGRA notes that data for these strata are available from existing databases. Note that the 3-D seismic survey provided a detailed mapping of the steeply dipping fault(s) that would need to be considered in geotechnical design for construction.

The major findings of NAGRA's geotechnical clay rock characterisation research are:

- The deformation behaviour of the Opalinus Clay is complex.
- The Opalinus Clay in the Zürcher Weinland is slightly overconsolidated (OCR is between 1.5 and 2.5).
- The deformation behaviour is strongly dependent on the water content of the rock.
- The deformation and fracture behaviour is dominated by the internal, anisotropic structure of the Opalinus Clay.
- Hydro-mechanical and thermo-mechanical couplings must be taken into account.
- Time-dependent deformation is characterized by compaction and consolidation of the rock (volumetric deformation and displacement of the pore water), by a 'flow break' or cataclastic flow process and by the swelling process.
- Time-dependent deformation can be described adequately by a stationary creep model. Nearly stationary creep rates are observed only at high deviatoric stresses.
- For construction purposes, deformation and fracturing behaviour can be described adequately by an elasto-plastic, anisotropic Mohr-Coulomb model with hydro-mechanical coupling.
The conceptual description of the long-time behaviour of the clay is elasto-plastic and interpretation with a modified Cam-Clay model was considered satisfactory.

NAGRA observed that the deformation process is largely plastic at high water content. Lower water contents and also drying out can lead to a significant increase in rock strength. Partial drying out of the rock close to underground openings will contribute to stabilisation. Resaturation leads to time-dependent deformation and probably self-healing of the rock.
5  GDF CASE STUDIES: EVAPORITES

5.1  WASTE ISOLATION PILOT PLANT (WIPP), US

5.1.1  BACKGROUND

This section provides a summary of work that was undertaken by Sandia National Laboratories to evaluate geotechnical and thermal parameters for the Waste Isolation Pilot Plant (WIPP) repository near Carlsbad in New Mexico, US. Experimental studies were undertaken at the site from 1975 and construction of the repository began in the 1980s. The repository has been operated by the US Department of Energy (DOE) since 1999 and receives transuranic (TRU) radioactive wastes from US defence clean-up sites.

The WIPP repository is at a depth of 655 m in a thick low-permeability salt rock horizon, the Permian-age Salado Formation, and comprises a series of disposal panels with each panel containing seven rectangular disposal rooms (Figure 5.1). Tunnels link the waste disposal region to the operations region where four shafts (a waste handling shaft, a salt handling shaft, an air intake shaft, and an exhaust shaft) connect to the ground surface. Remote-handled TRU containers are emplaced in holes in disposal room walls and contact-handled TRU waste packages are stacked in the disposal rooms (Hicks et al., 2008).

The WIPP repository is in a tectonically stable and aseismic setting, and given the viscoplastic behaviour of salt, the in situ stress condition in salt is lithostatic (i.e. the stresses in all directions are equal to the pressure of the overburden) (Hanson and Leigh, 2011). Differential stresses cause elastic deformation and time-dependent inelastic viscoplastic flow and deformation (creep behaviour).

5.1.2  BACKGROUND ON GEOTECHNICAL AND THERMAL INVESTIGATIONS

A substantial experimental and modelling programme was undertaken from 1975 through the 1980s and 1990s to understand processes pertinent to transuranic radioactive waste disposal at the WIPP facility. For example, by 1983, some 78 boreholes had been specifically drilled for WIPP investigations (Borns et al., 1983). Tyler et al. (1988) presented a summary of the research undertaken through to the late 1980s, including work to understand the thermal parameters relating to the behaviour of the salt host rock. The work involved a progressive programme of
experiments, from laboratory tests to small-scale field tests in salt and potash mines, to full-size in situ tests at the WIPP facility. Specifically, a programme of experiments was designed to develop an understanding of the thermal and structural integrity (TSI) of the rock.

The TSI programme involved the following:

- Laboratory tests to determine material properties.
- Model development (constitutive and numerical models).
- In situ experiments (predictive capability validation database).

The main focus of these tests was to derive a constitutive model for salt creep, but tests were also undertaken to determine physical parameters such as thermal expansion, thermal conductivity and density. The acquisition and interpretation of thermal parameters are discussed in the following sub-sections.

In the early stages of the WIPP programme, materials were available only as cores from deep boreholes and from an exploratory shaft (construction and salt handling shaft) (Tyler et al., 1988). Consequently, the data for early development of the constitutive models were limited, with associated uncertainties. With the development of the underground facility, specimen material in significantly greater amounts became available from the exact WIPP horizon.

**Rock properties**

Laboratory tests formed the basis for the database from which constitutive models were developed and from which material parameters were determined. Materials response data obtained included physical properties (density, thermal conductivity, etc.) and mechanical properties (creep, quasi-static strength, etc.). The tests undertaken included:

- Compression testing under confining pressures to duplicate expected stress fields. Specific testing machines were built to handle the large specimens needed because of the large grain size.
- Quasistatic compression strengths.

The quasistatic tri-axial compression tests were undertaken to determine the ultimate strength of the salt, which is a time-dependent property (Tyler et al., 1988). The tests also gave the elastic moduli of the salt. These data were analysed in terms of an elastic-plastic constitutive description and the description of the failure surface of salt under quasi-static loading. Permeability testing of fractured core samples under pressure demonstrated the healing of fractures in rock salt.

Creep tests were undertaken at different temperatures, applied stresses and confining pressures. These tests yielded the structure factors, activation energies and stress power parameters for the WIPP creep model. With the underground access established, large quantities of WIPP specimens became available and creep tests on these materials were undertaken. These tests led to refinement of parameters determined from the borehole core material. The tests also provided information on the pressure sensitivity of the creep process.

Rock densities were determined using conventional laboratory procedures for determining bulk rock densities using the buoyancy method. Commercial grade kerosene was used for the buoyancy measurements (USDOE 1996).

**In situ stress**

The lithostatic stress was evaluated by considering the weight of the overburden rock above the site.

**Thermal properties**

The thermal conductivity, thermal expansion and specific heat of salt at the WIPP site were determined from borehole cores (Sweet and McCreight, 1980a and Sweet and McCreight, 1980b).

5.1.3 GEOTECHNICAL DATA INTERPRETATION AND MODELLING

**Rock Properties**

Krieg (1984) reported a WIPP halite density of 2,300 kg/m$^3$ and USDOE (1996) reported a density of 2,180 kg/m$^3$ for rock salt found at the WIPP horizon. An average density of 2,320 kg/m$^3$ was calculated for the overburden rock by averaging the measured density along the length of a borehole from the surface to the top of the reference
stratigraphy that includes the disposal horizon (a depth of about 598 m). The stress at this depth was thus calculated to be 13.6 MPa (Krieg, 1984).

Values for the elastic constants of halite, anhydrite and polyhalite are shown in Table 5.1 (Kreig, 1984) for 25 °C and 100 °C. Kreig (1984) considered the elastic properties of halite to be independent of temperature. Densities, elastic moduli and other physical and thermal properties of natural rock salts were also established from classical literature values (Clark, 1966).

Table 5.1. Elastic constants for materials at the WIPP site.

<table>
<thead>
<tr>
<th></th>
<th>Halite</th>
<th>Anhydrite</th>
<th>Polyhalite</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25 °C</td>
<td>25 °C</td>
<td>100 °C</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>31.0 GPa</td>
<td>75.1 GPa</td>
<td>51.0 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
<td>0.35</td>
<td>0.26</td>
</tr>
<tr>
<td>Bulk modulus</td>
<td>20.7 GPa</td>
<td>83.4 GPa</td>
<td>35.4 GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>12.4 GPa</td>
<td>27.8 GPa</td>
<td>20.2 GPa</td>
</tr>
</tbody>
</table>

Rock creep

Various constitutive models for the creep deformation behaviour of salt were proposed and tested during the TSI programme. The model eventually developed was based on the results of laboratory testing of small-size rock specimens subjected to stress and thermal loadings. The tests established that the WIPP salt creep rate is about three times more rapid than models predicted prior to the TSI programme.

The constitutive model selected for salt creep is a thermally-activated Arrhenius steady-state creep law with stress raised to a power (as used for metal creep) (Tyler et al., 1988). The model gives the steady state creep rate as:

$$\frac{\partial \varepsilon_s}{\partial t} = A \exp\left(-\frac{Q}{RT}\right) S^n$$

where $A$ is a structural factor, $Q$ an activation energy, $R$ the universal gas constant, $T$ the absolute temperature, $S$ the stress and $n$ the power exponent.

The creep data were first interpreted by Munson and Dawson (1979; 1982) and the material parameters for transient and steady state creep models were determined. A deformation-mechanism map was produced, as shown in Figure 5.2 (Munson, 1979), and Munson and Dawson (1979) developed a constitutive model for the steady state creep of salt for the range of stress and temperature expected for the WIPP facility. As indicated in Figure 5.2, three deformation mechanisms were identified as being relevant to radioactive waste disposal (dislocation glide, dislocation climb and an undefined mechanism). The other deformations identified in Figure 5.2 were considered not to occur under expected stress and temperature conditions at the WIPP. The diffusional creep mechanisms (Nabarro-Herring creep and Coble creep), as well as the potential for deformation by pressure solution, are discussed by Borns et al. (1983).
The steady-state strain rate is given by (Tyler et al., 1988):

$$\frac{\partial e_s}{\partial t} = \sum_{i=1}^{3} \frac{\partial e_{s_i}}{\partial t}$$

and the three individual mechanisms (dislocation climb, undefined and dislocation glide) have steady-state creep given by:

$$\frac{\partial e_{s_i}}{\partial t} = A_i \exp\left(-\frac{Q}{RT}\right) S_i^n$$
\[
\frac{\partial e_{e_2}}{\partial t} = A_2 \exp\left(-\frac{Q_2}{RT}\right) S^e
\]
\[
\frac{\partial e_{e_3}}{\partial t} = 2B_1 \exp\left(-\frac{Q_1}{RT}\right) + B_2 \exp\left(-\frac{Q_2}{RT}\right) \sinh(q/\mu(S - S'))\]

where \(q\) is an activation volume, \(\mu\) is the shear modulus, \(S'\) is a cut-off stress, and the \(A_s\) and \(B_s\) are constants. Borns et al. (1983) reported a theoretically calculated strain rate of \(3.2 \times 10^{-16}\) s\(^{-1}\) at the WIPP site.

Transient strain was treated as a strain potential modifier to the steady-state function with a work hardening branch, an equilibrium branch and a recovery branch:

\[
\frac{\partial e}{\partial t} = F \frac{\partial e_*}{\partial t}
\]

where

\[
F = \begin{cases} 
\exp(D(1 - x / e_*)) \\
1 \\
\exp(d(1 - x / e_*)) 
\end{cases}
\]

with the different \(F\) representing work-hardening, steady state and recovery, respectively. The \(D\) and \(d\) are work-hardening and recovery constants and \(e_*\) is the transient strain limit. The transient strain limit is expressed as:

\[
e_* = K \left(\frac{S}{\mu}\right)^m
\]

where \(K\) is an empirically determined temperature dependent parameter and \(m\) is a material parameter.

The parameter values for the creep model were summarised by Fossum and Frederich (2002) and are as follows: \(A_1 = 8.386 \times 10^{22}\) s\(^{-1}\); \(A_2 = 9.672 \times 10^{12}\) s\(^{-1}\); \(Q_1 = 25\) Kcal/mol; \(Q_2 = 10\) Kcal/mol; \(B_1 = 6.086 \times 10^6\) s\(^{-1}\); \(B_2 = 3.034 \times 10^{-2}\) s\(^{-1}\); \(K = 6.275 \times 10^5\); \(d = 0.58\).

Fossum et al. (1993) also used the results of triaxial compression creep tests to derive probability distribution functions assuming that certain parameters (\(K, A_2\) and \(D\)) are random variables. The database comprised 29 tri-axial compression creep tests on cylindrical core specimens of salt from the WIPP (100 mm diameter by 200 mm long) at different temperatures and stresses. The distribution functions were used as input in calculations to determine the distribution function for room closure.

Fracture properties

Plastic deformation occurs in rock by dislocation motion within the salt lattice and includes processes of dislocation multiplication, glide, cross slip, and climb. Damage occurs in the form of time-dependent initiation, growth, and coalescence of microfractures when the deviatoric stresses are relatively high compared to the applied mean stress. Fracture growth around an opening is anisotropic with fractures aligning parallel to the most compressive stress.

Salt will heal previously damaged areas when the magnitude of the deviatoric stress decreases relative to the applied mean stress. The healing mechanisms include microfracture closure and bonding of fracture surfaces. Microfracture closure is a mechanical response to increased compressive stress applied normal to the fractures, while bonding of fracture surfaces occurs either through crystal plasticity, a relatively slow process, or pressure solution and redeposition, a relatively rapid process. Evidence for healing has been obtained in laboratory experiments, small-scale tests, and through observations of natural analogues (Hanson and Leigh, 2011).

Hanson and Leigh (2011) noted that analysis had been undertaken to evaluate volumetric strain in terms of principal stresses. Stress states that led to dilation were defined in terms of the first invariant of the traditional Cauchy stress tensor, \(I_1\), and the second invariant of the deviatoric stress tensor, \(J_2\). These invariants are related to mean (or confining) stress and deviatoric stress, respectively as follows:

\[
I_1 = \sigma_1 + \sigma_2 + \sigma_3
\]
\[ J_{1/2}^2 = \left( \frac{1}{6} \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right] \right)^{1/2} \]

where \( \sigma_1, \sigma_2, \) and \( \sigma_3 \) are the three principal stress components for a particular type of test.

Hanson and Leigh (2011) noted that a delineation exists in the \( J_1 - J_2 \) stress space between conditions that cause dilation and those that do not, regardless of the type of salt or type of test considered. An empirical relationship to divide dilating stress states from non-dilating stress states was reported:

\[ J_{1/2}^2 = 0.27 I_1 \]

This relationship is called the stress-invariant model and has been used at the WIPP site to predict the occurrence of disturbed or damaged zones around disposal rooms.

Hansen and Leigh (2011) reported that laboratory tests have been undertaken to understand the damage evolution process in salt. Short-rod fracture toughness strength tests were performed to determine the tensile load required to initiate and propagate fractures in salt. The fractured test specimens were subsequently compressed hydrostatically and then retested in a second series of fracture toughness tests. The study showed that the healed specimens regained from 70% to 80% of their initial fracture toughness strength.

Hanson and Leigh (2011) also note the analysis undertaken by Brodsky (1990) using compressional ultrasonic wave measurements to study crack initiation, propagation and healing of salt. In general, velocity decreases with increases in rock dilation and increases in response to crack closure and/or porosity reduction. Microcracks were introduced into the salt specimens during constant-strain-rate, triaxial compression tests conducted at low confining pressure. After specified damage levels were induced, hydrostatic compressive stresses were applied to promote healing. During both the loading and healing stages of the tests, ultrasonic wave transducers monitored wave velocity parallel and perpendicular to the maximum principal compressive stress. During loading, wave velocity parallel to the maximum principal stress decreased 1% to 2%, while velocity perpendicular to the maximum principal stress decreased by as much as 10%. In contrast, velocity measurements made during healing (or fracture closure) increased, with the rate of increase depending on the initial level of damage and the magnitude of the applied hydrostatic stress. Velocity recovery generally occurred within a few days.

5.1.4 THERMAL DATA INTERPRETATION AND MODELLING

Laboratory studies served to set the material thermal properties for the WIPP Project. The scalability of thermal conductivity values was confirmed by bench-scale studies with blocks of salt and field measurements using a thermal probe. Small-scale field tests were also undertaken in a potash mine to evaluate thermal conductivity (Tyler et al., 1988). The thermal properties of materials at the WIPP site were summarised for the reference stratigraphy by Krieg (1984) and these are listed in Table 5.2.

Thermal conductivity measurements were fitted to the following equation:

\[ \lambda = \lambda_{300} \left( \frac{300}{T} \right)^\gamma \]

where \( T \) is the temperature (Kelvin) and parameter values are as given in Table 5.2 (Sweet and McCreight, 1980a; Krieg, 1984).

Note that USDOE (1996) reported a thermal conductivity of 5.75 W/m-K for rock salt found at the WIPP horizon. Thermal conductivities of rock salt samples were determined using longitudinal heat flow apparatus designed specifically for use with geologic core sections. Constant power is supplied to a heater at one end of the specimen until thermal equilibrium is established, allowing the thermal conductivity to be calculated.
Table 5.2.  Thermal properties for materials at the WIPP (Krieg, 1984).

<table>
<thead>
<tr>
<th>Material</th>
<th>Specific heat (J/kg·K)</th>
<th>Coefficient of linear thermal expansion (K⁻¹)</th>
<th>( \lambda_{300} ) (W/m·K)</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>Halite</td>
<td>860</td>
<td>45 x 10⁻⁶</td>
<td>5.0</td>
<td>1.14</td>
</tr>
<tr>
<td>Argillaceous salt</td>
<td>860</td>
<td>40 x 10⁻⁶</td>
<td>4.0</td>
<td>1.14</td>
</tr>
<tr>
<td>Anhydrite</td>
<td>860</td>
<td>20 x 10⁻⁶</td>
<td>4.7</td>
<td>1.15</td>
</tr>
<tr>
<td>Polyhalite</td>
<td>860</td>
<td>24 x 10⁻⁶</td>
<td>1.4</td>
<td>0.35</td>
</tr>
</tbody>
</table>

5.1.5   DATA INTERPRETATION TOOLS

For the WIPP performance assessment, a porosity surface representing potential changes in gas pressure and porosity in disposal rooms over a 10,000-year simulation period was calculated to indirectly couple mechanical closure and gas generation with two-phase fluid flow calculations. The porosity surface approach was used because it was not possible to fully-couple creep closure, waste consolidation, brine availability, and gas production and migration calculations while maintaining computational efficiency. The calculation of the porosity surface involved closure calculations using the SANTOS code, a quasi-static, large deformation finite-element structural analysis code (Stone 1997).

Modelling the complex characteristics of the materials in the WIPP and its surroundings require the use of specialised constitutive models that have been built into the SANTOS code over a number of years. These models have been developed through initiatives such as the TSI programme, starting from a theoretical background and parameterised from surface-based investigations, but developed later through comparison with in situ tests and measurements. On the basis of this development, appropriate mechanical material response models and their corresponding property values are assigned to each region of the configuration (Butcher 1997). These models include:

- A combined transient-secondary creep constitutive model for clean and argillaceous halite.
- An inelastic constitutive model for anhydrite.
- A volumetric plasticity model for waste.

SANTOS is maintained by Sandia National Laboratories.

5.1.6   LESSONS LEARNT

Coordinated activities in laboratory database generation, constitutive model formulation, and numerical code capability improvement aided the development of thermal/structural codes to predict the creep deformation of the WIPP disposal rooms in bedded salt deposits. Following underground excavation, the codes were validated by testing against large-scale underground structural in situ tests. This validation allowed further development and confidence-building.

The development of SANTOS by Sandia National Laboratories was a major component of the research undertaken in support of the WIPP performance assessment and site licence application. The work involved a highly skilled research team and there is a limited capacity for such modelling worldwide.
6 SOILS AND NEAR-SURFACE CONDITIONS

Shallow ground conditions in many parts of the UK comprise drift deposits, which may be of considerable thickness and ranging from normally consolidated alluvial sediments to Glacial Till. The underlying solid geology can often have properties more akin to soil than rock (e.g. Gault Clay and London Clay), is intrinsically weak, or weakened by weathering, often to the consistency of a soil. Drift deposits and weathered bedrocks by their nature are heterogeneous, with often significant variations in engineering properties and lithology over short distances both vertically and laterally. Current UK Standards which cover the investigation and assessment of soils and weathered rocks comprise:


The primary focus of a ground investigation is generally to obtain the following properties of the shallow soils and rocks;

- Classification data;
- Shear strength parameters;
- Stiffness/consolidation properties;
- Permeability;
- Stiffness and damping parameters for prediction of dynamic behaviour; and,
- Ground aggressivity.

Figure 6.1 shows the idealised framework for derivation of characteristic and design values for geotechnical properties (BS EN 1997-2:2007).

Information on data acquisition techniques to measure soil properties has been compiled by Golder Associates (2008). The report contains a summary of methods for determining baseline engineering parameters for rocks and soils through in situ and laboratory testing for host rocks, access tunnels and shafts and associated surface infrastructure. Typical methods used to obtain and interpret these data for soils are presented in Table 6.1.

Raw laboratory data are seldom used directly in engineering assessments. The results will often be subdivided into relevant strata. Parameters are typically plotted initial against depth or elevation for each stratum. Plots of data are useful in establishing ranges of values which may demonstrate trends with depth and in addition, they may also identify anomalous data, which may be removed from the interpretation process.

Since the purely statistical approach defined by EC7 has inherent difficulties when selecting characteristic geotechnical parameters it also defines the characteristic value as “a cautious estimate of the value affecting the occurrence of the limit state” (BS EN 1997-2:2007). Therefore, the characteristic value adopted for the corresponding design situation is based on a cautious estimate, taking into account;

- Sampling and testing limitations;
- The quality of the data (e.g. empirical correlations would not normally take president over in situ testing results or direct laboratory measurement;
- The nature of the structure and its serviceability requirements; and,
- Engineering judgment.
It should be noted that in most respects, parameter determination and data manipulation is as previous UK good practice. A consequence of the above definition of a characteristic parameter is that the characteristic value can only be selected during the design of the structure.

Figure 6.2 presents a typical illustration of the interpretation of data derived from multiple sources for undrained cohesion of a uniform Glacial Till. The chosen characteristic design line can be seen to be essentially a cautious estimate of the spatial mean, which takes into account the two direct testing methods used (field vane and laboratory triaxial testing) and values derived using the correlation between plasticity index, $c_u$ and SPT $N_{60}$ (CIRIA 1995).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Typical method of data acquisition</th>
<th>Typical data interpretation techniques</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classification</td>
<td>Moisture content, plasticity index, plastic limit particle size distribution, bulk density tests on samples recovered from exploratory holes.</td>
<td>Plots of classification parameter versus depth or elevation. Plasticity data commonly plotted on standard plasticity chart with different strata often represented by different data groupings.</td>
</tr>
<tr>
<td>Shear strength – total stress</td>
<td>In situ testing within boreholes and trial pits e.g. Standard Penetration Test (SPT) and field vane). Recovery and testing of undisturbed (Quality Class 1) samples e.g. thin-walled undisturbed (UT100), cored sample and piston sample. Static cone penetration testing (CPT).</td>
<td>Assessment of undrained cohesion ($c_u$) using the relationship with SPT $N_{60}$ (corrected for hammer efficiency) and plasticity (CIRIA 1995). Laboratory triaxial testing will provide direct measurement of $c_u$. Field vane tests also measure $c_u$ directly in the field, but values will normally be corrected for the soils plasticity. Prior to interpretation values obtained directly and indirectly will usually be amalgamated and plotted versus depth or elevation. CPT tests, with appropriate assessment can provide continuous plots of $c_u$ versus depth.</td>
</tr>
<tr>
<td><strong>Parameter</strong></td>
<td><strong>Typical method of data acquisition</strong></td>
<td><strong>Typical data interpretation techniques</strong></td>
</tr>
<tr>
<td>---------------</td>
<td>--------------------------------------</td>
<td>-------------------------------------------</td>
</tr>
<tr>
<td><strong>Shear strength – effective stress</strong></td>
<td><em>In situ</em> testing (e.g. SPT). Classification tests. Shear box testing. Triaxial testing (with measurement of porewater pressure).</td>
<td>Use of relationship between and SPT $N_{60}$ (SPT N corrected for effective stress and hammer efficiency) in granular soils (CIRIA 1995). Interpretation from classification test data (BS8002:1994. “Code of Practice for Earth Retaining Structures” [Incorporating amendments Nos. 1 and 2]. B.S.I., UK). Method applicable to both granular and cohesive soils. Values obtained from plots of normal stress versus shear stress (shear box), or $P'/q'$ (consolidation stress/deviator stress) plots (e.g. Consolidated Undrained/Consolidated Drained tests). Test methods often require a significant degree of interpretation to obtain typical design parameters.</td>
</tr>
<tr>
<td><strong>Stiffness and compressibility</strong></td>
<td>Appropriately staged one-dimensional consolidation tests. In situ testing (SPT). Plate load testing. Static Cone Penetration Testing (CPT) with pore pressure sensor. Pressuremeter/high pressure dilatometer testing.</td>
<td>One dimensional consolidation coefficients can be determined over appropriate stress ranges. Over-consolidation ratio (OCR) can be determined directly from void ratio versus effective stress plots. Correlation between SPT $N_{60}$ and coefficient of volume compressibility ($m_v$) and Undrained/Drained Young’s Modulus ($E_u/E'$) (CIRIA 1995). Plate load tests will provide values $E_u/E'$ at shallow depth. CPT dissipation testing provides in situ readings of Time factor ($T_{50}$) used in the assessment of settlement time. Assessment of $m_v$ and $E'/E_u$ lack reliability. Pressuremeter testing provides <em>in situ</em> measurement of soil stiffness.</td>
</tr>
<tr>
<td><strong>Dynamic behaviour</strong></td>
<td>Surface or borehole geophysical methods. Seismic CPT. Bender element tests. Resonant column tests. Cyclic triaxial test. Published relationships between $G_{max}$, void ratio, OCR, $c_u$ and effective stress.</td>
<td>Seismic methods provide an estimate of dynamic shear modulus ($G_{max}$) and values will need to be used in conjunction with typical curves of shear modulus versus stress. Laboratory methods can be tailored to assess reduction in shear modulus with increasing strain and damping over a range of stresses. Published correlations show a wide variation and should be treated with caution and preferably used with other, more accurate methods.</td>
</tr>
<tr>
<td><strong>Permeability</strong></td>
<td>In situ tests in boreholes (e.g. falling/rising/constant head, packer tests). Soakaway testing in trial pits. CPT dissipation tests. Laboratory permeameter tests.</td>
<td>Cautious appraisal of <em>in situ</em> test results taking into account accuracy of test method and potential for vertical and lateral variability. Cautious assessment of laboratory data taking into account the limitation of the test method and scale effects.</td>
</tr>
<tr>
<td>Parameter</td>
<td>Typical method of data acquisition</td>
<td>Typical data interpretation techniques</td>
</tr>
<tr>
<td>-------------------------</td>
<td>---------------------------------------------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Ground aggressivity</td>
<td>Laboratory tests on recovered samples of soil and groundwater (pH total/water soluble sulphate, chloride). Resistivity by field survey or laboratory testing.</td>
<td>Results assessed in accordance with guidance provided in BRE Special Digest 1 (BRE 2005). Resistivity tests results generally interpreted by a specialist.</td>
</tr>
</tbody>
</table>

Figure 6.2. Typical Plot of Undrained Shear Strength versus Depth.

Undrained Shear Strength vs Depth (Glacial Till)
7 DATA INTERPRETATION AND MODELLING IN OTHER SECTORS

A review of interpretation and modelling of rock mechanical and thermal data in other sectors has been undertaken with the aim of identifying methods and tools that could be utilised in the UK site characterisation programme. The review has considered the civil engineering, oil and gas and geothermal sectors with the aim of identifying methods that have not already been described in the GDF site characterisation case studies.

7.1 CIVIL ENGINEERING SECTOR

7.1.1 ROCK MECHANICS MODELLING

Sommer and Wittke (2012) favoured the use of rock mechanical models based on the results of geotechnical investigations as well as engineering and geological judgment rather than rock classification systems in tunnel construction (Figure 7.1). The proposed approach is based on structural models describing the structure and texture of the intact rock and the discontinuities. The corresponding rock mechanical models include the parameters that describe the stress-strain behaviour and permeability of the rock mass for all units that the planned tunnel is passing through. Furthermore, the in situ stress state must be considered. Amongst other parameters, the orientation of the discontinuity system with respect to the tunnel considerably influences the type and amount of support measures. Thus, anisotropy is an important aspect of every model.

As an example, a schistose rock (clay slate) and the corresponding rock mechanical model is shown in Figure 7.2. The clay slate is separated by three sets of discontinuities, D1, D2 and Sch, the orientations of which are found to be approximately vertical (D1 and Sch) and horizontal (D2). The intact rock has a planar grain structure caused by the schistosity Sch, which in this particular case has the same orientation as the bedding.

On the basis of such rock mechanical models, stability analyses are carried out considering parameter variations. The rock–structure interaction and anisotropic rock mass behaviour are accounted for using a suitable analysis method such as the finite element method (FEM). Supplementary analyses with regard to the stability of rock wedges adjacent to excavation surfaces may also be necessary. Based on the results of the analysis, the design is completed, followed by construction of the tunnel.

7.1.2 ROCK CLASSIFICATION

Rock classification systems generally are strongly subjective. They are based on coefficients by means of which it is attempted to account for the rock mechanical parameters, in situ stresses and groundwater conditions (Figure 7.1). Tables, formulas, diagrams and combinations are used to obtain ratings for these coefficients. By means of empirical functions of the coefficients, the so-called "rock mass rating indexes" are calculated. Based on such single indexes, design recommendations for support measures denoted as "support classes" are given. The basis of each classification system is empirical. The selection of the influencing variables, their rating and the recommended support classes are therefore based on the experience of the developer gained from practical cases. One example is the determination of the RQD-value, which is common to most of the classification systems. It is dependent on a number of factors such as the orientation of the corresponding borehole with respect to the discontinuity system, the drilling method, the core quality, and the core diameter.

In Figure 7.3, the result of an optical scan of a 100-cm long wall of a borehole is compared with the photo of a corresponding core obtained from the same depth. The borehole scan reveals two discontinuities with spacing of more than 10 cm resulting in a RQD value of 100 %, whereas the corresponding core is separated into pieces of <10 cm due to the drilling process resulting in a RQD value of 0. This example indicates that the RQD evaluated for a rock mass can be completely different depending on the method of exploration. Furthermore, the RQD value depends on the orientation of the borehole with regard to the discontinuity system.
Figure 7.1. Design based on rock mechanical models and rock classification systems (Sommer and Wittke, 2012).

Figure 7.2. Example of a rock mechanical model (Sommer and Wittke, 2012); a) clay slate wall; b) corresponding rock mechanical model.
7.1.3 DEFORMATION ANALYSIS

InSAR (Interferometric Synthetic Aperture Radar) can be used to detect and monitor surface movements covering very large areas (several thousands of km$^2$) with millimetre precision (de Faragó and Cooksley 2012). The approach is based on the use of data from very high resolution satellites and involves comparing the distance between the satellite and the ground in consecutive satellite passes over the same area or point on the Earth’s surface. Radar satellite images record, with very high precision, the distance travelled by the radar signal that is emitted by the satellite. When the distance between the satellite and a certain point is compared at different times, highly accurate ground deformation measurements can be acquired (see Figure 7.4). However, the technology is typically used to detect movements before and during tunnel construction, or natural variations in vertical elevation caused by soil deformation or larger rock block movements. Applications of the technology may be limited in GDF site characterisation.

Figure 7.4. Basic principles of radar interferometry (de Faragó and Cooksley, 2012).
7.2 OIL AND GAS SECTOR

7.2.1 ROCK PROPERTIES

Data collection and site surveys conducted in the oil sector most often focus fully on reservoir characterisation and therefore involve seismic, electromagnetic and gravimetric methods. Seismic, electric and radiometric methods are the most common in well-logging. Wireline logging has also been used to estimate rock mechanical properties in some recent cases (Ameen et al. 2009) where rock mechanical parameters were determined to be functions of porosity and to a lesser extent of mineralogy, texture and pore fabric.

This analysis was done through empirical correlation of compressional P-wave velocity ($V_p$) laboratory measurements and ambient porosity that enabled the generation of porosity pseudo-logs from $V_p$ logs. Logs of static and dynamic elastic constants were generated from these porosity correlations using simultaneous laboratory measurements. Rock strength parameters were also derived from porosity correlations. The layer-specific pseudo-logs were found to be in fairly good agreement with laboratory tests. It should be noted that although indirect correlation methods such as this allow for geotechnical modelling and interpretation of the bedrock under study, indirect methods are prone to inaccuracies (Khaksar 2009), which can only be resolved with adequate calibration data sets from core testing. Porosity determination and its link to rock mechanical properties is fairly quick and inexpensive and measurements can be conducted down-hole.

7.2.2 GEOMECHANICAL MODELLING

Development of geomechanical models has been an important activity in the oil and gas industry for reservoir management (Herwanger and Koutsabeloulis 2011). Models are created from 3-D seismic data and borehole logging results. The models describe the initial data pore pressures, the spatial distribution of subsurface material properties, reservoir and overburden geometry (layers, bodies, faults) and tectonic and overburden stresses. One major requirement is to determine the elastic rock properties from the seismic data. This is achieved using the AVO inversion (amplitude variation with offset) technique utilising P-wave and S-wave velocities and densities and amplitude variation dependence on angle of incidence.

A second important issue is dynamic-to-elastic property correlation, because seismic surveys map dynamic parameter values (Young’s modulus and Poisson’s ratio), but static elastic moduli provide the input. Herwanger and Koutsabeloulis (2011) list a three phase procedure involving 1) fluid correction for the difference between incompressible and compressible pore space, 2) lithology classification based on logs and inverted seismic volumes, and 3) applying lithology specific correlation functions.

Geomechanical and hydromechanical models can be used to predict changes in the stress state during reservoir production. Deformation gives rise to anisotropic wave velocity changes, which are used to gain an understanding of the induced stress changes during production. Various models have been developed and applied to stress analysis in oil and gas reservoirs; examples are Poly3D, Visage and ANSYS as described below. These techniques have also been applied to analyse unconventional reservoirs such as gas shales (Wikel 2011).

The Poly3D software (distributed by Schlumberger [Schlumberger 2013]) is a three-dimensional stress modelling tool (based on the boundary element method) that is used in reservoir drilling planning and geomechanics analysis. Poly3D can be used to calculate heterogeneous stress fields in oil reservoirs models containing many faults and discontinuities and stress field perturbations associated with active faulting (see Figure 7.5).

Zhang et al. (2007) presented the results of hydro-mechanical modelling of a faulted reservoir using a finite element modelling approach (the commercially available VISAGE code) to investigate the concept that the natural state of stress is close to a critical point in a significant proportion of the earth’s lithosphere. Specific boundary conditions were applied to represent the conditions of a faulted reservoir, including the far field stress regimes and fault systems (See Figure 7.6).

This study demonstrated that a critical stress state around a faulted reservoir prior to production and injection plays an important role in the hydro-mechanical responses during production. Under such a stress state, a small change in the effective stress due to fluid pressure changes in the reservoir is likely to trigger global hydro-mechanical reaction of the reservoir, irrespective of whether the change was at a local scale or a global scale. Such responses include fault reactivations, volumetric and shear strain changes, induced deformation evolution and permeability changes. In contrast, if the in situ stress is not at a critical state, the reservoir reacts locally. In this case, the deformation is mainly elastic, and no permeability enhancements take place. Therefore, the impact of inelastic geo-mechanical interactions (particularly shear deformation) at a critical point is likely to be very influential on reservoir fluid flow.
Frischbutter and Henk (2010) reported a study on the potential for geomechanical reservoir models to predict stresses and fracture networks in order to provide necessary geotechnical information prior to reservoir drilling operations. They presented a workflow describing the various steps and data requirements to set up, run and calibrate a geomechanical model (Figure 7.7). An example was considered in which the reservoir geometry is constrained by 3D seismic data. Stress and fracture data from three wells were used to check the model predictions. Modelling was carried out as a history match to mimic the increase in information acquired during the exploration and appraisal stage. The case study showed that a robust prediction of the stress field, including its local perturbations near faults, can be based primarily on the reservoir geometry. Fracture prediction is more complex and requires well data for calibration because the model has to use several poorly constrained parameters, such as the magnitude of the paleo-stresses to infer the fracture orientations.

Frischbutter and Henk (2010) described two distinct modelling approaches: static and dynamic. Static models are based on the present-day reservoir geometry and the present-day ambient stress field. Modelling results provide a quantitative basis to predict the recent stress distribution within the reservoir, particularly the local perturbations near faults. The corresponding stress tensor data can then be used to calculate, for example, shear and normal stresses relative to the existing fault and fracture surfaces and infer the corresponding slip and dilation tendencies, respectively. In contrast, fracture formation and possible reactivation typically took place under stress conditions which were different from the present-day situation. Therefore, dynamic models have to account for the tectonic evolution of the reservoir, and temporal changes in regional stress fields and/or reservoir geometry have to be incorporated. If the reservoir geometry has been modified substantially, the tectonic evolution can be divided into several modelling stages with different model geometries.
Frischbutter and Henk (2010) used the Finite Element (FE) technique and the commercial FE code ANSYS (Ansys Inc., Houston, USA), respectively, in their analysis. The model geometry is based on a boundary representation of faults and lithological horizons, which can be constructed using data sets such as fault maps. The ideal data base utilizing the full power of the modelling approach would be a reservoir model based on 3D seismic data and geometrically consistent with all available data. Some manual editing is usually required to honour the specific needs of the FE technique, particularly the representation of the existing major faults. Fault friction coefficients can be assigned to the contact elements, which will slip if the shear strength described by the Mohr-Coulomb law is exceeded. Material properties are assigned to the elements representing the various lithologies. The FE models can describe elastic and plastic rock deformation. If ductile rheologies like salt are involved, their plastic deformation can be approximated by temperature and/or strain rate-dependent creep laws.

Modelling results include the full stress tensor for each part of the model, which can also be used to infer the slip and dilation tendency of faults and fractures. In addition, stress and strain information can be combined to predict fracture types and fracture orientations and to provide fracture intensity maps throughout the reservoir. The case study indicated that a robust prediction of the stress field, including its local perturbations near faults, can be based primarily on the reservoir geometry. Fracture prediction is more complex and requires well data for calibration because the model has to use several poorly constrained parameters such as the magnitude of the paleo-stresses and the paleo-pore pressures to infer fracture orientations.

7.2.3 DATA VISUALISATION

Geotechnical data interpretation can be aided by the use of tools such as RockWorks for subsurface data visualisation (Rockware, 2013). A borehole manager feature allows soil boring, sampling, well screening and other information to be imported into RockWorks to enable models of different geotechnical attributes to be built as part of the data interpretation process (see Figure 7.8). Such tools are commonly used in the oil and gas industry as well as in civil engineering and mining.
7.3 GEOTHERMAL SECTOR

7.3.1 STRESS FIELD

Determining the in situ stress field is important in geothermal reservoir investigations. The stress field affects the stability of the boreholes, production and injection rates, reservoir behaviour, induced seismicity and fault reactivation potential. Geothermal wells, with a minimum set being a pair, are typically drilled to depths of more than 2000 m for heat generation and 5000 m for electricity generation in an environment of typical low thermal gradient (20 – 30°C/km). The iterative steps of stress determination are described in the flow-chart shown in Figure 7.9 according to Moeck (2012). The early exploration phase mainly uses the regional data available: stress maps, structural geology, and known rock densities. After drilling, the stress magnitude and direction results can be interpreted from well image logs (optical, electric, acoustic), cores, mini-frac testing and density logs. Cores are measured for unconfined compressional strength, tensile strength and friction coefficient. Note that the concept of limiting stress ratios in Figure 7.9 refers to calculation of the effective horizontal stresses by using the following formula:

$$\frac{\sigma_{1\text{eff}}}{\sigma_{3\text{eff}}} = \frac{\sigma_1 - P}{\sigma_3 - P} = \left(\sqrt{\mu^2 + 1} + \mu\right)^2$$

where $\sigma_{1\text{eff}}$ and $\sigma_{3\text{eff}}$ are the maximum and minimum effective stresses, $P$ is the pore fluid pressure and $\mu$ is the friction coefficient.

Hydraulic interconnection of the wells is necessary for geothermal production. Geomechanical parameters are important for stimulation design and to optimise injection/production rates determined by the rock porosity, fracturing and faults. Hydraulic or shear stimulation is used with seismic monitoring to locate induced fractures. One standard test is the leak-off test of the well section from which minimum horizontal stress is determined. For a fault, reactivation potential is calculated and verified by the slip tendency technique and microseismicity data.
7.3.2 TEMPERATURE FIELD

Remote sensing technology is used to detect temperature anomalies in geothermal energy prospecting. For example, Peng et al. (2013) discussed application of the technology to investigate geothermal energy potential in China. The approach involves processing and interpreting Landsat Thematic Mapper (TM) images (information from satellites) in combination with existing geological and geographical data. Analysis of the distribution of land surface temperatures (LSTs) and geological data enables identification of prospective geothermal areas. The image characteristics of different structures are represented by different colours, shapes and textures (see Figure 7.10). LST anomalies may indicate the major faults in a geothermal system and ground temperature is usually lower in a recharging area than in a discharging area. However, applications of the technology may be of limited value in GDF site characterisation.
Figure 7.10. Example of distribution of land surface temperature anomalies superimposed on a simplified geologic map (Peng et al. 2013).
8 DATA INTERPRETATION AND MODELLING RESOURCES

8.1 DATA
All the reviewed case studies have collected site-specific geomechanical and thermal data through a combination of laboratory testing and borehole measurement. However, at the early stages in an investigation and programme and, indeed, for comparison of candidate sites, regional and generic data have been used. The use of regional data is particularly relevant to determining properties such as in situ stresses and the geothermal gradient. Such data exist at a high level throughout the UK. A more detailed picture may be developed for some areas from, for example, resource exploration boreholes. However, it is beyond the scope of this review to consider the variable availability of such data across the UK.

8.2 SOFTWARE
Software-based tools noted in this review have been used primarily to:

- Derive properties or parameter values from measurements.
- Derive parameter values from statistical analyses.
- Model data.
- Visualise spatial variation.

This excludes more general software for managing databases. Visualisation software is generally applied more widely, merging geotechnical and thermal data with data from other disciplines such as the geological structure. Radioactive waste management organisations such as SKB have developed their own software (SKB's Rock Visualisation System) for this purpose, although commercial packages to support industries such as mining and oil exploration are also available.

Derivation of parameter values (e.g. thermal conductivities) and statistical analysis of data can generally be achieved using commercially-available software, such as Excel, GSLIB, ISATIS and MATLAB. Within this software, however, programme-specific algorithms and routines are used. For example, in Finland, a MATLAB application has been developed to analyse thermal data from the TERO borehole tool and an Excel application has been developed to process information from the CSIRA stress measurement interpretation code. Therefore, while the software environment can be purchased, it will become more programme-specific and less transferable over time.

A common suite of commercial codes for geomechanical analysis and modelling is provided by Itasca (an international engineering company), including FLAC, PFC and DEC in 2-D and 3-D, which have a strong pedigree having been used worldwide over many years. The PFC codes concern micromechanical analysis of geomaterials and particulate systems, while the FLAC and DEC codes are used to simulate the larger-scale behaviour of structures under loads. The distinction between FLAC and DEC is continuum versus discrete element modelling. The former is more generally applied in the lower-strength rock environments, but all of the codes have been applied across all of the rock types. Geomechanical and hydromechanical models such as Poly3D, Visage and ANSYS are used in the oil and gas industry to predict changes in the stress state during reservoir production.

Site-specific proprietary software has been used for site-specific models, developed for complex geomechanical simulations. The models tend to rely on observations in underground excavations for parameterisation and/or verification and, as such, may be developed to a large degree later in a site investigation programme. The Nagra rock mechanics model was informed by observation and experiment in the Mont Terri tunnel, while the WIPP rock mechanics model (SANTOS) was developed significantly through experiments and testing in underground excavations after its initial derivation from borehole core analysis.

8.3 PEOPLE
Application of regional geomechanical and thermal data to consideration of particular sites should be within the capability of a competent geoengineer. However, geotechnical modelling and use of software such as FLAC3D and 3DEC requires a significant degree of experience and there may be a lack of such experience specifically within the nuclear decommissioning industry in the UK, as there has been little recent need for such experience. The necessary experience does exist in other industries, such as the oil and gas industry, mining and civil engineering, and should be
readily transferable, but there are general concerns regarding the depth and availability of such resources when required for GDF site investigation in the UK.

Concerns about shortages of scientists and engineers in the Earth sciences have been raised in the context of the future requirements of the UK oil and gas industry. For example, a joint submission to the House of Commons Science and Technology Select Committee in the UK in 2011 from the Petroleum Exploration Society of Great Britain (PESGB), the British Geophysical Association (BGA) and the Geological Society of London (GSL) commented on the effect that the science and research budget allocations have had on the provision of university courses in petroleum geoscience and the likely consequences for the oil and gas industry\(^1\). The submission argued that there is a risk of serious oil and gas industry market failure if there isn’t a provision of suitably qualified scientists and engineers. The submission highlighted that there is a reducing number of university courses in some specialist areas associated with resource exploration and drilling, where many specialists are relatively near to the end of their careers.

It was noted that there are similar risks to future supply of suitably qualified personnel in other sectors dependent on Earth science and engineering skills. For instance, it was estimated that a commercial scale carbon capture and storage industry could require thousands of trained geoscientists. The submission acknowledged that a large number of Earth scientists was being produced in emerging economies, but considered that there is not expected to be any surplus in global supply.

This shortage of skills in the Earth sciences could affect the progress of a GDF site investigation programme in the UK. Further, a GDF site investigation programme will need to compete for qualified scientists and engineers against the backdrop of potentially attractive salaries offered by the oil and gas industry.

\(^1\) Submission to House of Commons Science and Technology Select Committee call for evidence: science and research budget allocations, 2011/12 – 2014/15, 27 April 2011.
9 STRATEGIC APPROACH TO INTERPRETATION AND MODELLING

RWMD plans to take an iterative approach to site characterisation as discussed in NDA (2011, §3.3). The iterative approach involves comparing results of various stages of investigation with predictions made based on the understanding gained from earlier stages of investigation. In this way, the understanding of site conditions is updated until uncertainties have been reduced sufficiently for investigations to be considered complete. The various stages in site investigations will be undertaken in Stages 4, 5 and 6 of the MRWS process, as indicated in Figure 9.1. Stage 5 will involve surface-based investigations at candidate sites and this report is concerned with the interpretation of geotechnical and thermal data acquired in this stage to support the development of the SDM. Interpretation and modelling work will run in parallel with site investigations.

The general requirements of the geotechnical components of the SDM in terms of providing information on the host rock formation, cover sequence and soils are set out in Section 2.2. The requirements of the thermal model in terms of providing information on the thermal properties of the rock are set out in Section 2.3. However, the extent of information required will to some extent depend on the type of host rock being investigated. This review has considered data interpretation and modelling techniques associated with site characterisation for facilities in higher-strength rocks, lower-strength sedimentary rocks and evaporites.

Geotechnical and thermal data interpretation and the SDM components will be refined as site characterisation proceeds. Data from early investigations would be used to establish the main features and properties of the site that might impact its suitability to host a GDF. Later investigations will address information gaps identified in the preliminary investigations and the interpretation will assess remaining uncertainties in the geotechnical and thermal descriptions of the site.

Figure 9.1. Stages of site characterisation (NDA 2012, §3.3).
9.1 GEOTECHNICAL DATA INTERPRETATION

Geotechnical data will be obtained from seismic surveys, borehole investigations, down-hole logging and rock sampling for laboratory tests. The data interpretation that would be required in conjunction with this data acquisition is described in Table 9.1. Additional interpretation approaches required for investigations in lower-strength sedimentary rocks and evaporites are discussed in Table 9.2 and Table 9.3, respectively. A key difference between the rock types is that evaporites may not contain water, which will influence the interpretation approach.

Table 9.1. Geotechnical data interpretation from surface-based investigations.

<table>
<thead>
<tr>
<th>Test</th>
<th>Parameter</th>
<th>Interpretation method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock stress measurements</td>
<td>Magnitude and direction of the stress field.</td>
<td>Standard interpretation techniques on cores assuming CHILE or DIANE conditions depending.</td>
</tr>
<tr>
<td>(overcoring, hydraulic fracturing and hydraulic tests on pre-existing fractures, borehole breakouts, mapping core discing, focal plane analysis).</td>
<td></td>
<td>Standard interpretation of field tests. Could use supplementary method to evaluate stresses induced by coring, overcoring or fracture growth (e.g. using FLAC3D as done by Posiva). Larger-scale numerical modelling to derive a stress model that respects geological structure, such as deformation zones. Possible approaches include using UDEC (SKB) or a model such as RMM (Posiva).</td>
</tr>
<tr>
<td>Rock properties from laboratory tests</td>
<td>Young’s modulus, Poisson's ratio, rock strength, sonic velocity data, including P- and S-waves, bulk density.</td>
<td>Standard interpretation methods on core samples. Evaluate mean and standard deviation of mechanical properties. Analysis could be supported by the use of a particle flow code such as PFC3D (Posiva) to aid understanding of laboratory tests, especially the effects of anisotropy. Use interpretations in combination with the geological model to estimate properties at a larger scale. Empirical approaches based on rock classification indexes or numerical models such as 3DEC (SKB) are available.</td>
</tr>
<tr>
<td>(uniaxial compression tests, triaxial compression tests, Brazilian test).</td>
<td></td>
<td>Standard interpretation methods involving consideration of failure criteria. Use interpretations as input to geological modelling.</td>
</tr>
<tr>
<td>Fracture properties from laboratory tests</td>
<td>Tensile strength, normal stiffness, shear strength and shear stiffness.</td>
<td>Standard interpretation methods involving consideration of failure criteria. Use interpretations as input to geological modelling.</td>
</tr>
<tr>
<td>(normal loading tests on fractures, shear tests on fractures).</td>
<td></td>
<td>Standard interpretation methods involving consideration of failure criteria. Use interpretations as input to geological modelling.</td>
</tr>
</tbody>
</table>
Table 9.2. Geotechnical data interpretation from surface-based investigations in lower-strength sedimentary rocks (in addition to that undertaken in higher-strength rocks).

<table>
<thead>
<tr>
<th>Test</th>
<th>Parameter</th>
<th>Interpretation method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock stress measurements</td>
<td>Magnitude and orientation of the stress field.</td>
<td>Interpretation should take account of pore pressure using elasto-plastic or poro-elastic analysis.</td>
</tr>
<tr>
<td>Rock properties from laboratory tests.</td>
<td>Young’s modulus, Poisson’s ratio, rock strength, creep rate, variation with loading rate and parameter distributions.</td>
<td>Interpretation should consider the effects of water content on rock properties. Analysis could be supported by the use of a particle flow code such as PFC2D (Nagra) to aid understanding of the effects of consolidation and anisotropy. Derive creep models from consolidation experiments. Use interpretations in combination with the geological and hydrogeological model to estimate behaviour at a larger scale. Numerical models such as FLAC (Nagra) are available.</td>
</tr>
</tbody>
</table>

Table 9.3. Geotechnical data interpretation from surface-based investigations in evaporites rocks (in addition to that undertaken in higher-strength rocks and lower-strength rocks).

<table>
<thead>
<tr>
<th>Test</th>
<th>Parameter</th>
<th>Interpretation method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock properties from laboratory tests (compression tests at different temperatures)</td>
<td>Creep rate.</td>
<td>Derive creep models from compression tests.</td>
</tr>
</tbody>
</table>

9.2 THERMAL DATA INTERPRETATION

Thermal data will be obtained from down-hole logging of boreholes and rock sampling for laboratory tests. The data interpretation that would be required in conjunction with this data acquisition is described in Table 9.4.

Table 9.4. Thermal data interpretation from surface-based investigations.

<table>
<thead>
<tr>
<th>Test</th>
<th>Parameter</th>
<th>Interpretation method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature from borehole Logging.</td>
<td>Temperature of rock mass with depth.</td>
<td>Evaluate mean and standard deviation based on data from all boreholes.</td>
</tr>
<tr>
<td>Thermal conductivity from borehole measurements.</td>
<td>Thermal conductivity of rock mass over large volumes.</td>
<td>Use analytical or numerical inversion methods.</td>
</tr>
<tr>
<td>Laboratory tests on core samples (TPS method and calorimetric method).</td>
<td>Thermal expansion coefficient, density, thermal conductivity, and heat capacity.</td>
<td>Evaluate mean and standard deviation of thermal properties to address uncertainty. Also use density measurements and understanding of the thermal properties of constituent minerals to estimate thermal conductivity. Determine heat capacity indirectly from thermal conductivity and diffusivity measurements. Undertake stochastic simulations of lithology and thermal conductivity to derive realisations of thermal conductivities for each rock domain (geo-statistical approach).</td>
</tr>
</tbody>
</table>
10 SUMMARY AND CONCLUSIONS

RWMD is undertaking work to develop an understanding of approaches to carrying out a site characterisation programme. RWMD proposes to develop and present the information derived from site characterisation activities for a geological disposal facility in the form of a single integrated SDM, which will describe the geometry, properties of the bedrock and water, and the associated interacting processes and mechanisms, which will be used to address the information requirements of all end users. The SDM will be divided into parts comprising clearly defined disciplines:

- Geology;
- Hydrogeology;
- Hydrochemistry;
- Geotechnical;
- Radionuclide Transport Properties;
- Thermal Properties; and
- Biosphere.

Work is being undertaken to provide descriptions of the techniques available for the modelling and interpretation of site characterisation data in each of these disciplines. This report provides a review of approaches that have been adopted to geotechnical and thermal data interpretation and modelling.

10.1 CASE STUDIES

The approaches to geotechnical and thermal data interpretation and modelling at a number of sites have been reviewed. These case studies have been presented in terms of the three generic host rock types being considered by RWMD, which are:

- Higher-strength rocks: SKB (Forsmark, Sweden); Posiva (Olkiluoto, Finland) and UK Nirex (Sellafield);
- Lower-strength sedimentary rocks: NAGRA (Benken, Switzerland); and

Key findings of the reviews are as follows:

- Care should be taken in deciding the simulation scale and the simulation volume given the different scales of measurements and rock volumes of importance to rock mechanics and thermal modelling. That is, it is important to understand the upscaling factor, especially concerning rock strength.
- For geotechnical site understanding, a strong link to the geological model which provides the geometric framework for expanding point values into volumes is important.
- Understanding the rock stress regime at a site may be complicated if the site has a complex deformation history.
- Statistical and geostatistical approaches are useful for determining the geotechnical properties at a site and understanding scale effects.
- In lower-strength sedimentary rocks, interpretation and modelling is likely to focus on the development of models that describe rock behaviour by using micro-mechanical models, short and medium-term deformation models and long-term deformation models (creep) as well as linear and bi-linear rock mechanical models.
- High-resolution 3-D seismic reflection surveys can be used to characterise and interpret geotechnical and thermal parameters, especially in a homogeneous formation.
- Hydro-mechanical and thermo-mechanical couplings should be taken into account in the interpretation of data from lower-strength sedimentary rocks.
- Laboratory analysis, constitutive model formulation, and numerical modelling are likely to be required in the development of thermal/structural codes to understand the creep deformation of evaporites. The models are likely to evolve significantly as information from underground excavations becomes available.
A review of interpretation and modelling of rock mechanical and thermal data in other sectors was also undertaken with the aim of identifying methods and tools that could be utilised in the UK site characterisation programme. The review considered the civil engineering, oil and gas and geothermal sectors. A range of different data interpretation approaches and tools has been identified, but these methods generally focus on understanding specific characterisation pertinent to the engineering or resource sector to which they are being applied. For example, in the oil and gas industry, geomechanical models have been developed specifically to understand how stress states respond to oil and gas production. Interpretation of thermal properties in geothermal energy prospecting is concerned with identifying thermal anomalies. These specialist tools have limited applicability to GDF site investigation and SDM development, but they may be applicable to later stages of GDF excavation, assessment and monitoring.

10.2 DATA INTERPRETATION AND MODELLING TOOLS

Software-based tools used in geotechnical and thermal data interpretation have been used primarily to:

- Derive properties or parameter values from measurements.
- Derive parameter values from statistical analyses.
- Model data.
- Visualise spatial variation.

Derivation of parameter values and statistical analysis can generally be achieved using commercially-available software, such as Excel, GSLIB, ISATIS and MATLAB. Within this software, however, programme-specific algorithms and routines are used. Therefore, while the software environment can be purchased, it will become more programme-specific and less transferable over time.

A common suite of commercial codes for geomechanical analysis and modelling is provided by Itasca (an international engineering company), including FLAC, PFC and DEC in 2-D and 3-D. The PFC codes concern micromechanical analysis of geomaterials and particulate systems, while the FLAC and DEC codes are used to simulate the larger-scale behaviour of structures under loads. Also, geomechanical and hydromechanical models such as Poly3D, Visage and ANSYS are used in the oil and gas industry to predict changes in the stress state during reservoir production.

Site-specific proprietary software has been used for site-specific models, which have been developed for complex geomechanical simulations. The models tend to rely on observations in underground excavations for parameterisation and/or verification and, as such, may be developed to a large degree later in a site investigation programme. The Nagra rock mechanics model was informed by observation and experiment in the Mont Terri tunnel, while the WIPP rock mechanics model was developed significantly through experiments and testing in underground excavations after its initial derivation from borehole core analysis.

10.3 PEOPLE

Use of the modelling tools described above for interpreting geotechnical data from surface-based site investigations requires a significant degree of experience and there may be a lack of such experience specifically within the nuclear decommissioning industry in the UK. The necessary experience does exist in other industries, such as the oil and gas industry, mining and civil engineering, and should be readily transferable, but there are concerns regarding the depth and availability of such resources when required for GDF site investigation in the UK. Limited science and research budget allocations for UK university courses in petroleum geoscience and specialist research areas in Earth sciences have led to concerns in the oil and gas industry about the provision of suitably qualified scientists and engineers in the future, especially in specialist disciplines where many scientists are relatively near to the end of their careers. A GDF site investigation and characterisation programme in the UK will place even greater demands on the supply of these resources. Although a large number of Earth scientists is being produced in emerging economies, there is not expected to be any surplus in global supply.

Broadly, the skills required to interpret thermal and geotechnical data are similar for the range of GDF host rock types being considered by RWMD. There are distinctions between codes such as FLAC and DEC (continuum versus discrete element modelling), with the former being more generally applied in lower-strength rock environments. However, the skills required to use these codes are similar and they have been applied across all of the rock types. The development of SDMs for evaporite and lower-strength sedimentary rocks specifically requires an understanding of the elasto-plastic behaviour of rocks and specific skills in the development of rock creep models. For example, the creep model developed at the WIPP site involved a highly skilled research team and there is a limited capacity for such modelling worldwide.
10.4 STRATEGIC APPROACH

The approach to site characterisation will depend on the host rock selected and the extent to which the proposed site has undergone previous investigation and characterisation. It is anticipated that data will be obtained from seismic surveys, borehole investigations, down-hole logging and rock sampling for laboratory tests. A strategic approach to interpreting and modelling geotechnical and thermal data has been set out that describes the different techniques to be used to derive each parameter.

Many of the techniques to derive rock mechanics and thermal properties involve the use of standard, well-established data interpretation methods. As data become available, large-scale numerical modelling would be undertaken to derive geotechnical and thermal parameters on a large scale that respects geological structure, such as deformation zones and accounts for uncertainty and spatial variability.
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Sections 1 and 2


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Section 3.2 – Finland


Section 3.3 – UK


Section 4 – Switzerland


Section 5 – USA


Section 6 – Soils

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Section 7 – Other Sectors


