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Site Investigations

Strategy for Rock Mechanics Site Descriptive Model

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Preface

This Report is based on the results of a suite of projects with the main objective to establish a systematic strategy for development of a Rock Mechanics Site Descriptive Model. This model is a part of the geoscientific site description for the Site Investigations that SKB plan to carry out. The Site Description forms a base for the Design and Safety Assessment works that have to be undertaken for each of the studied sites to enable a comparison and evaluation.

The suite of work that forms the input to this Report is given in the following Table:

SKB report No.	**Authors**	**Title**
R-02-01	Röshoff K, Lanaro F, Jing L	Strategy for a Rock Mechanics Site Descriptive Model: Development and Testing of the Empirical Approach
R-02-02	Staub I, Fredriksson A, Outters N	Strategy for a Rock Mechanics Site Descriptive Model: Development and Testing of the Theoretical Approach
R-02-03	Hakami E, Hakami H, Cosgrove J	Strategy for a Rock Mechanics Site Descriptive Model: Development and Testing of an Approach to Modelling the State of Stress
R-02-04	Editor J A Hudson	Strategy for a Rock Mechanics Site Descriptive Model: A Test Case based on data from the Äspö HRL
R-02-11	Makurat A, Løset F, Hagen A W, Tunbridge L, Kveldsvik V, Grimstad E	Äspö HRL A Descriptive Rock Mechanics Model for the 380–500 m level
IPR-02-11	Pinnaduwa H S W Kulatilake, Jinyong Park, Jeong-Gi Um	Estimation of rock mass strength and deformation in three dimensions for four 30m cubes located at a depth region of 380–500 m at Äspö HRL
IPR-02-12	Pinnaduwa H S W Kulatilake, Jeong-Gi Um	Fracture network models in three dimensions for four 30m cubes located at a depth region of 380–500 m at Äspö HRL

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Abstract

As a part of the planning work for the Site Investigations, SKB has developed a Rock Mechanics Site Descriptive Modelling Strategy. Similar strategies are being developed for other disciplines. The objective of the strategy is that it should guide the practical implementation of evaluating site specific data during the Site Investigations. It is also understood that further development may be needed. This methodology enables the crystalline rock mass to be characterised in terms of the quality at different sites, for considering rock engineering constructability, and for providing the input to numerical models and performance assessment calculations. The model describes the initial stresses and the distribution of deformation and strength properties of the intact rock, of fractures and fracture zones, and of the rock mass. The rock mass mechanical properties are estimated by empirical relations and by numerical simulations. The methodology is based on

- estimation of mechanical properties using both empirical and theroretical/numerical approaches; and
- estimation of in situ rock stress using judgement and numerical modelling, including the influence of fracture zones.

These approaches are initially used separately, and then combined to produce the required characterisation estimates. The methodology was evaluated with a Test Case at the Äspö Hard Rock Laboratory in Sweden. The quality control aspects are an important feature of the methodology: these include Protocols to ensure the structure and coherence of the procedures used, regular meetings to enhance communication, feedback from internal and external reviewing, plus the recording of an audit trail of the development steps and decisions made. The strategy will be reviewed and, if required, updated as appropriate.

Summary

The Swedish Nuclear Fuel and Waste Management Co (SKB) is responsible for the handling and final disposal of the nuclear waste produced in Sweden. In 2002, SKB plans to start site investigations using deep boreholes at different sites. As a part of the planning work, SKB has developed a Rock Mechanics Site Descriptive Modelling Strategy. Similar strategies are being developed for other disciplines. The objective of the strategy as presented in this report is that it should guide the practical implementation of evaluating site specific data during the Site Investigations. It is also understood that further development may be needed.

There are several requirements for the strategy. Most of them are general to all disciplines formulating the Site Descriptive Modelling Strategy. The strategy:

- is developed for needs connected to siting and building a KBS-3 type repository in crystalline rock,
- should be adapted to the iterative and integrated character of the Site Investigation and Site Evaluation programme,
- should allow full transparency of data gathering, management, interpretations, analysis and the presentation of results, and
- should make use of experiences gained, not only in the recent project, but also from practical experiences and observations, e.g. from the SKB Äspö project.

It should also be noted that the Rock Mechanics Site Descriptive Model concerns prediction of parameters. Evaluation of stability or other rock mechanics modelling connected to Design or Safety Assessment is done elsewhere and is not part of the Site Descriptive modelling.

Components of the Rock Mechanics Site Descriptive Model

The rock mass is a mechanical system that is normally in static equilibrium under the prevailing loads. Disturbances to this equilibrium may be caused by load changes, for example due to excavation of cavities in the rock, or to changes in mechanical properties of the rock by alteration over a period of time. Instability leads to deformation of the rock mass and failures can occur if the strength is reached. Such failures do not, however, necessarily imply serious instability.

The Rock Mechanics Site Descriptive Model should include the rock mechanics parameters needed for design and safety assessment. The model should describe the initial stresses and the variability of rock mechanics properties such as the deformability and strength properties of the intact rock, of fractures and fracture zones, and of the rock mass viewed as a unit consisting of intact rock and fractures. The parameters for the rock mass should primarily be provided on the ‘tunnel scale’, considered here as a cube with a 30 m edge length capturing the mechanical processes that may be expected around underground openings of the sizes planned for the KBS-3 repository.

In the characterisation approach, it is the engineering and safety assessment impacts of the property values and their ranges that are the key consideration. Thus, the property estimations should be made bearing in mind the use to which the values are to be put and the practicalities of estimation. There is no need for absolute certainty: it is only an adequate prediction that is required.

Approach to the strategy – the Test Case

It is intended that the development of the Rock Mechanics Site Descriptive Modelling should provide a systematic description of the theories, data and interpretation methods used to develop the model, allowing full transparency of data gathering, management, interpretations, analysis and the presentation of results. A variety of quality control items are necessary and were implemented in the development to progress the work in line with the product realisation principle. Furthermore a Test Case was applied were the developed Rock Mechanics Site Descriptive Model modelling technique were tried on a data set measured at the Äspö Hard Rock Laboratory. Well-defined procedures (Protocols) were established for the components of the Test Case in order that the step-by-step procedures led to the objectives being achieved. Each Protocol consists of three sections: Objectives, Procedures and Products. An auditing trial was established through the project to allow full transparency of the development of the strategies, as well as of the results from the Test Case.

The mechanical properties of the rock mass

The two basic rock mechanics properties of the rock mass, as distinct from intact rock, that are required for initial design are the three dimensional distribution of the rock mass deformation modulus and the rock mass strength at the 'tunnel scale'. These properties of the rock mass are impossible to measure directly during Site Investigation when the site specific data are limited to surface and borehole information. Thus, these properties have to be assessed, rather than measured directly. Empirical relations and numerical simulations have been used for this purpose, although both approaches contain significant uncertainties.

Data potentially available for the rock mechanics modelling comprise both primary data from measurements of the geometrical and mechanical properties of the rock, and processed data (i.e. models) produced in other aspects of the Site Descriptive Model (mainly the geological model). The primary data ('measurements') usually need further evaluation before they can be used for predicting rock mass properties.

A logical step in evaluating the distribution of mechanical indices such as Q and RMR in the three-dimensional rock mass is to determine the variation of these indices along the available boreholes. Some of the component parameters needed may not be readily obtainable and have to be assessed from other sources. The rock mass characterisation is sensitive to the technique adopted for isolating homogeneous sections of borehole and the selection of Q or RMR components has elements of arbitrariness to it, even when the rock properties are reasonably well known.

The rock quality description should be able to characterise, describe and present data uncertainty, spatial variation and confidence. But the available site-specific data will only cover a small portion of the volume to be characterised. The main challenge then is how to extrapolate information measured at the surface and in a few boreholes into a three-dimensional distribution within the model volume. The cornerstone of the SKB strategy for this modelling is the geometrical division in the Geological Model of the rock into fracture zones and rock units, which in turn are grouped in rock domains. In this context, the following specific conclusions were reached in the Test Case during the work to estimate the spatial distribution of rock mechanics parameters.

- Understanding and evaluating the Geological Model from a rock mechanics point of view is essential and should form the basis for the spatial distribution of the rock mechanics properties.
- Visualisation and geostatistical analyses should be undertaken in support of the three-dimensional modelling. However, during site investigation, the distance between boreholes may be larger than the statistical correlation distances.
- Stress dependency should not primarily be handled by sub-dividing rock units into sub-units. It is a much better approach to describe the dependency directly and not via such sub-unit partitioning.
- The rock mechanics modelling should consider whether further sub-divisions of domains are needed to make an adequate description, but it may in fact be better to retain few domains and then increase property uncertainty within the domains.

One approach for determining rock mass properties is to use empirical relations based on different rock mass classification systems, such as Q and RMR. The general assumption with the approach is that such empirical classification indices have a simple relation with the rock mass properties. With this assumption, the spatial distribution and uncertainty of the indices should reflect the spatial variability and much of the uncertainty of the rock mass properties.

For a specific situation, a good correlation between Q or RMR and the mechanical properties could be obtained by adjusting the parameters comprising the indices and/or their rating values. However, this approach would be less useful if additional adjustments were needed for every single site and application. The empirical relations are not derived from basic mechanics. Their validity can thus only be ascertained in situations similar to those on which they are based. For any new construction problem, and in particular when constructions are relatively unique, in the current case a deep repository, the empirical approach cannot be verified beforehand. This does not mean that empirical evaluations would not give valuable insight to potential stability problems, but their application is judgmental, relying on the appropriate choice of formulae and advice from experts. Thus, the empirical approach requires supplementary considerations.

An alternative to the empirical relations for assessing rock properties is to calculate the rock mass properties from known properties of the components of the rock mass, i.e. from the mechanical behaviour of the intact rock, the mechanical behaviour of fractures, and the geometry of fractures. A workable scheme for such a theoretical approach has been developed. However, here are several questions and uncertainties related to the theoretical/numerical approach as implemented. Uncertainties concerning the material model for the intact rock, depth and stress dependence and influence of the domain size and applied boundary conditions are generally manageable. However, the 2D representation of the 3D fracture model, used because of computer resource limitations, is less straight-forward, although it has been established that if the 2D section is in the plane of the weakest direction of the 3D model, i.e. in the direction of the least stability, the results could well be similar. Also, the uncertainty and spatial variability in input data are again an important source of uncertainty. These and other issues need to be explored when judging the overall uncertainty and confidence in the input data to the modelling.

Considering the uncertainties with both the empirical and theoretical approaches, it is evident that one single approach cannot be recommended. Even so, for the Test Case the two approaches provided fairly similar results for the rock mass deformation modulus.

The best approach to the rock mechanics modelling is to apply different methods for estimating the rock mass mechanical properties and then devise a procedure for making an overall judgement. After a first set of different modelling attempts (which could be empirical and theoretical), a stage of harmonisation and amendment should follow with the purpose of identifying and correcting errors, establishing and agreeing on non-method specific assumptions (such as geometry), and making different experts more familiar with each other's approaches.

A combined prediction should be targeted, based on consensus discussion involving relevant expertise at the different stages of the Site Investigation. Four main decision factors should be used in arriving at the consensus range:

- the overlap of the individual predictions;
- the confidence of the individual predictions;
- relevant engineering experience; and
- the engineering significance of differences between different predictions.

This procedure was tested successfully within the Test Case work.

It must also be remembered that the primary aim of the Site Investigation is to find rock volumes suitable for the repository construction. In high quality rock, differences between different methods may be less significant. When identifying volumes of good rock, many of the difficulties discussed in this Section could be of second order.

Initial stress

A necessary component of the rock mechanics site descriptive model is the specification of the pre-existing state of stress in the rock mass because a knowledge of the stress state is required for both analytical and numerical modelling of the stresses induced by excavation of a repository.

The pre-existing rock stress is caused by the combined effect of gravitational and tectonic forces. But, several factors affect the rock stress; in particular, the overall stress state can be locally perturbed by the presence of fractures or fractured zones at various scales. Also, residual stresses, water and temperature can have local influences on the overall stress state. Rock stress estimation is not straightforward because the stress field is likely to vary within the rock mass, and the measurement methods require skill and careful quality control. The modelling strategy attempts to handle these problems.

The stress data that are already available are of three types: global information, Fennoscandian information, and local information (data obtained from an actual site nearby). Studies of rock stresses on a global scale are valuable for considering whether the estimations are in line with the continental or regional trends and for making decisions on model boundary conditions. Collated information of rock stress measurement data in Fennoscandia is available in various databases. Local information will be in the form of stress measurement results in boreholes, usually obtained by overcoring and hydraulic fracturing methods.

The primary information should be interpreted within the context of the geology at the site being studied: this provides an enhanced understanding of the existing stress information and provides guidance for the stress estimation strategy. It is necessary to understand how fractures at various scales might interact with and modify the current regional stress field, and hence advise on the construction of a numerical model to determine the state of stress in the crust, constraining the models using geologically realistic boundary conditions. Furthermore, an understanding of the geological history of the study area is useful for establishing the evolution of the stress regime.

Numerical modelling methods may be used to investigate possible mechanisms responsible for a certain stress pattern at the site. For the Test Case several published examples of such modelling were studied and then also numerical modelling of the structures in the Äspö area was undertaken. The rock mass was modelled as a continuum, and the major fracture zones were modelled as planar, single fractures with Coulomb slip properties. Making the assumption that the latest tectonic movements (fracture movements) in the area were caused by a regional stress field similar to that of today, the stresses in the model can provide an indication of the possible prevailing stress field variations.

Modelling cannot be used to establish the general magnitude of the stress field, but it can assist in interpolating between boreholes where stress measurements have been taken. Also, by changing the model parameters and performing sensitivity studies, modelling can also help in the estimation of the possible stress variation in the area, Alternative geological conceptual models (different fracture zone geometry, mechanical properties and loading conditions) may also be analysed and compared.

Based on the experiences gained, a stress model approach was developed as an integrated approach combining stress measurement information, geological factors, numerical modelling results, and consideration of the uncertainties involved. The approach involves different steps starting with a preliminary stress estimation and followed by interpretation of site specific information. If the stress pattern and structural geology of the site are complex, including major fracture zones intersecting the area, numerical analysis of the stress field is recommended.

Stress measurement results and observations from the site concerning slip directions must be used in the evaluation of the modelling. This is a most difficult step since the stress field in the models will depend on the boundary conditions, the loading sequence and the geometrical and strength properties of the zones. Therefore the different assumptions made for each model should be compared with the input information concerning rock mass and fracture zone geometrical and mechanical properties. The aim of this step is to judge which one of the possible models best represents the actual stress field.

Simple stress estimation models are preferred to complicated models, i.e. a simple linear function of stress versus depth should be used if the underlying reason for the depth variation is not known. It is not recommended that non-linear curves (exponential, logarithmic or polynomials) be fitted directly to measurement data and used as 'models' because, in such cases, there is no mechanical explanation for the observed stresses, and therefore the observation should not be used for stress estimation in areas distant from the measurements. It is important to note clearly within which area of the region and, even more importantly, at which depths, a certain estimation is made. The stress models must therefore be used carefully and not for estimation deeper than the deepest boreholes.

The stress estimation should include a quantitative estimation of the uncertainty and the variability. The confidence in the prediction of the stress magnitudes will be dependent on the measurement results and the complexity of the site. Inside, and also in the vicinity of major fracture zones, both the stress magnitudes and stress orientation are expected to vary significantly from point to point. The estimation of the mean stress inside a fracture zone is therefore more uncertain and the associated estimated local variation will be larger.

The mean orientation for the maximum principal stress may be estimated with a fairly high degree of certainty because both the regional stress pattern and the site-specific measurements can be used. The same general trend, 135º–165º, and 0º ± 10º plunge, is expected for the whole of central and southern Sweden, but local deviations caused by topography and faults could exist. This estimate applies to rock mass blocks away from major fracture zones. The local spatial variation around the mean can be predicted based on measurement data.

Quality Assurance and Interactions with other disciplines

The Site Investigation will be carried out in steps, with data being produced in batches for each site. Consequently, the further development of the Rock Mechanics Site Descriptive Model will be progressed in a stepwise manner as well. Updates of the Rock Mechanics Description will be co-ordinated with the overall revision of the Site Descriptive Model. This means there are many aspects of concern in the Quality Control System to be considered for the Site Investigations and the modelling within the various geoscientific disciplines. 'Technical Auditing' (TA), i.e. examining the technical content to establish if it is adequate for the purpose, and 'Quality Assurance' (QA), i.e. checking that procedures are followed in line with the product realisation principle, plus the review of anomalous results, are essential tools for quality control.

For the development of a Rock Mechanics Model the key aspects are as follows.

- Does the Modeller understand the Geological Model?
- Is the rock mechanics conceptualisation of the Site realistic?
- What kind of instinctive assumptions are made by the Modeller?
- Are reasons for scattering in input data understood?

It is a fundamental principle of the Site Descriptive modelling that there should be consistency between the different discipline descriptions (e.g. the geological, rock mechanics, hydrogeological and hydrogeochemical descriptions). Much of the rock mechanics description builds on the geological and hydrogeological models. These links should be acknowledged, but the rock mechanics modelling also provides important feedback to the geological and hydrogeological modelling. To allow for full traceability in the modelling works, the modeller must consider control of input data, interpretation of input data and documentation of the modelling decision process.

Conclusions and further devlopment needs

The overall approach for developing the Rock Mechanics Site Descriptive Modelling Strategy is judged very successful. The application of more than one method for modelling was shown to be important. This provided insights into the benefits and pitfalls of other approaches and demonstrated the fact that complex problems may have more than one solution. The method of achieving consensus, harmonisation and amendments proved to be essential and gave extra insight.

The use of a Test Case was invaluable for developing the strategy. It forced generic predictions to be specific. It highlighted the need for consensus and demonstrated what can and cannot be predicted. It was also an effective means for others to understand what has been done. Technical Auditing, as well as the Protocols developed, are judged to be potentially powerful Quality Control instruments. These tools were developed in parallel with the methodology and Test Case modelling and, because the strategy development was a learning exercise for everyone involved, strict application of the Quality Control procedures was not possible for all aspects of the development work. Further development of these tools is therefore needed.

The strategy as presented in this Report is judged sufficiently developed for guiding the practical evaluation of rock mechanics data to a rock mechanics site description. The current Report provides the foundation for the approach, but more development will be needed for application during the later stages of the Site Investigation. This strategy ought to be fully reviewed not later than the completion of the planned Initial Site Investigations (i.e. after the first two years of the SKB programme for Site Investigations). Further development should focus on

- Enhancing the Quality Control procedures and the associated scope and content of the Protocols.
- Improving the input and output (post-processing) routines for stress modelling.
- Developing rock mechanics modelling for Design and Safety Assessment in order to create a feedback loop to the property estimation strategy.
- Preparing for possible modification to the approach during the first stages of Site Descriptive Modelling during the Site Investigations.

As the planned Site Investigations proceed and more experience is obtained and related worldwide activities progress, there should also be a mechanism for updating the rock mechanics approach. The updating of the approach should consider, not only advances in knowledge and techniques, but also the implications of the longer term use of the rock mechanics information.

Sammanfattning

Svensk Kärnbränslehantering AB (SKB) ansvarar för hantering och slutförvaring av det kärnavfall som kraftindustrin i Sverige producerar. SKB planerar att år 2002 på flera platser påbörja platsundersökningar, som omfattar djupa borrhål. Som en del i planeringsarbetet har SKB utvecklat en strategi för platsbeskrivande bergmekanisk modellering. Liknande strategier utvecklas för andra discipliner. Målet med den strategi som presenteras i denna rapport är att ge vägledning vid utvärdering av platsspecifika data under platsundersökningen. Det bör noteras att ytterligare utveckling kan komma att behövas.

Det finns ett flertal krav på strategin. De flesta är generella och är giltiga för alla discipliner som tar fram en strategi för platsbeskrivande modellering. Strategin:

- utvecklas för behov relaterade till lokalisering och byggande av ett KBS-3-förvar i kristallint berg.
- ska anpassas till de stegvisa och integrerade platsundersöknings- och platsutvärderingsprogrammen.
- ska tillåta full öppenhet vad gäller datainsamling, styrning, tolkning, analys och presentationen av resultat.
- ska utnyttja erfarenheter, inte bara från detta projekt, utan också från praktiska erfarenheter och iakttagelser, exempelvis från Äspölaboratoriet.

Det bör också noteras att den bergmekaniska platsbeskrivande modellen omfattar uppskattning av värden på parametrar. Utvärdering av stabilitet eller annan bergmekanisk modellering som hör samman med projekteringen av förvaret eller med säkerhetsanalysen görs på annat håll och är inte en del av den platsbeskrivande modelleringen.

Den platsbeskrivande bergmekaniska modellen

Bergmassan är ett mekaniskt system som normalt är i statisk jämvikt vid de belastningar som råder. Denna jämvikt kan påverkas av förändringar i belastningen, till exempel på grund av utbyggnad av bergrum eller att bergets mekaniska egenskaper förändras med tiden. Belastningsförändringar kan leda till deformation av bergmassan och brott om bergets hållfasthet överskrids. Sådana brott behöver dock inte betyda en allvarlig instabilitet.

Den bergmekaniska platsbeskrivande modellen inkluderar de bergmekaniska parametrar som behövs för projektering av förvaret och säkerhetsanalys. Modellen ska beskriva initiala bergsspänningar och variationen i bergmekaniska egenskaper såsom deformerbarhet och hållfasthet i det ostörda berget, i sprickor och sprickzoner samt för bergmassan som en enhet bestående av ostört berg och sprickor. Parametrar för bergmassan tas huvudsakligen fram i ”tunnel skala”. Vilket här betyder en kub med sidlängden 30 meter, för att omfatta de mekaniska processer som förväntas kring bergrum med en storlek som motsvarar de som planeras i KBS-3-förvaret.

Vid karakteriseringen av bergmassan, är det byggnationen och säkerhetsanalysen som är avgörande för vilka egenskaper värden behöver mätas och för vilket intervall. Uppskattningen av egenskaperna ska därför göras med tanke på vad data kommer att användas till och vad som är praktiskt möjligt att bestämma. Absolut visshet behövs inte, det som krävs är en tillräckligt god uppskattning.

Angreppsätt – Testfall

Syftet med utvecklingen av en strategi för den platsbeskrivande bergmekaniska modelleringen är att ge en systematisk beskrivning av teorierna, data och utvärderingsmetoderna som använts för att utveckla modellen. Detta ska ge full öppenhet vad gäller datainsamling, styrning, tolkning, analys och presentationen av resultat. Ett flertal punkter för kvalitetskontroll krävs. Dessutom har den utvecklade tekniken för den platsbeskrivande bergmekaniska modelleringen applicerats på ett testfall med data uppmätta vid Äspölaboratoriet. Väldefinierade tillvägagångssätt (Protokoll) har tagits fram för testfallets olika delar, för att säkerställa att den stegvisa proceduren leder till att målen uppnås. Varje Protokoll består av tre delar: mål, procedurer och produkter. En prov granskning upprättades under projektet för att erhålla full öppenhet av utvecklingen av strategierna såväl som av resultaten från testfallet.

Bergmassans mekaniska egenskaper

De två mekaniska egenskaper hos bergmassan, till skillnad från ostört berg, som behövs för den tidiga initial projekteringen är den tredimensionella fördelningen av bergmassans elasticitets moduler och bergmassans hållfasthet i ”tunnel skala”. Dessa egenskaper är omöjliga att mäta direkt under platsundersökningen eftersom platsspecifika data i detta skede begränsas till information från ytan och från borrhål**.** Dessa egenskaper måste därför uppskattas i stället för att mätas. Empiriska förhållanden och numeriska simuleringar har används för detta ändamål, även om båda tillvägagångssätten är behäftade med betydande osäkerheter.

Data som möjligen är tillgänglig för den bergmekaniska modelleringen består av primära data från mätningar av de geometriska och mekaniska egenskaperna hos berget, och bearbetade data (dvs modeller) producerade för andra syften i den platsbeskrivande modellen (huvudsakligen den geologiska modellen). Primära data (mätta parametrar) behöver normalt utvärderas innan de kan användas för att förutsäga bergmassans egenskaper.

Ett logiskt steg vid utvärdering av fördelningen av mekaniska indikatorer såsom Q och RMR i den tredimensionella bergmassan är att bestämma variationen av dessa indikatorer längs tillgängliga borrhål. Några av de parametrar som behövs kan eventuellt inte erhållas och måste därför uppskattas från andra källor. Karakteriseringen av bergmassan är känslig för tekniken som används för att isolera homogena sektioner av borrhålet och valet av Q och RMR vilka är behäftade med en viss godtycklighet, även när bergets egenskaper är relativt väl kända.

Beskrivningen av bergets kvalitet bör kunna karakterisera, beskriva och presentera osäkerheten, rumsvariationen och konfidens i data. Tillgängliga platsspecifika data kommer bara att täcka en liten del av den volym som ska karakteriseras. Den huvudsakliga utmaningen ligger i hur information uppmätt vid ytan och i enstaka borrhål ska extrapoleras till en tredimensionell fördelning inom modellvolymen.

Hörnstenen i SKB:s strategi för denna modellering är den geometriska modellens indelningen av berget i sprickzoner och bergenheter, vilka i sin tur grupperas i domäner. I detta sammanhang nåddes följande specifika slutsatser under arbetet med att bestämma den rumsliga fördelningen av bergmekaniska egenskaper för testfallet.

- Förståelse och utvärdering av den geologiska modellen ur en bergmekanikers synvinkel är väsentlig och bör utgöra grunden för den rumsliga fördelningen av de bergmekaniska egenskaperna.

- Visualisering och geostatistisk analyser bör genomföras som stöd för den tredimensionella modelleringen. Vid platsundersökningen kan dock avståndet mellan borrhål vara större än den statistiska korrelationslängden.

- Spänningsberoende bör inte huvudsakligen hanteras genom att dela in bergenheter i mindre delar. Ett mycket bättre angreppssätt är att om möjligt beskriva beroendet direkt och inte via en sådan indelning.

- Vid modelleringen av bergmekanikiska förhållanden bör övervägas huruvida en ytterligare indelning av bergenheter behövs för att få en tillräcklig beskrivning. Det kan vara bättre att behålla ett fåtal domäner och öka egenskapernas osäkerhet inom domänen.

En metod för att bestämma egenskaper hos bergmassan är att använda empiriska förhållanden baserade på olika system för klassificering av bergmassa, såsom Q och RMR. Det allmänna antagandet för metoden är att sådana empiriska klassificerings indikatorer har ett enkelt förhållande till bergmassans egenskaper. Indikatorernas rumsliga fördelning och osäkerhet bör med detta antagande återspegla den rumsliga variationen och osäkerheten i bergmassans egenskaper.

För en specifik situation kan en bra korrelation mellan Q eller RMR och de mekaniska egenskaperna erhållas genom en justering av de parametrar som bestämmer indikatorerna och deras utvärdering. Denna metod skulle dock vara mindre användbar om ytterligare justeringar skulle behövas för varje enskild plats och tillämpning. De empiriska förhållandena är inte framtagna från grundläggande mekanikiska samband. Deras giltighet kan därför bara garanteras i situationer som liknar de för vilka de är framtagna. För varje nytt konstruktionsproblem, och då speciellt när konstruktionerna är relativt unika, i detta fall ett djupförvar, går det inte att på förhand verifiera den empiriska metoden. Detta innebär inte att empiriska utvärderingar inte skulle ge värdefull information om potentiella stabilitetsproblem, men deras tillämpning baseras på bedömningar och beror på urvalet av empiriska formler och expertråd. Det empiriska angreppssättet kräver därför ytterliga överväganden.

Ett alternativ till att använda empiriska förhållanden för att uppskatta bergets egenskaper är att beräkna dessa från kända egenskaper hos bergmassans delar, dvs från det mekaniska egenskaperna hos ostört berg, det mekaniska egenskaperna hos sprickor samt sprickornas geometri. Ett användbart system för en sådan teoretisk metod har utvecklats. Här finns det dock flera frågor och osäkerheter som har att göra med det teoretiska/numeriska angreppssätt som har används här. Osäkerheter beträffande materialmodellen för ostört berg, beroende och påverkan av djup och bergsspänningar på domänens storlek och de randvillkor som tillämpas är i regel hanterbara. 2D representationen av 3D-sprickmodellen, som har används på grund av begränsningar i datorkraft, är dock inte lika enkel att hantera. Det har dock fastslagits att om 2D sektionen ligger i samma plan som riktningen för den lägsta bergspänningen i 3D-modellen, det vill säga i den riktning som har minsta stabiliteten, kan resultaten från

2D-modellen mycket väl vara liknande de från en 3D-modell. Osäkerheten och den rumsliga variationen i indata är en viktig källa till osäkerhet. Dessa och andra frågor måste utredas när den totala osäkerheten och tilltron till indata till modellen bedöms.

Med tanke på osäkerheterna i både det empiriska och det teoretiska angreppssättet är det uppenbart att det inte går att rekommendera det ena eller det andra sättet. För testfallet, gav båda angreppssätten tämligen likartade resultat för bergmassans elasticitetsmodul.

Det bästa angreppssättet för att modellera bergmekaniken är att tillämpa olika metoder för att bestämma bergmassans mekaniska egenskaper och därefter ta fram ett förfarande för att göra en helhetsbedömning. Efter en första serie av olika modelleringsförsök (vilka kan vara empiriska eller teoretiska) bör en fas kännetecknad av harmonisering och förbättring följa i avsikt att identifiera och korrigera fel, fastslå och överenskomma om icke-metodspecifika antaganden (som geometri) samt sprida kunskap experter emellan om olika angreppssätt.

En sammanvägd förutsägelse av bergets mekaniska egenskaper ska vara målet, baserad på ett samstämmigt resultat av en diskussion mellan experter inom platsundersökningens delar. Fyra huvudsakliga beslutsfaktorer bör användas för att nå samstämmighet:

- överlappning av de olika förutsägelserna
- konfidensen av de olika förutsägelserna
- relevant ingenjörsmässig erfarenhet
- den ingenjörsmässiga betydelsen av skillnaden mellan olika förutsägelser

Detta tillvägagångssätt testades framgångsrikt inom arbetet med testfallet.

Man måste ha i åtanke att det huvudsakliga målet med platsundersökningen är att hitta en bergvolym som är lämplig för djupförvaret. Skillnader mellan olika metoder för detta bör ha mindre betydelse för berg av hög kvalitet. Många av de svårigheter som har diskuterats kan därför vara av underordnad betydelse vid identifiering av områden i bra berg.

Initialspänningar

En nödvändig del av den bergmekaniska platsbeskrivande modellen är en beskrivning av spänningar i den ostörda bergmassan. Kännedom om detta spänningstillstånd krävs för både analytisk och numerisk modellering av de spänningar som induceras vid byggandet av ett förvar.

Bergspänningar i det ostörda berget orsakas av den kombinerade effekten av gravitations- och tektoniska krafter. Många faktorer påverkar dock bergspänningarna; i synnerhet störs det totala spänningstillståndet lokalt, i olika skalor, av förekomsten av sprickor eller sprickzoner. Residualspänningar, vatten och temperatur kan också ha lokal inverkan på det totala spänningstillståndet. Uppskattning av bergspänning är inte enkelt eftersom spänningsfältet troligtvis varierar i bergmassan, och mätmetoderna kräver skicklighet och noggrann kvalitetskontroll. I modelleringsstrategin försöker man hantera dessa problem.

De spänningsdata som finns tillgängliga är av tre typer: global information, Fennoskandisk information och lokal information (data som har erhållits från en faktisk plats i närheten). Studier av bergspänningar i en global skala är värdefull för att göra en bedömning av huruvida uppskattningarna är i linje med kontinentala eller lokala trender och för att bestämma randvillkor till modellen. Som jämförelse finns information om bergspänningar i Fennoskandia tillgänglig i olika databaser. Lokal information kommer att finnas i form av resultat från spänningsmätningar i borrhål, normalt erhållna genom överborrning och hydraulisk spräckning.

Den primära informationen ska tolkas med hänsyn tagen till geologin på den plats som studeras: Detta ger en förbättrad förståelse för den existerande spänningsinformationen och det ger vägledning till strategin för spänningsbestämning. Det är nödvändigt att förstå hur sprickor i olika skalor kan interagera med och modifiera det nuvarande regionala spänningsfältet. Detta ger vägledning för skapandet av en numerisk modell för att bestämma spänningstillståndet i jordskorpan samt begränsa modellerna via geologiskt realistiska randvillkor. En förståelse för den geologiska historien i den studerade regionen är dessutom värdefull för att fastställa utvecklingen av spänningsfältet.

Numeriska modelleringsmetoder kan användas för att undersöka möjliga mekanismer som påverkar spänningsmönstret på platsen. Flera publicerade exempel på sådan modellering studerades för testfallet, och dessutom genomfördes numerisk modellering av strukturerna i Äspöområdet. Bergmassan modellerades som ett kontinuum och de större sprickzonerna modellerades som plana, enstaka sprickor med Coulumb-egenskaper. Baserat på antagandet att de senaste tektoniska rörelserna (sprickrörelser) i området orsakades av regionala spänningsfält liknande de som finns idag, kan spänningarna i modellen ge en indikation på möjliga variationer i det spänningsfält som råder.

Modellering kan inte användas för att fastställa magnituden på spänningsfältet, men det kan vara ett hjälpmedel för att interpolera mellan borrhål där spänningsmätningar har genomförts. Genom att ändra modell parametrar och genomföra känslighetsstudier kan modellering också vara till hjälp för att bestämma de spänningsvariationer som kan råda i området. Alternativa geologiska konceptuella modeller (olika sprickzongeometri, mekaniska egenskaper och belastningar) kan också analyseras och jämföras.

Baserat på de erfarenheter som har byggts upp, utvecklades ett integrerat angreppsätt som kombinerar information från spänningsmätningar, geologiska faktorer, resultat från numerisk modellering och som tar hänsyn till osäkerheter. Angreppssättet omfattar olika steg och inleds med en preliminär spänningsbestämning följt av tolkning av platsspecifik information. Numerisk analys av spänningsfältet rekommenderas om spänningsmönstret och platsens strukturella geologi är komplex och större sprickzoner korsar området.

Resultat från spänningsmätningar och observationer från platsen om deformations-riktningar måste utnyttjas i utvärderingen av modelleringen. Detta är ett mycket svårt steg eftersom spänningsfältet i modellerna kommer att bero på randvillkor, belastningssteg, samt zonernas geometriska egenskaperna och hållfasthet. De olika antaganden som har gjorts för varje modell bör jämföras med data för bergmassan samt sprickzonernas geometriska och mekaniska egenskaper. Målet med detta steg är att bedöma vilken av de möjliga modellerna som bäst representerar det faktiska spänningsfältet.

Enkla modeller för uppskattning av bergsspänningar är att föredra framför mer komplicerade modeller, det vill säga en enkel linjär funktion mellan spänning och djup bör användas om den förhållandena som ligger till grund för djupvariationen är okänd. Icke-linjära kurvor (exponential, logaritmiska eller polynom) bör inte anpassas direkt till mätdata och användas som ”modeller” eftersom det i ett sådant fall inte finns någon mekanisk förklaring till de observerade spänningarna. Observationen bör därför inte användas för att bestämma spänningarna i områden långt från området där mätningarna genomfördes. Det är viktigt att tydligt notera inom vilket område av regionen och, än viktigare, på vilket djup en viss mätning har gjorts. Spänningsmodellerna måste användas med försiktighet och inte för uppskattningar djupare än det djupaste borrhålet.

Spänningsuppskattningar ska omfatta en kvantitativ uppskattning av osäkerheten och variabilitet. Osäkerhetsnivån i förutsägelsen av spänningens storlek kommer att vara beroende av både mätresultaten och platsens komplexitet. I, och även i närheten av, större sprickzoner förväntas både spänningens storlek och riktning variera avsevärt från punkt till punkt. Bestämningen av medelspänningen i en sprickzon är därför osäkrare och den tillhörande uppskattade lokala variationen kommer att vara större än motsvarande bestämning i bergmassan.

Medelriktningen för den maximala huvudspänningen kan bestämmas med en relativt hög säkerhet eftersom både regionala spänningsmönster och platsspecifika mätningar kan användas. Samma allmänna bäring, 135°–165° och 0° ± 10° stupning förväntas för hela centrala och södra Sverige, men lokala variationer orsakade av topografi och förkastningar kan förekomma. Denna bestämning gäller för block av bergmassa på avstånd från större sprickzoner. Den lokala rumsvariationen kring medelvärdet kan uppskattas från mätdata.

Kvalitetssäkring och växelverkan med andra discipliner

Platsundersökningen kommer att genomföras i steg, data tas fram i kampanjer på varje plats. Den fortsatta utvecklingen av den bergmekaniska platsbeskrivande modellen kommer följaktligen även den att fortskrida stegvis. Uppdateringar av den bergmekaniska beskrivningen av platsen kommer att koordineras med den övergripande uppdateringen av den platsbeskrivande bergmekaniska modellen. Detta innebär att det är många aspekter i kvalitetskontrollsystemet som måste beaktas vid platsundersökningarna och modelleringen inom de olika geovetenskapliga disciplinerna. Viktiga verktyg för kvalitetskontrollen är: ”teknisk granskning”, dvs undersökning av det tekniska innehållet för att fastställa om det är lämpligt för sitt ändamål, ”kvalitetssäkring”, dvs kontroll av att procedurerna för genomförandet följs, samt granskning av anomala resultat.

De viktigaste aspekterna vid utvecklingen av en bergmekanisk modell är följande:

- Förstår modellören den geologiska modellen?
- Är den begreppsmässiga beskrivningen (konceptualiseringen) av platsens bergmekanik realistisk?
- Vilken typ av omedvetna antaganden har modellören gjort?
- Är orsakerna till spridning i indata kända?

Det är en grundprincip i platsbeskrivande modellering att det ska vara konsistens mellan de olika disciplinbeskrivningarna (t.ex. geologiska, bergmekaniska, hydrogeologiska och hydrogeokemiska). En stor del av den bergmekaniska beskrivningen bygger på de geologiska och hydrogeologiska modellerna. Man ska vara medveten om dessa kopplingar, men den bergmekaniska modelleringen ger också viktig återföring till de geologiska och hydrogeologiska modelleringarna. För att erhålla full spårbarhet i modellarbetet måste modellören tänka på kontroll av indata, tolkning av indata och dokumentation av modelleringens beslutsprocess.

Slutsatser och behov av ytterligare utveckling

Det övergripande angreppssättet för att utveckla strategin för platsbeskrivande bergmekanisk modellering bedöms vara väldigt lyckosam. Tillämpningen av mer än en metod för att modellera har visat sig vara viktig. Detta gav insikt om fördelarna och nackdelarna med olika angreppssätt och visade att komplexa problem kan ha mer än en lösning. Metoden för att uppnå samstämmighet, harmonisering och förändringar visade sig vara viktig och gav ytterligare insikt.

Testfallet var ovärderligt för att utveckla strategin. Generiska uppskattningar tvingades ersättas med platsspecifika uppskattningar. Det belyste behovet av samstämmighet och visade vad som kan och vad som inte kan förutsägas. Det var också ett effektivt sätt för andra att förstå vad som hade gjorts. Den tekniska granskningen, liksom de utvecklade protokollen, bedöms vara viktiga verktyg för kvalitetskontrollen. Verktygen utvecklades parallellt med metodiken och testfallsmodelleringen. En strikt tillämpning av procedurerna för kvalitetskontroll var inte möjlig för alla delarna av utvecklingsarbetet på grund av att strategiutvecklingen var en lärorik övning för alla inblandade. Ytterligare utveckling av dessa verktyg krävs.

Såsom strategin presenteras i denna rapport bedöms den vara tillräckligt utvecklad för att kunna vägleda den praktiska utvärderingen av bergmekaniska data till en bergmekanisk platsbeskrivning. Denna rapport ger grunden för angreppssättet, men ytterligare utveckling kommer att behövas för tillämpningen under de senare stegen av platsundersökningen. Strategin bör vara färdiggranskad senast vid slutet av den inledande platsundersökningen (det vill säga efter de första två åren av SKB:s program för platsundersökningar). Den fortsatta utvecklingen bör fokuseras på att:

- förbättra procedurerna för kvalitetskontroll samt protokollens omfattning och innehåll.
- förbättra rutinerna för in- och utdatahantering vid spänningsmodellering.
- utveckla bergmekanisk modellering för projektering och säkerhetsanalys för att skapa en återkoppling till strategin för att uppskatta egenskaper.
- förbereda för möjliga modifieringar i angreppssättet under de första stegen av den platsbeskrivande modelleringen under platsundersökningarna.

När de planerade platsundersökningarna fortskrider och mer erfarenhet erhålls samt att liknande arbete i övriga världen utvecklas så bör det också finnas en mekanism för att uppdatera angreppssättet för bergmekaniken. Uppdateringen bör beakta, inte bara framsteg inom kunskap och teknik, utan också innebörden av att använda bergmekanisk information i ett längre perspektiv.

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1 Introduction

The Swedish Nuclear Fuel and Waste Management Co (SKB) is responsible for the handling and final disposal of the nuclear waste produced in Sweden. In 2002, SKB plans /SKB, 2000a/ to start site investigations from deep boreholes at three different sites. The site investigation phase, which will be carried out in different stages /SKB, 2001/, shall provide the broad knowledge base that is required to evaluate the suitability of investigated sites for a deep repository.

The interpretation of the measured data is made in terms of a *Site Descriptive Model* covering geology, rock mechanics, thermal properties, hydrogeology, hydrogeo-chemistry, transport properties and surface ecosystems /SKB, 2001/. The Site Descriptive model is the cornerstone for the understanding of the investigated site and forms a basis for subsequent planning of the repository design, as well as for Safety Assessment studies. The current report concerns the strategy for predicting the *Rock Mechanics* aspects of this Site Descriptive Model. The strategy builds on experiences gained from a project run as a Test Case during 2001 /Hudson, 2002/. The strategy is developed for the needs connected to siting and building a KBS-3 type repository in crystalline rock but may, with modification, be applicable to other projects and geological conditions as well.

1.1 Objectives and Scope

The objective of this report is to present a strategy for developing the Site Descriptive Rock Mechanics Model within the SKB Site Investigation Programme. The strategy as presented in this report should guide the practical implementation of evaluating site specific data during the Site Investigations, although it is understood that further development may be needed.

There are several requirements for the strategy. Most of them are general to all disciplines formulating the Site Descriptive Modelling Strategy.

- The strategy is developed for needs connected to siting and building a KBS-3 type repository in crystalline rock, with focus on the conditions to be expected at the sites selected for site investigations /SKB, 2000a/. The strategy should provide (predict), in three dimensions, the site specific properties needed for design and safety assessment. However, the currently reported strategy for the Rock Mechanics Description focuses on the needs for design.

- The strategy should be adapted to the iterative and integrated character of the Site Investigation and Site Evaluation programme /SKB, 2000b/. It should be able to incorporate a gradual increase in measured data, so that early predictions are revised when new data become available. Predictions should be consistent with those made in other disciplines (mainly geology and hydrogeology). The strategy shall also guide in establishing when the Site Evaluation, based on investigations from the surface, has fulfilled the characterisation phase to a sufficient degree that the Sites can be compared to each other as a basis for the final decision on siting the deep repository.

- The strategy should allow full transparency of data gathering, management, interpretations, analysis and the presentation of results. The interpreted parameters should cover the entire model domain, not just in the proximity of measuring points. Spatial variability, as well as conceptual and data uncertainty due to sparse data, errors and lack of understanding should be handled and illustrated.

- The strategy should make use of experiences gained, not only in the recent project, but also from practical experiences and observations, e.g. from the SKB Äspö project.

It should thus be noted that the Rock Mechanics Site Descriptive Model concerns *prediction* of *parameters*. Evaluation of stability or other rock mechanics modelling connected to Design or Safety Assessment is not part of the Site Descriptive modelling.

1.2 Rock Mechanics Properties to be Predicted

Before establishing a prediction methodology, it is necessary to establish what needs to be predicted, on what scale and with what accuracy and precision. The SKB selection of parameters to be studied during the site characterisation rests on several assessments of what is required for safety assessment and design. The general objectives of the rock mechanics support for the design activity during the site investigation phase are outlined in /SKB, 2000b/. When the site investigations are finished, the activity design shall have:

- presented one site-adapted deep repository facility among several analysed, and proven its feasibility,

- identified facility-specific technical risks, and

- developed detailed design premises for the detailed characterisation phase.

The site-specific properties in the Site Descriptive Model should allow such analyses to be possible. Based on this reasoning, the general site characterisation programme /SKB, 2001/ lists relevant rock mechanics parameters to be determined (Table 5.2 in /SKB, 2001/). This list needs further specification.

1.2.1 Rock mechanics aspects of constructability and safety

All potential SKB repository sites are in crystalline rock formations within the Baltic Shield, which is known to be a particularly stable part of the Earth's crust. The crystalline rock is composed of minerals, which combine to form different rock types, for example granite. The rock is intersected by discontinuities at various scales, from microcracks of a size less than a mineral grain up to large fracture zones hundreds of metres in extent. The rock material is the intact rock between the discontinuities, while the total medium containing all the different types of discontinuities is referred to as the rock mass. The properties of the rock material and the discontinuities determine the strength and the deformability of the rock mass.

The rock mass is subject to a state of stress. Stress is a second-order tensor quantity requiring six independent values for its specification at a point in a solid. At some depth below the ground surface, the in situ stress field can be approximately described with one vertical and two horizontal components. The vertical stress component is mainly due to gravitation and can in general be assumed to be the product of the depth and the unit weight of the overlying rock mass. Thus, simple depth-stress relations apply for the vertical stress. The horizontal stress components depend not only on the rock overburden, but also on tectonic forces and on glacial rebound effects, and are therefore more difficult to predict than the vertical stress. Inside blocks of intact rock there may also be ‘residual’ stresses. In addition, discontinuities at various scales can have a local influence on the in situ state of stress /Martin et al, 2001/.

The rock mass is a mechanical system that is normally in static equilibrium under the prevailing loads. Disturbances to this equilibrium may be caused by load changes, for example due to excavation of cavities in the rock, or to changes in mechanical properties of the rock by alteration over a period of time. Equilibrium disturbances lead to deformations and, if the rock mass strength is reached, to failures. Therefore, deformability and strength are fundamental rock mass properties for analysing the consequences of loading the repository host rock. Failures do not necessarily imply serious instability. Small deformations, without any consequences for performance and safety, may be sufficient to restore equilibrium to the system.

Constructability

It is noted in the SKB report on what requirements the repository has of the host rock /Andersson et al, 2000/ that the major rock mechanics aspect to consider for the Deep Repository is the risk of spalling during construction. Even if extensive spalling may not have any effect on the long-term safety, it has a significant influence on the constructability – mainly because of the possible hazard for workers, and for the time and costs of construction. This aspect of constructability indicates the importance of well-founded predictions or estimates of stresses and stress variability within the repository host rock volume. It also points to the importance of estimates of intact rock strength and intact rock strength variability.

For construction, there is also a need for estimates of tunnel scale rock mass deformability and rock mass strength, and their variability. These equivalent rock mass properties are needed for assessing the overall stability of rock chambers and rock pillars, and are also used as input in mechanical numerical analyses undertaken in support of the design work.

Long term safety

For assessment of the long term safety, it is necessary to be able to predict and describe the mechanical development of the repository rock mass on all scales. Therefore, the fundamental equivalent continuum properties, i.e. deformability and rock mass strength, are essential, but the intact rock properties and the fractures are also important.

Numerous numerical analyses, both mechanical and thermo-mechanical, have shown that the mechanical integrity of the waste canisters is not threatened in any direct way by processes that can be described by use of equivalent continuum rock mass properties, given that realistic ranges of parameter values are assumed. The importance of the rock mechanics properties to the long term safety is more general, since they govern the overall response of the repository to future loads, and therefore also the scope and extent of mechanically-induced changes in the geosphere retention properties. In particular, during the approximately 5000 years of elevated temperatures following closure of the repository, these properties will be important – because they govern the rock response to thermal stresses. More detailed analyses of the long term mechanical stability of deposition holes would require information on the intact rock strength and deformability, the geometry of the fracture systems and the mechanical properties of these fractures.

1.2.2 Rock properties

Based on the above discussion, the Site Descriptive Model should include the initial stresses and mechanical properties of the rock mass. Thus, the main focus for the prediction is the rock mass properties, with the intact rock properties and the fracture properties being used as input for determining the rock mass properties. However, the predicted three-dimensional distributions of 'measured' data (intact rock properties, fracture statistics, etc) and 'processed' data (empirical indices of rock mass quality, fracture statistics, etc) are by themselves important results. For many purposes, such an intermediate prediction would be sufficient.

Table 1-1 specifies properties to be predicted in the Site Descriptive Rock Mechanics Modelling Strategy. The selection builds on the above reasoning and focuses on the needs for design. For the early stages of site investigation, the current property specification is judged sufficient also for the Safety Assessment needs. However, further development is needed for specifying these needs when applied at the later phases of the site investigation.

Table 1-1 also indicates which scale is needed for describing the parameter. This scale should be consistent with the intended use of the parameter, see Section 1.2.3 below. An initial estimate of the acceptable range for the values is also given in the table, see Section 1.2.4 below. However, these estimates may need re-evaluation when applied as a general methodology during the site investigations.

Table 1-1. Listing of rock mechanics parameters to be predicted by the Rock Mechanics Model, with initial suggestions for acceptable uncertainty values (building on Table 5.2 in /SKB, 2001/).

Rock mass			
Parameter (generally a function of location)	**Scale**	**Units**	**Acceptable estimation**
Orientation of in situ principal stresses	tunnel scale (30 m)	degrees, azimuth/dip	± 20º (if anisotropic otherwise less strict)
Magnitude of in situ principal stresses	tunnel scale (30 m)	MPa	± 20% but high accuracy and precision are required for judging whether σ_1 < 60 MPa
Intact rock strength	detailed scale (m)	MPa	Conclusions concerning whether there is risk for substantial rock failure (e.g. spalling) should be accurate. Such evaluations may, e.g., be made using Figure 5-1 in /Andersson et al, 2000/
Rock mass modulus, E_m	tunnel scale (30 m)	GPa	± 15% if 15 GPa< E_m< 45 GPa less than ±10% if E_m> 45 GPa
Rock mass strength	tunnel scale (30 m)	MPa	Conclusions concerning overall stability of rock pillars etc

1.2.3 Relevant scales

The investigated volume shall cover at least the possible location for the repository, which may involve a surface area of 2–4 km^2. The disposal area is the largest part, but there are many other openings to be built as well. Rock mechanics considerations for a deep KBS-3 repository will have to consider three different scales, Figure 1-1.

1. The repository scale, including the entire rock mass around the repository and up to the surface.

2. The tunnel scale, in practice capturing the mechanical processes that may be expected around any underground opening. These processes are normally limited to within 1.5–2 times the diameter of the opening. Of special concern for a KBS-3 repository, with deposition holes for vertical emplacement in the floor, is the volume closest to the deposition tunnels and holes.

3. The local scale, the volume closest to any opening where the mechanical effects are greatest. Of special concern for a KBS-3 repository is the volume within less than one diameter from the deposition holes (D = 1.75m)

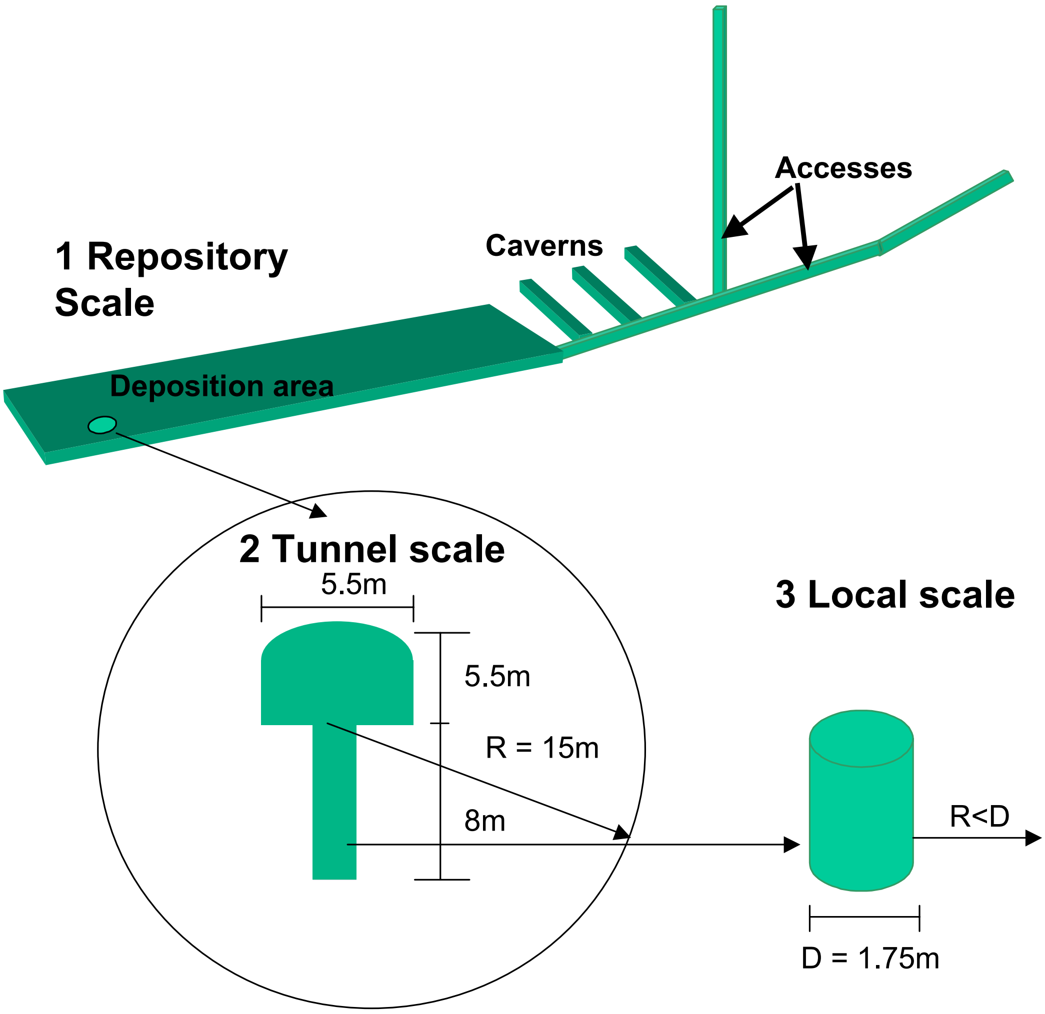

***Figure 1-1.** Illustration of the various scales of importance for the rock mechanics considerations for siting and constructing a KBS-3 repository.*

Some of the caverns that are planned for auxiliary systems may have dimensions up to 15–18 m in span or height. It is, however, estimated that 'the tunnel scale', as defined here, captures the key questions for constructability for the bulk of the tunnels, i.e. the deposition tunnels, with a minimum element size in the model of 30 m by 30 m by 30 m. This scale is also sufficient to cover rock mechanics aspects at 'the repository scale'. For a more detailed scale analysis and for estimating the risk of spalling, a higher resolution may be needed. This is the reasoning for the scales provided in Table 1-1.

The density of measurement points during site investigation will not allow for a precise description at the detailed scale; such a characterisation can only be achieved underground, but the spatial variability can still be well estimated. Furthermore, typical data are measured on a small scale (borehole cores), whereas the main modelling focus during site investigation is on the properties at the tunnel scale. This means that a method needs to be developed to handle the 'upscaling' of a few detailed measurements into rock mass properties at the tunnel scale.

1.2.4 Uncertainty and required prediction ranges

A means of representing uncertainty is needed for each parameter in Table 1-1. Furthermore, all parameters listed are a function of location in space and thus exhibit spatial and directional variability. The methods for dealing with property uncertainty must consider the following at a minimum.

- Lack of knowledge, including conceptual model uncertainty.
- The choice of model representing the rock mass.
- Natural variation of rock properties, including the effects of inhomogeneity, anisotropy, fractures and scale.
- Measurement inaccuracy and imprecision.

Representation of uncertainties in the models of mechanical properties and the state of stress are discussed in Chapter 3 and Chapter 4 respectively. The subject is also discussed in /SKB, 2001/. For example, it is pointed out that uncertainty in the interpretation shall be assessed and quantified after each investigation step. The amount of statistical spread or the validity of many different interpretations based on the same measurement information indicates the degree of uncertainty. Another indication of confidence in the description is the extent to which measurement results from later investigation steps confirm predictions made in earlier steps. Good agreement between prediction and measurement results is a sign that the models are reasonable in terms of the comparison in question, and that there is a reasonably good understanding of the Site. Poor agreement suggests the converse, and the possible need for more data.

Predictions will always imply uncertainties, but this does not necessarily mean the predictions cannot be useful. The properties in the model need only be predicted within an appropriate range. The following overall principles apply when specifying property ranges and when dealing with property uncertainties.

- There is no need for absolute certainty: it is only an adequate prediction that is required, not an exact prediction. If uncertainties are bounded and shown to be acceptable for the performance issues at stake, the model may be sufficient.

- It is the engineering and safety assessment impact of the property range that is the key consideration – through asking questions of the type, "What engineering or safety assessment *decisions* would be altered if the rock mass properties or stresses take different extreme values within the predicted uncertainty (ignorance) range?". Specific attention should be given to considering whether the predicted uncertainty ranges are too wide to be of any engineering value.

- The predictions should be made bearing in mind the use to which the property values are to be put and the practicalities of prediction. The predictions only need to be adequate in the sense that they are commensurate with the purpose of obtaining the property value.

Based on such arguments, Table 1-1 also contains an indication of the precision required in the estimation of parameter values. However, these estimates may need re-evaluation when applied as a general methodology during the site investigations.

1.3 Modelling Approach

The Rock Mechanics Site Descriptive Modelling Approach is part of the overall approach for Site Descriptive Modelling. Furthermore, the approach has to fulfil the needs as specified in Section 1.2.

1.3.1 Model requirements

The Model must meet the following requirements to be able to serve as a basis for design and safety assessment and the associated analyses.

- Ensure that the necessary variables, mechanisms and parameters have been included.

- Allow full transparency of data gathering, management, interpretations, analysis and the presentation of results.

- Provide interpreted rock mechanics data (properties and stresses) for the entire Model.

1.3.2 General approach to site specific modelling

The general characterisation program /SKB, 2001/ describes the objectives of the different programme stages, methods for characterisation, the different characterisation stages and the interaction required between different disciplines. Work on the Design and Safety Assessment, as well as studies of the Environmental Impact Assessment, will be carried out in parallel and will thus allow feedback both to the characterisation programme and to the Site Descriptive Modelling efforts, as discussed in /SKB, 2000b/.

Structure

Figure 1-2 illustrates the general structure of the site descriptive modelling. The investigations result in primary data (measurements and directly calculated values) that are collected in a database. These data are used as input for predicting parameters in three dimensions. The various components of the site description (geology, rock mechanics, hydrogeology etc) are developed in an integrated fashion. Furthermore, there is considerable feedback (not shown in the figure) after each stage of site investigation.

The Site Description is thus represented by the combination of the geometrical framework and predicted property values within this geometrical framework. The Geological Descriptive Model forms a base for the other models, and it is a general ambition that the models should be mutually consistent. This implies that developing the site descriptive models will rely on substantial interaction between the different disciplines.

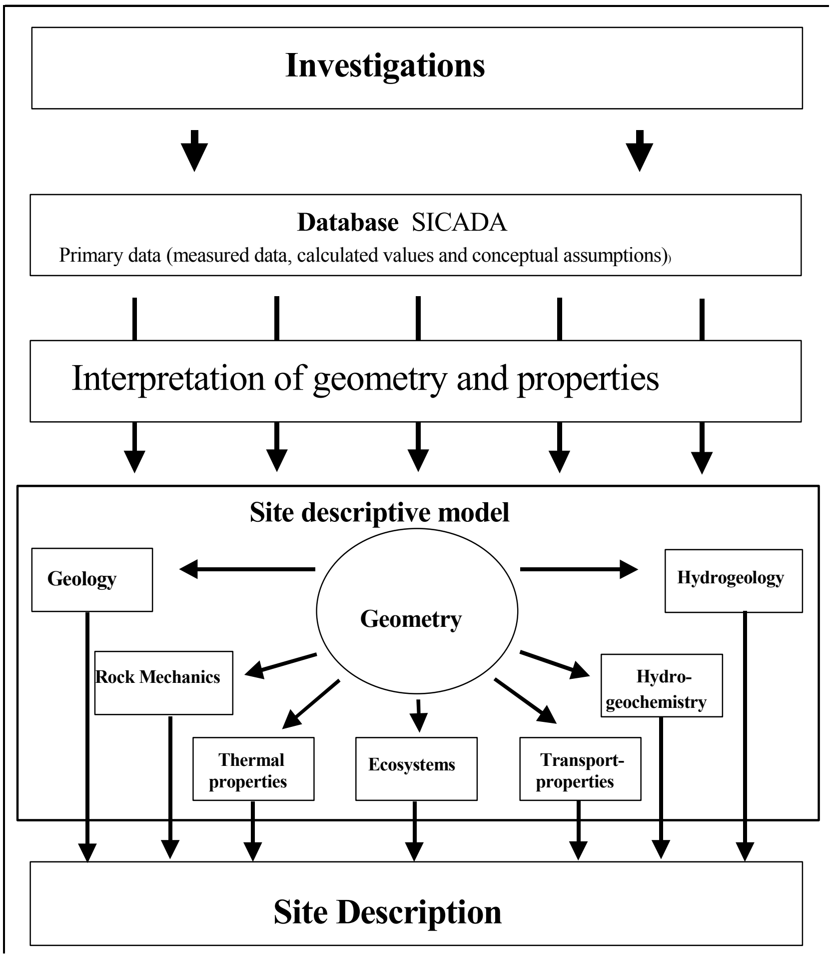

Figure 1-2. *The overall information flow from site investigations to site description and associated databases. (Note that the substantial feedback and integration needed between different disciplines, e.g. geology, rock mechanics, hydrogeology etc, is not fully illustrated) (Figure 2-4 in /SKB, 2001/).*

Components

SKB has also presented a methodology to construct, visualise and present the Site Descriptive Models /Munier and Hermanson, 2001/. These aspects are further discussed in Chapter 5. The methodology, which undergoes gradual development, has four main components as follows.

1. Construction of a geometrical model of the interpreted main rock components (fracture zones and different volumes between fracture zones) at the Site.
2. Description of the geoscientific characteristics of the rock components.
3. Description of the geometric uncertainties in the interpreted model structure.
4. Quality system for handling the geometrical model and the associated database.

The modelling work starts out from the primary data measured at the site. It is fundamental for quality assurance that data for modelling are taken from the SKB Site Characterisation Database (SICADA). Storing and retrieving data are controlled by strict QA-procedures. The main tool for interpreting and visualising geometrical information is the Rock Visualisation System (RVS) – a Microstation-based 3D CAD software package developed by SKB.

The geometrical modelling (Steps 1 and 3) is mainly the concern of the geological modellers. Other disciplines will also have an impact on the geometrical modelling results. All disciplines are faced with describing the discipline-specific rock conditions in the three-dimensional model domain (Step 2). The main challenge is how to extrapolate information from the surface and a few boreholes into three-dimensional form in the model volume.

Documentation

The Site Descriptive Model is not restricted to the geometrical representation in RVS, nor to the parameter distributions in the rock domains. An essential part is the documentation of the Model. The documentation relies on the following.

- A description of the information flow from primary data in SICADA and use of other discipline models into the final geometrical and parameter distributions in the Site Descriptive Model.
- A description of how uncertainties were estimated.
- An account of the arguments in support of and against the confidence in the model, or parts of the model.

Thus, the description of uncertainty is an integrated part of the Site Description.

Handling uncertainty

There are always uncertainties in interpreting measurements and rock parameters which vary in space. The three-dimensional description should present the parameters with their spatial variability over a relevant scale, with the uncertainty included in this description. In addition, it is also necessary to describe the confidence in the model predictions. The site descriptive modelling should deal with conceptual uncertainty, data uncertainty, spatial variability and confidence. Even if these concepts are related, it is useful to keep them separated.

Conceptual uncertainty concerns the uncertainty originating from an incomplete understanding of the structure of the analysed systems and the constituent interacting processes. The uncertainty is comprised both of lack of understanding of individual processes and the extent and nature of the interactions between processes. For the Site Descriptive Model, incomplete understanding of the basic geometrical structures of the rock (i.e. is there such a thing as a 'fracture zone') is also a conceptual uncertainty. Within geology it is also quite common to denote uncertainties in the geometrical model – such as the number and position of fracture zones – as conceptual uncertainties, but these uncertainties are related to data uncertainties (see below). The use of 'conceptual uncertainty' in this context is not supported and should be avoided.

Data uncertainty concerns uncertainty in the values of the parameters of a model. Data uncertainties may be caused by, for example, measurement errors, interpretation errors, or the uncertainty involved in extrapolation when the parameter varies in space and possibly also in time. Conceptual uncertainty can cause data uncertainty.

Spatial variability concerns the variation in space of a parameter value. Spatial variability is not uncertainty *per se* because it can be well recognised and understood, but it is often a cause for data uncertainty. Parameters with strong spatial variation are difficult to evaluate beyond the local region of their measurement.

Scale concerns the spatial resolution of the description. For a spatially varying property, the scale is the size of the domain over which properties are averaged. Spatially varying properties will manifest different values when described at different scales. For example, using a high resolution description, i.e. at the 'small scale', intact rock and fractures would be described as individual entities but, at the larger scale, the descriptions would be combined into a 'rock mass' value. Scale should not be confused with accuracy or precision.

The ***confidence*** in a descriptive model is the total assembly of motives, indications, and arguments in support of the model. High confidence is not the same as low uncertainty. If the uncertainty description is well founded, the confidence can be high in the model. Conversely, if a model description with low uncertainty has a poor foundation, the confidence in the model should be low.

1.3.3 Approach to the Rock Mechanics Site Descriptive Modelling

In meeting the model requirements, as specified in Section 1.2, a rock mechanics modelling methodology has been identified and subsequently evaluated within a Test Case applied with data from the Äspö HRL, see further in Chapter 2. The components of the Rock Mechanics Site Descriptive Model, as well as the proposed flow chart to achieve a complete Model, are illustrated in Figure 1-3. The two important components are the Geometrical and Mechanical Characterisation Model and the In Situ Rock Stress Model.

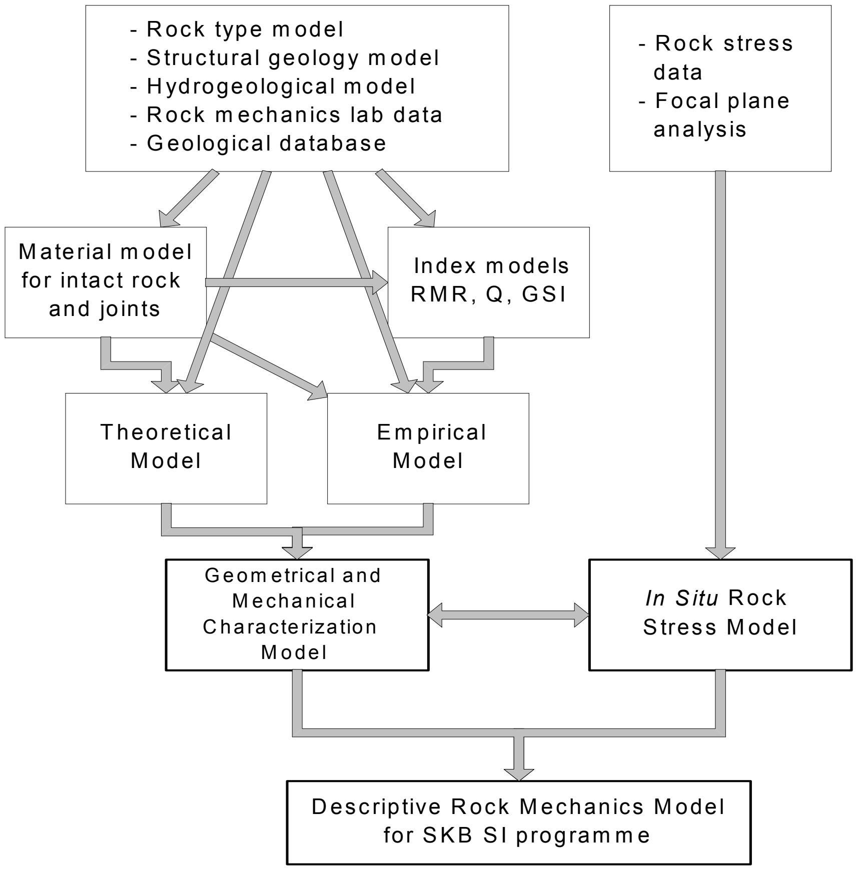

Figure 1-3. *Input data and flowchart for the Rock Mechanics Site Descriptive Model.*

The modelling approach is built around a set of protocols describing what to model, how to model it and how to assess the quality of the results, thereby allowing technical auditing of the modelling work. The concept of protocols and their value is discussed in Chapter 2. Furthermore, a guiding principle in developing the model strategy is that the answers to "What are the relevant questions for rock mechanics modelling?" may vary with time. A programme run under a systematic quality system, for example in accordance with ISO 9001, must be improved over time. Such a process is outlined in Chapter 7.

1.4 This Report

The structure of this report is as follows. Chapter 2 outlines the approach to the strategy and presents the work performed to develop it. To a large extent, this strategy builds on a *Test Case* established to develop and then apply the modelling technique on a limited data set measured at the Äspö Hard Rock Laboratory, see /Hudson, 2002/. Chapter 3 presents the approach for predicting rock mass mechanical properties. It is based on the application of empirical methods /Röshoff et al, 2002/ and numerical simulation methods /Staub et al, 2002/, but the findings of these analyses are further developed and adapted to the overall needs of the Site Descriptive Modelling in the SKB programme. Chapter 4 presents the approach for predicting the state of stress (prior to excavation).

It builds on the work of /Hakami et al, 2002/, but further development and thoughts are also included. Subsequent chapters cover the following: Chapter 5, Documentation and visualisation; Chapter 6, Integration with other disciplines for the site descriptive model; and, Chapter 7, Continuing improvement of the strategy. Final conclusions and recommendations are presented in Chapter 8.

2 Approach to the Strategy

The purpose of this Chapter is to provide a description of the way in which the approach to the strategy was developed using three converging methods and a Test Case structured by Protocols. The emphasis in the Chapter is on the Quality Control items used to ensure that the work was well structured and that the required end product was achieved. Following this Chapter, the actual techniques used for estimating rock mass properties and rock stress are described in Chapters 3 and 4.

It was important at the outset to implement an appropriate approach based on the following factors:

- a plan to ensure that the required end product was in fact achieved;
- initial studies to verify that the intended approach was suitable;
- robust and continuous Quality Control to avoid being side-tracked and to ensure transparency;
- a test application of the developed procedures;
- a method for co-ordinating the conclusions; and
- a method for presenting the information.

In this way, the strategy for the approach to the Rock Mechanics Site Descriptive Model would be robust. Also, the approach concentrated on the basic rock mechanics properties which will definitely be required for studies relating to site assessment, repository design and PA/SA. Each of the subjects in the bullet points above is described in the following sections of this Chapter.

The Quality Control items were mainly based on existing SKB company procedures, but some additional items related to quality assurance of geoscientific investigations during SKB's site investigations were included. The SKB approach encompasses quality strategy and quality documents, as well as the parts played by the SICADA database, the RVS modelling tool and GIS in the storage of site-specific information and the associated requirements in respect of traceability and the management of different versions of documents. In line with this guidance, we used project plans and activity plans, concentrating on the required product and the associated documentation. The items that are standard SKB procedures are included here for the sake of completeness in presenting the Quality Control instruments used in the approach.

We concentrated on

- input data: the data used in the activity must be unambiguously identified,
- the process: performance of the activity must be completely documented, and
- output data: the results produced by the activity must be unambiguously identified.

The core of the developmental plan for the approach to the Rock Mechanics Site Descriptive Model incorporated the use of three converging methods and a Test Case with Protocols, as described in Section 2-1. The Quality Control instruments are described in Section 2-2. The procedure for improving the strategy is described later in Chapter 7.

2.1 Development Plan

The development plan incorporated the use of three converging methods and a Test Case with Protocols leading to documentation.

2.1.1 Constraining the approach and achieving the objective

The Rock Mechanics Site Descriptive Model, being part of the Site Description explained in Chapter 1, will form the basis for continuous presentation of the mechanical properties and state of stress for any of the Sites studied. The approach should therefore concentrate on a systematic description of the theories, data and interpretation methods, allowing full transparency of data gathering, management, interpretations, analysis and the presentation of results.

The Model will be for a given volume of rock and it will describe the initial rock stresses and the distribution of the rock mechanics characteristics, such as the deformability and strength of

- the intact rock,
- fractures,
- zones of weakness, and
- the rock mass as a whole, including the intact rock and fracture components.

The site characterisation method should describe development of conceptual models for an area of the order of 100 km^2 to an area of 5–10 km^2. The Model will refer to rock depths to a maximum of about 1000 m. The Model should also describe the rock mass quality in order to support design and constructability analyses.

Furthermore, the purpose of developing the approach to the Rock Mechanics Site Descriptive Model is to have a predictive capability: the suitability of a site or a repository design must be based on some model, whether conceptual, empirical or numerical. This means that the development of the approach must include ensuring that the necessary variables, mechanisms and parameters have been included. This leads to Quality Control requiring

- that the right things are done, Technical Auditing (TA), and
- that things are done right, Quality Assurance (QA),

to ensure a comprehensive, reliable, useful and assured product (cf. Section 2.2.8). Given these requirements and the inherent difficulties in characterising rock masses, it was necessary to constrain the work with Technical Auditing in the early stages with the converging model approaches and then with a series of Protocols for the Test Case.

Throughout, there was emphasis on achieving the required end product within the context of the proposed site investigation over the next few years, as outlined in Chapter 1.

The method should provide interpreted rock mechanics data (properties and stresses) for the entire model and should handle uncertainty due to sparse data, irregular distribution of data and interpretative issues. It was necessary to consider

- uncertainties in measuring parameters and their representativeness,
- uncertainties in the application of specific theoretical models,
- uncertainties in the applicability, relevance and choice of rock mass rating methods,
- uncertainties in links between empirical indices and other models, and
- uncertainties in interpretation, use and judgement of data.

The documents used for reporting, quality assurance and reviews of the work are presented as part of the approach to the Model, as well as a systematic document trail to ensure full document and data control.

2.1.2 The use of converging developmental approaches

The strategy for the approach to the Rock Mechanics Site Descriptive Model content and development has been presented in Figure 1-3. The mechanical properties were estimated using both empirical and theoretical methods as defined below, and subsequent harmonisation of the two approaches was then established.

The flowchart in Figure 1-3 provides an outline of the approach with the three model components.

- ‘Theoretical model’ – the use of numerical models

The development task was to review and test different methods for computing rock mass parameter values, in particular deformation modulus values, using statistically generated fracture networks.

- ‘Empirical model’ – the use of rock mass classification methods

The development task was to review and evaluate the applicability of different rock quality indices and different methods for converting quality indices into rock mass parameter values, in particular the deformation modulus and strength values. For single fractures, empirical methods of obtaining parameter values for fracture deformation and strength models were evaluated.

- ‘Stress model’ – the use of geological evidence and numerical models

The stress analysis approach aimed at generating representations of numerically computed initial stresses. The development task was to find, review and test methods of taking into account structural features and, possibly, the geological history in such a way that reasonable agreement between measured and computed stresses could be obtained.

The results of the three work modules were assembled and converted into a strategy, which was tested, evaluated, modified as needed, and then documented.

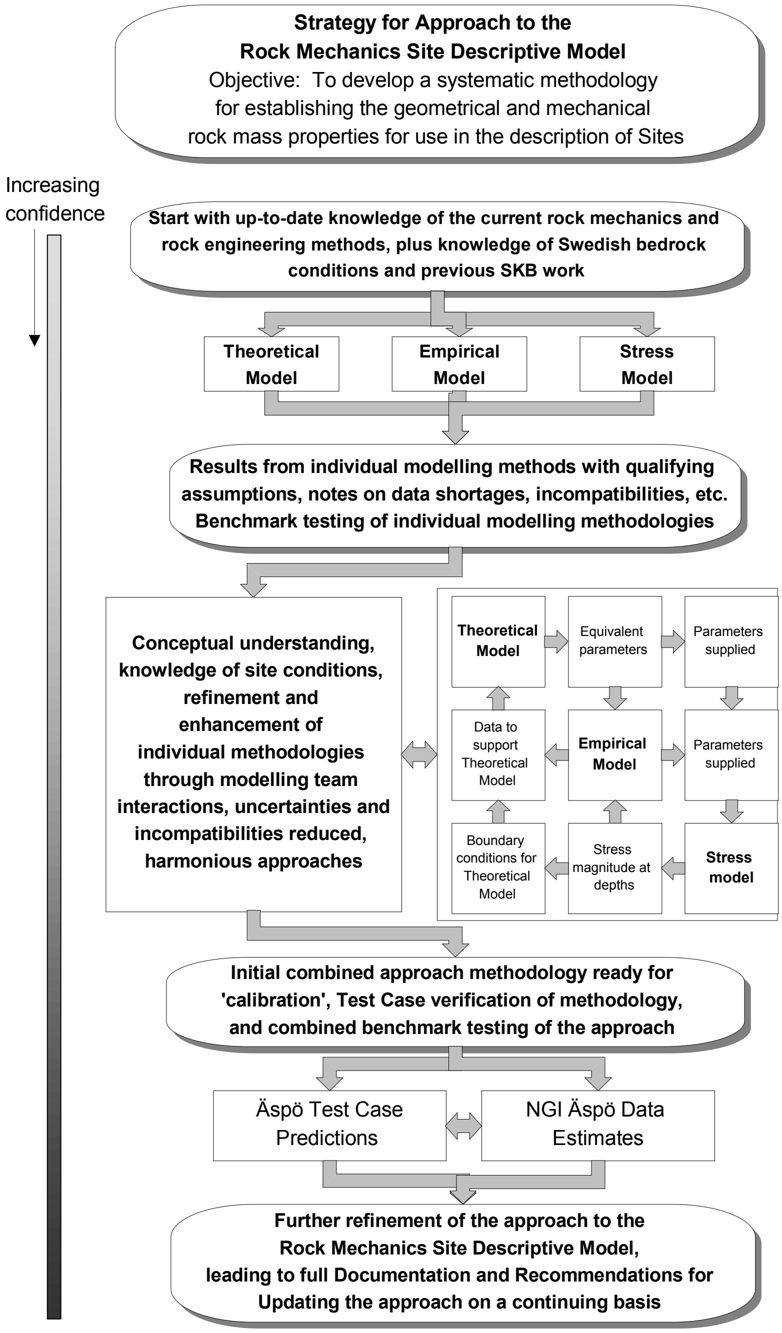

Figure 2-1. *The developmental methodology for ensuring a converging approach to the required Rock Mechanics Site Descriptive Model***.**

The flowchart in Figure 2-1 indicates how the three modelling methods start with up-to-date knowledge of the science and the tools available, plus knowledge of Swedish conditions and previous SKB work. Initial studies were conducted to consider all the factors involved and undertake preliminary modelling. Then, the individual modelling methodologies were developed, followed by consideration of how to combine the three models in order to provide the integrated approach to the Rock Mechanics Site Descriptive Model. Once this had been done, the methodology was evaluated with a Test Case involving the prediction of the rock properties in a rock mass block at the Äspö HRL. This led to further refinement of the overall approach and to the lower box in Figure 2-1, which includes the preparation of this document. The approach was one of continual enhancement of confidence in the methodologies and overall approach, as indicated by the left-hand shaded column in Figure 2-1.

In this way, the method was developed by completing the necessary components and testing them individually and together, see Figure 2-1. The application of the Technical Auditing (TA) and Quality Assurance (QA) procedures was designed to help ensure a comprehensive, reliable, useful and assured product.

The development of the approach was designed not to depend on one person or one resource, and to include full traceability. The development used, with the three modules and the document trail, ensured that this was the case. For the Quality Control instruments, with the Technical Auditing and Quality Assurance components, a 'common platform' was established for input data, report format, presentation, interpretation, supporting data, checklists, etc.

2.2 Quality Control Methodology

2.2.1 Requirements

Given the importance of ensuring that the approach to the Rock Mechanics Site Descriptive Model was appropriately developed, that the work should indeed be completed within the necessary time-frame, and the inherent difficulties in characterising the stress, geometry and mechanical properties of rock masses, it was essential that the work followed a clear developmental plan and that a robust Quality Control structure was installed. For this reason, and in anticipation of future requirements, the philosophy used by the Quality Assurance standard ISO 9001:2000 was adopted. Central to this newer, more flexible and less prescriptive standard, which has replaced the previous version ISO 9001:1994, is that a product starts out as an idea, and then the idea is realised or actualised by following a set of product realisation processes. Product realisation refers to interconnected processes that are used to bring products into being.

The Quality Control instruments used for this project are listed in Table 2-1.

Table 2-1. Quality control items used in the development of the approach to the Rock Mechanics Site Descriptive Model.

Quality Control Item	Method
Initial considerations	Early discussion on methods and an initial plan
SKB contracted teams	Contracts to the Theoretical, Empirical and Stress Teams
Overall Project Plan	Project Plan produced and refined
Schedule	Schedule with sequence of items and completion dates
Activity plans	Documents describing the work with activity sheets
Handling uncertainties	Emphasis on the methods for dealing with uncertainties
Initial technical auditing	Discussions to alert teams to the main modelling problem areas
Regular team meetings	Meetings of complete team to ensure interaction
Internal reviewing	Continuous internal review of method development
Test Case Project Plan	Test application of the developing Rock Mechanics Site Descriptive Model
Protocols used for the Test Case	A set of nine protocols for testing and refining the Rock Mechanics Site Descriptive Model
Workshop	Two-day in depth discussion of model development
External reviewing	Three meetings of five-member International Panel
Minutes of all main meetings	Minutes distributed and agreed record produced
Production of documents	Emphasis on regular and punctual production of documents
Lessons learnt	Incorporation of lessons in updating approach (in Chapter 7)

Each of the items in Table 2-1 is explained in the following sub-sections. The related items that are recommended for the continual updating of the approach in the years ahead are listed in Chapter 7.

2.2.2 Initial considerations

In the latter half of 2000, SKB discussed how to approach a Rock Mechanics Site Descriptive Model, as outlined in the then recently completed "General investigation and evaluation programme" /SKB, 2000b, TR-00-20/ and further developed in the "Investigation methods and general execution programme" /SKB, 2001, TR-01-29/. These discussions led to the first version of the Project Plan. It was intended that the method would provide interpreted rock mechanics data (properties and stresses) and would handle uncertainty due to sparse data, irregular distribution of data and interpretative issues.

2.2.3 SKB contract teams

Following the initial studies and the establishment of the strategic structure shown in the flowchart in Figure 1-3, contracts were established with Golder Associates, BBK and ITASCA to develop the theoretical, empirical and stress models respectively. The existence of these contracts, with the associated work plans and number of days specified, provided a specific working framework for each team which could be monitored and amended if necessary.

These contracts over the 2001 calendar year included all the phases in Figure 2-1, i.e. not only the development of the individual models, but also creation of the combined model, plus using the respective models for the Test Case, leading to the final reports.

2.2.4 Rock Mechanics Site Descriptive Model Project Plan

The Project Plan concentrated on the objective of the Site Descriptive Rock Mechanics Models for sites considered in the SKB Site Investigation programme, to be able to compare Sites and help estimate the extent of expected stability problems for different design and layout solutions.

The final Project Plan emphasised the following points.

- The method for development of the Site Descriptive Rock Mechanics Model should provide a systematic description of the theories, data and interpretation methods used to develop the Model.
- The methods used should allow full transparency of data gathering, management, interpretations, analysis and the presentation of results. The development of the method should ensure that the necessary variables, mechanisms and parameters have been included.

- The method should provide interpreted rock mechanics data (properties and stresses) for the entire model and will handle uncertainty due to sparse data, irregular distribution of data and interpretative issues.
- The documents used for reporting, quality control and reviews of the work should be presented as part of the Model, as well as a systematic document trail to ensure full document and data control.

A brief summary indicating the work requirements is given in Table 2-2.

Table 2-2. The work components of the Theoretical, Empirical and Stress Models as support for the approach to the Rock Mechanics Site Descriptive Model.

Theoretical model /SKB R-02-01/
Review of literature on constitutive laws for intact rock and joints, statistical representation of the joint geometry, constitutive laws for jointed rock masses, procedures to generate non-persistent joint block models for numerical codes.
Test a distinct element method in 3-D to perform stress analyses in rock masses containing non-persistent joints. Do load tests in the computer to determine the rock mass deformability and strength. Interface with the other models.
Test the method on Äspö data.
Write manual for description of methods.
Empirical model /SKB R-02-02/
Literature research, both of presented cases histories as well as research projects and verify the use of the rating systems and establish their limitations.
Also required are methods for upscaling of empirical parameters and the compatibility and linking techniques for interfacing with the theoretical and stress models in the programme, and development for statistical handling of data in general for the empirical parameters.
Integrate the proposed methodology with the other two models, and make recommendations for its application to the Rock Mechanics Site Descriptive Model.
Test the method on Äspö data.
Write manual for description of methods.
Stress model /SKB R-02-03/
Review of literature concerning tectonic modelling, or modelling of in situ state of stress in crystalline bedrock. Also review of previous modelling work. Exemplifying numerical models will be built where different base cases are tested. The interpretation approach of the results and the sensitivity of the model set-up may be tested on these example cases. Identification and description of the types of structural features that, if found at a site, would contribute to the building of a conceptual in situ stress model.
Formulate a strategy for how to select optimal locations for stress measurements, initially based on the basic conceptual stress model, and later based also on the results from first measurement results and preliminary modelling results. The aim is to obtain an in situ stress model with a low uncertainty level in the area where stresses are of importance in the Safety Assessment.
Test the method on Äspö data.
Write manual for description of methods.

2.2.5 Schedule

The Program Plan generated the schedule as listed in Table 2-3.

Table 2-3. Schedule of the phases in developing the approach to the Rock Mechanics Site Descriptive Model.

Phase and description	Time period
Phase 0 – Write programme and plans.	Late 2000
Phase 1 – Development of methods, identification of required investigations, compilation of basic theories, formulation of hypotheses and requirements for verification of theories. Development of quality control procedures and management of uncertainties procedures, etc.	Start December 2000, finish end of March 2001
Phase 2 – Preparation of activity plans for application of draft methods for Äspö HRL Test Case, using Phase 1 results as point of departure. Select data for the Test Case.	During April, 2001
Phase 3 – Testing of proposed draft method for Äspö HRL Test Case using subset of SICADA database.	May–June, 2001
Phase 4 – Complete application of approach to Rock Mechanics Site Descriptive Model for the Äspö HRL.	Autumn, 2001
Phase 5 – Complete documentation of methods generated for the Rock Mechanics Site Descriptive Model as internal SKB documents, external review and completion (this report).	Finish end of 2001/beginning of 2002

The whole development of the approach to the Rock Mechanics Site Descriptive Model was thus completed in just over one and a half calendar years.

2.2.6 Activity plans

Activity plans set out detailed instructions for how work was to be carried out and documented, as needed for ‘doing the work correctly’. Their most important part is the ‘Controlling activity table’, which summarises performance of the activities (broken down into suitable sub-activities/stages, referring where necessary to more detailed instructions), clarifies responsibilities and specifies expected results/deliveries etc for each sub-activity/stage.

The three contractors were requested to make detailed activity plans and to fill out Activity Sheets. An example of an Activity sheet is given in the Appendix, Section 1.

2.2.7 Handling uncertainties

A key aspect of the work was the treatment of uncertainties. In the context of the three sub-models used for the approach (the theoretical model, empirical model and stress model), work was performed to:

- identify the types and sources of uncertainties introduced into the approach from the three respective models;

- identify the sources of parameter variabilities;
- establish methods to treat the uncertainties and parameter variabilities, and estimate their impact on the performance and reliability of the approach.

It was noted that there is an important difference between the uncertainty and the variability (see also Chapter 1).

Uncertainty concerns something which is unknown today and can arise from assumptions adopted in the models for processes, properties, and conditions. Examples are uncertainties about the correctness of assumed constitutive models, inaccuracy in details of fracture geometries, unknown rules for upscaling, etc. Uncertainties can only be reduced by a certain extent, often by consensus.

Variability concerns changes in conditions, most often related to property and loading parameters, over space or time. We may know that they change, but we may not know accurately how they change. Therefore, variability cannot be characterised with 100% certainty, and is often treated using stochastic processes (Monte Carlo simulations) and sensitivity analysis – so that its impact on the performance of the models could be estimated with a known degree of certainty.

Because the uncertainty and variability will be functions of the subject being studied and the approach, they were initially treated separately in each model component of the approach. Then, there was consideration of how to integrate uncertainty and variability issues into the overall descriptive rock mechanics methodology.

The final uncertainty in the approach to the Rock Mechanics Site Descriptive Model is that the three models, theoretical, empirical and stress, may not be fully reconcilable. However, practical methods should be found to enable matching methods to be developed. The advantages of having the three models far outweighs the final potential difficulty of ensuring perfect matching.

2.2.8 Technical auditing during the formulation of the modelling techniques

The term 'Technical Auditing' refers to the process of checking that the technical content of a procedure or model is adequate for the purpose. /Stille et al, 1998/ in their paper on quality systems and risk analysis point out that 'doing the right things' is not the same as 'doing things right'. One needs to ensure that the right things are being done (through Technical Auditing) before monitoring that these right things are, indeed, being done right (through Quality Assurance).

Thus, it was necessary at the outset in developing the approach to the Rock Mechanics Site Descriptive Model to ensure that the necessary variables, mechanisms and parameters had been included and that the structures of the individual models were sound. This provided confirmation that the essence of the model would successfully capture the rock reality. An associated audit trail was generated through the record of the technical auditing meetings with the three modelling teams.

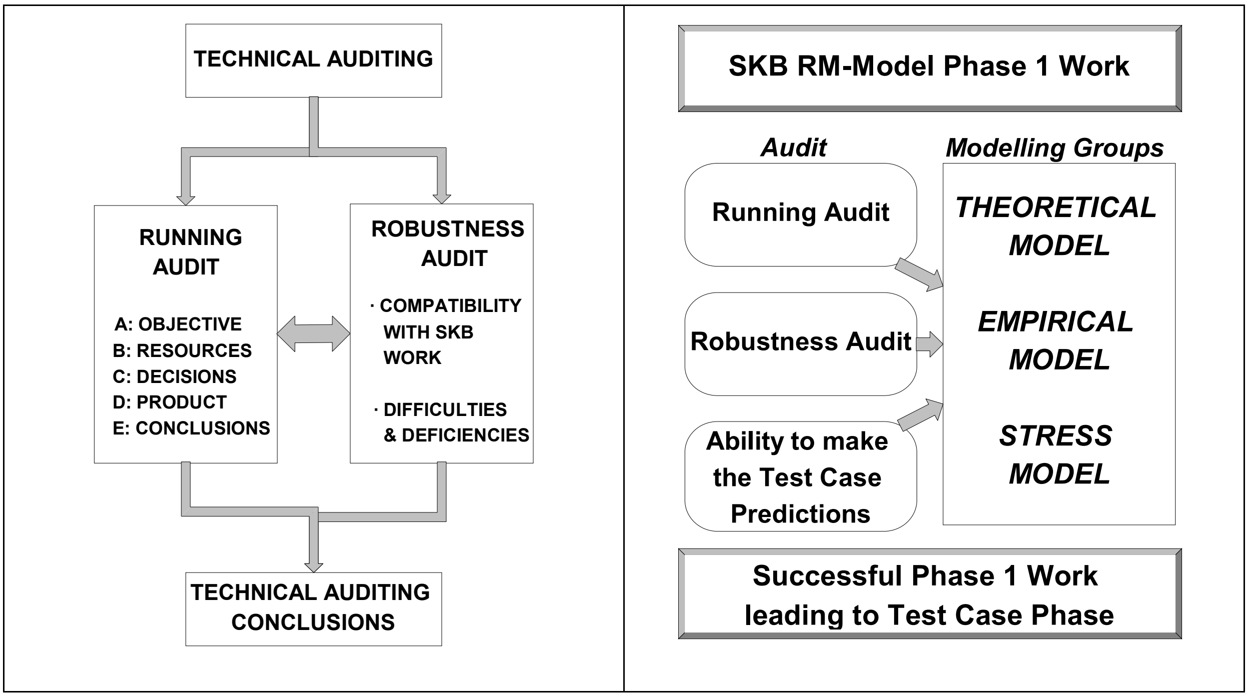

Figure 2-2. *The running audit and robustness audit components of Technical Auditing.*

An example of a Technical Auditing Sheet is given in the Appendix, Section 2. The sections A to E of the 'Running Audit' sheets (see left-hand side of Figure 2-2) were completed for the three model components. An additional audit component was the 'Robustness Audit' (see Figure 2-2) to ensure that potential areas of difficulty were being appropriately addressed. This ensured that both the Test Case and the approach to the Rock Mechanics Site Descriptive Model would be robust and less susceptible to difficulties in the future.

The potential areas of difficulty relate to the discontinuous, inhomogeneous, anisotropic and inelastic nature of rock masses, scale dependency of rock properties, difficulties in measuring rock properties, the lack of comprehensive testing standards, and the need for modelling verification and validation. Particular problem areas are

- using structural geology information,
- establishing rock domains,
- local in situ stress variations,
- characterising post-peak rock behaviour,
- obtaining fracture stiffnesses,
- obtaining fracture data for flow models,
- presenting the content of numerical codes,
- characterising construction effects,
- identifying hazards.

The most important of the issues were considered through a list of twenty-four technical questions, given in the Appendix, Section 3, that covered the well-known difficulties in rock mechanics and rock engineering, /Harrison and Hudson, 2000/. It was not expected that the modelling work would solve the problems, but there should be agreed procedures for dealing with the difficulties. The Technical Auditing approach alerted the three teams to the main problem areas so that these issues would be taken into account during the modelling exercises. However, flexibility in the approach was necessary because there is no clear consensus on some of the issues. The key point is that the modelling teams were aware of these problems and developed appropriately justified approaches.

The 'Ability to Make the Test Case Predictions Audit', see the right-hand side of Figure 2-2, checked that the Phase I approach work would lead smoothly to the Test Case Predictions and into the series of nine Protocols that were developed for the Test Case structure (see 2.2.12). The work associated with these Protocols was clear at an early stage, except for the case of Protocol 5 – Combined Model Predictions. Thus, this third component of the overall Technical Auditing highlighted the need to establish a method by which a combined Theoretical-Empirical-Stress modelling set of rock property predictions could be made.

2.2.9 Regular joint team meetings

There were regular meetings of the three modelling teams throughout the year. These meetings provided the opportunity for communication and resolution of difficulties. The main advantages of the meetings were

- problems could be explained and discussed by all the participants, with the internal reviewers also contributing to resolution of any difficulties, and
- the forum provided an opportunity to check on progress, to alert the teams to the next set of requirements, to implement items of quality control, and to foster a good 'group spirit'.

2.2.10 Internal reviewing

There had to be strict application of management and Quality Control instruments. There were three internal reviewers who led the work, prepared the plans, and ensured that the Quality Control aspects were in place.

This internal reviewing also included commenting on the way in which the science was being conducted. An external Review Panel was also asked to comment on the work (see Section 2.2.14), so the precursor internal reviewing enabled the work of the Review Panel to be most effective because the members did not then have to concentrate on details that should have been resolved beforehand.

2.2.11 Test Case Project Plan

The Test Case component of the approach to the Rock Mechanics Site Descriptive Model is indicated in Figure 2-1. The intent of the Test Case was to apply the Rock Mechanics Site Descriptive Model modelling technique to a limited data set measured at the Äspö Hard Rock Laboratory, to make predictions about the rock mass mechanical state at the tunnel scale for a volume at 400 to 500 m depth, and to compare the model predictions with an assessment of the mechanical conditions in this region based on a much wider and more detailed data set independently studied by an external group (NGI) /Makurat et al, 2002/.

The specific objectives of the Test Case were to

- test the draft modelling manuals developed within the project, to the extent that these manuals would be available at the time of the Test Case, and thereby establish a reference point for future improvements of the methodology,
- trigger the development of practical means for representing uncertainty,
- explore confidence in the modelling technique, by evaluating to what extent theoretical and empirical models arrive at similar conclusions and to what extent they are in agreement with observation (bias and precision),
- evaluate both the practical experiences of the model application and the confidence in its application and thereby document potential problems/difficulties and suggest improvements and adjustments of the modelling technique, and
- explore to what extent increased input data density may enhance predictions.

Hence, the Test Case produced results to be considered in the final production of the manuals describing the approach to the Rock Mechanics Site Descriptive Model. In addition, the conclusions from the Test Case exercise are being documented separately /Hudson, 2002/, with the aim of being able to demonstrate to the professional community, if required, that SKB has a tested methodology for development of the rock mechanics site description and characterisation.

Protocols (well-defined procedures) were established for the components of the Test Case such that the step-by-step procedures led to the objectives being achieved. These Protocols are described in the next Section. Even though this initial Test Case only included a sub-set of the required rock mechanics properties, it was important that the work be structured and subjected to Quality Control through the application of Protocols.

2.2.12 Protocols used in the Test Case

Protocols (a set of detailed rules to be followed in order to attain a desired outcome) were drawn up before starting the actual Test Case task. Protocols may be based on knowledge gained from previous tasks and hope to represent best practice.

The Test Case Protocols developed specifically for the Test Case are shown in Figure 2-3 and are described in detail in the companion Test Case report /Hudson, 2002/.

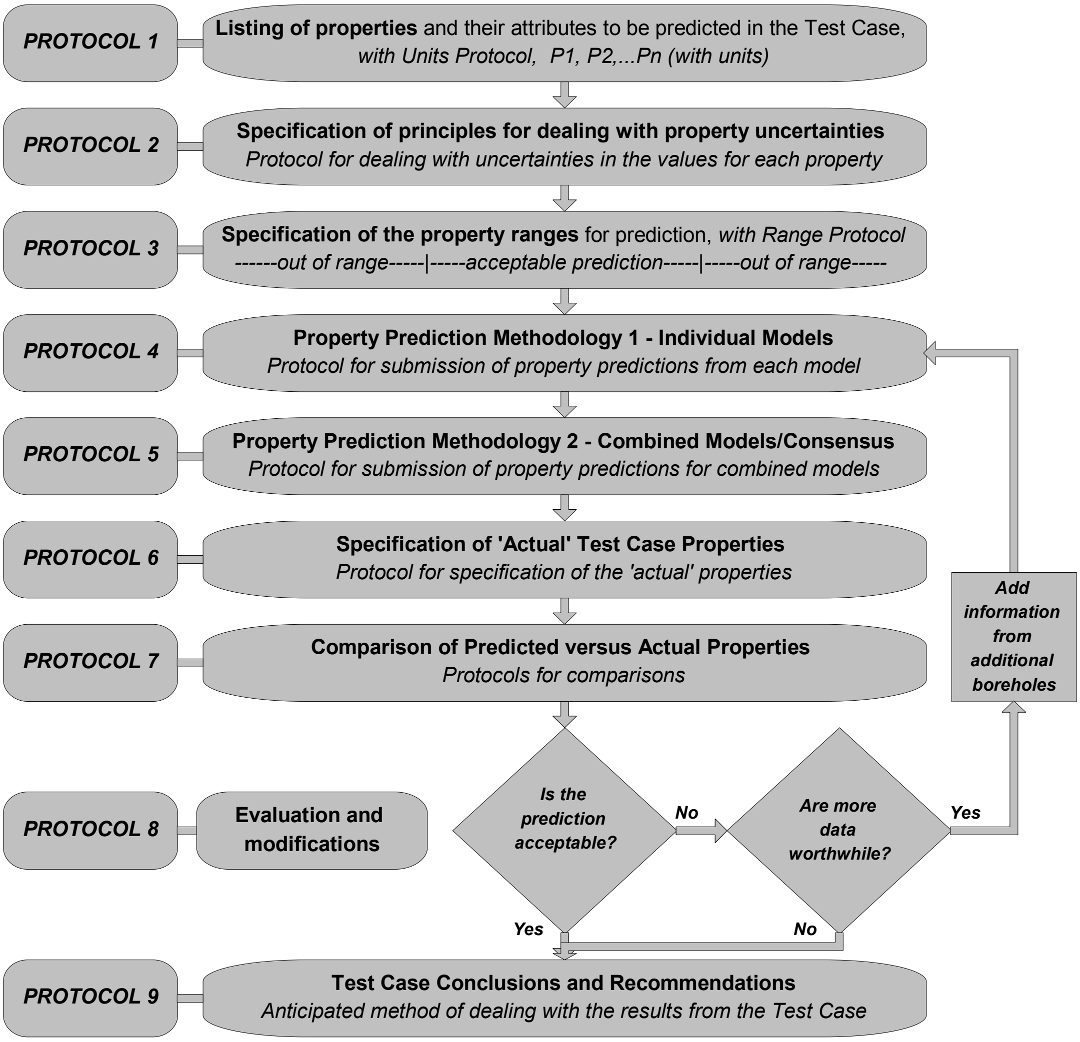

Figure 2-3. *Protocol structure used to guide the developmental work through the Test Case evaluation shown in Figure 2-1.*

These Protocols allowed for flexibility in approaching individual subjects, but ensured that the necessary developmental milestones would be passed and that the objective would be achieved within the requirements already outlined. Thus, the Protocols were a valuable structure at all stages, from the initial listing of the rock properties, through the process of obtaining a combined model using the three basic models, to analysis of the results and making the final conclusions.

Each Protocol consists of three sections.

- **Objectives**: Listing of the one or more objectives associated with each step of the Test Case work.
- **Procedures**: Listing of the procedures to be adopted to achieve the objectives.
- **Products**: The products that will be generated by each step of the Test Case work.

An example of a Protocol Sheet is given in the Appendix, Section 4, and the use of the individual Protocols is also described in the Appendix, Section 5. Similar Protocols are envisaged for continuous updating of the approach to the Rock Mechanics Site Descriptive Model in the years ahead, see Chapter 7, and indeed could be the procedural basis for the future site investigation techniques themselves.

2.2.13 Workshop

As highlighted in the central portion of the Figure 2-1 flowchart, a key component of the approach to the Rock Mechanics Site Descriptive Model was the aim to combine the three converging modelling methods into one model, and for the Test Case to be able to integrate the individual sets of predictions into one set of predictions. Accordingly, and to discuss the issues arising and the remainder of the work, a 'Combined Rock Mechanics Model' Workshop was held for the whole Rock Mechanics Site Descriptive Model approach group. A brief summary of the activities is given in the Appendix, Section 6. The value of the Workshop was demonstrated by the ideas and requirements that emerged.

2.2.14 External reviewing

In addition to the thirteen preceding 'within-development team' Quality Control items already discussed, it was necessary for the approach to be subject to external scrutiny and comment. Accordingly, an external, independent and international Review Panel was formed with representatives from Canada, Finland, Norway, Sweden, and the UK. Three Review Panel meetings were held in 2001.

In order to consider the contribution that the Review Panel has made to the Quality Control, Table 3 in Appendix Section 7 summarises the comments made by the Review Panel in the five categories: use of words and English; communication, presentation and documentation; completeness of parameters and obtaining data, and operation of the model. Also, at the end of this table there is a list of the main oral summary points made by the Review Panel members and points made in written review contributions.

In Figure 2-4, the reviewing points are tabulated simply by the number of comments made in each category. It can be seen that the majority of points related to the science (81%) and the remainder to essentially 'editorial' matters (19%). The Review Panel meetings were successful, not only through the receipt of these comments, but also because they provided a forum where the results to date had to be presented in a formal setting. Also, the teams explained at successive meetings how they had responded to the earlier Review Panel contributions.

Table 2-4. Overall Summary of the Types of Comments Listed in the Appendix, Section 7, Table 3.

(The numbers refer to the total comments in each category, i.e. both the oral and written comments for each Review Panel meeting)

Subject Area	Review Panel Meeting			Total	
	RP1	RP2	RP3		
Use of words, English, etc	13	4	1	18	19%
Communication, presentation, and documentation	12	1	2	15	
Completeness of parameters, obtaining data	15	12	16	43	81%
Content and completeness of model	15	33	8	56	
Operation of model	10	22	14	46	
Total	65	72	41	178	100%

2.2.15 Minutes of all main meetings

A record was made of all the main meetings to remind everyone of what was said and to confirm the actions that needed to be taken. The minutes formed part of the audit trail. All the meetings held in 2001 contributed a wealth of scientific discussion, managerial operations and Quality Control implementation.

2.2.16 Production of documents

The production of documents is a crucial aspect underlying SKB's management system. For example, site investigations will be carried out in the form of projects based on SKB's requirements for project. Special procedures will be established prior to site investigations, dealing with aspects such as organisation, responsibility and authority for various posts, reporting procedures etc. Thus, the documentation describing the approach to the Rock Mechanics Site Descriptive Model is the product of the work.

2.2.17 Lessons learnt from the development work

Considerable experience was gained in the application of the Quality Control measures during the 2001 development of the approach to the Rock Mechanics Site Descriptive Model. All the measures were required for the development phase. During the site investigations and the application of the approach, it will be necessary to continue using most of these Quality Control items to ensure continuous updating of the approach. This is described in more detail in Chapter 7.

2.3 Method of Co-ordinating the Conclusions and Presenting the Information

The Quality Control information presented in Section 2.2 and the Test Case activities described in the Appendix provide the basis for co-ordinating the conclusions. The Test Case experience was particularly successful in firming up the conclusions.

The information resulting from the development of the approach to the Rock Mechanics Site Descriptive Model is being presented in a series of reports, of which this is the 'top document'. There will be six reports, as indicated in Figure 2-4.

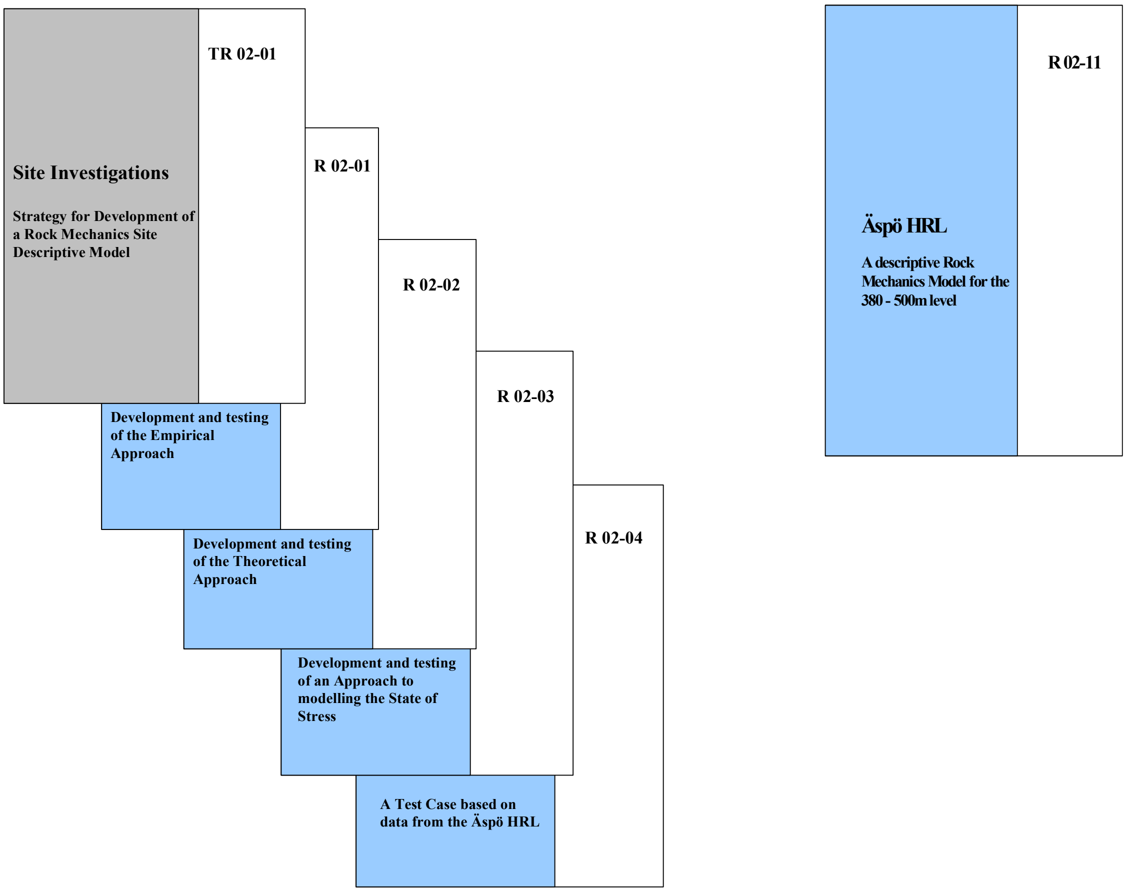

Figure 2-4. *Documentation describing development of the approach to the Rock Mechanics Site Descriptive Model.*

3 Strategy for Estimating Mechanical Properties

This Chapter concerns methods for estimating spatial distribution of intact rock strength and mechanical properties of the rock mass in 3-D. This involves extrapolating measurements made in a few boreholes throughout the rock mass volume of interest. Because of the large scale, rock mass properties cannot be measured directly. Empirical relations and numerical simulations may be used for the estimation, but both approaches contain significant uncertainties. Final selection of parameters for the Site Descriptive Model involves evaluation of the results from different approaches combined with judgement.

3.1 Introduction

As discussed in Chapter 1 (Section 1.2), the focus of the modelling for the approach to the Rock Mechanics Site Descriptive Model is to determine the three dimensional distribution of the rock mass deformation modulus and the rock mass strength at a 'tunnel scale', thus enabling the mechanical responses to a repository tunnel and the vertical deposition holes to be evaluated, Figure 1-1 (i.e. at a scale of the order of 30 m). These are the basic rock mechanics properties of the rock required for initial design. In addition, small-scale properties, the intact rock strength in particular, are important. While the intact rock strength can be readily determined by tests on borehole cores, there remains a prediction problem with regard to its distribution in space. Furthermore, as noted in Chapter 1, the identified rock mechanics modelling requirements may need updating when the site descriptions are first used in design and safety assessment. These aspects are further discussed in Chapter 7. It should also be noted that the rock property estimations only require limited accuracy. As explained in Chapter 1, uncertainties in the order of 15% are certainly acceptable.

Because the rock mechanics properties of the rock mass are difficult to measure directly, rock mass properties have to be assessed by other means. Empirical relations and numerical simulations may be used for this purpose, but both approaches contain significant uncertainties. Before applying these relations or simulations, the 3-D distribution of the input parameters must be determined as well.

3.1.1 Definitions

Rock mass deformation modulus

The deformation modulus of the rock mass, E_m , is defined as the ratio of the axial stress change to axial strain change in the rock mass induced by a normal stress change. The rock mass strain is the combination of deformation, both elastic and inelastic, of the intact rock and fractures in the strained rock volume. Fractures typically form an inhomogeneous and isotropic network, and so the deformation modulus should be expected to vary with location and with direction of loading.

Rock mass strength

In geotechnical analyses, a failure criterion is used to describe the rock mass strength. Two well known failure criteria are /Hoek-Brown (H-B), 1977/ and Mohr-Coulomb (M-C).

The /Hoek and Brown, 1977/ failure criterion relates the maximum and minimum principal stresses at failure via the following formula:

$$\sigma_1 = \sigma_3 + \sigma_{ci} (m_b \cdot \sigma_3 / \sigma_{ci} + s)^a \qquad (3\text{-}1)$$

where σ_{ci} is the uniaxial compressive strength of the intact rock and m_b, s and a are rock mass dependent properties. In the generalised Hoek-Brown failure criterion for jointed rock masses as defined by equation 3-1, the values of the model constants should be determined by analysis of sets of stresses (σ_1 and σ_3) at failure. The range of minor principal stress values, σ_3, over which these combinations of stresses are given is critical in determining reliable values for the constants.

The Mohr-Coulomb criterion can be expressed as

$$\sigma_1 = \sigma_3 \cdot (1 + \sin \phi) / (1 - \sin \phi) + 2 \cdot c \cdot \cos \phi / (1 - \sin \phi) \qquad (3\text{-}2)$$

where c (cohesion) and ϕ (friction angle) are rock mass strength dependent properties. Most geotechnical software is written in terms of the Mohr-Coulomb failure criterion (equation 3-2). The linear relation between the major and minor principal stresses, respectively σ_1 and σ_3, for the Mohr-Coulomb criterion is illustrated in equation 3-2. By setting $\sigma_3 = 0$ in equations 3-1 and 3-2, one obtains σ_1 for failure at zero confining pressure, i.e. σ_{cm} the rock mass uniaxial compressive strength. This measure is important for judging tunnel stability because σ_3 is close to zero at unsupported excavation surfaces.

The uniaxial strength obtained from the H-B failure criterion can be quite different to that obtained from the M-C failure criterion, see e.g. Figure 3-1, even if the failure envelopes are similar over other ranges of confining stress. The rock mass strength is discussed by, for example, /Martin et al, 2001/.

Intact rock uniaxial compressive strength

The compressive strength is determined by uniaxial tests on drill cores. This strength can also be formulated through the Hoek-Brown failure criterion using equation 3-1, by setting a=0.

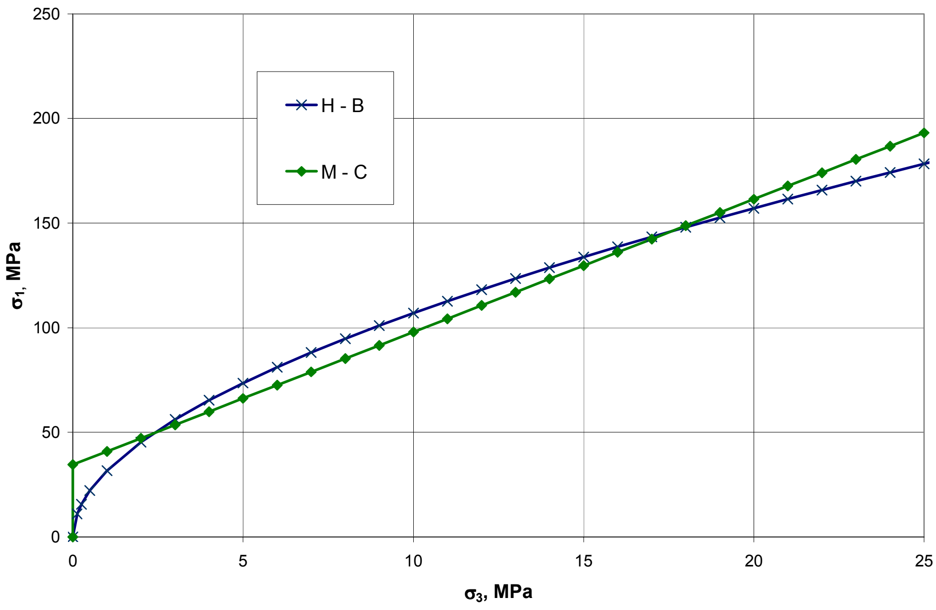

Figure 3-1. *Examples of the M-C and H-B strength envelopes.*

3.1.2 Empirical classification methods

One possible approach for determining rock mass properties is to use empirical relations based on different rock mass classification systems. These schemes are developed based on case studies. The most commonly used systems are the Rock Mass Rating (RMR) system, /Bieniawski, 1989/ and the Tunnelling Quality Index (Q) system, developed by /Barton et al, 1974/. Section 3.5 discusses how these classification values may be used for estimating rock mass deformation modulus and rock mass strength. There are also other rock mass classification systems such as the RMS /Stille et al, 1982/, the Geological Strength Index (GSI), Rock Mass Index (RMi), and the Ramamurthy formula, as discussed by /Röshoff et al, 2002/.

The empirical relations are based on experience, and are not derived directly from basic mechanics. For this reason, the validity of the empirical relations can only be assumed for circumstances similar to those from which they were originally developed. Despite this, rock mass classification systems have been widely and successfully applied in civil engineering for the design of underground excavations, especially tunnels, over a long period of time. However, the experience from mines and hydropower tunnels constructed at the anticipated repository depths, i.e. of the order of 500 m in Fennoscandian type crystalline rock, is more limited.

Nevertheless, given the impracticality of direct measurement and the difficulties with theoretical modelling, classification schemes are an important means of characterising the mechanical properties of the rock mass. Thus, it is judged meaningful to describe the rock mass in terms of the spatial distribution of Q and RMR and to use them for estimating rock mechanics properties. Given the uncertainties, it is also clear that alternative means of estimation are additionally required.

The Q system

The Q-classification system developed by /Barton et al, 1974/ is given by the relation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (3\text{-}3)$$

where the parameters are:

RQD (0–100%) – rock quality designation;

J_n (0.5–20) – joint set number;

J_r (0.5–4, or 0.5–5 when joint set spacing is >3 m) – joint roughness number;

J_a (0.75–20) – joint alteration number (related to friction angle);

J_w (0.05–1) – joint water reduction number;

SRF (0.5–400) – stress reduction factor.

The Q-value ranges typically from extremes of about 0.001 to 1000.

The six Q-parameters developed by /Barton et al, 1974/ were the end product of six months of repeated trial-and-error fitting of different ratings, to match 212 case records of tunnel and cavern support needs, resulting in the final Q-value. /Grimstad and Barton, 1993/ published an updated set of support recommendations based on 1050 new case records. Three changes to the strength/stress SRF ratings for massive, highly stressed rock were made. Later studies (e.g. /Barton, 2000/ or /Barton, 2002/) have led to further understanding of how to apply Q.

Barton (pers. comm.) notes that the application of high SRF ranges are only for stress-slabbing and rock burst situations but, because of the increasing use of rock mass classification methods for estimating input data for modelling, various SRF recommendations have arisen (including /SKB, 2001/), e.g. putting SRF = 1, and J_w = 1 when using Q′, as opposed to Q, for characterisation (in the absence of excavation effects).

The argument that 'stress and water pressure are boundary conditions' is a logical recommendation applied in numerical modelling of underground openings. However, especially with the Q-system, the recommendation of a 'dry and stress-less' Q′ value (assuming SRF = 1, and J_w = 1), to be used with a separate effective stress 'correction', introduces pitfalls. One is that the presence of water (with or without pressure) may have a negative effect on stability, e.g. when clay-coatings or clay-fillings are present. Part of the Jw rating (the last parameter added to Q) was specifically needed to correct for this. The second pitfall is that the SRF term tries to 'correct' for several 'rock mass fragmentation' effects. The presence of weakness zones (shear zones or faults), crossing or close enough to influence the excavations, is one of these. Setting SRF = 1 in such circumstances is inappropriate.

/Barton, 2002/ makes the following recommendation as an alternative to Q´:

- ***J_w***. For general characterisation of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33 etc as depth increases from say 0–5 m, 5–25 m, 25–250 m to > 250 m is recommended, assuming that RQD/Jn is low enough (e.g. 0.5 to 25) for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterisation values of SRF. Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

- ***SRF***. For general characterisation of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0, and 0.5 is recommended as depth increases from say 0–5 m, 5–25 m, 25–250 m to > 250 m. This will help to adjust Q for some of the effective stress influences, in combination with appropriate characterisation values of J_w . Correlations with depth-dependent static deformation modulus and seismic velocity will then follow the practice used when these were developed.

The RMR system

The RMR rating system by /Bieniawski, 1989/ can be expressed as the sum of ten factors:

$$RMR = RMR_{\substack{strength \\ of\ intact\ rock}} + RMR_{RQD} + RMR_{\substack{fracture \\ spacing}} + RMR_{\substack{fracture \\ length}} + RMR_{\substack{fracture \\ weathering}}$$
$$+ RMR_{\substack{fracture \\ aperture}} + RMR_{\substack{fracture \\ roughness}} + RMR_{\substack{fracture \\ infilling}} + RMR_{water} + RMR_{\substack{joint \\ orientation}} \quad (3\text{-}4)$$

The five “fracture terms” relating to length, weathering, aperture, roughness and infilling are usually combined into a single term in most RMR formulations).

This index is generally regarded as varying between 0 and 100, but wider ranges are theoretically possible.

The RMR system has been developed for tunnel design where specific ranges (classes) of the indices correspond to different potential problems for tunnel construction. The RMR ratings for general characterisation should not, therefore, consider water inflow or fracture orientations in relation to a tunnel. Thus, when RMR is used for characterisation, it is suggested to set RMR_{water} = 15 (the maximum value) and $RMR_{joint\ orientation}$ = 0, /SKB, 2001/.

Relation between Q and RMR values

The two classification systems use some of the same parameters, but there are some important differences. A principal difference is that RMR uses the uniaxial compression strength of the rock as a basis for one of the parameters, whereas the Q index uses an indirect stress-strength relation, the SRF factor. For design, RMR uses a parameter for the orientation of fractures related to the tunnel direction, while in the Q-system the J_r and J_a values are used for the fracture set considered as most unfavourable for the stability of the tunnel.

3.1.3 The theoretical approach – calculating rock mass properties

An alternative to the empirical relations would be to calculate the rock mass properties from known properties of the components of the rock mass. A general rock mechanics model should consider the

- the mechanical behaviour of intact rock,
- the mechanical behaviour of fractures, and
- the geometry of fractures.

However, such calculations are not trivial – due to the complex geometry and the non-linear and spatially varying small-scale properties of rock. For the Test Case work, /Staub et al, 2002/ attempted such an analysis by stochastic numerical simulations. It is evident that the method would require additional improvement, but the approach is nevertheless an important complement to the empirical relations discussed above. The approach is further discussed in Section 3.6.

3.1.4 Need for judgement

Given the uncertainties and difficulties with the different suggested techniques, there is a need for a combined evaluation of all results and other potentially available evidence relating to the rock mass properties. This is discussed in Section 3.7.

3.2 Identification and evaluation of input data

Defining the input data is an essential component of the modelling strategy. In developing the Test Case, a special Protocol (Protocol 4A) was designed for this purpose (see Chapter 2 and the Appendix). After establishing the available data, the next step in the modelling is conversion of these data to a format suitable for the subsequent three-dimensional modelling. The current section builds on the ideas developed within the Test Case project but is adapted to the conditions likely to occur during the Site Investigations.

3.2.1 Overview of available data

The general programme /SKB, 2001, TR-01-29/ provides an overview of when different data and model versions will be available during the site investigation. Table 3-1 lists data potentially available for the rock mechanics modelling. Such data comprise both

- processed data (i.e. models) produced in other aspects of the Site Descriptive Model (mainly the geological model) and
- primary data from measurements of the geometrical and mechanical properties of the intact rock.

The Site Descriptive Model comprises descriptions of geology, hydrogeology, rock mechanics, hydrogeochemistry, transport properties and ecosystems. These descriptions are developed jointly and iteratively during the different stages of the site investigation (see Chapter 6 and /SKB, 2001/) and care must be taken not to unnecessarily duplicate efforts and to provide mutually consistent descriptions. For any given iteration (version) of the Descriptive Model, the Geological Model is updated first. The geological, and to a minor extent the hydrogeological, descriptions contain information of direct consequence for the mechanics modelling. The primary data ('measurements') usually need further evaluation before they can be used for predicting rock mass properties in the three-dimensional Site Descriptive Model.

Table 3-1. Data and descriptive models used as input for the mechanical property model.

Type of data	Source	Description
Geological description	Site Descriptive Model produced jointly with Rock Mechanics Model	Fracture zones and rock domains with rock type distribution and fracture statistics. Geological evolution
Hydrogeological description	Site Descriptive Model produced jointly with Rock Mechanics Model	Distribution of permeability, groundwater flow and pressure distribution
Stress distribution	Model estimation	see chapter 4
Mechanical properties of intact rock and fractures	Measured on some borehole cores and on fractures in the cores	Laboratory tests for determining strength, elasticity, E-modulus, Poisson's ratio, seismic velocity. Laboratory tests for normal and shear stiffness and strength
Core logging information	Measured along boreholes	Fracture, fracture frequency, fracture properties etc along boreholes, direct RMR or Q logging
Surface information	Sampled at outcrops etc	Sampling fracture statistics (frequency, size, properties at the surface)…
Seismic information	Seismic profiles in some bore holes	Primarily seismic tests providing seismic velocity
Experiences from underground construction	Collection of experiences	Records and experiences from underground construction in the vicinity of the site or in similar rock types and at similar depths

3.2.2 Geological description

The geological modelling is outlined in /SKB, 2001/ and further described by /Munier and Hermanson, 2001/. The geological description component of the Site Descriptive Model covers the geometry of regional and local major fracture zones, as well as the geometry of other rock units. The geological model is visualised in the RVS-system. However, the RVS-representation is just one aspect of the model: it also needs supporting documentation and justification.

Fracture zones and fractures

The crystalline rock mass contains deformation zones on a wide range of scales, from micro-cracks in the 'intact rock', individual visible joints, to regional fault zones. According to SKB nomenclature, see e.g. /SKB, 2000/, all deformation zones with essentially brittle deformation history are called 'fracture zones'. However, the Geological Site Descriptive Model only explicitly (deterministically) describes the fracture zones with a size larger than 1 km. Such zones are called 'regional zones' and 'local major zones'. The remaining zones are described statistically within each rock unit, see next paragraph.

Fractures not described as deterministic geometric features are described statistically. The statistical description is based on discrete fracture network terminology and typically comprises the following.

- Fracture orientation distribution for each fracture set, usually inferred from orientation measurements along scanlines (including boreholes) properly corrected for orientation sampling bias, e.g. /Terzhaghi, 1965/.
- Spatial distribution of fractures, i.e. random (Poisson distribution) or models implying statistical dependence among fractures (usually inferred from fracture positions/spacing measurements along sampling lines or from two-dimensional rock exposure mapping).
- Fracture size distribution. Fracture size is also usually inferred from fracture positions/spacing measurements along sampling lines or from two-dimensional rock exposure mapping. Fracture zone sizes are inferred from lineament maps. The size distribution should capture all fracture zone sizes ranging from lineaments not included in the deterministic model down to individual fractures. A lower size cut-off is needed, but the allowed cut-off depends on the use of the information and may be different for different applications (hydrogeology or rock mechanics).
- Volumetric fracture intensity. The fracture intensity, P_{32}, is the ratio between total area of all fractures to the total volume inside which the fractures are generated. P_{32} is linearly correlated to fracture frequency along a borehole or on the rock exposure. The proportionality constant depends upon fracture orientation and distribution of the fracture size. It is a much better measure than fracture frequency (which is orientation dependent) or fracture centre density (which depends on fracture sizes).

The statistical fracture parameters (the DFN-model) can be used to produce realisations of fracture networks using software such as FracMan or NAPSAC, but the DFN-model is the statistical input, not the individual realisations themselves.

Rock domains

The geometrical distribution of rock properties and fracturing is described using the concepts of *rock units* and *rock domains*. A rock unit is a volume judged to have a reasonably statistically homogeneous distribution of lithology and fracturing statistics. (Fracture zones are special cases of rock units). A rock unit may contain several different rock types judged to be similar. A rock unit may also contain small scale inclusions of very different rock types. Each rock unit is defined by its location and is described in terms of rock type distribution and fracture and fracture zone statistics. In addition, several rock units, e.g. those just separated by different fractures zones, may have similar properties. This information is also handled by logical connections in the geological model, where several rock units are assembled into *rock domains*. A rock domain is a region of the rock mass for which the properties can be considered essentially the same in a statistical sense. In fact, experience suggests that the rock mechanics modeller should perhaps pay more attention to the rock domains than to individual rock units, especially since the rock domains are defined by a rock mechanics objective, rather than by geology alone, /Hudson and Harrison, 2002/.

It is likely that early versions of the geological description will contain few fracture zones and a limited number of rock domains; whereas variability within rock domains will be quite large. Later versions of the model, based on more data, will potentially lead to a further subdivision of the model volume into more domains (with more homogeneous properties).

Geological Evolution

The geological description also includes a description of geological historical evolution. This evolution is crucial for the understanding of the site and in particular for allowing the geologist to make statements on the size of rock domains or the potential historical deformation of major fracture zones.

3.2.3 Hydrogeological description

The hydrogeological description primarily contains information on the permeability distribution at various scales (see /SKB, 2001/) for the rock units. The distribution builds on the geological description of the rock domains and associated rock units, but the hydrogeological evaluation of data may lead to further divisions into different units, or that geologically distinct units are combined into hydraulic domains with the same (statistically) hydraulic properties. The hydraulic description may in turn be used for simulating groundwater flow and groundwater pressure distribution.

The information is potentially important, both as regards design issues (e.g. $J_w \neq 1$, see Section 3.1.2) and for considering coupled hydrological-mechanical processes. Information on the degree of open, water-bearing fractures may assist in judgements on fracture mechanics properties in both the empirical and theoretical approaches (see Sections 3.5 and 3.6 respectively). Local minor hydraulic conductive zones may also guide in interpretating the normal stress magnitudes across such features.

3.2.4 Properties of intact rock and fractures – laboratory data

During the site investigation, direct mechanical tests will be carried out on borehole cores of intact rock and on fractures found in borehole cores. The planned tests are outlined in the SKB investigation methods and execution programme /SKB, 2001/.

Laboratory tests are used for determining strength, E-modulus, Poisson's ratio, seismic velocity of intact rock and for determining shear stiffness and shear strength of individual fractures. Only approved methods, such as for example the Suggested Methods published by the ISRM and ASTM standards, are considered.

3.2.5 Core logging data

Several data samples are taken along the boreholes, see /SKB, 2001/, including the various geophysical logs, fracture frequency logs and geological core logs. These data are mainly used for developing the Geological Description, but some are also used for rock mechanics interpretation of single holes, see Section 3.3.

3.2.6 Surface data

There will be few, if any, direct rock mechanics measurements made on samples taken from the surface, but the surface information is essential for the construction of the geological model of the site. Surface mapping, topography and various geophysical methods are important inputs to the geological model. The primary data are interpreted to lineament maps, rock type maps, and fracture and lineament statistics before being used in the Geological Model. Clearly, the Rock Mechanics Model should not re-interpret these data, but instead use the already processed information.

It is also necessary to bear in mind that the surface information may or may not indicate the properties at depth. Some words of caution are required. Surface mapping is the only source for fracture size distribution during the site investigation phase. Fracture data sampled from boreholes provide fracture intersections and directions, but cannot provide estimates of fracture extent. Lacking fracture size data at depth, the only solution is to use the size distribution obtained from the surface, but the fracture frequency and directions measured in the boreholes can still be used to make the fracture intensity and orientations depth dependent. There are several judgmental aspects in producing a fracture network model of the site. This means that the rock mechanics modeller should interact with the geological fracture modeller in order to make sure that the fracture network model reflects the properties and trends of importance for the Rock Mechanics Model. See also Chapter 6 in this connection.

3.2.7 Seismic data

Seismic measurements in the rock mass may be used to estimate seismic velocities (v_p – compressional wave velocity and v_s – shear wave velocity) through the rock mass. Seismic measurements may offer another way of estimating the dynamic rock mass deformation modulus and dynamic Poisson's ratio, see Section 3.5.2. It should be noted that the dynamic modulus is not expected to have the same value as the 'static' modulus.

3.2.8 Experiences from underground construction

A part of the preliminary assessment phase of the site investigation is assembling records of experiences from underground construction in the vicinity of the sites. Such information needs to be considered with caution because the facilities will usually be at different depths, and the similarity of the rock conditions at the investigated site is not easy to demonstrate. Nevertheless, past experience is a valuable tool for evaluation of the modelling results, as discussed in Section 3.7.2.

3.3 Single hole interpretations of mechanical indices

A logical step in evaluating the distribution of mechanical indices such as Q and RMR in the three-dimensional rock mass is to determine the variation of these indices along the available boreholes. The Q and RMR systems were primarily developed for tunnel mapping and some of the parameters needed are not available boreholes, or are uncertain in the boreholes. These parameters must be assessed from other sources.

For the Test Case work, /Röshoff et al, 2002/ partitioned each borehole into a number of core sections with homogeneous RQD and fracture frequency values. For each core section, three data processing sheets were developed as follows.

- Input data form. This sheet contains all basic information for parameterisation of both Q and RMR systems.
- Data remark form. This sheet contains mainly information and file sources and comments on the fracture conditions.
- Data processing form. This sheet is the parameterisation form for both Q and RMR.

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3.3.1 Estimating Q from borehole parameters

To some extent, Q can be estimated directly from the primary borehole data in the SKB SICADA database. These data are routinely sampled in each borehole during the site investigations. The following scheme for Q-estimates along the borehole was developed as part of the empirical approach /Röshoff et al, 2002/ to the Test Case. Some modifications are introduced here as a result of subsequent discussions within the project.

RQD

The RQD is defined as the percentage length of the pieces of intact core longer than 10 cm for a specific core length. If spacing is distributed according to a negative exponential distribution, $RQD=100e^{-0.1\lambda}(0.1\lambda+1)$, indicating that RQD can be calculated from fracture frequency, /Harrison and Hudson, 2000/. As spacing is directionally dependent, this means that RQD and also Q and RMR are directionally dependent.

Information on fracture frequency is obtainable from the SKB SICADA Database, but care is needed when using the data since there are different types of fractures in SICADA, and sections designated as crushed rock usually do not have fracture frequencies assigned to them. In analysing the frequency data, /Röshoff et al, 2002/ divided the borehole into different sections where RQD was judged to be fairly similar. An alternative would be to take RQD-values from the fracture statistics interpreted for the geological model (the DFN-data). This would ensure consistency and proper attention to biases etc. Clearly, the rock mechanics modeller needs to make sure that this input makes proper allowance for potential depth changes.

J_n (Joint set number)

The number of fracture sets can be estimated from statistics of fracture orientations along the borehole. However, estimating fracture orientations along a single hole involves directional biases and the question also arises regarding the length of borehole over which to conduct the averaging. Judgement and assumptions are needed. For each borehole, /Röshoff et al, 2002/ calculated J_n for each rock unit intersected by the borehole. As an alternative, J_n could be based on the fracture statistical description provided in the geological model analysis (the DFN-data). This may ensure better consistency and proper attention to biases etc.

J_r (Joint roughness number)

The J_r values concern the effect of fracture roughness. There are no logs of fracture roughness for all fractures encountered in a borehole and a mix of information sources would be needed. /Röshoff et al, 2002/ based their assessment on three sources: JRC values determined by laboratory shear tests, borehole logging information and direct site observations by the team at the Äspö HRL site.

J_a (Joint alteration number)

The J_a parameter concerns the conditions of the fracture surfaces, mainly coating, infilling, shear history and the residual friction angle. With the shear history largely unknown, /Röshoff et al, 2002/ determined J_a using mainly the residual friction angle determined from laboratory tests of fractures (a few samples) and coating/infilling conditions from borehole logging records. Only the latter gives the full distribution along the borehole. Data concerning the difference in strength between the fracture surface, which was tilt tested, and the intact rock of the samples were also used.

J_w (Joint water reduction factor)

For general characterisation of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33 etc as depth increases from say 0–5 m, 5–25 m, 25–250 m to > 250 m is recommended /Barton, 2002/, assuming that RQD/J_n is low enough (e.g. 0.5 to 25) for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effects, in combination with appropriate characterisation values of SRF. (see also Section 3.1.2).

SRF (Stress reduction factor)

The SRF concerns the effect of stress. For general characterisation of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0, and 0.5 is recommended /Barton, 2002/ as depth increases from say 0–5 m, 5–25 m, 25–250 m to > 250 m. This will help to adjust Q for some of the effective stress effect, in combination with appropriate characterisation values of J_w. (See also Section 3.1.2).

3.3.2 Estimating RMR along a borehole

The parameterisation for the RMR-system is similar in technique to that for the Q-system.

Rock strength for RMR

The rock strength rating for each core section is determined using the uniaxial compressive strength data from different sources. A few core samples from the borehole may have been tested in the laboratory, but these tests will not cover the entire length of the hole. The core log provides a qualitative estimate of the strength test results. Additional data may be taken from generic sources, i.e. data from measurements made on 'similar rock', but this increases the uncertainty.

RQD for RMR

The same RQD values used for the Q-ratings are used, see above. This means that RMR is directionally dependent. The rating is calculated using Chart A in /Bieniawski, 1989/.

Fracture spacings for RMR

The fracture spacing is also used directly in RMR. The mean spacing of the fractures is calculated, with the reciprocal being the mean frequency, for each core section. An alternative would be to calculate the spacing from the fracture statistical description provided in the geological model analysis (the DFN-data). This would ensure consistency and proper attention to biases etc provided the derivation of this model considered potential depth changes of the fracture statistics. The rating is determined using Chart B in /Bieniawski, 1989/.

Fracture length for RMR

The fracture extent, considered as a length, is not used in Q, but is required in the RMR ratings. Length data are not available in the borehole – these need to be taken from other sources. For the site investigation, this information is given by the size information in the DFN-model to be developed as part of the geological description. (This model will be based on surface exposures.)

Fracture aperture for RMR

It should be possible to obtain some estimate of fracture aperture from the BIPs logs (see /SKB, 2001/). However, this is often difficult: the aperture varies considerably and can be affected by the presence of the borehole. For the RMR rating, aperture may be divided into three classes: very tight fractures with aperture 0–0.1 mm, tight fractures with aperture 0.1–0.5 mm, and moderately open fractures with aperture 0.5–1 mm. These values can be used to determine the rating using Chart E in /Bieniawski, 1989/.

Fracture roughness for RMR

The same fracture roughness estimation that is used for the Q ratings is used for the RMR system, with the most representative category being ‘slightly rough’ surface. The RMR rating is determined according to Chart E in /Bieniawski, 1989/.

Fracture infilling for RMR

The cores are logged for fracture infilling. The RMR rating for fracture infilling has five classes (no infilling, hard infilling of < 5 mm thickness, hard infilling of > 5mm thickness, soft infilling of < 5 mm thickness and soft infilling of > 5mm thickness). The RMR rating is determined according to Chart E in /Bieniawski, 1989/.

Fracture weathering for RMR

The cores are logged for fracture weathering. The RMR rating is determined according to Chart E in /Bieniawski, 1989/.

Groundwater for RMR

The RMR ratings for the characterisation purpose should not consider water inflow. Thus, the suggested setting is RMR_{water}=15 (the maximum value) when RMR is used for characterisation (see Section 3.1.3). The rating for design is determined using either inflow data or the ratio of fracture water pressure to the major principal stress.

Fracture orientation for RMR

The RMR ratings for characterisation also should not consider fracture orientations in relation to the tunnel. It is thus suggested to set $RMR_{joint\ orientation} = 0$ when RMR is used for characterisation (see Section 3.1.3). For design, the RMR rating should consider the proposed tunnel orientations.

3.3.3 Remarks

Direct estimates of Q or RMR along cores

The Q or RMR distribution along a borehole may also be directly estimated by specially devised core mapping. In the Test Case, some of the boreholes were logged in this way /Makurat et al, 2002/. The basic problems experienced in direct Q-logging, directional bias etc, should be the same as when calculating Q from more basic logs, but judgement and experience may still cause differences. A potential disadvantage with the direct logging is that estimates of directions or RQD that are too judgmental. For example, it is recognised that logs have to be made over core sections of several metres in length, because one or two fracture sets might be missing in shorter sections. A potential advantage could be that an experienced interpreter may make an overall judgement of the rock 'quality', thereby assigning reasonable Q values, regardless of the results of arithmetic operations, but this potential advantage needs to be balanced against the obvious risks of subjective bias being introduced.

For the Test Case, /Röshoff et al, 2002/ compared the direct characterisation of Q and RMR for the three boreholes with their calculated distributions. It was found that the Q value is generally higher for the ratings made by /Makurat et al, 2002/ compared to ratings by /Röshoff et al, 2002/. However, both ratings range within the same rating class of Q=10–40. It is suggested that the differences mainly depend on different values used for J_n (fracture set number) and J_r (joint roughness number).

The RMR ratings have a difference of 15–20 points, but the rating is lower for results by /Röshoff et al, 2002/. This difference is due to the fact that Röshoff has used rating values for $RMR_{water} < 15$ and $RMR_{joint\ orientation} = -5$. /Makurat et al, 2002/ used the values RMR_{water}=15 and $RMR_{joint\ orientation}$=0, i.e. the values suggested for characterisation, see Section 3.1.2. If these ratings are changed, the resulting ratings are almost equal for the two approaches.

Consequently, direct Q or RMR-characterisation of borehole cores may not be necessary.

Relation between Q and RMR

/Makurat et al, 2002/ conclude that it is difficult to decide on the best general conversion equation between the two systems. Several attempts have been made in the literature to develop such an equation, even though different sets of parameters are used in the two systems. A certain conversion equation may only be valid for a special rock type or for a limited range of Q or RMR values. Therefore, a single conversion equation cannot be used indiscriminately, /Harrison and Hudson, 2000/.

/Röshoff et al, 2002/ compared their RMR and Q estimates for the Test Case and found that the relation between Q and RMR derived for the Äspö Test Case, and for design purposes, closely resembles the published ones. On the other hand, the same relations seem to underestimate the RMR determined for the Äspö Test Case as a function of Q, when the results concerning the characterisation of the site are considered. The relations apply only for the design versions of the classification indices considered in the present study, and different equations should be developed for the characterisation versions.

One could also note that high Q values often are related to high RMR values (e.g. for Q>10, RMR>70), which indicates that the selection of classification method (Q or RMR) is not important for high quality rock. However, high Q values being the result of high stress regimes (i.e. low SRF) may not be related to RMR in this fashion.

Conclusions

Classifying borehole sections for Q or RMR should be undertaken based on the techniques described in this Section. However, there are uncertainties involved in this undertaking. Apart from the questions concerning the validity of empirical relations, which are further discussed in Section 3.5.4, the following can be noted (partly based on issues identified by /Röshoff et al, 2002/).

- Rock mass characterisation based on core samples is sensitive to the technique adopted for isolating homogeneous sections of borehole for which the characterisation is performed, and also the borehole orientation relative to the dominant fracture sets. Depending on the geological parameter used for identifying homogeneous sections, the length of the sections can vary markedly. Moreover, the choice of the ranges of variation of each parameter that identifies homogeneous sections has strong effects on the length. In principle, long borehole sections tend to smooth out the local variations, so that the characterisation gives averaged results. On the other hand, some parameters are scale dependent (fracture density) so that the results indicate different statistics when changing the length of the analysed sections.

- The wealth of information (and its quality) may differ between different boreholes and in different parts of the boreholes. Any lack of proper documentation of the location of the measurements for mechanical properties of intact rocks and fractures makes the rating and estimation of spatial variability of the properties difficult.

- Selection of Q factors or RMR terms has elements of arbitrariness to it, even when the rock properties are reasonably well known.

- Several of the parameters in Q or RMR concern fracture geometry. It should be recognised that a model of the fracture geometry is captured in the DFN-model produced as a part of the geological description. It builds on fracture trace mapping on the surface and on fracture logging in boreholes, taking size and directional biases into account. Consequently, this information should be taken from the DFN-model, but the rock mechanics modeller needs to make sure that the geological modelling produces DFN-models adapted to the needs of the mechanics modelling.

- Q and RMR are directionally dependent and may thus vary depending on the borehole direction. This needs to be considered in characterisation, where the direction of characterising boreholes may not coincide with the direction of future tunnels. The degree of directional dependency should be explored, e.g. by exploring the directional dependence of RQD and fracture spacing given by the fracture statistical description in the DFN-model.

When making Q or RMR classifications of the boreholes, these and other uncertainties should be considered and explored. The uncertainties will impact on the overall uncertainty evaluation of the empirical modelling, see e.g. Section 3.5.4. Some of these problems may be resolved by further development efforts – but not all. The classification methods have their inherent problems, /Harrison and Hudson, 2000/, and there are fundamental problems in interpreting three-dimensional data along a borehole.

3.4 Estimating the spatial distribution of parameters

The first assessment of rock mechanics data only concerns a limited portion of the rock volume to be characterised, and the rock mechanics data only exist at relatively few points. Estimation of Q or RMR, or other indices, concerns profiles along some boreholes. Based on the measured data, the next logical step is to make a 3-D distribution of Q, RMR, and the primary information, such as intact rock strength and the mechanical properties of the fractures. One should also note that these predictions of the spatial distribution of rock properties are modelling results (endpoints) in themselves, in addition to the need to provide them as input for deriving rock mass parameters.

3.4.1 General problem

The available site specific data will only cover a small portion of the volume to be characterised. This is the general case in rock engineering where it is always less than 1% of the rock mass that is being investigated. As noted in /SKB, 2001/, the main challenge is how to extrapolate information measured at the surface and in a few boreholes into a three-dimensional distribution within the model volume. There are always uncertainties in interpreting such measurements and rock parameters vary spatially. The three-dimensional description needs to evaluate the parameters, with their spatial variability, over a relevant scale and to describe the uncertainty in this description. The description should be able to characterise, describe and present data uncertainty, spatial variation and confidence (see Chapter 1).

Extrapolation or interpolation in space is based on a conceptual model of the spatial variability structure. Models for spatial variability include:

- designated volumes having a statistically homogeneous distribution of the parameter (rock domains),
- continuous changes, linear or non-linear trends, of a parameter (disseminated inhomogeneity),
- discontinuous variation, such as fractures (local inhomogeneity)

The cornerstone of the SKB strategy for this modelling is the geometrical division in the Geological Model of the rock into fracture zones and rock units, which in turn are grouped in rock domains. They may contain several different rock types, but with properties judged similar for the hydrogeological and rock mechanics applications. The rock domain division should thus be developed jointly among the geological, hydrogeological and rock mechanics modellers, see also Chapter 6.

Ideally, the rock domains should describe regions of statistical homogeneity of a parameter. If rock domains were truly of this character, the extrapolation problem would simply be to estimate the statistical distribution of the parameter in samples from the volume and then assign this distribution to the rock domain. There are, however, problems with this procedure:

- The geometrical description in the geological model is uncertain. Furthermore, small scale fracture zones are not evident in the geological model, other than as part of the statistical fracture zone distribution, but such zones may show important imprints on data along a borehole. These uncertainties will cause a general uncertainty in the position of different rock properties, and are also a difficulty when interpreting borehole data. The rock mechanics modelling should consider these uncertainties.

- Even if the rock units in a rock domain may be viewed as homogeneous from a geological perspective, they may not be homogeneous as regards mechanical properties. Stress dependent mechanical parameters, which vary with depth as stress varies with depth, are a clear example. The mechanics modeller can possibly handle this situation by further subdivision of the rock units. An alternative would be to keep the rock units, but introduce trends (stepwise, linear, non-linear) with depth. The rock mechanics modelling should address the inhomogeneity and spatial variability within the rock domains.

- Another problem is that different parameters may have different spatial distributions; rock domains established for one parameter may not be the same as those established for another parameter. This can be overcome by the use of a 'mapping function', as described in /Hudson and Harrison, 2002/, through which the parameters are considered together in the form of an index based on an equation, which could be RMR, Q or any function appropriate for the characterisation and design objectives.

- For extrapolations, it is necessary to characterise the (auto-) correlation structure. This structure varies between different parameters. Fractures manifest themselves in a wide range of scales, ranging from regional fracture zones down to small micro-fractures. Capturing the scale relations is an essential part of the statistical fracture modelling, see Section 3.2.2. The rock type distribution may, on the other hand, be seen more as a volume property where geostatistical tools, such as variograms, may be used for expressing the correlation.

- The amount of measurement data from different rock domains varies. Some domains may be well penetrated by boreholes; others will only have limited data; and some may be without any direct measurement. The inequality in the amount of data needs to be considered.

All these issues are indeed difficult, and are typical of site investigations for all underground excavations. There is no one correct way of handling them. The overall approach recommended in the general execution programme /SKB, 2001/ is to assess and quantify uncertainty and confidence after each investigation step. The size of the statistical spread and the difference between different interpretations based on the same measurement information indicate the degree of uncertainty. An indication of confidence in the description is the extent to which measurement results from later investigation steps confirm predictions made in earlier steps.

Important initial attempts at handling these issues were tried as part of the Test Case (see Chapter 2). These attempts, and some possible developments, will be described in the following subsections.

3.4.2 Assigning 'typical' distributions to individual rock domains

For the Test Case, /Röshoff et al, 2002/ developed means for characterising each rock unit in the Geological Model. The characterisation was based on the Q and RMR evaluation along the boreholes, see Section 3.3, and focused on rock units intersected by such boreholes, but it also dealt with rock units not being intersected by boreholes.

Rock domains intersected by boreholes

For rock domains intersected by boreholes, /Röshoff et al, 2002/ have developed a statistics analysis sheet for calculating the following.

- Arithmetic and weighted (against core length) mean values of ratings,
- Standard deviation of ratings,
- Maximum and minimum ratings,
- Mean, maximum and minimum values of parameters (RQD, etc),
- Histograms of all ratings and parameters.

These calculated properties may then be assigned to the rock domain.

Rock domains not intersected by boreholes

Some rock domains may not be intersected by boreholes. Some of these domains may be exposed on the surface, but one may also conceive a situation where there are regions in the geological model without any information at all. Röshoff et al, decided to deal with this information as follows:

- assigning surface interpretations of Q/RMR to domains where surface data were available;
- comparing 'known' domains which have partly surface data with domains with only surface data and thereby making a judgement about whether the domain would be similar – thus allowing use of the 'known' data for the domain without borehole measurement;
- inspecting the Geological Model to find rock domains without data, which may be modelled as similar to domains with 'known' data; and;
- leaving parameterisations as 'blank' for regions in the model where there was judged to be too little support for any prediction.

It is evident that this approach is judgmental. However, when there is a lack of data, some predictions are needed, even if the confidence in prediction may be low. Clearly, if further evaluation by the users of the model (i.e. for design) suggests that the confidence is too low, additional measurements may be contemplated in order to enhance confidence.

Uncertainty and confidence

The uncertainty in Q and RMR predictions in the different rock domains is assessed from the statistical distribution of the supporting data. Confidence in the prediction is based on the amount of data support. Also the following difficulties affect confidence and uncertainty.

- The rock domains in the Geological Model can consist of a mixture of rock types. From a mechanical standpoint, it may be required to distinguish between rock types not separated in the rock domain, and in the extreme to subdivide the domain into sub-domains, but this was not considered within the Test Case work.
- The rock domains and associated rock units in the Geological Model essentially concern lithology and fracturing, which means that stress dependent trends will not be captured within the unit. Such trends, if found to be needed, have to be introduced by the rock mechanics modeller. This can be done by the use of the mapping function mentioned in the second set of bullet points in Section 3.4.1
- The geometrical rock unit division is uncertain; this may result in inconsistencies between borehole data and the perceived model. For the Test Case, the validity of the Geometrical Model was not to be questioned, but this will not be true for the site investigation sites, where feedback from rock mechanics modelling into the geological modelling is essential, see Chapter 6.
- The distribution of different rock types and fracture properties can vary within a rock domain, and can vary among different boreholes intersecting the domain. This spatial variability should be properly described – and incorporated as updates of the domain division, if appropriate.
- The Q and RMR classification along the boreholes is uncertain, see Section 3.3.
- The Geological Model may contain information, such as the percentages of rock types in different types of rock units, but these distributions may not necessarily be sampled when analysing available borehole data. The question then arises whether one should use the overall Geological Model or whether predictions should be based on what was experienced in the borehole. For the Test Case, /Röshoff et al, 2002/ found examples where distinct large sections of different rock types, sometimes over 100 m in one direction, existed with different mechanical and fracture characteristics, in the same rock unit. However, such local differences are to be expected within a rock unit: the homogeneity of a unit concerns reasonable statistical homogeneity; it does not mean that there will be no variation at all.

These issues are further discussed in Section 3.4.4 below.

3.4.3 Visualisation and geostatistical analysis

A formal way of extrapolating point information into a volume is to make use of geostatistical theory and search for the correlation structure. This technique was not fully tested within the Test Case /Hudson, 2002/, but is nevertheless judged powerful if combined with modern visualisation software techniques and if sufficient data are available. Figure 3-2 shows the distribution of interpreted RMR along three boreholes within a rock unit in the Geological Model of the Test Case. This illustration provides directly an impression of good coverage of the borehole inside the rock unit and relative homogeneity, supporting the representativeness of the borehole information for this unit.

The information along the borehole could also be analysed geostatistically by calculating variograms and other measures. The variogram γ(h) of a property p(x), see e.g. /Journel and Huijbregts, 1978/ is expressed as:

$$\gamma(h) = \left(\frac{1}{2n}\right)\sum_{i=1}^{n}[p(x) - p(x+h)]^2 \qquad (3\text{-}5)$$

where γ(h) is the semi-variogram statistic for samples distance h apart, n is the number of sample pairs, p(x) is the rock property value at location x, and p(x+h) is the value at location x+h.

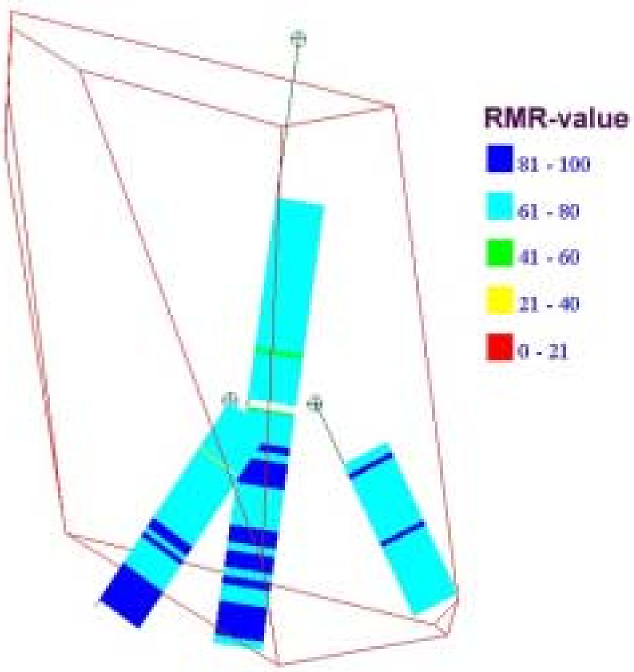


Figure 3-2. *Distribution of interpreted RMR along three boreholes inside a rock unit of the geological model.*

Figure 3-3 shows the variogram of RMR along the borehole. It exhibits a ‘nugget effect’, i.e. that there is a short range variability in the data, and a short range (20–30 m) correlation scale (autocorrelation). This means that RMR values in two adjacent blocks are correlated, but that the ‘statistical memory’ does not extend to further distances. Clearly, this variogram is only an example, and another resolution of the input data (e.g. a 10 m moving average instead of 1 m) may result in a different correlation structure. However, the variogram displays some typical properties of crystalline rock, that the ‘memory’ of a property is short-ranged, and that one is not necessarily constrained by values at a measuring point which is distant from the point. On the other hand, the level of variation is not unlimited. The variogram levels off into the overall statistics of the rock block at some distance.

The variogram can also be used for kriging interpolation, see /Journel and Huijbregts, 1978/, to the blocks in the target areas inside rock unit H. The kriging estimate can also be combined with estimates of the kriging variance, which is calculated from the variogram, see Figure 3-4. This diagram also illustrates that outside the correlation range the specific borehole information does not affect the prediction. There the kriging variance is the same as the ensemble variance. For the Test Case, both /Röshoff et al, 2002/ and /Staub et al, 2002/ reached similar conclusions as regards confidence in the results (i.e. only blocks directly connected to the boreholes were considered high confidence predictions). The geostatistical evidence illustrates that the implicit assumptions made by the teams regarding the correlation structure, of about 30 m, in fact are supported by the data.

The geostatistical analysis outlined here could be used for the values of any number of measured parameters, or any function of these, such as results from uniaxial testing as well as values based on empirical rating systems.

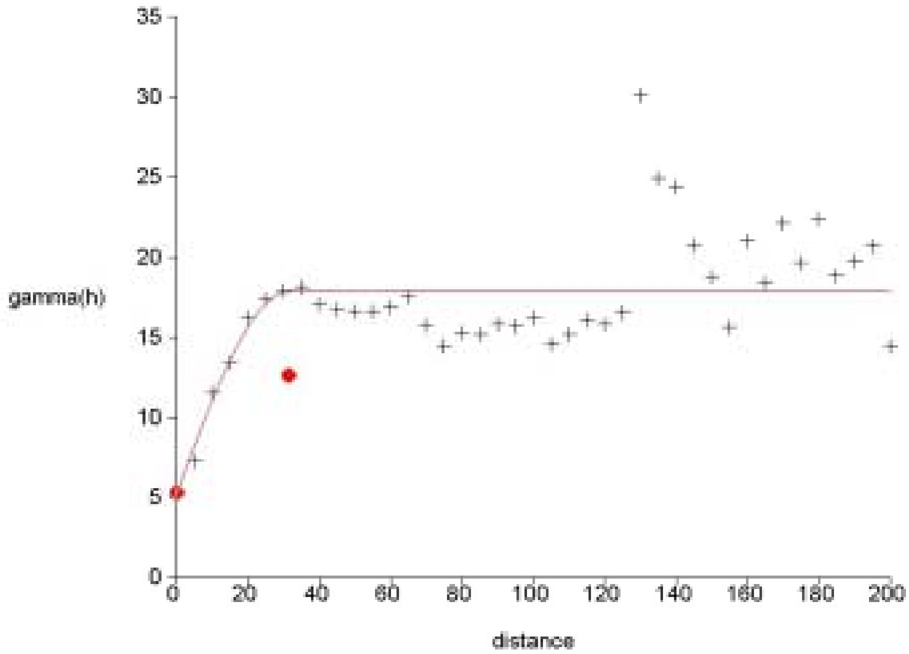


Figure 3-3. *Example of variogram of RMR along a borehol.e*

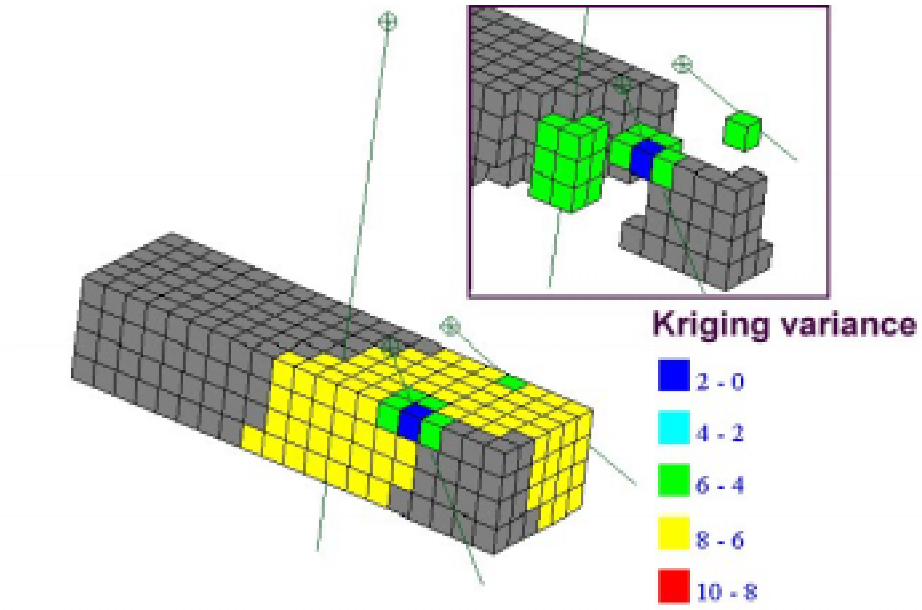


Figure 3-4. *Example of visualisation of kriging variance in block modelling.*

3.4.4 Conclusions

Extrapolation of the rock mechanics data into three-dimensions is a necessary step in the estimation of the mechanical properties of the Site Descriptive Model. The resulting three dimensional distributions of intact rock and fracture mechanical properties, as well as the estimated three dimensional distribution of classification indices such as Q or RMR are important results, regardless of the subsequent estimation of rock mass properties.

Given the limited mathematical correlation structure in rock data, other methods are needed to justify the extrapolation of information from few measurements into the full description of the different rock units in the model domain. Such information may be obtained from the Geological Model as it not only builds on strict data interpolation, but also on reasoning regarding the geological evolution, thus potentially justifying the statistical homogeneity of considerable volumes. For various reasons, partly due to incompleteness of the Geological Model, this approach was not fully adopted within the Test Case, but will probably be essential during Site Investigations.

The following specific conclusions were reached.

- Understanding (and evaluating) the geological model from a rock mechanics point of view is essential.

- Visualisation and geostatistical analyses should be made in support of the three-dimensional modelling. However, during site investigation, the distance between boreholes is likely to be much larger than the statistical correlation distances. Three-dimensional modelling needs to resort to other methods, by identifying a priori distributions for different rock domains. If later uncertainties are judged too high, additional boreholes at critical locations may provide information for updating the description locally.

- It is important to separate the fracture zones from the typical rock mass. It may not be practical or necessary to assign mechanical properties to the fracture zones from the limited information provided. However, it is important to identify where the fracture zones are located. On the other hand, for the rock units, rock mechanics characterisation is essential and good data support for the predictions is needed. The information on the fracture zones should already exist in the Geological Model. The rock mechanics modelling just needs to consider whether the zones in this model are reasonable (see also Chapter 6).

- The rock units in the Geological Model may contain minor fracture zones, not described explicitly. Considering the tunnel scale of modelling, the imprints of these zones should be filtered out from the distributions (indicated by bimodal distributions).

- The rock units and the rock domains are selected to represent volumes of statistically homogeneous lithology and fracturing. Even so, detailed logs along a borehole through such a unit may show spatial variation. The rock units will consist of mixtures of rock types and different samples taken from within a rock unit will never exactly match this mix. In the same way, different samples of fracturing will show different orientations and intensity. It is necessary to understand that differences between different samples may be treated as random variations within the overall distribution, rather than be used directly as a reason for further subdivision of the rock units.

- Even if different rock units are separated by fracture zones, there may be good geological arguments for assuming that some rock units should have similar geological properties, i.e. belong to the same rock domain. Such arguments can be based on the geological history combined with evidence from the surface. It will never be possible to penetrate all rock units with boreholes, but this does not mean that it is impossible to have well founded ideas on how different rock units form a single (or a few) rock domains.

- The Geological Model contains information concerning the distribution of rock types and fracture statistics (expressed as DFN-parameters), which may be readily interpreted for related factors and terms in Q or RMR. The rock mechanics modelling should use the existing data in the Geological Model instead of creating a new assessment of fracture statistics and lithology distributions.

- Rock mass mechanical properties are generally stress dependent. The rock units themselves are not suitable for representing stress variation, as stress is a separate and continuously distributed property to some extent independent of the rock characteristics (see Chapter 4). Stress dependency, which may be manifested as depth dependency, should not primarily be handled by sub-dividing rock units into more sub-units. It is a much better approach to describe the dependency directly and not via sub-unit partitioning.

- The rock mechanics modelling should consider whether further sub-divisions of rock domains are needed to make an adequate description. In such cases, the information needs to be co-ordinated with the geological modelling in order to ensure consistency and use of the proper tools (e.g. the CAD based Rock Visualisation System, RVS) and methodology, see Chapter 5. The potential benefit of further sub-division should be weighed against the acceptable uncertainty of the model (see Chapter 1) and the substantially increased uncertainty in the geometry of the sub-domains. It may in fact be better to retain few domains and then increase property uncertainty inside the domains.

It is essential that the three-dimensional rock mechanics model builds on the three-dimensional structure offered by the Geological Model; however, the confidence in the three-dimensional description then rests entirely on the confidence in the Geological Model. For this reason, the rock mechanics modeller needs to have an understanding of the confidence in the Geological Model and to interact with the geological modelling teams (see also Chapter 6).

3.5 Empirical methods for predicting rock mass properties

The empirical classification systems were primarily developed as a means of classifying a rock mass for tunnel design work. However, as discussed previously, the empirical indices, if properly adapted to the characterisation situation, have also been empirically related to the rock mass mechanical properties. Clearly, the validity of the relations can be questioned if used outside their respective case history databases, but the methods are still valuable, especially in an early phase as a systematic approach to describe mechanical properties.

3.5.1 Overview of methods and assumptions

Within the Test Case, /Röshoff et al, 2002/ evaluated and applied several empirical relations for predicting rock mass mechanical properties. Additional evaluations were made by /Makurat et al, 2002/. An overview of the approach is given in

Figure 3-5. Note, that some of the actions listed may not be relevant for application within the Site Investigation.

The general assumption with the approach is that indices such as Q and RMR have a simple relation with the rock mass properties. With this assumption, the spatial variability, and some of the uncertainty, of the rock mass properties would be reflected by the spatial distribution, and uncertainty, of the indices, see Section 3.4. Clearly, for a specific situation, a good correlation between Q or RMR and the mechanical properties could always be obtained by adjusting the parameters comprising the indices and/or their rating values. Indeed, some adjustments are already made for the characterisation purpose, see Section 3.1.2. However, the approach would be less useful if additional adjustments were needed for every single site and application.

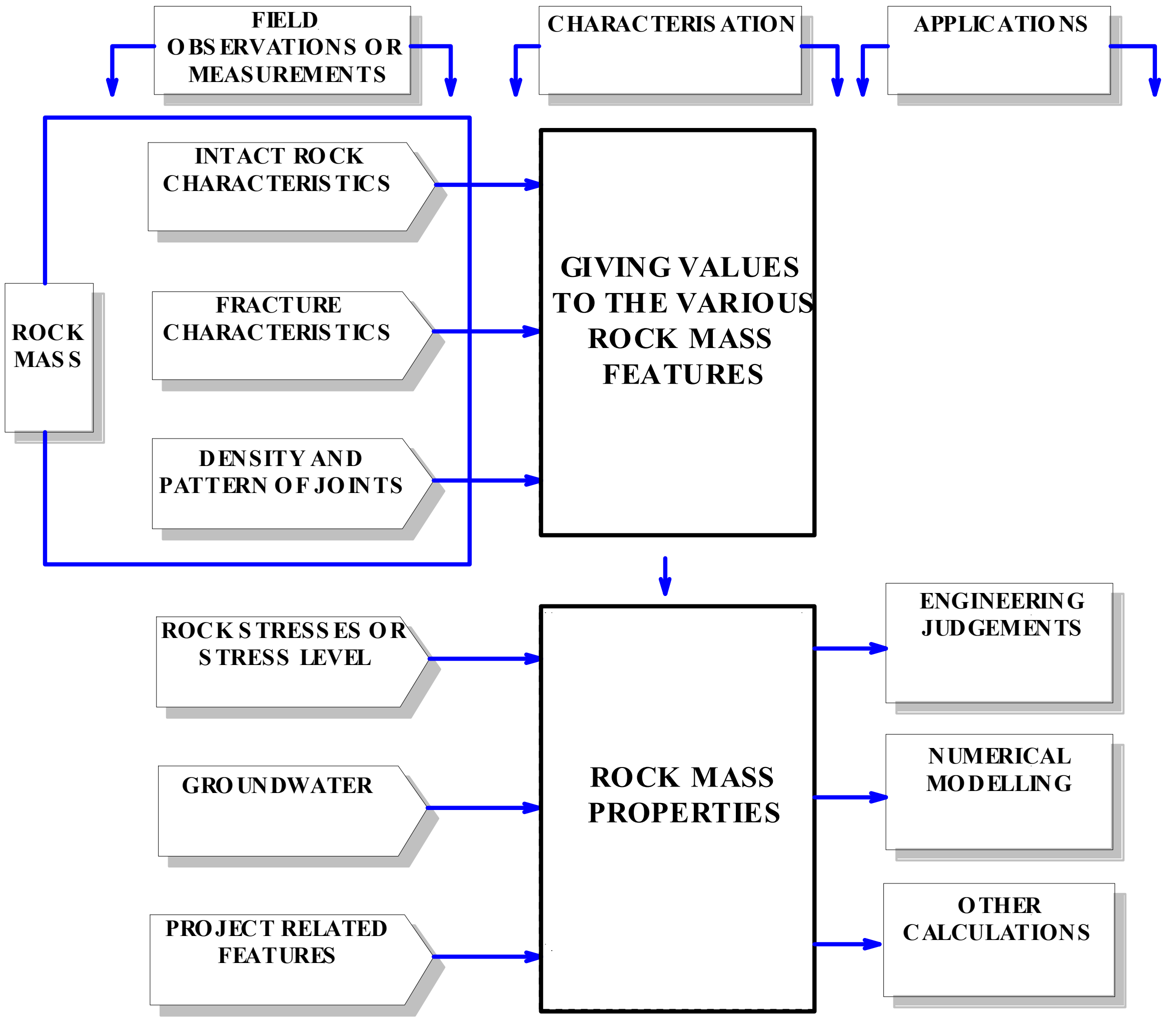


Figure 3-5. *Flow chart for the empirical approach, modified after /Palmström et al, 2001/.*

3.5.2 Deformation modulus and Poisson´s ratio

In the literature, there are several suggested relations between rock mass deformation modulus and Q or RMR. While developing the Test Case predictions, /Röshoff et al, 2002/ mainly used the relation suggested by /Serafim and Pereira, 1983/:

$$E_m = 10^{\frac{RMR-10}{40}}, \text{ for RMR} > 50 \qquad (3\text{-}6)$$

When Q values were available, /Makurat et al, 2002/ used a relation suggested by /Barton, 1995/:

$$E_m = 10Q^{1/3} \qquad (3\text{-}7)$$

When only RMR values were available /Makurat et al, 2002/ used the relation suggested by /Serafim and Pereira, 1983/. However, there are several other relations suggested in the literature, although many of them are developed for other types of rock.

/Röshoff et al, 2002/ demonstrate the differences between different relations by plotting resulting E_m values in the Test Case Predictions for four different relations, see Figure 3-6. The spread in predicted E_m is wide. Results from Bieniawski's and Serafim and Pereira's methods are similar to one another, and they range between the extreme values provided by other relations. For this reason, /Röshoff et al, 2002/ applied the equation by Serafim and Pereira for characterising the target cells in the Test Case. Comparison of results between /Röshoff et al, 2002/ and /Makurat et al, 2002/ suggests that the /Barton, 1995/ relation produces results in a similar range, see /Hudson, 2002/. However, this does not mean that these relations can be used without caution.

Over the years, different relations have been developed for different situations and the application of a relation outside the range of its 'database' is questionable. Common sense is needed when applying empirical relations, and results need to be compared with other available information. This is discussed further in Section 3.7.

The Poisson's ratio, ν, is often assumed to take the same value in the rock mass as in the intact rock. On the other hand, it might be expected to be less because of the effect of the fractures. The dynamic Poisson's ratio may be estimated from the seismic velocities through the rock mass. For the Test Case, /Makurat et al, 2002/ applied the following formula presented by /Goodman, 1980/:

$$\nu = (v_p^2/v_s^2-2)/(2(v_p^2/v_s^2-1)) \qquad (3\text{-}8)$$

where v_p is the compressional wave velocity and v_s is shear wave velocity. The seismic velocities may also be used to estimate the dynamic deformation modulus. In /SKB, 2001/ it is suggested that

$$E_m = v_p^2 \cdot \rho \cdot (1+\nu) \cdot (1-2\cdot\nu) / (1-\nu) \qquad (3\text{-}9)$$

where ρ is the rock mass density.

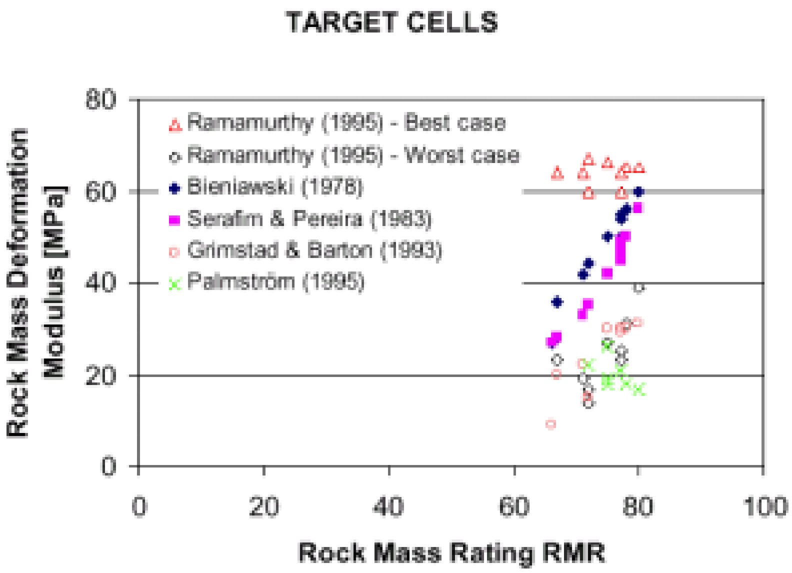


Figure 3-6. *Different relations produce a wide range of predicted E_m values, and the applicability of the relations depends on the specific problem and geological setting (from /Röshoff et al, 2002/, Figure 6.15).*

/Barton, 1991/ proposed a correlation between the seismic velocity v_p and Q values:

$$Q = 10^{\frac{V_p - 3500}{1000}} \tag{3-10}$$

/Röshoff et al, 2002/ used this relation to compare the Q ratings with the available seismic velocity data in the Äspö area. For good rock quality, a better fit was obtained using the equation /Barton, 1991/:

$$Q = (v_p - 3600)/50 \tag{3-11}$$

In conclusion, it seems that seismic data may offer an additional source of information. However, its direct applicability to calculating rock mass mechanical properties needs further evaluation.

An update of Q correlations with rock mass properties is given in /Barton, 2002/. This article also discusses the distinction between rock mass classification specifically for tunnel design and generally for rock characterisation.

3.5.3 Rock mass strength

The literature also provides several relations between the empirical indices and rock mass strength.

For the test case, /Röshoff et al, 2002/ mainly used the /Hoek and Brown, 1988, 1997/ relations. The generalised empirical Hoek and Brown strength criterion for jointed rock masses is given as:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \tag{3-12}$$

where σ'_1 and σ'_3 are the major and minor effective stresses, respectively, σ_{ci} is the uniaxial compressive strength of the intact rock material, and m_b, s and a are parameters characterising the rock mass. These parameters may in turn be related to the GSI (Geological Strength Index). Furthermore, GSI is equivalent to RMR for characterisation, i.e. where the groundwater rating is set to 15 and the adjustment for orientation to zero, /Bieniawski, 1989/. The parameters in equation 3-12 are then estimated from the following relations:

$$m_b = m_i e^{\frac{GSI-100}{28}}$$

$a=0.5$ and $s = e^{\frac{GSI-100}{9}}$ for GSI>25

$$a = 0.65 - \frac{GSI}{200} \text{ and } s=0 \text{ for GSI<25} \tag{3-13}$$

m_i is the constant for intact rock. As an alternative, a may be set to 0.5 for hard rock masses. /Martin et al, 1996/ showed that, in order to fit the Hoek-Brown criterion to brittle failure, the value of m_b had to be reduced to unconventional low values, i.e., close to zero, with a value of s = 0.11 (1/3σ_{ci}).

Other relations were applied by /Makurat et al, 2002/. They note that the number of fracture sets and the properties of the fractures have a strong influence on the rock mass compressive strength (σ_{cm}). They also note that, according to /Singh et al, 1992/, σ_{cm} can be estimated from the equation:

$$\sigma_{cm} = 0.7\gamma Q^{1/3} \text{ (MPa)} \tag{3-14}$$

where γ is the rock mass specific weight in kN/m^3. According to /Makurat et al, 2002/, this equation underestimates the σ_{cm} values for hard rocks with high Q values. /Grimstad and Bhasin, 1996/ have therefore modified the equation for hard rock masses by incorporating the uniaxial compressive strength (σ_c) of the rock in the following way:

$$\sigma_{cm} = (\sigma_c/100)\ 0.7\gamma Q^{1/3} \text{ (MPa)} \tag{3-15}$$

For the Test Case, /Makurat et al, 2002/ used this relation for the rock mass and the original Singh et al, relation for the fracture zones.

When comparing results in the Test Case, the estimates made by from /Röshoff et al, 2002/ usually provided the lowest values, /Makurat et al, 2002/ usually provided the highest and /Staub et al, 2002/ (see below) came somewhere in-between. Some of these differences were due to different means of estimating the rock mass strength, using the linear Mohr-Coulomb or non-linear Hoek and Brown failure criteria. Most of these differences illustrate the difficulties in estimating the rock mass strength and the large variation between different empirical relations. It seems evident that any predictions of the rock mass strength need to be assessed on the basis of multiple lines of evidence. This is further discussed in Section 3.7.

3.5.4 Uncertainties

There are several questions and uncertainties related to the empirical approach, as implemented by /Röshoff et al, 2002/ and /Makurat et al, 2002/. Below is a short compilation of issues identified and explored during the work and review of the Test Case work, /Hudson, 2002/. Additional questions may also be raised (see Chapter 7).

The uncertainties in the predictions of the rock mass mechanical properties originate both from uncertainties in the methods of estimating properties with known input data and from the uncertainties in the three-dimensional distribution of the input. The latter issue is generally discussed in Section 3.4. When applying the methods, it is important to consider all uncertainties, assess how they would influence prediction ranges and make an overall judgement on confidence in the predictions.

Validity of empirical relations

The empirical relations are not derived from basic mechanics. Their validity can thus only be ascertained in situations similar to those on which they are based. For any new construction problem, and in particular when constructions are relatively unique, in the current case a deep repository, the empirical approach cannot be verified beforehand. This does not mean that empirical evaluations would not give valuable insight to potential stability problems, but their application is judgmental, relying on the appropriate choice of formulae and advice from experts. Thus, the empirical approach requires supplementary considerations.

Stress dependent parameters

None of the empirical relations utilises the rock stress values directly. Instead, the Q system accounts for stress dependence though the SRF factor, see section 3.1.2; the RMR system does not include rock stress. The resulting parameters, for example the E-modulus, can then be questionable when used in numerical simulations where stress levels may change.

Uncertainty and spatial variation in input data

There are several uncertainties involved in estimating the Q or RMR distributions in the rock. They are discussed in Section 3.4. The spatial variability in Q and RMR predictions in the different units is assessed from the statistical distribution of the supporting data. Furthermore, confidence in the prediction is judged on the basis of the amount of data support.

3.6 Theoretical-numerical derivation of rock mass properties

An alternative to the empirical methods is to calculate rock mass mechanical properties from the mechanical behaviour of intact rock, the mechanical behaviour of fractures and the geometry of fractures. However, such calculations are not trivial – due to the complex geometry and the non-linear and spatially varying small-scale properties of rock. Within the Test Case, /Staub et al, 2002/ attempted such an analysis by stochastic numerical simulations. Another approach was attempted by /Kulatilake et al, 2002/. It is evident that the methods will require additional improvement, but the approach is nevertheless an important complement to the empirical relations discussed above.

3.6.1 Overview of method and assumptions

Within the Test Case, /Staub et al, 2002/ attempted a numerical analysis approach aimed at calculating rock mass properties from fracture geometry and the mechanical properties of the fractures and the intact rock. The general approach is summarised in Figure 3-7. The computations of the mechanical properties of the rock mass are based on multiple stochastic realisations. Multiple realisations reflect the variability and possible distribution of input parameters to the model and permit a statistical analysis of the results.

Realisations of the fracture network are produced from the fracture statistical data using the discrete fracture network generator FracMan /Dershowitz et al, 1998/. These 3-D realisations are transferred to two dimensional block networks, to be analysed by the mechanical code UDEC /Cundall, 1980/.

For a single realisation, UDEC is set up as a mechanical test in order to evaluate rock mass deformation and strength properties. Each realisation is treated independently in UDEC.

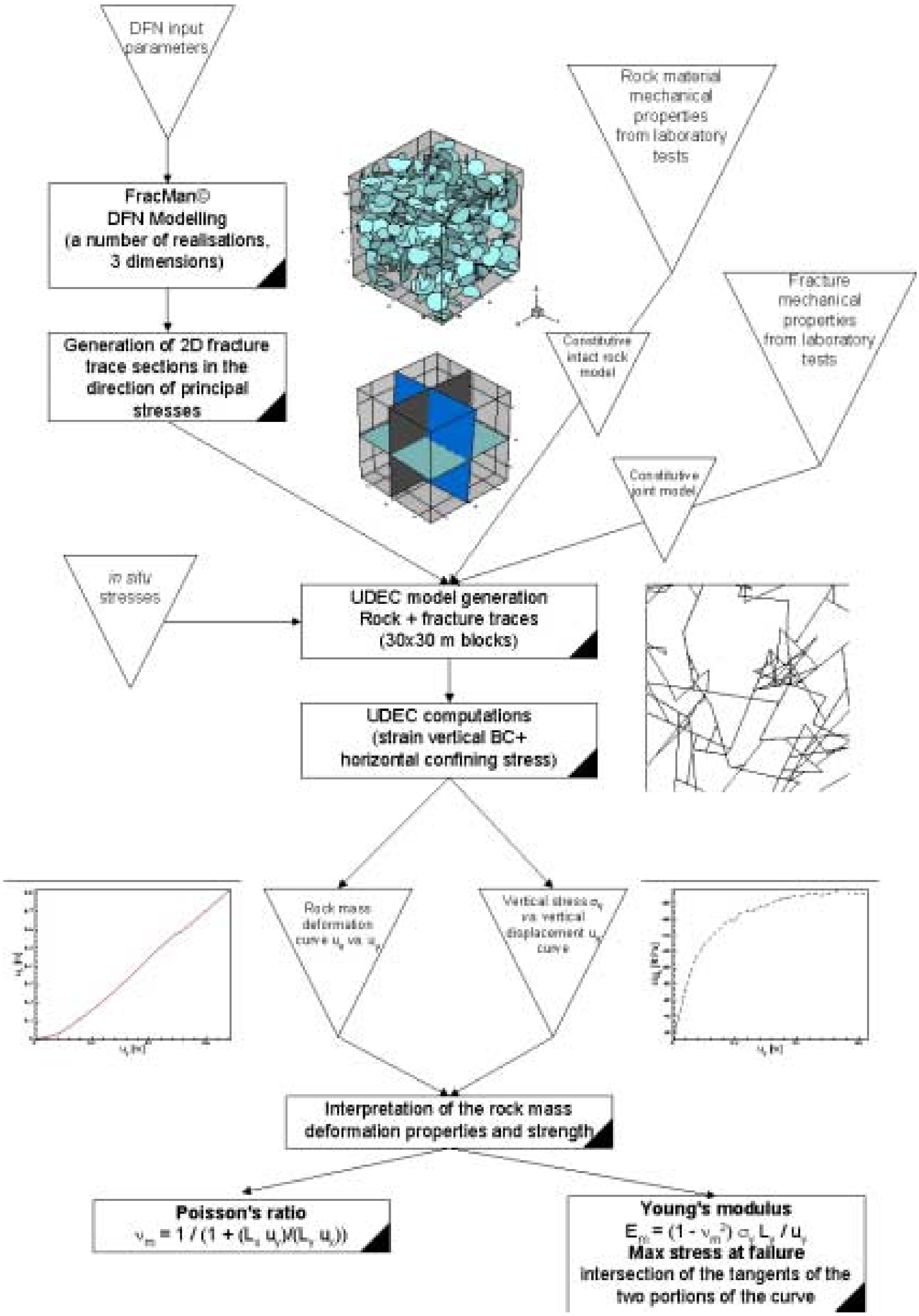


Figure 3-7. *Flow chart for the numerical modelling – taken from /Staub et al, 2002/.*

Data needs

The analyses generally require information on the fracture geometry, and on the spatial distributions of the mechanical properties of the intact rock and of the fractures. These distributions generally should be obtained from the three-dimensional extrapolation of properties, as described in section 3.4.

In the analyses by /Staub et al, 2002/, these needs were simplified. It was assumed that, apart from the fracture zones, a single DFN-model was applicable for the entire rock volume (with no change in fracturing with depth, etc). Furthermore, the evaluation of deformation modulus and strength were made for one set-up of intact rock strength and deformation modulus, representing a single rock type. In order to obtain a 3-D distribution in rock domains with several rock types, /Staub et al, 2002/ built on the rock type distribution of the rock domain as provided by the Geological Model, by mixing the calculated distributions according to the rock type mix in the Geological Model.

Using this method means that some interpretations have to be made. It is assumed that all the types of rock can occur in a 'sufficiently' large volume, where 'sufficiently' is a function of how large the volumes are that are required to be modelled.

Fracture network generation

Based on the fracture statistics of the modelled volume, a three-dimensional fracture network is generated using the discrete fracture network code FracMan /Dershowitz et al, 1998/. The development of this rock mechanics model requires the identification of different fracture sets that can exhibit different mean fracture orientations, different fracture sizes, different properties, different fracture densities and different fracture termination modes. The fracture statistical input to DFN-modelling is outlined in section 3.2.2.

Adapting a 3-D fracture network to a 2-D UDEC geometry

Preferably, the mechanical analyses should be performed directly on the generated three-dimensional fracture network, but /Staub et al, 2002/ judged that only 2-D mechanical analyses were feasible. There was no automated interface between the fracture network generator and a suitable 3-D mechanics code. Furthermore, with the given dimensions and fracture intensities, the demands on computer time and memory would have been substantial. Instead, some verification runs were made, based on a simpler fracture geometry (see section 3.6.4). Nevertheless, it would be better to use just 3-D models because there is significant uncertainty introduced by switching between the 2-D and 3-D models.

For the Test Case, /Staub et al, 2002/ applied the following procedure for generating two-dimensional representations of the generated three dimensional fracture networks. Since the boundary conditions of the UDEC code are preferably set to normal loading only (no shear conditions), fracture traces should be obtained in planes aligned with the in situ principal stress. The fracture traces are obtained by introducing 2-D planes in the 3-D model and then calculating the intersections between fractures and this plane. Three different fracture trace planes from the 3-D FracMan model, adapted to the UDEC modelling size and aligned with the in situ principal stress field, were identified by this procedure.

Material models for intact rock and fractures

For the Test Case, /Staub et al, 2002/ usually worked with an equivalent 30 m x 30 m block model in the UDEC code. A block model consists of intact rock and fractures which form a pattern of rock blocks of different shapes and sizes. First, a single rock block of the size of the model is created. Then the fracture traces are applied to the original block, defining different rock sub-blocks. For the Test Case, /Staub et al, 2002/, further simplified the analysis, by deleting fracture traces terminating in the intact rock. If such deletions are made in future application, justification of the procedure will be required.

The mechanical properties of the intact rock and of the fractures have to be specified. For the Test Case, /Staub et al, 2002/ assumed a Mohr-Coulomb model for the intact rock. Tests were also performed with a Hoek and Brown model, providing very similar results. The Mohr-Coulomb model was selected for further analyses since it is easier to apply in UDEC. /Staub et al, 2002/ also tested different constitutive models for rock fractures and found the Barton-Bandis joint model /Barton, 1982; Bandis et al, 1985/ to be the most appropriate assumption for the mechanical properties of the fractures.

Table 3-2 summarises the data input needs.

Table 3-2. Intact rock and fracture data required as modelling input.

Parameter	Description
Intact rock properties	
E_i	Young's modulus of the intact rock
ν_i	Poisson's ratio of the intact rock
c_i	Cohesion of the intact rock
ϕ_i,	Friction angle of the intact rock
σ_T	Tensile strength of the intact rock
Barton-Bandis joint properties	
jk_n	Joint normal stiffness limit
jk_s	Initial joint shear stiffness limit
ϕ_r	Residual angle of friction
σ_c	Intact rock uniaxial compressive strength (back calculated for 50 mm diameter samples)
JRC_o	Lab-scale roughness coefficient
JSC_o	Lab-scale joint wall compressive strength
l_o	Lab-scale joint lengths
a_p	Aperture for zero normal stress

Stress/depth dependence

The in situ stress conditions are reproduced in the model by applying vertical stress to the right vertical boundary of the model, see Figure 3-8. The stress conditions are provided by the Stress Modelling (see Chapter 4). For the Test Case, /Staub et al, 2002/ assumed that the stress directions are the same for the whole model domain, but this assumption is not critical to the method. It was made to reduce the number of Monte-Carlo simulations.

Numerical analyses – UDEC computations

The rock mass properties are obtained by numerical simulations with each realisation of the UDEC block model. The block model simulation in UDEC is similar to a plane strain test. The numerical model will simulate a plane strain load test of the rock mass with constant confining stress, see Figure 3-8. The model can be unloaded horizontally from the initial stress condition to simulate any confining stress.

The numerical mechanical testing is carried out by vertical loading at the top of the model with a constant velocity boundary displacement. The interface between the loading block and the rock mass block is assumed to have no friction. The vertical loading is applied to the model after it has reached equilibrium under pre-loading conditions. The vertical loading is applied to the model beyond the elastic behaviour of the components of the model (rock material and fractures) so that the estimation of the rock mass strength can be made. The influence of the boundary conditions has also been investigated, see Section 3.6.4.

The following parameters are monitored during the loading test:

- Vertical displacement and stress along a horizontal profile at the top of the model,
- Horizontal displacements along a vertical profile at the right boundary of the model

The monitoring profiles consist of 25 monitoring points. A mean value for the monitored variable is determined at each loading step.

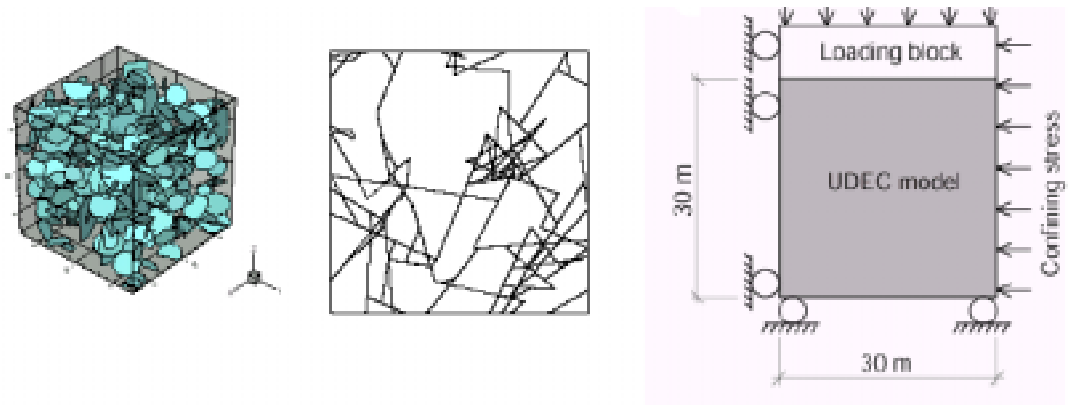


Figure 3-8. *Basic set-up of UDEC simulations (based on /Staub et al, 2002/).*

Determination of rock mass deformation properties and strength

At the end of each realisation, deformation curves of horizontal displacement, dx, and vertical stress are plotted against vertical displacement, dy, the independent variable. These curves are used for the calculation of Young's modulus, Poisson's ratio and the strength properties of the rock mass (see Sections 3.6.2 and 3.6.3). However, because much of the input is statistical, in particular the fracture geometry, multiple realisations are performed and the result obtained as statistical distributions of the rock mass properties.

3.6.2 Predictions of E-modulus and Poisson's ratio

During the numerical load test, the average vertical stress, $\delta\sigma_v$, and average horizontal deformation, δx, are recorded as a function of the vertical deformation, δy. The Poisson's ratio, ν, and the Young's modulus of the rock mass, E_m, are calculated from the following equations:

$$\nu = 1 / (1 + (l_x\ \delta y)/(l_y\ \delta x)) \qquad (3\text{-}16)$$

$$E_m = (1 - \nu^2)\ \delta\sigma_v\ l_y / \delta y \qquad (3\text{-}17)$$

where l_x and l_y are the length over which δx and δy are measured. When evaluating the Poisson´s ratio and the Young´s modulus, δx, δy and $\delta\sigma_v$ are taken from the initial linear portion of the stress-deformation and x-deformation–y-deformation curves, see Figure 3-9.

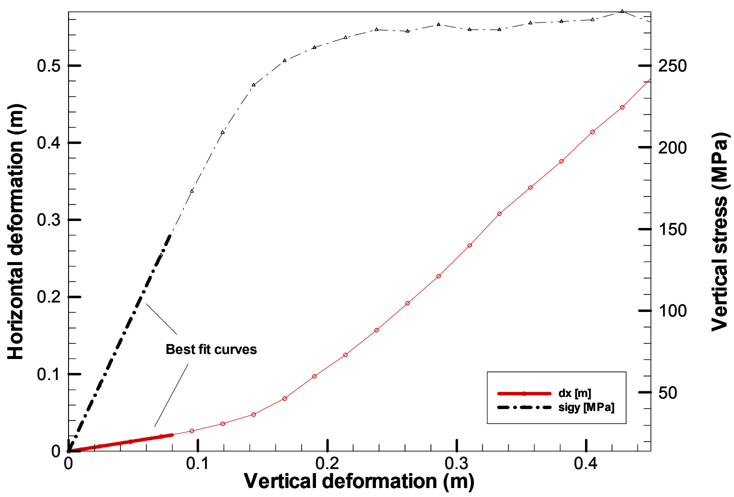


Figure 3-9. *Evaluation of deformation properties in the approach by /Staub et al, 2002/.*

3.6.3 Predictions of rock mass strength

The numerical model can also be used to estimate rock mass strength. For the Test Case, /Staub et al, 2002/ performed a set of simulations in order to evaluate rock mass strength relevant for a 30 x 30 m rock volume around the deposition tunnels. The confining stress around tunnels will range from zero at the tunnel wall to the horizontal initial stress at a distance of about five tunnel radii (about 15 m). To obtain an average, it was decided to run two loading tests in the numerical model. The first test was consolidated to the initial stresses and then loaded in vertical compression to failure. The second test was consolidated to the initial stresses, unloaded horizontally to one quarter of the initial horizontal stress (valid for a distance of about 0.2 radii from the tunnel wall), and then loaded in vertical compression to failure.

The numerical load tests provide two sets of principal stresses at failure, σ_{1a}, σ_{3a} and σ_{1b}, σ_{3b}. The value of σ_1 at failure is evaluated at the intersection of two straight lines. One is the initial linear part of the stress-deformation curve, and the second is the linear portion after failure on the same curve, see Figure 3-10.

The cohesion, c, the friction angle, ϕ, and the uniaxial strength, σ_{cm}, of the rock mass are calculated assuming a linear relation between σ_1 and σ_3 at failure, according to the following equations:

$$\phi = \arcsin (k - 1) / (k + 1) \tag{3-18}$$

$$\sigma_{cm} = \sigma_{1b} - k\, \sigma_{3b} \tag{3-19}$$

$$c = \sigma_{cm} (1 - \sin \phi) / (2 \cos \phi) \tag{3-20}$$

where $k = (\sigma_{1a} - \sigma_{1b}) / (\sigma_{3a} - \sigma_{3b})$.

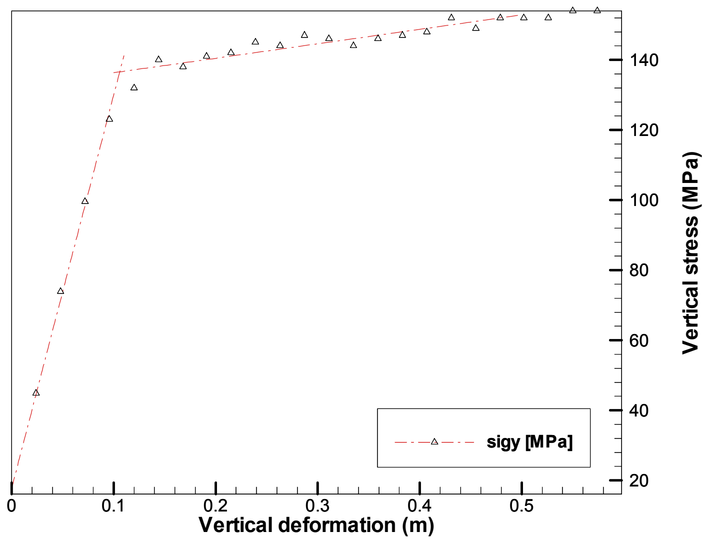


Figure 3-10. *Evaluation of the major principal stress at failure in the approach by /Staub et al, 2002/.*

The assumption of a linear relation between the principal stresses at failure, which is equivalent to assuming c and ϕ are stress independent, was criticised during the Test Case. However, additional runs by /Staub et al, 2002/ exploring the relation between σ_1 and σ_3 at other stress levels indeed showed a linear relation, but it was noted that this was possibly a circular result because of the assumed (stress independent) Mohr-Coulomb failure criterion for the intact rock. A sensitivity analysis for the failure criterion used is discussed in Section 3.6.4.

3.6.4 Uncertainties

There are several questions and uncertainties related to the theoretical/numerical approach as implemented by /Staub et al, 2002/. Below is a short compilation of issues identified and explored during the work and review of the Test Case. Additional questions may be raised (see also Chapter 7).

As for the empirical approach, uncertainties in the predictions of the rock mass mechanical properties originate both from uncertainties in the methods of estimating properties with known input data and from the uncertainties in the three-dimensional distribution of the input. The latter issue is discussed in Section 3.4. When applying the methods, it is important to consider all uncertainties, assess how they would influence prediction ranges, and make an overall judgement on confidence in the predictions.

The material model for the intact rock

The initial set-up of the method assumed an elastic-plastic material model for the intact rock, with a Mohr-Coulomb (M-C) failure criterion for the plastic component. The influence of using the Hoek-Brown (H-B) failure criterion or a strain-softening (S-S) model for the intact rock has also been tested. The three models give almost the same results for deformation properties, except at low stresses where the M-C model gives the lowest stress at failure and the H-B and S-S model higher values, and vice versa at high stresses. For more details, see /Staub et al, 2002/.

Depth and stress dependence

It should be noted that the methodology generally provides stress dependent properties because the confining stresses and in situ stresses are introduced as boundary conditions. However, the model results will of course only be able to account for the stress dependency embedded in the underlying models for fracture and intact rock deformation.

Influence of the domain size and applied boundary conditions

/Staub et al, 2002/ have explored the importance of the domain size and boundary conditions. Tested model sizes range from 20 m to 60 m side lengths, and boundary conditions were either a confining stress applied on two sides or a confining stress applied on one side. Each test was performed with the same fracture network. The results show minor effects on the value of E_m, but some reduction in rock mass strength as the size increases.

Influence of the discarded joints

The fracture traces that terminate in the intact rock and do not intersect other fracture traces are discarded when 'meshing' in UDEC. In order to quantify the influence of this process on the mechanical properties of the rock mass, /Staub et al, 2002/ conducted a special test on a selected fracture network pattern that had also been tested with the described set-up for modelling. In the new analyses, all fracture traces that terminate in the intact rock were extended until they intersected with other fracture traces or with the border of the model. This 'artificial' part of the fracture was given properties to mimic intact rock. The resulting deformation was very similar to the case without the added joints, and for the Test Case it seemed acceptable to discard these fractures. In other applications, this issue may warrant further consideration.

2-D representation from the 3-D model

The transfer of a 3-D fracture network into a 2-D trace network with the same overall mechanical properties is not trivial. For example, even if there are no blocks formed by the fractures in a 3-D fracture network, fracture traces in a two-dimensional cross-section may form 2-D blocks. The difference between the 3-D and 2-D model is likely to depend on the nature of the fracture network. For this reason, at the moment it is unrealistic to conduct 3-D simulations with real fracture networks, but /Staub et al, 2002/ performed some tests on simple fracture networks (with 6 fractures) comparing simulations having several intersection planes in UDEC and 3-D simulations using the 3DEC software /ITASCA, 1999/.

In Figure 3-11 there is a comparison of the calculated stress-deformation curves obtained from the UDEC and 3DEC runs, respectively. The Figure shows that the results of the 3DEC simulations are similar to the 'softest' two-dimensional runs, both in deformation modulus and principal stress at failure. The 2-D sections corresponding to these runs are in the axis of the most unfavourable stability situation in 3-D and the direction of most deformation. The fact that the principal stress at failure in the 3-D model is similar to the principal stress at failure of the weakest 2-D sections conforms to expectations.

Further studies should be orientated towards the behaviour of the 3DEC model close to failure. With few fracture planes in the model, the difference between 3DEC and 2-D simulations is probably large and ought to decrease with the number of fracture planes. Nevertheless, the simulations show that the discrepancy of results decreases when the 2-D sections are in the axis of the most unfavourable stability situation in the 3-D models. This illustrates that it is important to extract the 2-D vertical sections in different directions in relation to the fracture system when simulating with a 2-D program.

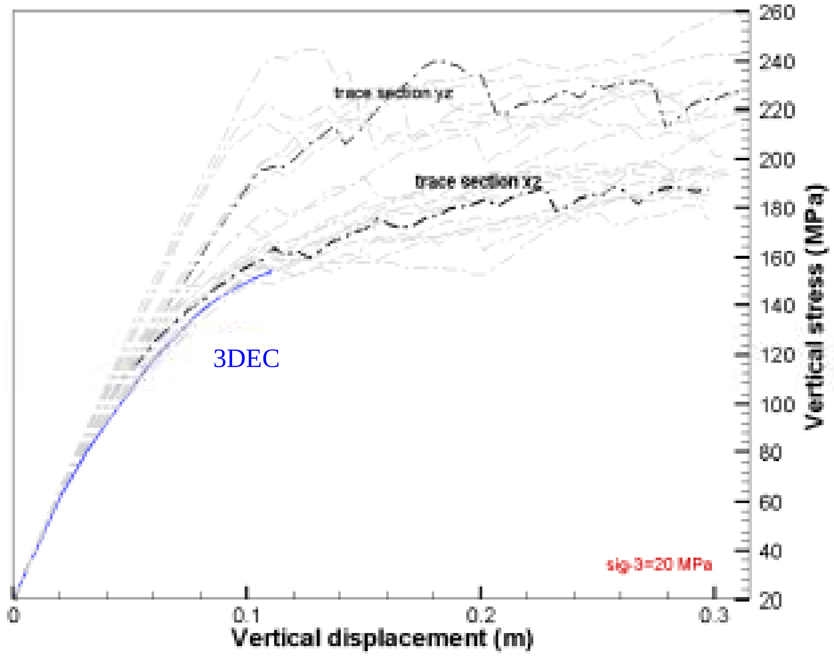


Figure 3-11. *Calculated vertical stress – vertical deformation curves. The 3DEC curve is for a triaxial loading situation and the UDEC curves for plane strain loading (from /Staub et al, 2002/).*

Uncertainty and spatial variability in input data

There are several uncertainties involved in estimating the spatial distribution of fracture statistics, intact rock mechanics properties and fracture mechanics properties. As already noted, in the Test Case /Staub et al, 2002/ essentially accepted the given geological model. The spatial variability, uncertainty and confidence in the geological model, including the DFN-model, is thus transferred into the mechanical description. In particular, one should note that the statistical nature of the DFN-simulations and the mix of rock types in the spatial domains are essentially measures of spatial variability. However, even accepting the fact that the rock mechanics modelling was based on the Geological Model, several issues may be raised. These include the following.

- Estimation and derivation of the DFN-model implies several uncertainties and includes several judgmental aspects (selection of statistical model for size distribution, treatment of sampling cut-offs, division into different rock units, depth dependence etc). The rock mechanics modeller needs to assess the basis for and uncertainty in the DFN-model provided within the Geological Model and make sure it is relevant to the rock mechanics work.

- Are measurements of the mechanical properties of intact rock representative of the rock types or are there other relevant factors (e.g. depth) influencing the intact rock strength? Connected to this are potential difficulties in obtaining representative (undamaged) core samples from high stress environments.

- What are the spatial distributions of fracture mechanics properties which to a large degree depend on the nature of the fracture fill? Are they related to rock type, depth, or something else?

These and other issues need to be explored when judging the overall uncertainty and confidence in the input data to the modelling.

3.7 Strategy for a Combined Rock Mechanics Property Model

Considering the uncertainties with both the empirical and theoretical approaches, it is evident that one single approach cannot be recommended. Instead, the rock mechanics modelling will need to apply different methods to estimate the rock mass mechanical properties and then devise a procedure for making an overall judgement. This step was also tried within the Test Case and Protocol 5 was specially developed for this. The following section builds on the Test Case Protocols and experiences, with adaptations for use of the approach in the site modelling.

3.7.1 Harmonisation and amendments

After a first set of different modelling attempts (which could be empirical and theoretical), a stage of harmonisation and amendments should follow. This step involves a series of meetings and other interactions between the modelling teams, essentially aiming at an 'internal review' with the purpose of

- identifying and correcting errors,
- identifying and agreeing on non-method specific assumptions (such as geometry),
- making different experts more familiar with each other's approaches.

The harmonisation step will probably indicate a desire/need to update individual predictions using various approaches, by adjusting assumptions, correcting errors etc.

After harmonisation and amendments there is need for further evaluation. Neither the empirical methods nor the current theoretical methods are fully adapted to the needs of the rock mechanics modelling. There is a considerable element of judgement involved in using the empirical relations, as already discussed, and the experience database for deep construction is limited. The theoretical methods also build on a set of assumptions and there are numerical limitations as to what can actually be calculated.

3.7.2 Establishing reasonable estimates and an overall uncertainty span

After harmonisation and amendments, the next step is to explore the possibilities for a combined prediction. The basic principle for achieving the combined prediction is to superpose predictions made by different methods. However, the ranges of predictions generated by, for example, two different methods may or may not overlap.

A combined prediction should be sought for in a consensus discussion involving relevant expertise at the different stages of the Site Investigation. Four main decision factors should be used in arriving at the consensus range:

- the overlap of the individual predictions;
- the confidence of the individual predictions (including identified short-comings in input data);
- relevant engineering experience; and
- the engineering significance of differences between different predictions.

The combined prediction should be presented in two forms.

- a **reasonable combined prediction estimate –** in the form of a range agreed by consensus, supported by the reasons for choosing this portion of the superposed individual modelling range predictions, and with the confidence stated.
- an **alternate combined prediction estimate –** which will be a wider range, reflecting the overall uncertainty in predictions. This could be the complete superposed range, a reduced part, or even a wider range, depending on the group discussions.

This procedure was tested successfully within the Test Case work, see /Hudson, 2002/.

It is the users, for design and safety assessment, who will determine how to value these two forms of predictions. For example, it is likely that the design process primarily would work with the reasonable estimate, but the design process would also assess to what extent the alternate prediction would imply severe problems. In such a case, there would be strong motivation to improve the estimate and reduce uncertainty.

3.7.3 Total evaluation of spatial variability, uncertainty and confidence

Considering all the different inputs and sources of uncertainty, the predictions of rock mass properties need to be combined with assessments of spatial variability, uncertainty and confidence. The following principles can be applied.

- Spatial variability for the part of the model volume lying outside the correlation distance from measurements should be assessed from the measured data variability within all rock units within a rock domain. In the theoretical analyses, the variability is also embedded in the stochastic nature of the analyses (i.e. different fracture network realisations represent the spatial variability of the fracture network).
- Volumes close to measurements (i.e. within a correlation distance) may be allocated the values actually at the measuring point.
- The modeller should discuss to what extent and where additional data (more boreholes, denser measurement points) would affect prediction uncertainty. Due consideration to the correlation structure of the parameters should be given in this assessment.

- Uncertainty predictions concern any additional numerically expressible data uncertainty in the predictions. For the empirical approaches, it can, for example, be useful to adopt different empirical relations, or to consider the quality in the Q or RMR estimations along the borehole. For the theoretical approaches, this concerns the effects of boundary conditions, unknown scaling rules or poor measurements, or the uncertainty in the DFN-model.

- Neither the empirical relations in Section 3.5, nor the attempted theoretical modelling in Section 3.6 consider coupled hydro-mechanical effects. While advanced modelling work is being developed, see e.g. /Stephansson et al, 1999/, it is difficult to utilise it in practice at this stage. The approach would instead be to be aware of scientific progress and use it as assistance in engineering judgements. Updating the approach to the Rock Mechanics Site Descriptive Model for such developments is discussed in Chapter 7. If coupled processes should be considered, it is likely that this can only be done using a mechanistic modelling approach (i.e. building on the theoretical modelling described in Section 3.6).

- The confidence in the predictions (including the predictions of spatial variability and uncertainty) is judgmental. The Test Case experience suggests it is practical to indicate the level of confidence in the prediction in different classes in the following form.

  1=value supported by local data or otherwise high confidence prediction,

  2= value produced by interpolation/reasoning, and

  3=value based only on generic information.

  It should be noted that different regions in the predicted domain may have different confidence levels.

All uncertainty predictions and uncertainty values should be described in written text.

3.8 Conclusions

Predicting rock mass mechanical properties in a three-dimensional volume based on information from the surface and from a few boreholes is indeed a challenge. Neither the empirical methods nor the theoretical analyses are ideal in this respect. The empirical methods involve substantial elements of judgement. The theoretical numerical modelling involves simplifications in terms of dimensions and complexity. Both methods share the difficulty of extrapolating information measured at or close to a borehole into predictions of properties far from the borehole.

However, it must be remembered that the primary aim of the Site Investigation is to find rock volumes suitable for the repository construction. The required and preferred rock conditions are discussed in /Andersson et al, 2000/. From a rock mechanics point of view, this is equal to a ‘fair’ or better rock mass quality according to the Q and RMR ratings. In high quality rock, differences between different methods are less severe, as indicated by Figure 3-12. In the range of ‘fair’ to ‘good’ rock, the empirical classification systems using Q and RMR indicate roughly the same rock mass deformation modulus. When identifying volumes of good rock, many of the difficulties discussed in this Chapter may be of a second order.

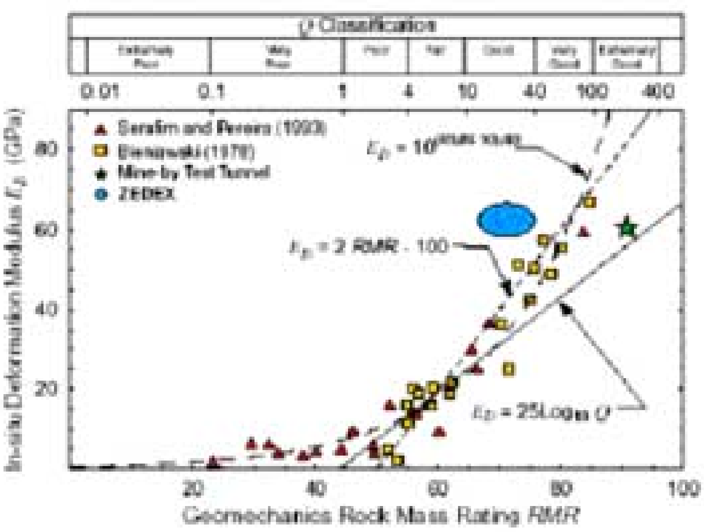


Figure 3-12. *Relations between the rock deformation modulus and the classification systems RMR and Q, (from /Martin et al, 2001/).*

The Rock Mechanics Site Descriptive Modelling should aim at applying several different methods. The empirical classification systems are especially useful in the early stages for identifying 'good rock' volumes, even if they should be applied with caution. The theoretical model needs further development, but has the merits that it is better suited for exploring different material models, can use more directly the information derived in the Geological Model, and may be less subjective.

The Test Case experience clearly showed the benefit of a combined, integrated approach. First of all, comparing results produced by the different teams and based on different assumptions was an effective opportunity for internal review. A harmonisation and amendment stage would be an important component also for the Site Descriptive Modelling. This ought to be combined with the integration process in other geoscientific disciplines, as discussed in Chapter 6. Furthermore, the results from various approaches should be combined with additional evidence and assessed in relation to the engineering needs. Such an evaluation would aim at combined predictions of the rock mass mechanical properties.

In conclusion, determination of rock mass mechanical properties cannot be made with high accuracy and precision. On the other hand, at least for design purposes, an uncertainty of say ± 15% (see Table 1-1 in Chapter 1) may not have a significant impact on the result in good rock conditions. Current methods as described in this Chapter, if combined with reason and experience, should be able to provide a picture of where the rock mass is likely to be relatively unproblematical for underground construction and where there is potential for difficulties. Also, in the future, there will be a stronger link between the site investigation procedures and the specific repository and PA/SA input requirements. This will include the associated data reduction and the type of interpretative studies described here. In this way, the work can be focussed more directly on the specific rock mass parameter requirements.

4 Estimating the State of Stress

4.1 Rock stress as a component of the descriptive model

A necessary component of the Rock Mechanics Site Descriptive Model is the characterisation of the pre-existing state of stress in the rock mass. This stress is caused by the combined effect of gravitational force and tectonic forces. A knowledge of the stress state is required for both analytical and numerical modelling of the stresses induced by excavation of a repository.

A brief aide-memoire follows on the tensor nature of stress. Given a rock mass as illustrated in Figure 4-1, the forces at a point within the rock mass are considered via the stress field.

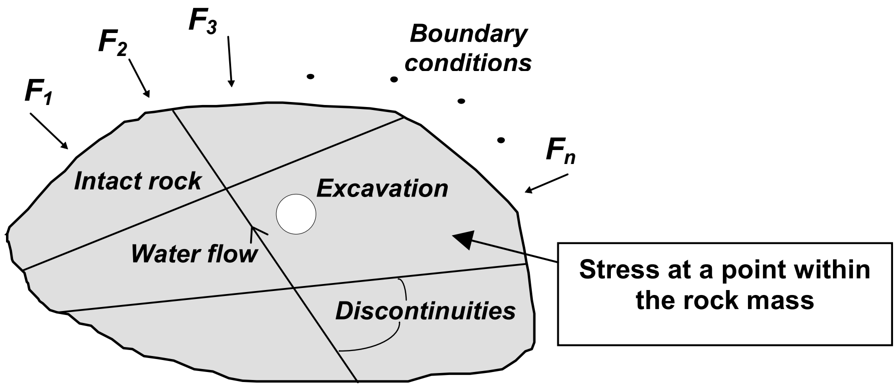


Figure 4-1. *The forces on and within a rock mass are considered using the concept of stress (from /Harrison and Hudson, 2000/).*

The normal and shear forces on an elemental cube are considered via the stress components, as shown in Figure 4-2 with reference to x, y and z axes.

The state of stress in a rock mass can only be correctly specified if there are six independent items of information given – because there are six independent components in the stress matrix. The components on the cube illustrated in Figure 4-2, i.e. with respect to the specified axes, are listed in the stress matrix as shown in Figure 4-3 (a). The matrix is symmetrical, giving the six independent components shown in Figure 4-3 (b). Principal stresses act perpendicular to planes on which there are no shear stresses present, the zeroes in the matrix in Figure 4-3 (c). Usually the estimated rock stress at a site is presented as the magnitudes and orientations of the principal stresses: σ_1, σ_2, and σ_3.

Thus, the measurement and specification of the stress at a point in a rock mass requires consideration of all the six components of the stress state. Moreover, the tensor nature of stress means that the components must be manipulated accordingly. For example, one should not take a direct average of the principal stress values when averaging several stress states.

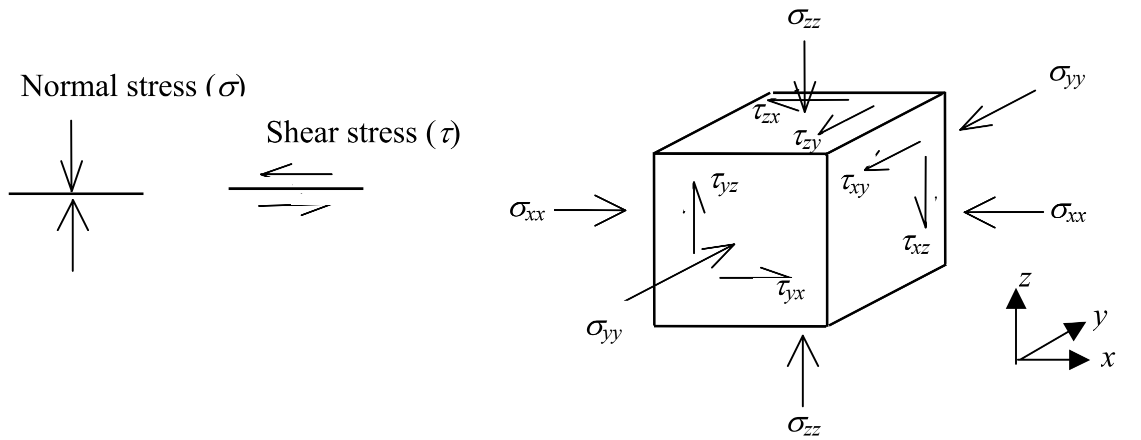


Figure 4-2. *The normal and shear forces acting at a point in the rock mass are represented by three normal and six shear stresses (from /Harrison and Hudson, 2000/).*

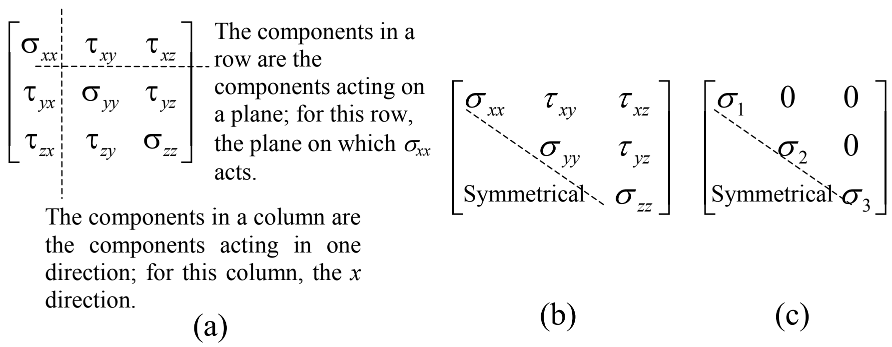


Figure 4-3. *a) All the components of the stress matrix; b) the six independent stress components; c) the three principal stresses.*

4.2 Factors affecting in situ rock stress and its measurement

There is no internationally agreed terminology for the terms describing in situ rock stress, but the descriptions below follow conventional usage /Harrison and Hudson, 2000/.

Natural stress:	the in situ stress which exists prior to engineering.
Induced stress[1]:	the natural stress state as perturbed by engineering.
Gravitational stress:	the stress state caused by the weight of the rock above.
Tectonic stress:	the stress state caused by tectonic plate movement.
Residual stress:	the stress state caused by previous geological activity.
Thermal stress:	the stress state caused by temperature change.
Palaeostress:	a previous natural stress that is no longer acting.
Near-field stress:	the stress state in the region of an engineering perturbation.
Far-field stress:	the stress state beyond the near-field.
Local stress:	the stress state in a region of interest.

For any rock mass, there is a composite overall rock stress caused mainly by gravity, and tectonic factors. However, at specific locations in the rock mass, this overall stress state can be locally perturbed by residual stresses, resulting from the tectonic, erosional and glacial loading history, and other factors such as water and temperature, and especially by the presence of fractures. Thus, these factors must be considered when estimating the rock stress state.

The influence of a fracture or fracture zone on the rock stress is illustrated in Figure 4-4. The stress state **A** indicates the two components of the pervasive 2-D stress state in this plane through the rock. Nearer the fracture, states **B** and **C**, the principal stress directions are rotated and the magnitudes of the principal stresses change, the maximum principal stress increasing. Imagining this type of effect adjacent to many fractures at all scales and in 3-D in a rock mass leads to the expectation that local values of in situ stress, and associated site investigation values, are likely to be variable.

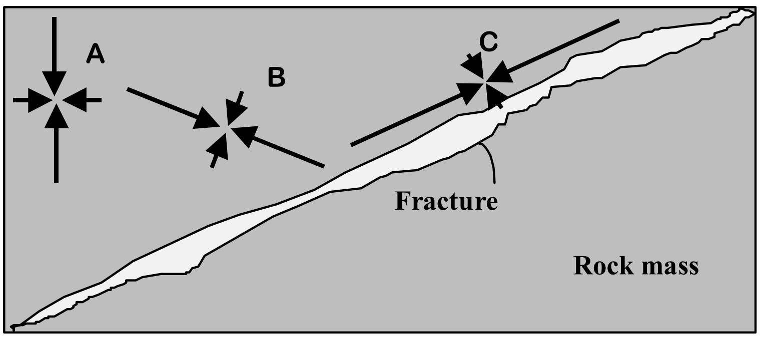


Figure 4-4. *Influence of a fracture on the rock stress state.*

[1] Some engineers use 'induced stress' to mean the actual stress after engineering; others use the term to mean the stress changes caused by engineering. It is important to be sure which definition is being used in any particular context.

The types of stress measurement and the associated geological and measurement factors are discussed in the book by /Amadei and Stephansson, 1997/. They note that important questions relating to the measurements are:

1. Do the different measurement methods yield comparable in situ stress fields?
2. What is the effect of geological structures on in situ stresses?
3. What is the importance of residual stresses, i.e. stresses locked into the rock mass?

Residual stresses occur when the fractured rock is loaded and then unloaded. Due to slip along fractures, stresses can become locked into the rock mass, even after the load has been removed. It is difficult, however, to establish the residual stress component of a measured in situ stress field.

Thus, rock stress estimation is not straightforward because the stress field is likely to vary within the rock mass, and the measurement methods require skill and careful quality control. Rock stress estimation requires:

- study of the existing data available,
- an understanding of the influence of geological factors on the rock stress,
- numerical modelling to support understanding of the likely variations in the stress field,
- careful control of the stress measurements with an understanding of their respective advantages and disadvantages,
- careful data reduction and presentation, and
- some form of test case calibration exercise if possible.

These aspects are covered in the following sections of the Chapter.

4.3 Existing rock stress information – primary data

It is of value to consider the stress data that are already available. This enables assessment of measured and modelled stresses in the context of known information. If the measured or modelled stresses are not in line with the known trends, study is required of the possible reasons – which could be the presence of a fault, topographic variations, residual stresses or incorrect data measurements or modelling.

The primary data can be categorised as three types: global information, Fennoscandian information, and local information (data obtained from an actual site nearby).

4.3.1 Global stress modelling and data collation

There has been considerable interest in the last decade in studying rock stresses on a global scale. For example the paper by /Gölke and Coblentz, 1996/ presents the results of finite element studies of the origins of the European regional stress field, based on the assumed tectonic plate forces. The modelling results in Figure 4-5 were constrained by the measurements in the European component of the World Stress Map illustrated in Figure 4-6.

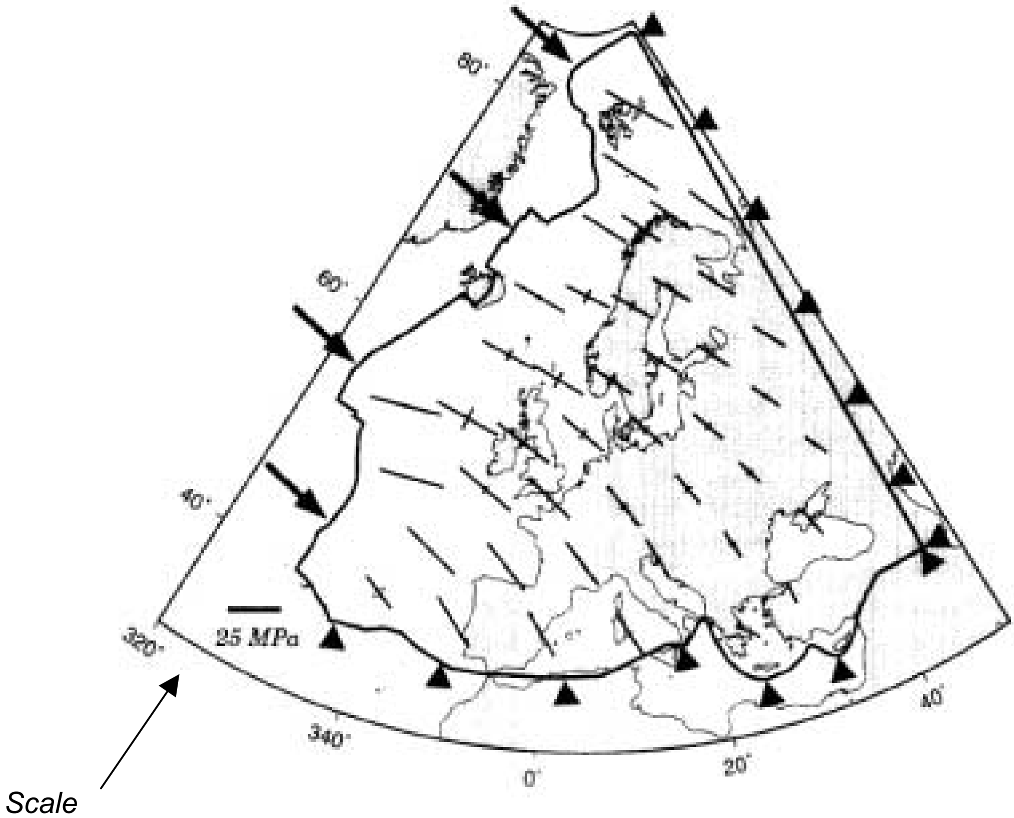


Figure 4-5. *Predicted European tectonic stresses based on mid-Atlantic ridge push from the north-west using one type of model (from /Gölke and Coblentz, 1996/). In this model, part of the boundary is locked (black triangles) and the active mid-Atlantic ridge push is represented by the arrows. The crosses indicate the orientations and values of the maximum and minimum principal stresses, as given by the directions and lengths of the arms of the crosses. Note the stress magnitude scale to the bottom left of the diagram.*

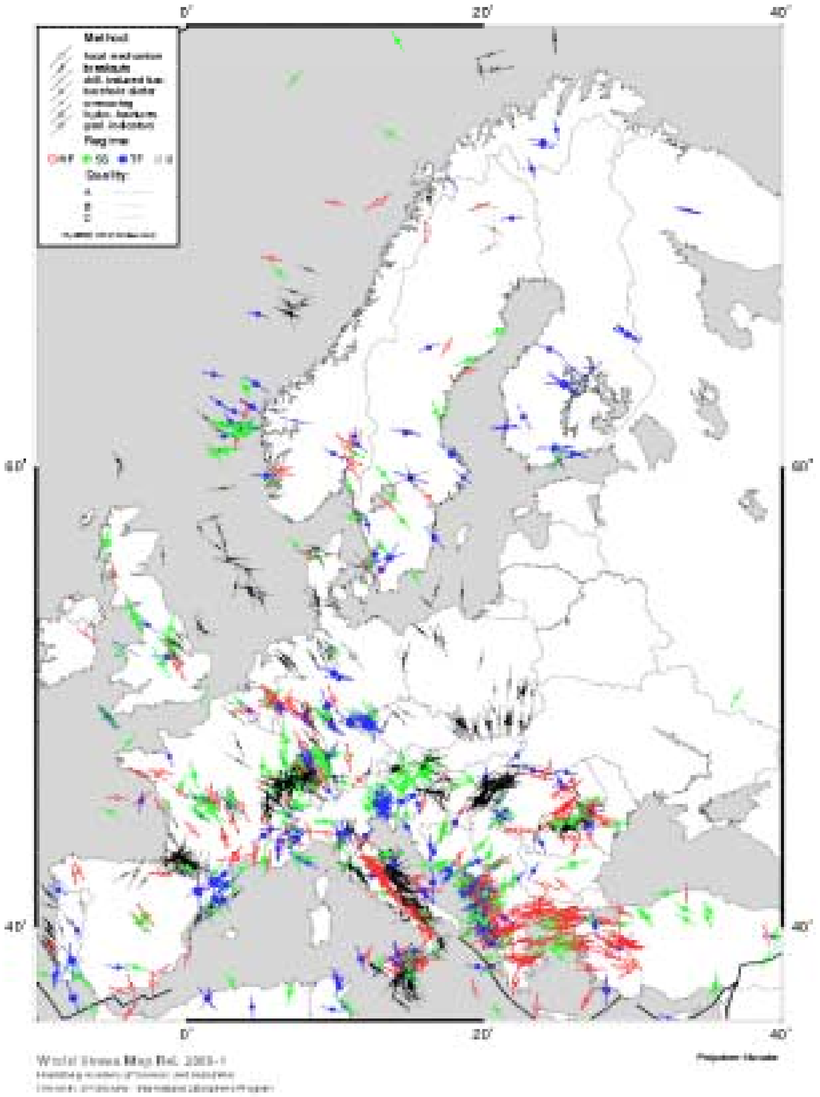


Figure 4-6. European component of the World Stress Map (from /Mueller et al, 2000/).

There is now strong evidence that across the north-eastern part of Europe we expect a regional stress field with the maximum principal stress being compressive, horizontal and orientated SE-NW. The component of the mid-Atlantic ridge push is estimated /Gölke and Coblentz, 1996/ at about 10–20 MPa. The stress in the region of interest will be a function of this component, plus gravity, and the local influences of faults, lithology, topography and possibly other factors such as residual stresses.

4.3.2 Fennoscandian information

Collated information of rock stress measurement data is available for Sweden and Finland. For example, the data in Figure 4-7 provide a good indication of the variation of the major principal stress values obtained from measurements in different parts of the Fennoscandian shield. These measurements were made with different techniques.

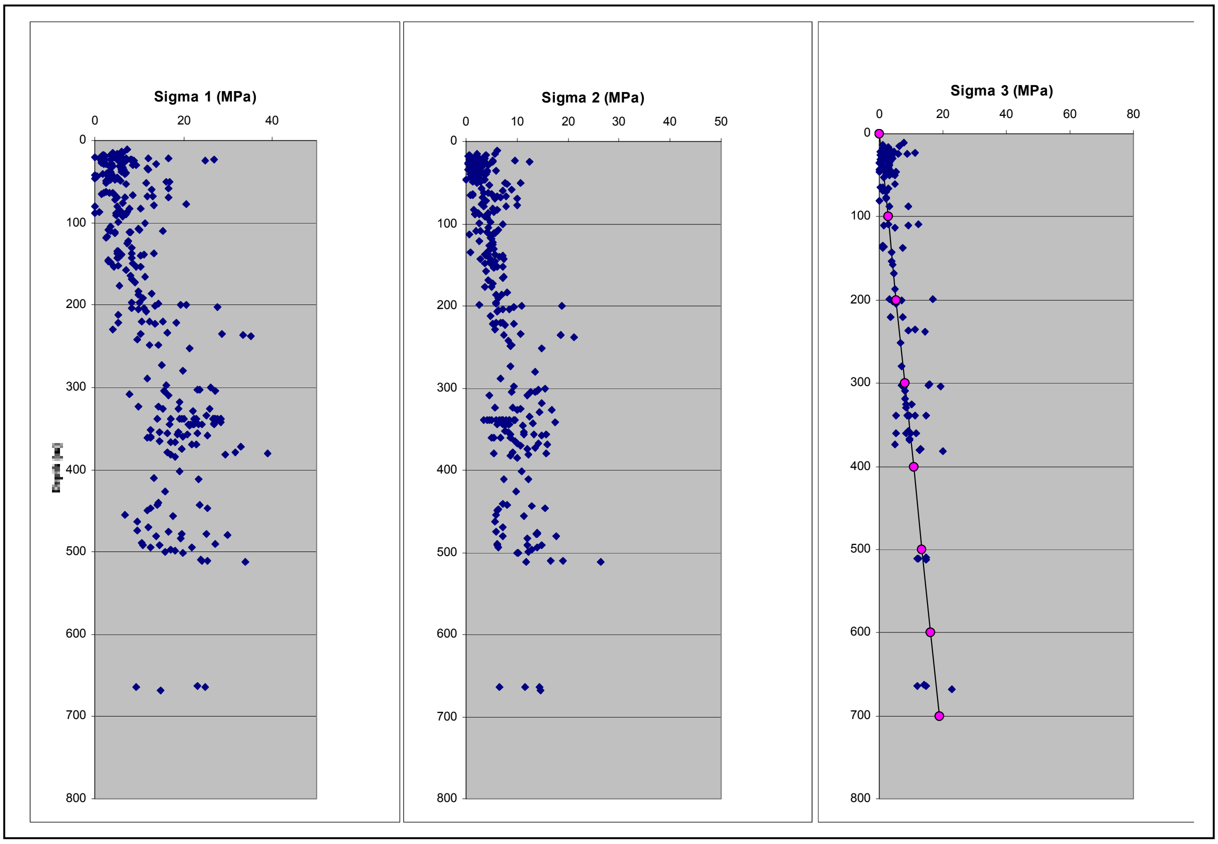


Figure 4-7. Maximum horizontal stress vs. depth. Data from measurements in gneiss, granite and diorite in different boreholes at different locations in Sweden and Finland /Martin et al, 2001/.

4.3.3 Local information

Local information will be in the form of stress measurement results. Stress can be measured in boreholes using two different basic techniques: overcoring methods (also called stress relief methods), and hydraulic fracturing methods. Both methods are described in /Amadei and Stephansson, 1997/.

With overcoring methods, the three principal stresses can be determined irrespective of the measurement borehole direction. This technique involves a small volume of the rock (borehole diameter scale), and several measurements are needed to obtain a good estimate of the six stress components. Hydraulic fracturing techniques are also useful, especially for determination of the minor principal stress. Both techniques require the rock at the measurement locations to be intact, although there is a hydraulic fracturing technique that can utilise pre-existing fractures.

4.4 Geological evaluation of rock stress

In this section, the use of structural geology in assessing the stress history and the present stress state of a site is described. The primary data should be interpreted within the context of the geology at the site being studied, because this provides an enhanced understanding of the existing stress information and gives guidance for the stress estimation strategy. In particular, there is a need to

- understand how fractures at various scales might interact with and modify the current regional stress field, and
- advise on the construction of a numerical model to determine the state of stress in the crust and to constrain the models using geologically realistic boundary conditions, as in Figure 4.5 but on a more regional and local scale.

Also, an understanding of the geological history of the study area is useful for establishing the evolution of the stress regime. This enables the likely pattern of fractures to be determined, a factor that may affect the local manifestation of current regional stresses (*cf.* Figure 4-4), and may enable the likely orientation of site stresses to be determined.

4.4.1 Gravitational and tectonic stresses

The two most important factors controlling the state of stress in the Earth's crust are gravity and tectonism. The former generates both vertical and horizontal stresses in the rocks as a result of the overburden load. The magnitude of these stresses depends on the thickness and density of the overlying rocks and the Poisson's ratio. The vertical stress (σ_v) generated by the overburden is given by $\sigma_v = z.\rho.g$, where z is the depth, ρ is the mean density of the overburden and g the acceleration due to gravity.

The Scandinavian shield has been subjected to several major glacial advances and retreats over the last 100,000 years. The effect has been to modify the vertical load, which can be incorporated into an analysis by either increasing or decreasing the vertical load resulting from the rock overburden. However, the addition or removal of a glacial load is achieved by the advancing or retreating of a glacier and this tends to load the crust asymmetrically – resulting in non-parallel uplift or depression of the crust. These asymmetric load boundary conditions may cause a flexing of the crust, which modifies the stress field and may result in the formation of fractures /Price, 1974; Price and Cosgrove, 1990/.

The stresses generated in the crust as a result of tectonism are more varied and depend on the tectonic setting of the part of the crust under consideration. The Scandinavian Shield is situated well away from present day plate margins. Nevertheless, as illustrated in Figure 4-5 and Figure 4-6, the stress field is dominated by the extensional regime of the Mid-Atlantic ridge and the compressional regime of the Alpine collision zone.

4.4.2 Heterogeneities and fractures

The stress state is frequently modified by local heterogeneities within the crust. These heterogeneities may be the result of intrinsic properties of the crust, such as vertical and/or horizontal lithological variations, or may be induced by the formation of tectonic structures, particularly fractures such as faults and joints (as illustrated in Figure 4-4).

Both the Young's modulus and the Poisson's ratio are important mechanical properties in this regard. A model of the crust, in which an upper layer with a high Poisson's ratio is situated above a layer with a low Poisson's ratio, will manifest a stress 'jump' of the horizontal stress at the boundary between the two, simply as a consequence of the effect of the overburden stress. Also, a weakness resulting from a relatively high density of fractures in the upper part of the crust could also cause such stress decoupling. This is because the properties of the fractured rock zone will be different from the underlying, more intact rock.

4.4.3 Geological conditions at the Test Case location

The study area for the Test Case used for the development of the Rock Mechanics Site Descriptive Model is situated on the west coast of the Baltic sea at Äspö Hard Rock Laboratory in southern Sweden. The site is on the western margin of the Baltic Sea basin, an inter-cratonic basin located on Early Proterozoic crust of the East European Craton (Figure 4-8). More details can be found in /Hakami et al, 2002/. Studies indicate the existence of an important and ancient fracture set and provide further support for the suggestion that the Swedish coast in the Äspö region is fault controlled.

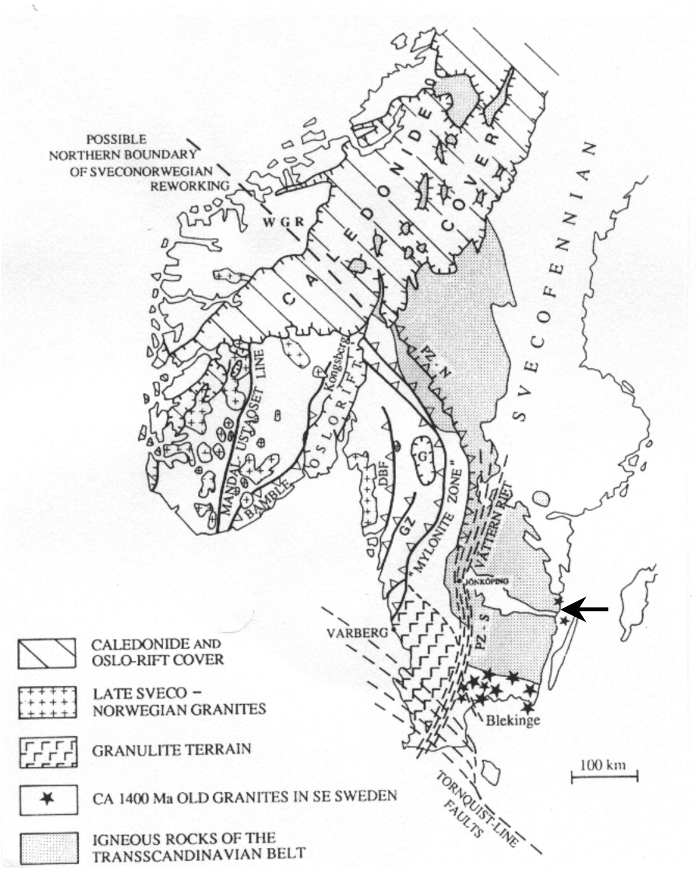


Figure 4-8. *Structure of the southwest Scandinavian domain. Of particular note is the major southeast-northwest trending crustal fracture zone, the Tornquist-line, and the occurrence of a major north-northeast south-southwest trending rift, the Vättern Rift, which parallels the Swedish coast in the vicinity of the ÄHRL site (marked with an arrow). From /Gorbatschev and Bogdanova, 1993/.*

The tectonic evolution of the Äspö region shows that through its long history the Scandinavian plate has been subjected to a series of major deformational events resulting in faults and joints. Thus, we can expect the current stress field to be partly a function of the rock structure. Accordingly, numerical modelling was performed to study the possible local effects at the Laboratory.

4.5 Numerical in situ stress modelling

Several published examples of such modelling were studied and then numerical modelling of the structures in the Äspö area was performed for the Test Case – as an illustration of a possible approach /Hakami et al, 2002/.

4.5.1 3DEC modelling of the Test Case area

A 3DEC model (a 3-D Distinct Element Code) was generated to illustrate the use of numerical modelling in the process of in situ stress analysis. The geometry of the model is shown in Figure 4-9. The rock mass is modelled as a continuum, but also incorporated are the major fracture zones, modelled as planar, single fractures. This is a simplification of the real geometry, but detailed knowledge concerning the fracture zone geometry and mechanical properties will normally not be available, and so a single plane with Coulomb slip properties representing a fracture zone is a pragmatic approach.

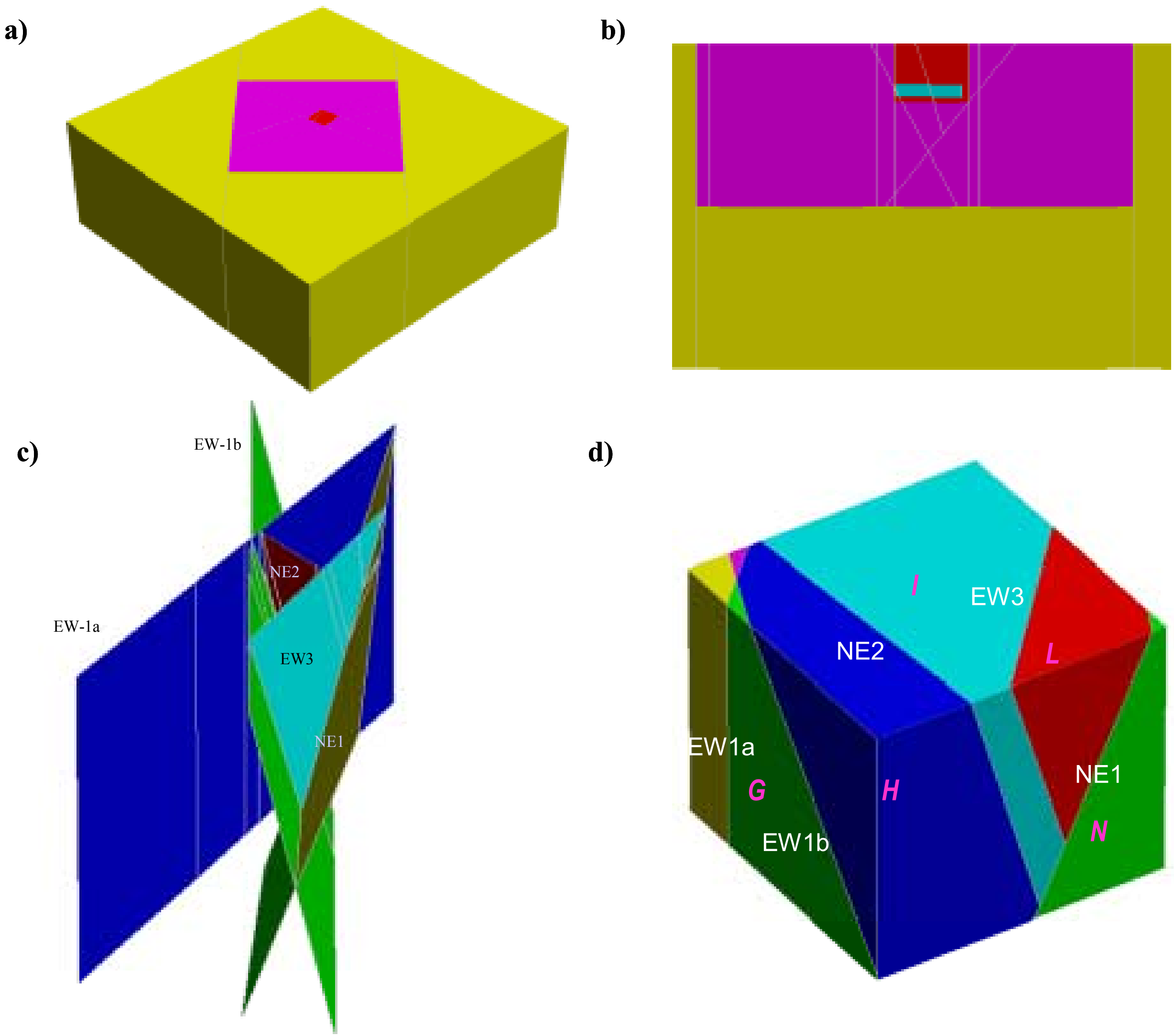


Figure 4-9. *3DEC model for the Test Case.*

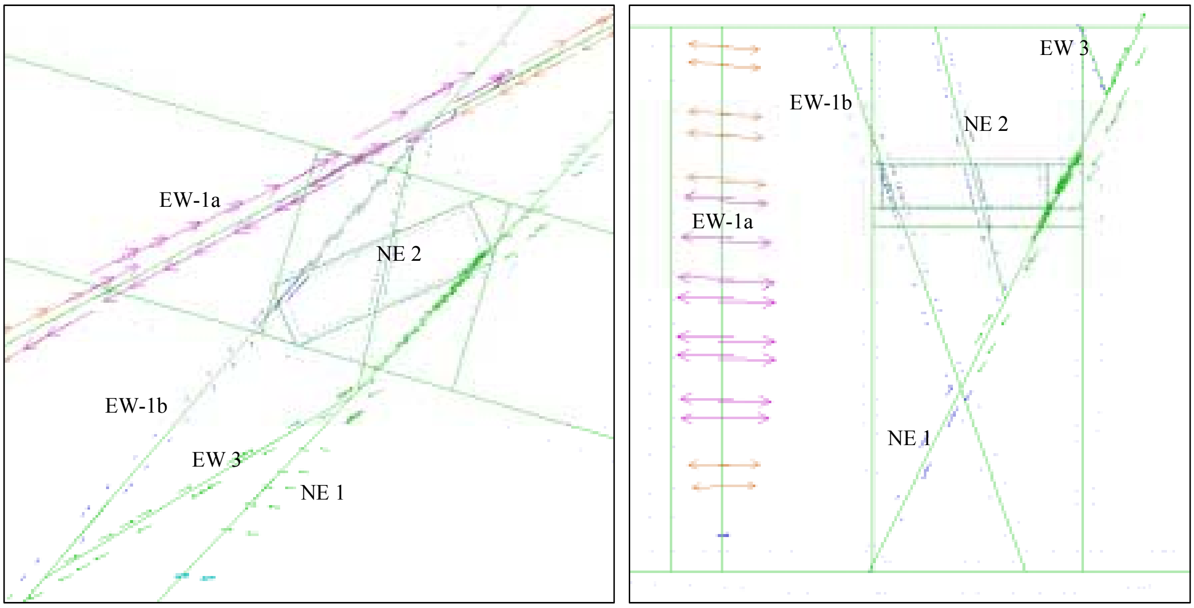
a) The whole block model (10,000 x 10,000 x 3000 m), with inner regions discretised.
b) Vertical section through centre of 3DEC model. The 550 m block is the red area. The purple region determines the size of the largest fracture zones.
c) Five major fracture zones in the ÄHRL are simulated, as labelled.
d) 3-D view of the 550 m block. The rock blocks between the fractures were labelled in the Test Case project (see the pink letters).

Making the assumption that the latest tectonic movements (fracture movements) in the area were caused by a regional stress field similar to that of today, the stresses in the model may provide an indication of the stress field variations prevailing. The present tectonic loading was simulated either by initiating a stress field in the model or by moving the model boundaries (horizontal compression in the NW-SE direction). The fracture shear movements and stress redistributions required to establish equilibrium were calculated. Depending on the fracture zone properties and the orientation of the applied stress field, different model shear displacements were developed. Further work is required to establish the best modelling method.

Figure 4-10 shows an example of results from the Test Case modelling. The fracture zone properties were the same for all fracture zones, but the shear displacement is larger for the larger zone with unfavourable orientation. The blocks inside the 550 m model block have moved upwards due to the compressional stresses, and this causes changes in the stress field when passing the fracture zone, See Figure 4-10.

Modelling alone cannot be used to establish the absolute magnitude of the stress field but, using primary and measurement data, the model can be calibrated to the region and to the site. The results can then assist in interpolating between boreholes where stress measurements have been taken. Thus, by these means and changing the model parameters and performing sensitivity studies, modelling can help in the estimation of the possible stress variation in the area.

Alternative geological conceptual models (different fracture zone geometries, mechanical properties and loading conditions) may also be analysed and compared (Figure 4-11). The example in this Figure, comparing results with and without depth-dependent fracture stiffness and E modulus, illustrates that the modelled maximum principal stress values do depend significantly on these assumptions at shallow depths, 0–200 m, but not so much at the greater depths where a repository is anticipated.



a) b)

Figure 4-10. *Example of calculated shear displacements along fracture zones in a) horizontal section and b) vertical section. The largest shear occurs for the fracture zone EW-1a (pink arrows). The movement is strike-slip and dextral in orientation. The rock wedge located between fracture zones NE1 and EW1b has moved slightly upwards. /Hakami et al, 2002/*

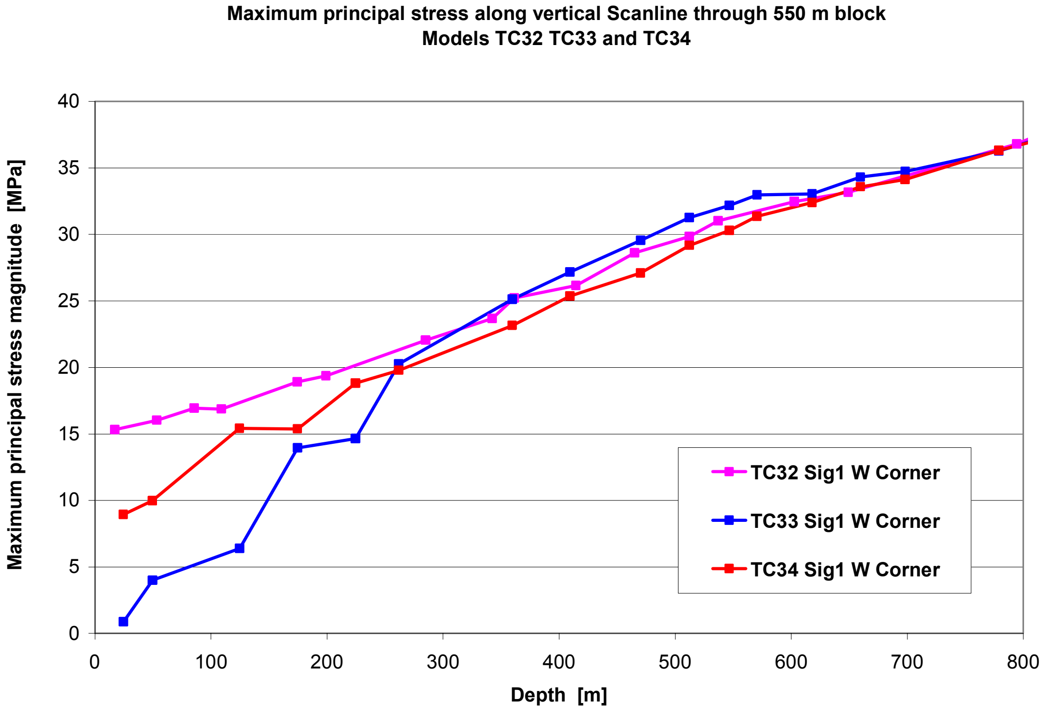


Figure 4-11. *Calculated maximum principal stresses along a vertical line for three different models. TC32 has a constant E-modulus through the whole block, TC33 has a fracture zone stiffness that depends on depth, and TC34 has an E-modulus as a linear function of the depth in the rock mass. Boundary conditions were similar. /Hakami et al, 2002/*

To select the model providing the best representation of the real stress field, the model results should be compared to stress measurement data from the site. As an example, Figure 4-12 presents calculated stresses along a borehole, together with actual measurement results from the same borehole.

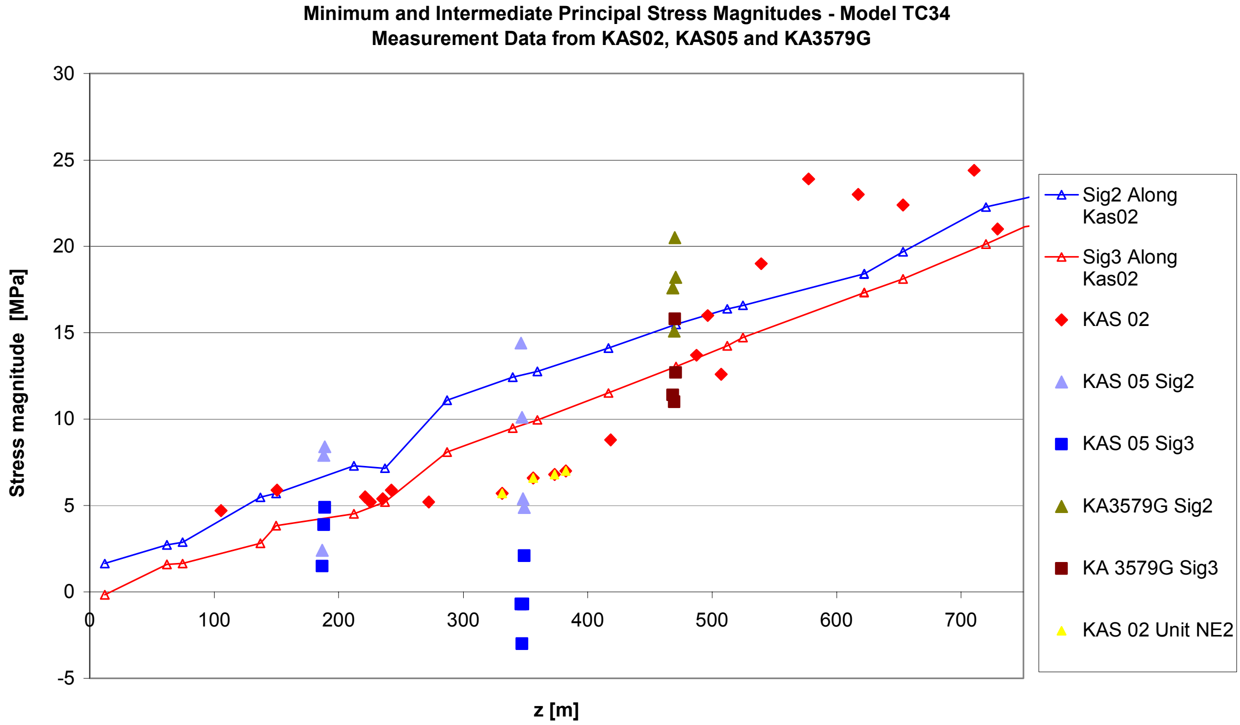


Figure 4-12. *Calculated minimum and intermediate principal stresses (the connected points) along a line parallel to borehole KAS02 in one of the 3DEC models and measurement data from the Test Case exercise (not all data from the ÄHRL). Data from KAS02 are hydraulic fracturing measurement results (minimum horizontal stress) and data from KAS05 and KA3579G are overcoring measurement results. Data points inside the fracture zone NE2 are marked. /Hakami et al, 2002/*

4.5.2 Conclusions from the stress modelling for the Äspö HRL Test Case

The available stress measurement data were not sufficient to identify a clear pattern in the stress variation at the Äspö Test Case site. This was because there was no obvious consistency between results from different methods, and there was a large uncertainty in the interpretation of part of the data.

Hydraulic fracturing measurement data were used, together with overcoring measurement data and 3DEC modelling, to make an estimation of the minimum and intermediate principal stresses. Hydraulic fracturing measurement data for estimating the maximum principal stress were judged too uncertain to form a basis for the stress estimation. This judgement was founded on theoretical arguments from the literature.

Overcoring measurements were used to predict the maximum principal stress. Additional overcoring data from the 300–400 m depth interval, at locations outside fracture zones, would be helpful in supporting/rejecting the final stress model.

The 3DEC modelling indicated a possible explanation for a change in minor and intermediate stress magnitudes when crossing a minor fracture zone (NE2) inside the Target volume. This is an upward movement of the blocks H, I and L (between EW1 and NE1) caused by the horizontal stresses (horizontal compression) in the region. The 3DEC models further suggest that a change in stresses occurs when passing the fracture zone EW1. This zone has undergone a dextral (sub-horizontal) slip movement. Figure 4-13, presents a simplified CAD-model of the fractured zones in the model and the Äspö HRL tunnel system.

The mean orientation of the maximum principal stress is predicted with a fairly high degree of certainty because both the regional stress pattern and the site-specific measurements indicate the same orientation: 150° ± 10° trend (azimuth) and 0° ± 10° plunge (dip). This estimation applies to the whole 550 m block away from the fracture zones. The local spatial variation around the mean is predicted to be within ± 15° for both trend and plunge.

The orientations of σ_2 and σ_3 are lying in the plane perpendicular to σ_1, but plunge and trend in this plane are uncertain for two reasons.

- The hydraulic fracturing measurement technique cannot provide any information on this matter and therefore the available data are few and also the overcoring data are inconclusive.
- The intermediate and minor principal stresses are expected to be of similar magnitude at some depth and, in this case, the stress magnitude is similar in all directions in the plane perpendicular to the maximum principal stress direction. σ_2 and σ_3 then have then no clearly defined directions.

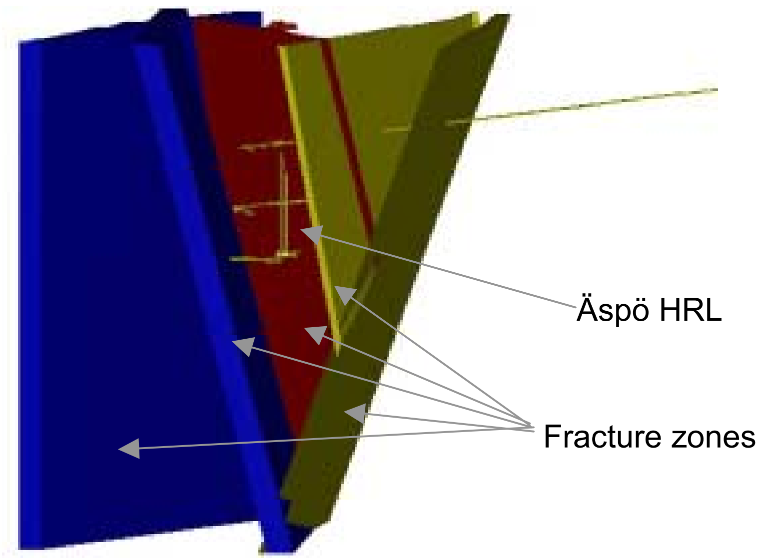


Figure 4-13. *Simplified CAD-model of the fracture zones and the Äspö HRL tunnel system.*

4.5.3 Practical aspects of the numerical modelling

The Test Case also provided practical experience with the use of a discrete element code for modelling the state of stress. Attention to many of the associated details is required; among these is establishing the procedure for importing the co-ordinates describing the fracture zones from the CAD-based geometrical model.

The calculated stresses represent a large amount of data, and the plotting of stresses, along scanlines simulating stress values along a borehole, or as contour maps in plan or section, provide helpful overviews. There is, however, a need for further post-processing of the results if mean stress profiles are to be presented. It is of interest also to explore further the possibility of exporting the stress values to geostatistical tools for the calculation of mean trends, and the identification of areas that indicate significantly different principal stress magnitudes or orientations due to the influence of fracture zones on the stress state in the rock mass.

4.6 Stress model approach – recommended method for the development of a stress model at a repository site

Bearing in mind all the factors that have been discussed, the information in the references, and worldwide experience, the stress model approach illustrated in Figure 4-14 was developed. This is an integrated approach combining stress measurement information, geological factors, numerical modelling results, and consideration of the uncertainties involved.

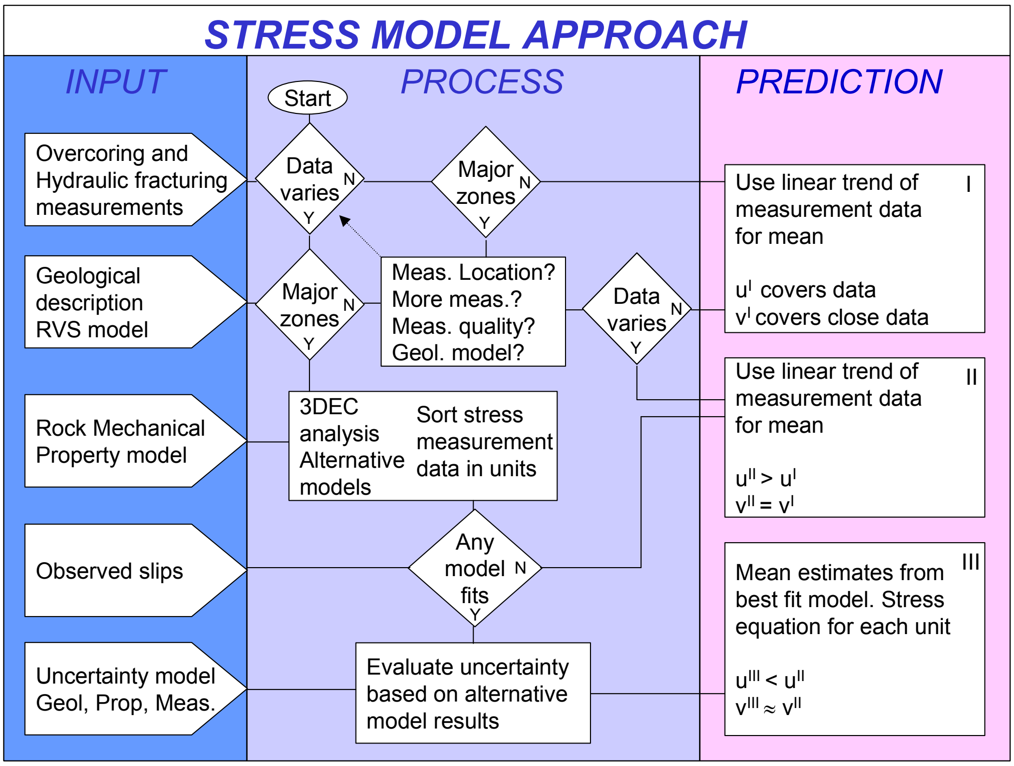


Figure 4-14. *Flowchart of the stress model approach (the u term refers to uncertainty of the data; the v term refers to the spatial variability of the data). /Hakami et al, 2002/*

4.6.1 Do the stress measurement data show variation with location?

The first step in the suggested approach in Figure 4-14 is to determine whether the site specific information indicates that there is a significant variation in stress within the area of interest. 'Variation' is understood here to refer to an in situ stress field that has a non-linear increase in stress magnitude with depth, or has different stress fields at different parts of the site, or in different rock domains.

4.6.2 Do major fracture zones exist in the area?

For Swedish geological conditions, the dominating mechanism explaining variation in the in situ stress field is likely to be the influence of fracture zones, not only the sub-vertical zones, but also the gently dipping and sub-horizontal zones. The next step, therefore, is to study the geological model of the site and determine whether the volume in question is intersected by any major fracture zones. For the design of a future repository, a location without major fracture zones will be preferable for design and safety reasons. However, since the repository will require a large horizontal extent, it may not be possible to locate it inside such an area.

If there is a variation in the stress data and there are also intersecting major fracture zones, the stress field should be analysed further with the help of a three-dimensional analysis tool. If there is no variation in the measurement data but there are major fracture zones in the area, the measurement data should be evaluated further before the estimation is made.

If there is no variation and no major fracture zones, the estimation may be established without further analysis. This estimation will then consist of principal stress values linearly increasing with depth, unless there is reason to consider the influence of any depth-dependent parameters. The same estimation should in this case be valid for the whole estimation volume.

4.6.3 Evaluation of measurement data

The measurement location in relation to the different rock units in the geological model should be determined. This is easily done by using the RVS system and the SICADA database. Also, all possible information should be gathered from the stress measurement programme, including the raw data if possible. Raw data, such as strain records during overcoring, biaxial test results and core descriptions, are valuable if the uncertainty of a single measurement needs to be examined. If a certain datum deviates considerably from the overall pattern, this measurement should be checked to see if there are any measurement-related explanations, or if it should be considered as a true stress value. This is discussed further in Chapter 5.

4.6.4 Three-dimensional modelling

When a varying in situ stress field is anticipated, a three-dimensional numerical analysis is a means to estimate the stresses in the volumes (rock blocks) in which no stress measurements have been performed or where the measurements are very uncertain. By using the geological model of the site that indicates the fracture zones, the expected stress distribution may be estimated. The main hypothesis behind such analysis is not only that the fracture zones have different stiffness properties, but that they are in equilibrium and previous slip is the main cause of the current stress variation.

It should be noted that the geology outside the volume being considered is also of interest. The structural model should preferably be large enough to include the whole extent of the fracture zones intersecting the area and also show the extent and orientation of the structures towards which they terminate. If there are other underground sites in the region, or if there are stress measurements in the region (but outside the site), they should also be considered.

4.6.5 Is there an appropriate model?

The most difficult aspect of the stress model development is the comparison between different modelling results and the input data. The stress field in the models will depend on loading sequence and the geometrical and strength properties of the zones. Therefore, the different assumptions of each model should be compared with the input information concerning rock mass and fracture zone mechanical and geometrical properties. The aim of this step is to judge which one of the possible models, i.e. which one of the different combinations of influencing factors (geometry, zone property, boundary conditions), best represents the actual stress field.

In this step, one should consider all the site specific input available and the current knowledge and understanding in the relevant subjects as reported in the scientific literature. Even though this comparison and judgement may appear to be a somewhat subjective step, it is an inevitable one and acceptable as long as the reasoning is documented properly, so that subjective decisions can be traced back by reviewers.

One way to quantify the difference between a certain stress model (for example a certain linear function with depth) and the measurement data is to calculate the sum of the differences point by point, or to use some other similar statistical method. However, it is recommended that the comparison is also made using judgement regarding the qualitative fit. This more subjective method is preferred because there are many different factors to consider in the comparison and some of them may not be correctly incorporated if only a calculated 'best-fit-factor' is used.

If several models provide a similar fit to the measurement data, the model that seems most reasonable in terms of the rock property assumptions made should be selected as the model for the estimation. If models with a similar fit to the measurements also indicate similar stress fields in areas outside the measurements, then there is no need to make any selection between the models – the aim of the modelling is to support a certain stress estimation and not to establish any 'true model'.

Simple models are to be preferred over complicated models, i.e. a simple linear function should be used if there is no other model that better explains observed features. It is not recommended that higher order terms be incorporated through non-linear curves (exponential, logarithmic or polynomials) fitted directly to measurement data and used as 'models', unless there is a mechanical explanation for the observed stresses. At the measurement location, the measurement results should of course be regarded as representative of the real stresses if the data are considered reliable.

It is important to note clearly within which area of the region and, even more importantly, within which depth interval a certain estimation is made. The stress models must be used carefully and not for estimation deeper than the deepest stress measurement.

4.6.6 Uncertainty parameter estimation

The sources of uncertainty in stress estimation are considered in two categories via the uncertainty u-parameter and the variability v-parameter. These parameters will determine two 'ranges' for each estimation (± u and ± v, respectively).

u-parameter

The first uncertainty range corresponds to the uncertainty in the estimation of an average principal in situ stress value (magnitude, trend and plunge). Here 'average' refers to the general stress situation expected within a certain specified rock volume, a 'rock unit'. For example, one rock unit could be the rock mass consisting of the same rock type and lying between the surface and three major deformation zones. In this volume, the geological model for the site states that the conditions should be fairly homogeneous. Further, the u-parameter covers the uncertainties in the geological model, the uncertainty concerning tectonic regimes prevailing, lack of measurements, measurement bias etc.

In the site investigations, the rock unit volumes will probably be of a large size, i.e. several cubic kilometres. A single principal stress value existing in the rock mass is defined as the load divided by the area over which the load is acting. The 'scale' of the stress parameter must therefore be chosen depending on the problem, and we have chosen to consider the 'local stress size' to be a loaded volume that would determine a stress measurement result, i.e. a local borehole scale volume in the order of 0.1–1 m^3. If all 1 m^3 size volumes in a certain rock unit are considered, the mean principal stress value in this rock unit is the mean value of all the 'local' stress values.

If there were a perfect measurement method that was able to measure stress on the large scale and with excellent accuracy and precision, then the uncertainty parameter u would become zero in rock units where such measurements were made. However, there are many different sources of uncertainty when performing and interpreting stress measurement results, and the u-parameter is also used to reflect this uncertainty.

The aim of the stress model is to minimise the u-parameter in areas where stresses are important.

v-parameter

The second uncertainty range, v, corresponds to the expected spatial variability of in situ stress around the average (magnitude and orientation). The cause of local variability of both principal stress magnitude and orientation is the inhomogeneous character of the rock mass at all scales. Even the most competent rock mass will include fractures of some size and heterogeneities, such that a 1 m^3 volume of rock should not be expected to have exactly the same stress as all the other 1 m^3 volumes in the same rock unit (*cf.* Figure 4-4).

As opposed to the u-parameter, the v-parameter is not meant to reflect the lack of knowledge or lack of data, but the expected actual variation in the parameter from point to point inside the volume it represents. Therefore, the value of the v-parameter will be dependent on the scale being considered. In a very large rock unit, the distance between different points will be larger and also the mechanical properties of the rock mass may be expected to vary more.

The larger the definition volume for 'local stress', the smaller the expected v-parameter becomes (as the local volume size approaches the size of the rock unit, v approaches zero). Also, the v-parameter is not necessarily expected to change with increased knowledge, increased number of measurements or improved measurement techniques. The quality of the stress model is not reflected in the v-parameter. The v-parameter is also not related to the assessment of the constructability at a site because small scale stress variation will not influence the design.

Use of the u and v parameters

A problem is establishing the cause of the scatter in stress magnitude values. If all the scatter in values (at the same depth) is caused by the local heterogeneity and fractures, then the scatter could be used directly to estimate the v-parameter. However, some of the scatter could be due to measurement errors. In particular, the non-systematic error components in measurements are impossible to separate from spatial variability components. The only way to obtain insight into this is to rely on research performed under known conditions.

Judgement must be used when finally selecting the values for u and v. They can be expressed either in terms of the stress units (MPa), or they can be expressed as a percentage of the average. One example of the suggested way of describing the stress estimation, say $\sigma_1 = 20$ MPa, with u = 20% and v = 10%, means that the average maximum principal stress in the rock unit is expected to lie in the range 16–24 MPa and that the local variation around the average is in the range 14.4–17.6 MPa (if the average is 16 MPa, 16±1.6 MPa), or in the range 21.6–26.4 MPa (if the average is 24 MPa). The predicted total possible range for a single σ_1 measurement value is thus the interval 14.4–26.4 MPa.

In cases where the rock units for estimation are located such that the stress variation with depth inside the unit cannot be neglected, the estimation of the average stress may be expressed as a function of the depth. In this case, the uncertainty and variability, if expressed as percentages of the average stress, will also be depth dependent.

4.7 Summary points for the stress model approach in the Site Descriptive Model

4.7.1 Estimating the rock stress

The main mechanism controlling the stress magnitudes in Sweden is plate tectonics, causing the stress field to show similarities to most parts of north-western Europe, having a maximum principal stress trend of SE-NW.

The **orientation** of the stress field is largely determined by the relative movements of the plates. However, the stress orientation may also be influenced by the presence of large regional weak zones, such as the Tornquist deformation zone that lies between Sweden and Denmark. The strike of the Tornquist deformation zone is parallel to the maximum principal stress as observed in central and southern Sweden.

The **magnitude** of the stress is more difficult to estimate, but the general pattern is an increase in magnitude with depth at least for the upper kilometres. To determine the stress magnitude at a certain site and depth, with reasonable certainty, stress measurement should be used.

4.7.2 Stress model methodology

A methodology for building a stress model has been proposed /Hakami et al, 2001/. It involves different steps starting with a preliminary stress estimation, followed by steps for interpreting site specific information. If the stress pattern and structural geology of the site are complex, including major fracture zones intersecting the area, numerical analyses of the stress field are recommended.

Different structural models (i.e. alternative geological concepts) should be analysed by numerical simulation to provide possible explanations for observed stress patterns. The orientation of fracture zones with respect to the applied stresses determines the direction of fracture zone deformation. Stress measurement results and observations from the site concerning slip directions must be used in the evaluation of the modelling.

The mean orientation for the maximum principal stress may be predicted with a high degree of certainty because both the regional stress pattern and site measurements can be used. The same general orientation, with a trend of 135°–165° and plunge of 0° ± 10°, is expected for the whole of central and southern Sweden, but local deviations caused by topography and faults could exist. This estimation applies to rock mass blocks away from major fracture zones. The local spatial variation around the mean can be predicted, based on measurement data.

The confidence in the estimation of the stress magnitudes will be dependent on the measurement results and the complexity of the site. Inside and proximate to a major fracture zone, both the stress magnitudes and stress orientations are expected to vary from point to point. The estimation of the mean stress inside a fracture zone is therefore more uncertain and the predicted local variation will be larger.

The stress estimation should include quantitative estimation of the uncertainty and the variability. Two parameters, u for uncertainty and v for local variability, are proposed. The aim of the stress model is to reduce the u-parameter to the value required in those rock units where the stress level is of importance for the design and safety assessment.

Recommendations on the stress measurement programme are given in SKB TR-01-29.

5 Documentation and visualisation

5.1 Overview of the SKB site modelling

5.1.1 The process of site understanding

The Site Investigation will be carried out in steps, with data being produced in batches for each site. Consequently the Rock Mechanics Site Descriptive Model will be developed in a stepwise manner as well. It is expected, see /SKB, 2001/, that at most two major revisions of the integrated model (see Figure 1-2) will be implemented during the initial site investigation stage. During the complete site investigation stage, the need for model revisions will be considered after each batch of investigations. Updates of the Rock Mechanics Description will be co-ordinated with the overall revision of the Site Descriptive Model. As regards the rock mechanics description, there are four types of input which would trigger the need for an update.

1. A significant update of the geological and/or geometrical model.
2. Significant new data from laboratory testing of the mechanical properties of intact rock or fracture specimens.
3. Significant new data from in situ stress measurements.
4. Fundamentally different conceptual understanding based on new findings within the rock mechanics community (cf. Chapter 7).

The updating of the Model could be scheduled with respect to the most suitable timing relating to data retrieval at the Site, see /SKB, 2001, TR-01-29/. Data from in situ stress measurements by means of overcoring will be available at an early stage of a site investigation data batch, but all other rock mechanics data are likely to be available towards the end of a data batch. Also, the receipt of data may overlap: if the next site investigation data batch is already in progress, the rock mechanics modeller may additionally have access to even more recent geological information. However, it is an over-riding principle of the Site Descriptive Modelling to strive for consistency between the geological, rock mechanics, hydrogeological and hydrogeochemical descriptions.

It is of fundamental importance to maintain an adequate record of the input data, and to take care to use the relevant version of, for example, the geological description. If new geological data appear, these data should be considered primarily by the geological modelling team, and not be used directly to develop alternative geological models for the rock mechanics analysis.

5.1.2 Management of model revisions

The development of the Site Description is a process which involves a number of geoscientific disciplines (cf. Chapter 1). All geoscientific models are part of a chain of sub-models that are mutually inter-dependent. Systematic management of Model versions and revisions is therefore of fundamental importance for Quality Assurance.

Any significant change in a model, such as the inclusion of newly discovered deformation zones or a significant alteration of the properties of an interpreted zone, will trigger the creation of a *major model revision*. The models are common to all geoscientific disciplines and will consequently affect a large proportion of the various sub-models. The number of major revisions should therefore, for practical reasons, be kept to a minimum, commensurate with the modelling objectives. Minor changes, such as an update of a set of parameters, that does not significantly affect neighbouring disciplines, will trigger a *minor model revision*. It is anticipated, in rock mechanics modelling, that a major revision might be required after each data batch; whereas, minor revisions can occur using data from the current and previous batches.

All model revisions will be kept in a model database – so that full traceability of the creation of the models is achieved and each revision can be inspected if the circumstances so require. Access to older revisions will be restricted to ensure that only the latest approved revision is used in subsequent modelling.

Uncertainties in interpretation and data might require the creation of alternative models. These are handled in the same way as the main models, using revisions, and the alternatives are stored in the same model database. Model versions, and their alternatives, are differentiated by a naming convention, illustrated schematically in Figure 5-1 (after /Munier and Hermanson, 1991/).

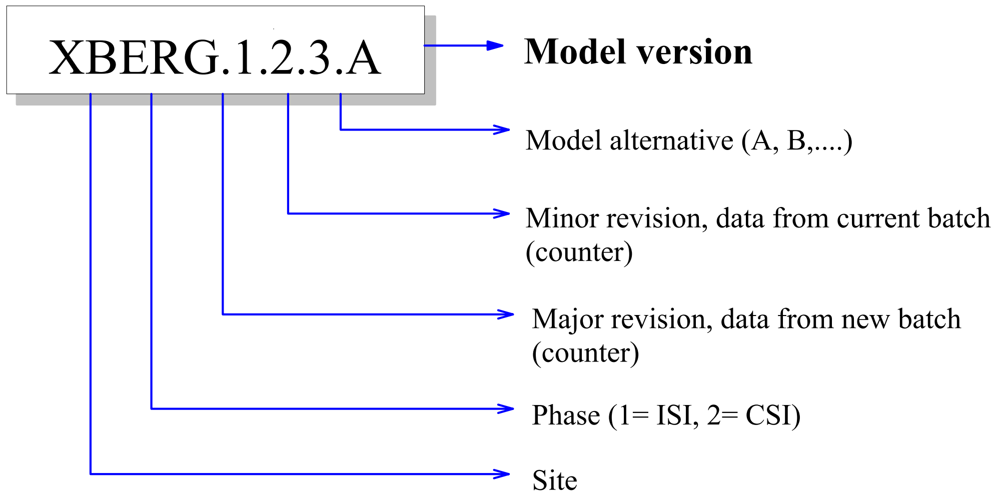


Figure 5-1. *Principles for nomenclature used for model revision. (ISI = initial site investigation, CSI = complete site investigation). Note that a simplified notation is used in /SKB 2001, TR-01-29/ only indicating the phase and major revision.*

5.1.3 Modelling strategy

SKB has presented a methodology to construct, visualise and present geoscientific descriptive models /SKB, R-01-15/. The descriptive model is a cornerstone for the understanding of the investigated site and forms a basis for subsequent planning of the repository layout, as well as for Safety Assessment studies. The methodology has four main components as follows.

5. Construction of a geometrical model of the interpreted main rock components (fracture zones and different volumes between fracture zones) at the Site.
6. Description of the geoscientific characteristics of the rock components.
7. Description and geometrical representation of the geometric uncertainties in the model.
8. Quality system for the handling of the geometrical model and the associated database.

The main tool for interpreting and visualising geometrical information is the Rock Visualisation System (RVS) – a 3-D CAD software package developed by SKB. The modelling approach is outlined in /SKB, 2000b, TR-00-20/ and /SKB, 2001, TR-01-29/, although additional method development is needed.

The modelling work starts out from the primary data measured at the site. Fundamental for quality assurance is that data for modelling are taken from the SKB Site Characterisation Database (SICADA). The data stored on SICADA, see Figure 5-2, are not only the primary results (measurement data, directly calculated values), but also the information relating to the activities that generate the data, as well as activities that might affect the measurement data. Each activity is documented in terms of where and when it was executed. The results are then linked to each activity, along with particulars on who executed the activity.

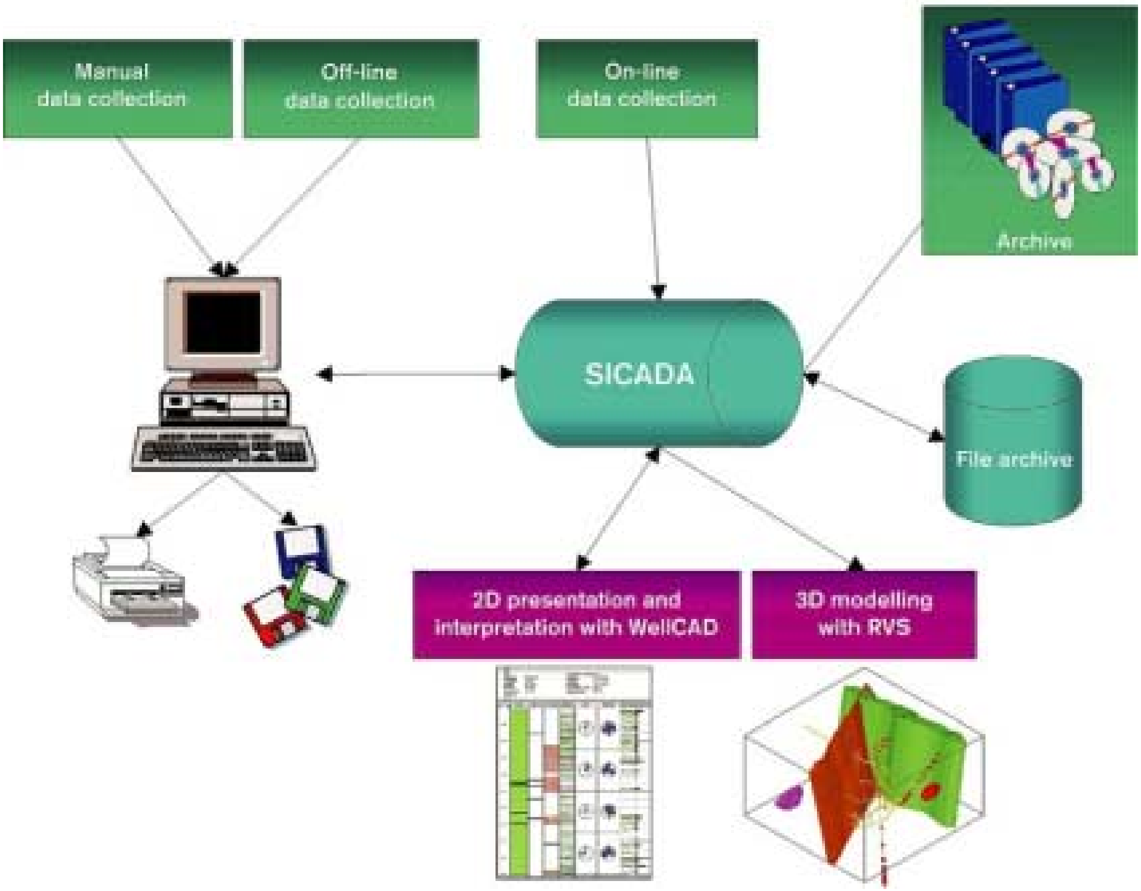


Figure 5-2. *SKB's database SICADA with associated functions (from Figure 6-2 in TR-00-20).*

There are two important rules for SKB's data management and use of SICADA. Firstly, only quality-controlled data may be stored in SICADA. Archived information must be maintained in accordance with quality assured procedures. Secondly, only data from SICADA may be used for interpretation, analysis and modelling of the investigated sites. Retrieval of data is followed by a log file, by means of which the same data set can be retrieved again from the database if necessary. This makes the origin of the data set unambiguous. In this way, the traceability of data processing can be assured – from original data source to finished results.

The geometrical modelling (Steps 1 and 3) is mainly the concern of the geological modelling approach and is described in a specific method document /Hermanson et al, 2002/. Other disciplines will also have an impact on the geometrical modelling results.

All disciplines are faced with describing the discipline-specific rock conditions in the three-dimensional model domain (Step 2). The main challenge is how to extrapolate information from the surface and measured in a few boreholes into three-dimensional form in the model volume. There are always uncertainties in interpreting measurements and rock parameters which vary in space. The three-dimensional description needs to describe the parameters with their spatial variability over a relevant scale and to describe the uncertainty in this description. In addition, it is also necessary to describe the confidence in the model predictions.

The Site Descriptive Model is thus not restricted to the geometrical representation in RVS, nor to the parameter distributions in the rock units.

An essential component is the documentation of the Model. The documentation relies on the following.

- A description of the information flow from primary data in SICADA and use of other discipline models into the final geometrical and parameter distributions in the Site Descriptive Model.
- A description of how uncertainties were estimated.
- An account of the arguments in support of and against the confidence in the model, or parts of the model.

Hence, the description of uncertainty is an integrated part of the Site Description.

5.1.4 Tools

To facilitate modelling and administration of geological interpretations and data, SKB has developed a CAD based tool for visualisation (Rock Visualisation System, RVS). RVS has three connected, but distinct uses.

- Creation and maintenance of 3-D geometries such as deformation zones, lithological boundaries, ground surface and tunnels.
- Storage and maintenance of interpreted data that are connected to the geometries in the model but not stored elsewhere (e.g. SICADA).
- Visualisation of 3-D interpretations and data. The visualised data may stem from the model or from other sources from which the model, or parts thereof, have been derived (e.g. SICADA, GIS).

The basic principles for geometrical modelling, not necessarily restricted to RVS, are given below (see /Munier and Hermanson, 2000/ for a detailed description).

A model is considered as a collection of interpreted geometries. Each geometry within the model is defined by its co-ordinates. In addition, each object contains a set of parameters that defines it, for example a geological feature (i.e. deformation zone, rock unit, etc). Other volumes might be defined that cannot be restricted to specific geological features, such as underground openings. Therefore the notation ‘object type’ has been introduced to cover a wider range of objects. An application for rock mechanics is schematically illustrated in Figure 5-3. Parameters are assigned to geometries by a set of dialogs.

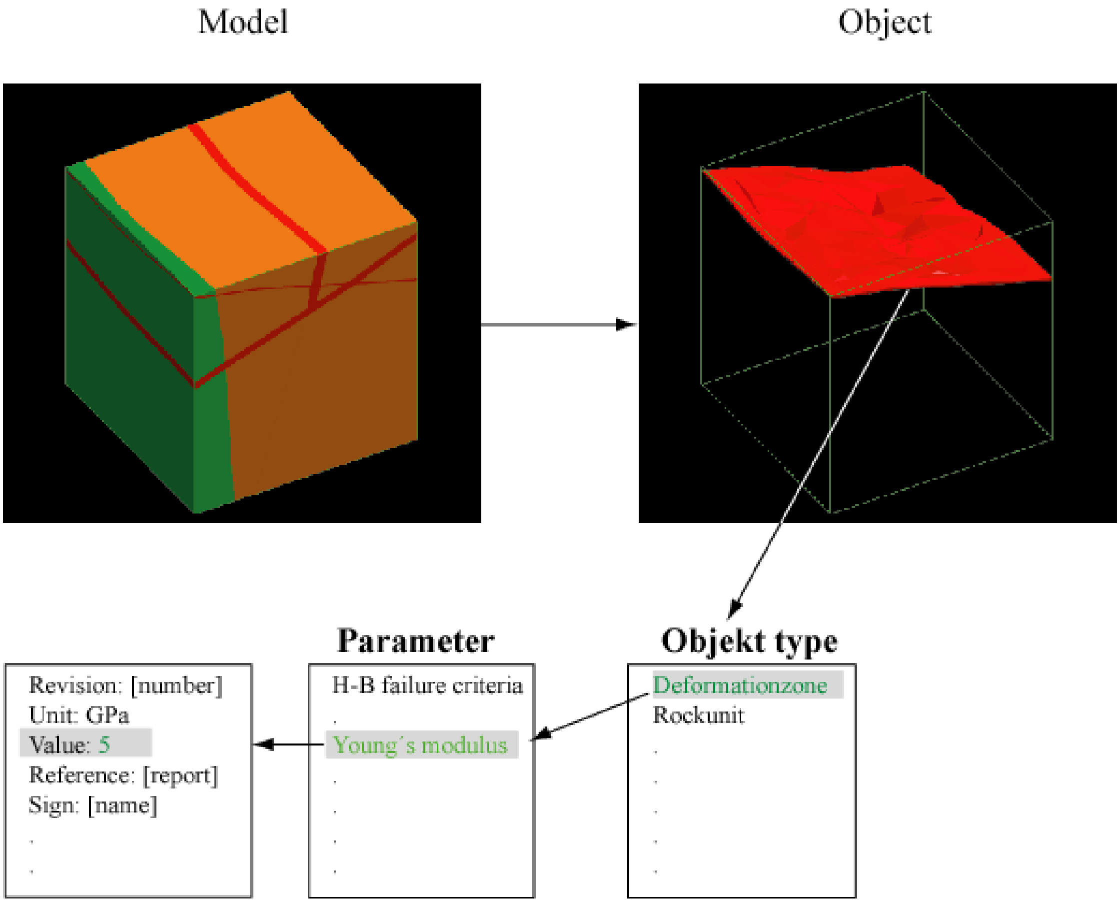


Figure 5-3. *Schematic illustration of the relation between model, object, type of object and parameter.*

Objects in the RVS model might have a complex geometry that, for practical or computational purposes, has to be simplified for use in numerical codes outside RVS. The principles of geometrical modelling can still be maintained with regards to traceability. The feedback to RVS will rarely, if ever, consist of alternative geometries, but rather as a set of calculated values or distributions. However, a calculated entity that varies continuously in space might still be imported to RVS as, for example, a set of iso-surfaces that outline blocks with specific properties.

Any other numerical tool could be used, so long as modelling of parameters in a rock unit can be defined in space by the co-ordinates. For example, numerical simulation of the stress field, as outlined in Chapter 4, is traceable by using the co-ordinates for nodes or central points of elements in the numerical model. Also, block modelling that visualises spatial information has utilised co-ordinates for each parameter value. It must, however, be considered that the RVS operates in a CAD package. This provides flexibility in presenting locations, for example for data sampling along a borehole.

Other numerical tools, especially numerical codes for calculating stresses in accordance with the description in Chapter 4, may not have such high flexibility for presenting the spatial distribution of results – because the grid used in the model must initially be adjusted to the geometrical model. There may be specific rules for how the grid in the numerical code can be established, e.g. the width-length ratios. Applied to a site with a complex geometrical model, the grid could be complex as well, as illustrated in Figure 5-4. The design of a grid for numerical modelling or voxels for block modelling must, as a minimum requirement, consider the need to be able to produce scanlines or contour maps to illustrate the modelling results to the user, who could be the designer.

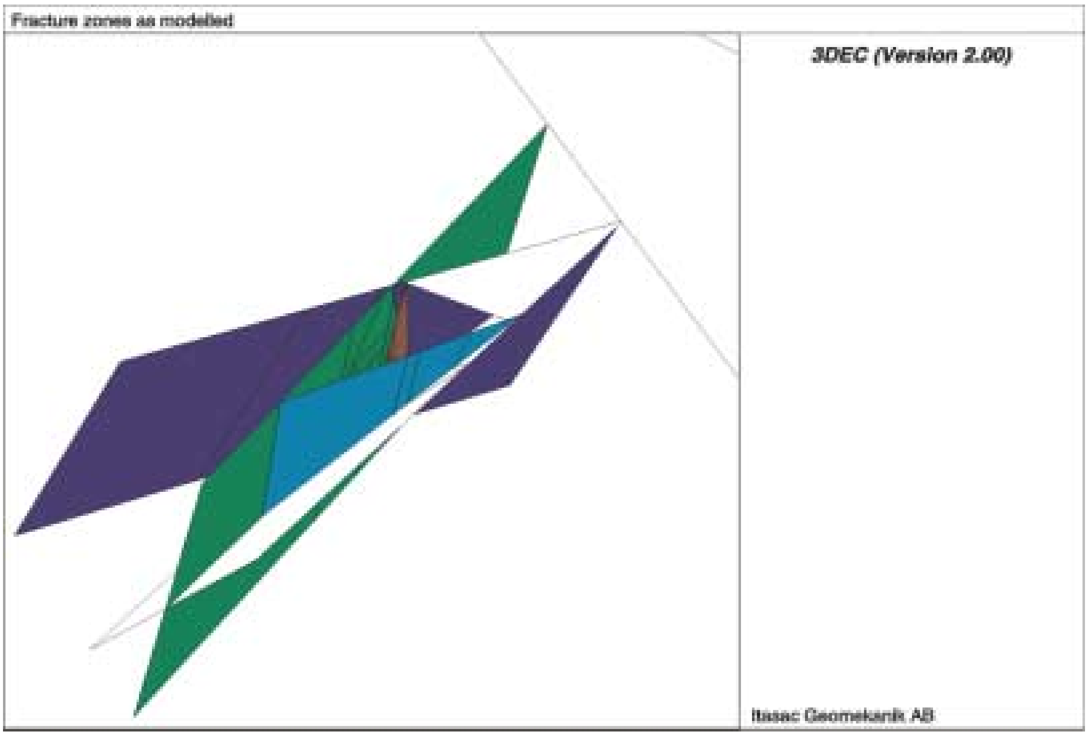


Figure 5-4. *The geometrical model for the Äspö Hard Rock Laboratory imported to 3DEC. The grid for discrete element modelling had to be adjusted to the complex geometry, /Hakami et al, 2002/.*

5.2 Traceability

5.2.1 Technical Auditing and Quality Assurance procedures

SKB follows the quality principles as defined in the standard ISO 9001:2000. There are many aspects of concern in the Quality Control System to be considered for the Site Investigations and the modelling within the various geoscientific disciplines. Some key aspects are, at least, but not necessarily limited to,

- that selected standards (for example for testing, established routines and other procedures that have been described in the overall programme) are followed,
- that results are systematically checked in appropriate steps,
- that there is a systematic way of monitoring any error or mistake of significance for the results and for the overall reliability for the SKB Site Investigation Programme, and
- that there are methods available for improvements, if necessary.

The term 'Technical Auditing" (TA) means examining the technical content to establish if it is adequate for the purpose. Quality Assurance (QA) refers to checking that pre-determined procedures are followed and to reviewing non-conforming results. However, the more recent version of the standard ISO 9001 contains both components because the emphasis is on product realisation, which requires both TA and QA. For the development of a Rock Mechanics Model, the key aspects are as follows.

- Does the Modeller understand the Geological Model?
- Is the rock mechanics conceptualisation of the Site realistic?
- What kind of instinctive assumptions based on experience are made by the Modeller – and does s/he understand this?
- Are reasons for scattering in input data understood?

In summary, the key word for both TA and QA in the case of rock mechanics modelling is 'traceability', of data, processes and results, and of the theories, conceptualisations and assumptions that form the basis for the modelling methods used and the conclusions drawn.

5.2.2 Control of input data

Only models from the model database can be used for rock mechanics modelling, and only data from SICADA can be used. In both cases, the input information quality is checked and approved.

All data that are imported from SICADA for the RVS modelling are recorded in a file. All files used and visualisations are stored as a project in the Microstation system. The project is protected from unauthorised users to secure full safety and traceability /Munier and Hermanson, 2001, R-01-15/.

The data from SICADA used for the specific rock mechanics modelling are of two types.

1. Results of in situ stress measurements.

2. Results of laboratory testing, whether intact rock or fracture samples.

The methods used for data collection are described in /SKB, 2001, TR-01-29/. The quality checks that are made on data prior to input to SICADA are based on SKB QA procedures.

In summary, the approval of data is based on the following activities.

- Only approved Contractors are used. A Contractor is approved after inspection of equipment and methods to ensure that standard or well-recognised procedures are followed, using adequate equipment and calculation methods with a good quality system for full traceability.

- In situ stress measurement results are checked with respect to the measurement objective, the relevance of the measurement method given the site conditions, the relevance of the results, and that the measuring and interpretation procedures follow the routines defined by the Contractor.

- Laboratory results are checked with respect to relevance of results, to the geological description of the core, that the measuring procedures follow the actual Standard/Suggested Method, and to any non-conforming report from the laboratory.

Despite a quality procedure for data quality control, problems or question marks regarding data quality may be raised when the modeller starts working with data. In such a case, it is important to discuss the possible problem and, if necessary, carry out a separate investigation. It is also important to keep a separate record of any potential problem with data quality or reliability.

To a large extent, the rock mechanics modelling will build, not only on data from SICADA, but on the interpretations made in the geological model and to some extent on the hydrogeological model. Then, the geological description will be stored in RVS, and it is essential to ensure that the rock mechanics model uses a consistent version of the geological model.

Modelling in RVS generates automatic registration of used data from SICADA /Munier and Hermanson, 2001, R-01-15/. This is not yet possible during when using other numerical tools. For these cases, separate records on input data and supporting model versions must be kept.

5.2.3 Interpretation of input data

Input data for rock mechanics modelling are in essence of two kinds.

1. 'Hard' data, based on measurement in situ or on a sample. These data always come from SICADA.

2. The geological (and hydrogeological) descriptions represented in RVS (which in turn are based on geological data from SICADA).

3. 'Soft' data, including experiences from underground construction in similar types of rock, etc.

Indirect or soft data have to be interpreted in various ways for the rock mechanics modelling. The first question to consider is the value of the information in the geological model. The geologist provides a model with different rock units, each one considered to be statistically homogeneous – from a geological point of view. It may be a realistic assumption that the homogeneity is valid for the mechanical properties of the rock mass within the unit as well, and the rock unit could be given, for example, an average of the estimated parameter in question.

An alternative assumption could, for example, be based on a conceptual model with stress dependence in the mechanical properties. Based on such an assumption, the rock mechanics model may possibly sub-divide the rock unit into a number of units or structural domains with depth, although it appears that a better alternative is to give the parameter in question a depth dependent equation.

The mechanical modelling may also imply needs for revisions of the geological description. The combined descriptions for a specific model version are produced in an integrated fashion, as described in /SKB, 2001, TR-01-29/.

/Staub et al, 2002/ give further examples concerning assumptions made during testing of the Theoretical Model, discussed in Chapter 3 using data from the Äspö HRL. Assumptions were made relating to the proposed geological model, the mechanical properties of the rock material and fractures, the numerical model used in UDEC, and the interpretation of the calculated rock mass properties. Two kinds of assumptions are needed. One kind concerns the conceptual model, i.e. the validity of a theory. This kind concerns a large spectrum of assumptions – from the estimation of parameters by laboratory testing to the applicability of numerical modelling. The other kind of assumptions concerns the need to make predictions in 3-D of spatially varying properties based on limited amounts of input data.

It is likely that the sparsely distributed data in an early phase of the site investigations may require more assumptions, compared to later phases. Reducing the number of assumptions would be a means for reducing uncertainty and enhancing confidence. For this reason, the study of alternative models based on alternative assumptions is recommended. The evaluation of alternative models should consider the acceptable property range (cf. Chapter 1). There is no need for absolute certainty: if uncertainties are bounded and shown to be acceptable for the performance issues at stake, the model may be sufficient. Specific attention should be given to considering whether the predicted uncertainty ranges are too wide to be of any engineering value.

It is fundamental for the traceability of the model that the assumptions relating to data (especially 'soft' data) are systematically recorded. This must be done in parallel with preparing the input data for modelling; this is because some of the assumptions may be adopted instinctively based on experience. A systematic checklist ought to be used for the full documentation of assumptions.

5.2.4 Documentation of the modelling decision process

The various approaches discussed in Chapters 3 and 4 indicate that a number of decisions may have to be made during modelling, based on, for example, assumptions relating to geological data or conceptualisations. This type of assumption could be related to, for example, assigning 'typical' mechanical parameters to a rock unit, or the simplified description of a fracture zone as it is input into the numerical stress modelling.

It is preferable for the design process to try to provide the 'best guess' for the entire model – if realistic and depending of the phase in the Site Investigation process, the density of input data and uncertainty in the geological model.

Decisions for modelling could be

- based on a specific conceptualisation, for example stress dependency,
- in the case of partly contradictory results, based on deciding to use only one type of data (at least for the actual modelling step), for example when results of stress measurements by overcoring and hydraulic fracturing give significantly different stress orientations, and
- based on the geological model or stochastic analyses, deciding if a rock unit with a limited amount of data could be regarded as similar to a rock unit with more data.

The documentation of these kinds of decisions is a 'Justification Statement'.

5.3 Modelling results

5.3.1 Geometric model and stress model

The main product is the description of the rock mechanics parameters in the various rock units of the geometrical model, as described in Section 5.1, and the in situ stress modelling results.

The simplest stress prediction is an average of the measured results, presented as stress magnitude profiles with depth with a statement on stress orientation. If modelling of the in situ stresses has been carried out with a numerical discrete element code, the grid should be set up in such a way that stress profiles and stress contour maps at specific levels could be produced.

5.3.2 Stochastic tools

Block modelling of mechanical parameters, as described in Section 3.4, is an additional possible product. The results are stored in the model database in a similar way to the geometrical model and associated parameter values.

5.3.3 Documentation trail

Based on the description in Section 5.2 the following records should be kept for Quality Control and justification reasons.

- Documentation of data quality control in accordance with Section 5.2.2, especially a separate record of any possible problem with data quality or reliability. In such a case, is it important to propose a separate investigation.
- Documentation of data used from SICADA and supporting model versions if other tools than RVS are used.
- A list of assumptions made for modelling.
- A justification statement for assumptions or decisions made during the modelling process.

5.3.4 Report

A comprehensive report should follow each Model version. The report should, as a minimum, include the following.

- Input data and supporting models.
- Evaluation of primary input data and input obtained from other aspects (geology and hydrogeology) of the Site Descriptive Model
- Reference to the generated files and their ID in the model database.
- The documentation trail.
- A summary description of the modelling results and major uncertainties.
- Recommendations for the next investigation step, if required.

6 Integration with other disciplines

It is a fundamental principle of the Site Descriptive modelling that there should be consistency between the different discipline descriptions (e.g. the geological, rock mechanics, hydrogeological and hydrogeochemical descriptions). Much of the rock mechanics description builds on the geological and hydrogeological models. These links should be acknowledged, but the rock mechanics modelling also provides important input to the geological and hydrogeological modelling. Thus, each official version of the Site Descriptive Model builds on interaction and integration of the different discipline teams.

6.1 The use of geological information

The rock mechanics modelling relies to a large extent on geological information, see Chapters 3, 4 and Section 5.2. This information provides the basis for considering how representative the rock mechanics parameters are when they have been sampled at points in the rock mass but are intended to characterise one or several similar rock units.

It is important that the rock mechanics modellers interact with the geologists at an early stage during the data collection phase, as well as later in the geological modelling. Rock mechanics engineers are familiar with using geological information for site description, based on data collection and interpretation. Classification systems based partly on geological data have been developed (cf. for example, Chapter 3). In this connection, the Äspö Test Case /Hudson, 2002/ raised a number of questions concerning the limited amount of geological information available. Some types of uncertainties based on geological conditions and related assumptions are given in Table 6-1.

Table 6-1. Examples of rock mechanics uncertainties and approaches considered in the Äspö HRL Test Case.

Type of uncertainty	Origin of the uncertainty	Tested approach
Size distribution of the fractures	Most of the fracture information comes from boreholes	Rely on DFN models
Spatial distribution of mechanical properties of intact rocks and fractures	Lack of systematic data distribution makes the unit ratings and estimation of spatial variability of the properties difficult	Assume that the mean values of these properties are valid for the whole rock unit where sampling and tests were reported
Rock unit information	Rock units with no geological data at all	Rely on the geologists' judgement concerning homogeneity
Lithological formations	Unknown number of lenses and dykes in the model area	Estimated as percentage of the rock mass, even if their exact locations and extents are largely unknown.
Fracture surface condition	The surface condition of the fractures, such as roughness, weathering, in-filling, coating, flow, wall strength and aperture, are important indices for both the Q and RMR rating systems.	Rely on the geologists' judgement concerning homogeneity
The 'water ratings'	The parameters J_w and RMR_{water} in the Q and RMR classification systems respectively concern the effects of water, and should be determined according to inflow and water pressure data	For design, the hydrogeological model could be used for predicting the water inflow to a possible tunnel location. For characterisation J_w should be set to 1 and RMR_{water} to 15 (see Chapter 3)
SRF in the Q-system	Subjective descriptive classification and large stepwise jumps of SRF values	• Use of SRF according to Q-system, based on the UCS and major principal stress values at different depths, or • let SRF=1.0 based on the reasons that: i) The effect of stress is not considered in RMR; ii) the SRF's unrealistically large effect on the Q-ratings at larger depth, or • let SRF < 1.0 based on knowledge of high confinement at large depths

The main objective of interaction between the geologists and the rock mechanics modellers is to ensure that the latter have a good understanding of the geological site model and that the rock mechanics characterisation is compatible with the geological representation. The key activity is to ensure that the geological description leads to an understanding in principle of the rock mechanics conditions. Examples of the related aspects are given in Table 6-2.

Table 6-2. Example of key rock mechanics questions related to geological descriptions.

Geological input	Rock mechanics question	Possible approach
Fracture zone	Actual extent in the Geometrical Model	Understand the 'geological width' and the geometrical uncertainty
	Mechanical properties in poor rock	Review primary geological information on fracture density and properties, alteration, clay, etc and compare with the results of empirical classifications
	Influence on in situ stresses	• Are measured stress orientations aligned with any significant lineament orientation? • Could inclined structures explain heterogenities in stress profiles?
Average rock unit	Homogeneity	• How complex is the lithological distribution? • What could be extracted from fracture statistics?
	Fracture distribution and density	• Number of fracture sets • Fracture frequency variation on outcrops • Fracture frequency with depth in boreholes
Geological development model	Stress boundary conditions	Interactive co-operation

Significant examples of the use of geological information have been published for various sites. Some are given by /Martin and Stimpson, 1994/ and /Read, 1996/.

6.2 The rock mechanics contribution to the site description

In the integrated study for the site description, the rock mechanics understanding can also contribute to the development of the geometrical model, as well as to the understanding of the geological evaluation (tectonic history).

The empirical classification methods are based on assigning a value to a scanline, such as a borehole, or a volume. These empirical values could strengthen the information given by, for example, just fracture frequency and thus help in developing the geometrical model. This is believed to be a valuable potential contribution, especially for modelling the location of fracture zones. SKB has limited experience to date in implementing this contribution in a systematic way, so interdisciplinary co-operation with the geological/geometrical modelling has to be developed further.

The understanding of the development of the Site is fundamental for the disciplines of both geology and rock mechanics. Also, the hydrogeologist may be dependent on this understanding for identifying the possible major conductive features. The stress modelling, as discussed in Chapter 4, could contribute significantly to the development of the tectonic history at a Site. The stress modelling may also be able to predict possible variations in stress magnitudes and stress ratios, indicating possible natural variations in the normal stresses caused by water-bearing fractures, leading to enhanced understanding of the stress influence on the hydraulic properties of the rock mass.

6.3 A co-operative work strategy

6.3.1 Field information

For the data collection, there are at least three main tasks in which the rock mechanics modeller should be involved.

1. Planning, fieldwork and interpretation of in situ stress measurements requires knowledge of the geological data collection and interpretation. Detailed geological descriptions of the test sections are required for evaluation of the reliability of the test results.

2. Selection of the core samples used for measuring the mechanical properties of the rock requires direct involvement in the core logging, plus the overall co-ordination of core samples for various test purposes. Detailed geological descriptions of mineralogy, grain size, foliation or fabric, etc are important for the full understanding of results from laboratory testing for the mechanical properties.

3. Direct involvement through rock mechanics logging of cores and boreholes is important for the full understanding of the site – not only for geological and empirical classifications, but also for screening of high stress indicators such as micro-cracked cores, core disking or borehole breakouts.

In addition, the general geological information collected on outcrops and in boreholes by geological and geophysical methods is available through direct observations and reports.

Thus, an interdisciplinary co-operative approach encourages a mutual and gradually improving understanding of the site. Of special concern for the rock mechanics modelling are questions related to homogeneity and spatial distributions, and trends versus depth for the rock units.

6.3.2 Regional geological model, geological evolution model and boundary stresses

The regional geological model and the geological evolution model probably constitute the most important geological input to the stress model. Based on a possible geometry, the boundary stresses can be discussed and modelled. A general understanding of the possible stress orientation is valuable in the early stage of modelling. If no nearby data are available, stress databases could be used, /Stephansson, 1993/.

The use of the geological information can be assessed by a series of questions. Based on the first overview of structures and possible stress orientations, the following questions can be asked:

1. Are there indications at the regional scale of a preferred orientation for fracture zones that could have a significant influence on the stress orientation?
2. Are there indications at the regional scale of a dominant fracture set that could have a significant influence on the stress orientation?
3. Could the actual fracture pattern at various scales, from the larger fracture zones down to the joint pattern, be explained by plastic or brittle failure caused by a paleostress field, perhaps similar to the one in place today but having higher stress magnitudes?

Interactive co-operation with the geologist for developing understanding of boundary stress conditions ought to be developed early, in order to attempt to identify the possible structures that may have a significant role on the current stress orientation. Bearing in mind that the state of stress in the old Scandinavian shield may have experienced many tectonic stages, the tectonic history is likely to be complex. However, a rational starting point, that enables some reduction in complexity, is the fact that geological observations indicate some fracture zones have been reactivated.

Stress modelling, in accordance with the descriptions in Chapter 4, should be planned on the basis of the answers to the three questions above. The modelling may provide indications as to whether the tectonic description of the site and the applied stress boundary conditions are realistic or if they have to be developed further. Examples of such indications are compatibilities of fracture zone displacement observations that have occurred over geological time and the modelled displacements of such movements. Clearly, other indicators are needed for inclusion or exclusion of fracture zones in the geological description, but the stress modelling can provide additional support for the model eventually selected.

6.3.3 The local geological model

The empirical classification systems can serve as a supporting measure for identifying the fracture zones in the local geological model (cf. Section 6.2). In the same way, rock mechanics studies of geological data may help to identify possible variations within a rock unit. The most significant requirement for the rock mechanics modeller is to find out if there are spatial variations in mechanical properties. Variation with depth, for example decreasing fracture frequency with depth, is one of the key aspects in describing the rock mechanics conditions. The geologists do consider such issues, but there is a need for a continuous development of co-operative discussions and interpretations with the users of the geological models.

Many of the aspects in the joint effort to develop the Rock Mechanics Site Descriptive Model based on geological inputs are indicated in Table 6-2. The aspects on traceability discussed in Sections 5.2.3 and 5.2.4 should also be considered.

7 Continuing improvement of the strategy

The purpose of this Chapter is to describe how the approach to the Rock Mechanics Site Descriptive Model will be updated during the future site investigations based on periodic reviews.

7.1 The future site investigation work

7.1.1 The context for updating the approach strategy

Site investigations will be conducted during the 5-year period 2002–2006. The purpose of the site investigations is to recommend one final site from the scientifically and technically feasible candidate sites selected from preliminary screening studies (cf. Chapter 1). The final site must not only satisfy all scientific/technical criteria for a permanent waste repository for spent nuclear fuel and other high level radioactive wastes, but also meet all political, social, legal and communal requirements. The philosophy is well outlined in SKB TR-01-03. The application for the licensing procedure is anticipated to be in the period 2006–2008, and the specific siting decision is expected at the end of 2008. The detailed rock mass characterisation at one site will then follow.

Thus, as the years pass and more experience is obtained during the SKB site investigations and from related worldwide activities, there should be a mechanism for updating the rock mechanics approach presented in the preceding Chapters.

7.1.2 The longer term use of the rock mechanics information

The updating of the approach should consider, not only advances in knowledge and techniques, but also the implications of the longer term use of the rock mechanics information as follows.

- For input to a reliable and efficient methodology for repository planning, design, construction, operation and post-closure monitoring, based on contemporary civil engineering technology and practice.
- As input to constantly evolving numerical tools, with properly constructed and validated constitutive models of materials, and incorporating the initial/boundary conditions and construction sequences and different problem scales. Problems to be analysed by model calculations are given in Table 5-4 in TR-01-29.
- For further developments of the necessary laboratory and field experimental techniques to measure the relevant data, the requirements for which may become more sophisticated during the site investigations.
- To ensure smooth execution of research and development of the site investigation programme after its implementation.
- For the development of a unified Site Descriptive Model for site evaluation.

7.1.3 The mechanism of an updating approach

The approach to the Model should utilise current experience and knowledge at the time it is being used. This means that the approach should be reviewed and, if required, updated as appropriate. A procedure is required, therefore, to review the approach strategy on a continuing basis – both during the 2002–2006 phase of the site investigations and beyond into the period when detailed rock mass property information is being obtained to support the rock engineering design and PA/SA requirements at the chosen site.

In the years ahead, there will be

- further developments at SKB, plus the experience gained directly from the site investigations themselves, and
- worldwide improvements in rock mechanics understanding and techniques.

It will be important to apply Quality Control measures to the incorporation of this knowledge into the approach strategy, in a similar way to that described in Chapter 2 for the initial approach strategy.

The next Section concentrates on the specific Quality Control issues required, including the recommendation for a 'Periodic Strategy Review'. In Section 7.3, there is discussion on how new worldwide information can be incorporated into the updating strategy. Also, there are known to be special areas of importance and difficulty in rock mechanics for site characterisation and repository design, so there should be additional emphasis on these in the continuing strategy improvement, as described in Section 7.4.

7.2 Quality Control of the improvement strategy

Quality Control applied to the improvement procedures has to account for the details of both the overall Rock Mechanics Site Descriptive Model and the specific components of the Model, plus the approach to the site investigations in the short and long terms. A summary description of the Rock Mechanics Site Descriptive Model is given in Table 7-1 (This is from Table 5-1 in TR-01-29).

Table 7-1. Purpose and content of the Rock Mechanics Site Descriptive Model.

Purpose of model
The parameters included in the model shall serve as a basis for design and safety assessment and the analyses performed in these steps. The model shall describe, for a given investigated volume, the initial stresses and the distribution of rock mechanics properties such as deformation and strength properties of the intact rock, of fractures and zones of weakness in the rock volume, and of the rock mass viewed as a unit consisting of intact rock and fractures. The model shall also describe the rock quality with regard to constructability.

Process description
Description of the processes that have given rise to the current distribution of rock stresses and properties in the area in question

The constituents of the model are listed as follows.

Geometric framework
The base for the geometric framework consists of the lithological and geological-structural models that are set up within the discipline of geology, together with the hydrogeological model that is developed within hydrogeology. With reference to the investigations conducted on the intact rock and the fractures, the geometric model can be further subdivided to obtain volume units with similar properties.

Parameters
Stresses: magnitude and direction of initial rock stresses, stresses in relation to tectonic structures.
Intact rock: deformation and strength properties, density, porosity, dynamic parameters, degree of weathering and degree of alteration.
Fractures: deformation and strength properties, statistical distribution of fracture geometries.
Rock mass: deformation and strength properties, seismic propagation velocity.
Rock classification system: such as Q, RMR, GSI, RMi indices.

Data representation
A uniform distribution of data is striven for within the volume in question. For the most part, however, constant parameter values and index values are associated with selected objects such as zones to represent and characterise the selected object in the rock volume. Statistical distributions are desirable where appropriate.

Boundary conditions
Initial rock stresses.

Numerical tools
RVS is used for interpretation and presentation of the constructed model. Numerical calculation models such as 3DEC are used to simulate the processes that have created the present-day distribution of rock stresses and properties.

Calculation results
Distribution of properties in accordance with the above parameter list plus distribution, magnitude and orientation of initial rock stresses within the area.

The continuing development of the approach to the Rock Mechanics Site Descriptive Model should therefore be geared to parallel these items.

7.2.1 Recommended Quality Control items

Table 7-2 shows the Quality Control measures recommended for the continuing development in the years ahead, especially in the years 2002–2006. These are those components of Table 2-1 in Section 2 that need to be retained for the updating procedure.

Table 7-2. Quality Control items for the updating strategy.

Quality Control Item	Method
Updating Project Plan	Updating Project Plan produced and refined (includes continuing assessment of the updating requirements)
Schedule	Schedule with sequence of items and annual dates
Activity plans	Documents describing the work with activity sheets
Worldwide literature	Scanning world literature for developments
Protocols	A set of protocols for the strategy of updating the Rock Mechanics Site Descriptive Model Approach
Handling uncertainties	Emphasis on the methods for dealing with uncertainties
Issues of special importance and difficulty	Define approach to these issues, see Section 7.4
Regular team meetings	Meetings of relevant personnel to ensure interaction
Minutes of all main meetings	Minutes distributed and agreed record produced
Internal reviewing	Continuous internal review of updating approach
Periodic Workshops	As appropriate, and to reach consensus on issues
External reviewing	Use of an International Review Panel
Production of documents	Periodic updating strategy documents

A brief summary of the reasons for the Quality Control strategy items in Table 7-2 is as follows.

- A Project Plan, Schedule and Activity Plan for the continuous improvement of the Model are necessary in line with SKB procedures and to ensure that the work is well planned, understood, and directly geared to the purpose, process description and constituents of the anticipated Model at the different time periods. Experts can be consulted at an early stage, as appropriate.
- There should also be a well-established method for regularly scanning the world information on factors relating to rock mechanics, rock engineering, repository site investigations, etc. Some of the information will come through SKB international meetings and personal contacts.
- The use of Protocols and/or activity plans is necessary so that the implementation of the work is well-structured.
- The handling of uncertainties is one of the most important issues and requires specific attention, see Chapter 2 from TR-99-09, Data and Data Uncertainties. Issues of special importance and difficulty should also be given emphasis (see Section 7-4).

- The results of ongoing modelling work should be screened on a regular basis, focussing on the need for further development. This includes team meetings, internal reviewing, periodic workshops, and external reviewing.
- Finally, there should be periodic memoranda presenting the updating approach to the Rock Mechanics Site Descriptive Model.

7.2.2 Periodic strategy review

In order to provide a forum for the work, it is recommended that meetings and/or Workshops be held periodically as appropriate to discuss the issues and the associated up-to-date recommendations. For example, possible timings are annually, or after the completion of the first two years of the Initial Site investigations, and at other suitable times related to the design and development stages anticipated in Section 5.2 of SKB TR-00-20. The record of these meetings and the co-ordinated recommendations will then form the updating strategy of the Rock Mechanics Site Descriptive Model.

In addition to SKB and contract personnel, there should also be international reviewers present who will have received documents for the meetings and Workshops and the periodic updating reports. They will present their review points at the meetings and Workshops prior to the finalisation of the periodic updating recommendations.

7.3 Incorporating worldwide developments in rock mechanics

It will be necessary to obtain information from

i) the general international literature on rock mechanics and rock engineering, and

ii) other international radioactive waste disposal programmes.

7.3.1 Incorporating improvements in worldwide rock mechanics knowledge

This is perhaps the easiest improvement to incorporate. Most major knowledge steps are published and there is now electronic access to the publications. The search facility at databases enables reference lists to be collated on any scientific and engineering subject. Electronic access to publications is accelerating rapidly and is the future for publishing material. However, the updating strategy has to include a method for actually incorporating the information into the Model.

7.3.2 Incorporating knowledge from international disposal programmes

Some in situ investigations and experiments around the world have been completed during the last few years, and some are still on-going. Examples are shown in Figure 7-1. The staff associated with these projects have dealt with many of the same issues in rock mass characterisation that are faced by SKB. The examples in Figure 7-1 do not represent a complete listing; there are other programmes being developed, for example in France.

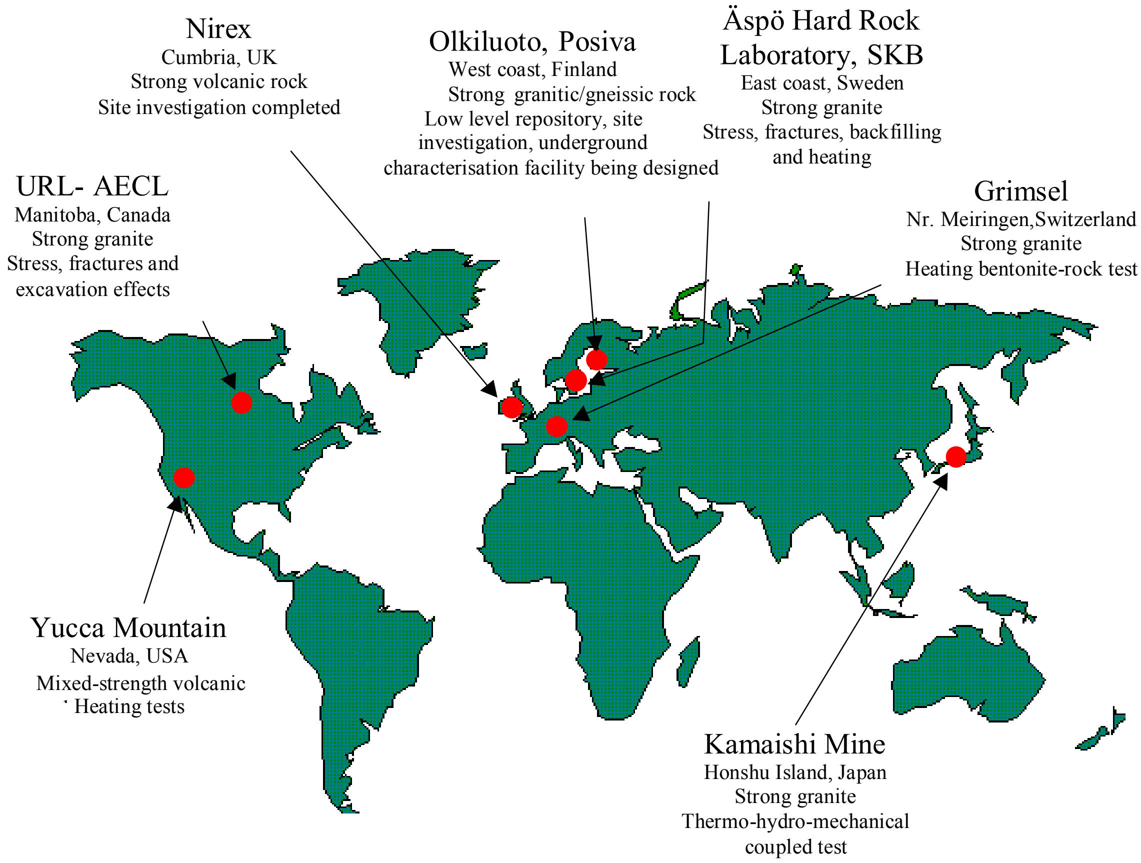


Figure 7-1. *Examples of Underground Research Laboratories (URLs) around the world from which key information is available.*

It is necessary, therefore, in updating the approach to the Rock Mechanics Site Descriptive Model, to consider what is generically useful information and what has to be site-specific information. Then the generically useful experiences and conclusions that can be transferred from the overseas activities should be included in the SKB approach. This applies also to the other five components of the geoscientic model – which indicates that an integrated approach to obtaining this information might be the most effective method. This is mainly achieved by international co-operation.

7.4 Issues of special importance and difficulty

In the Updating Project Plan and Protocols, special attention should be paid to the issues that are known to be of special importance and difficulty. These issues are noted in the following sections.

7.4.1 The stress conditions at the site

Since the potential repository zone is likely to be one or more rock blocks bounded by fracture zones, it is important to know whether the rock stress in such blocks can be represented by the regional stresses, or whether the local stresses in the individual rock blocks are significantly different in magnitudes and orientations.

An effort should also be made to establish whether significant residual stresses exist in the rock masses, and whether these stresses are important. (Residual stresses are 'locked in' stresses resulting from earlier geological activity, and hence are not necessarily related to the current tectonic stress regime.)

7.4.2 The characteristics of the rock mass (the intact rock, the fractures, and combined as the rock mass)

A fully-coupled thermo-hydro-mechanical-chemical rock mechanics model is not likely to be produced in the near future, and all the necessary supporting parameters could not be obtained in any case. Therefore, the analyses for repository design will be through rock mass classification methods and relatively simple numerical models. However, it is important that the operation of the model and the input parameters do have credibility in terms of representing the rock reality. The links between geology, hydrogeology and rock mechanics should be explored further and the fracture geometry and connectivity characterisation improved. The physical behaviour of large fracture zones should be taken into account, and upscaling and time effects should be incorporated.

7.4.3 The primary coupled effects, especially the links with the thermal and hydrological disturbances

In order to incorporate the coupled effects, it will be necessary for interaction to occur among the different disciplines supporting the Site Descriptive Model. Within each discipline, the activities are controlled by the common aims of the investigations. The detailed control of each discipline is obtained by specifying the discipline's principal tasks and activities based on the common aims. Thus, the rock mechanics properties are part of the wider scheme, and the geoscientific modelling and design will involve the linkages with other subjects, including consideration of the coupled effects. It is already known from international projects such as the completed 5-year Stripa project and the current DECOVALEX[2] and BENCHPAR projects that the most important mechanical linkages are with thermal processes and hydrogeology.

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[2] The terms DECOVALEX and BENCHPAR are acronyms for two related international projects concerned primarily with numerical coupled modelling of a repository and its host rock mass in the near-field and far-field.

7.4.4 The predicted mechanical effects of excavation in the rock mass, including the Excavation Disturbed Zone (EDZ)

The construction of a deep repository will lead to changes in the rock mass surrounding the excavation, resulting in localised mechanical deformation, alteration in the stress distribution and changes in the water flow and hydraulic properties of the surrounding rock volume. The zone of altered properties is termed the Excavation Disturbed Zone (EDZ). There are two types of disturbance caused by rock excavation:

a) the *inevitable disturbance* to the rock mass caused by the excavated space – rock movement, stress changes, and alteration of the hydrogeological circumstances; and

b) the *additional disturbance* to the rock mass caused by the excavation method – i.e. by the use of a tunnel boring machine or drilling and blasting.

These near-field effects of excavation are especially important for repository design, backfilling and radionuclide migration considerations. Methods to predict the effects and hence identify the necessary components of the Rock Mechanics Site Descriptive Model should be studied. This is an important component in describing the initial stage (after excavation and backfill/sealing) of the near-field characteristics, as an input to Safety Assessment.

7.4.5 Transparent, traceable and credible evidence that the information obtained is accurate and adequate

The importance of the Quality Control measures has already been stressed. It is critical that these measures are implemented in such a way that they do, indeed, operate as planned, and hence that the information in the Rock Mechanics Site Descriptive Model is credible. Quality Control of the site information and subsequent analysis should be conducted using appropriate Protocols and with supporting documentation to provide the necessary evidence. Experts in the relevant subjects should be involved at an early stage, as required.

Finally, it should be noted that the data and analyses do not have to be perfect: they should, however, be demonstrably accurate and adequate.

8 Conclusions

The development of the strategy and the experiences from the Test Case lead to several conclusions relating to the overall approach, the methodology for prediction, experiences from the Test Case, and continuing improvement of the strategy. These conclusions lead into specific recommendations for the SKB Rock Mechanics Site Descriptive Modelling.

8.1 Overall approach strategy and upgrading

The overall approach for developing the Rock Mechanics Site Descriptive Modelling Strategy is judged very successful. The Test Case application, see /Hudson, 2002/ demonstrates that it was possible to predict fairly well the conditions at Äspö, based on mechanical information from a limited number of boreholes.

The application of more than one method for modelling was shown to be important. This provided insights into the benefits and pitfalls of other approaches and demonstrated the fact that complex problems may have more than one solution. The method of achieving consensus, harmonisation and amendments proved to be essential and gave extra insight.

The use of a Test Case was invaluable for developing the strategy. It forced generic predictions to be specific. It highlighted the need for consensus and demonstrated what can and cannot be predicted. It was also an effective means for others to understand what has been done.

Technical Auditing, as well as the Protocols developed, are judged to be potentially powerful Quality Control instruments. These tools were developed in parallel with the methodology and Test Case modelling and, because the strategy development was a learning exercise for everyone involved, strict application of the Quality Control procedures was not possible for all aspects of the development work. Further development of these tools is therefore needed.

Finally, it should be stressed that understanding the geological model proved essential. Building the three-dimensional Rock Mechanics Site Descriptive Model should rest on the three-dimensional structure offered by the Geological Model, but also the rocks mechanics modeller should have an understanding of the confidence in the Geological Model and interact with the geological modelling teams.

8.2 Rock mass properties and state of stress

8.2.1 Rock properties

For the Test Case, two different approaches for estimating rock mass properties were considered.

- The use of empirical classification methods, as frequently applied in civil engineering in hard rock.
- The use of a theoretical approach, calculating the deformation and strength properties of the rock mass.

There are several uncertainties involved in estimating the properties and spatial distribution of input data for both the empirical and the theoretical models. Uncertainties originate both from uncertainties in the methods (empirical and theoretical) for estimating properties with known input data and from the uncertainties in the three-dimensional distribution of the input.

- The empirical relations are derived from experience, not from basic mechanics. The validity of the empirical relations can thus only be ascertained in situations similar to those to which they were originally fitted. For any new construction problem, and in particular when constructions are relatively unique, such as a deep repository, the validity of the empirical approach cannot be proved beforehand.
- The estimated rock mass strength values depend on the failure criterion chosen for the intact rock. However, the Mohr-Coulomb (M-C) plastic material model, Hoek-Brown (H-B) failure criterion or a strain-softening (S-S) model for the intact rock give almost the same results for deformation properties when compared with the theoretical approach.
- For the empirical Q system, stress dependence can be introduced by adjusting the SRF value, whereas the RMR system does not directly allow for such adjustment. Stress dependency can be introduced in the theoretical model through both the boundary conditions and the stress dependency embedded in the underlying input models for fracture and intact rock deformation and failure.
- The current application of the theoretical approach rests on 2-D mechanical analyses. The transfer of a 3-D fracture network into a 2-D trace network with the same overall mechanical properties is not a trivial exercise. However, comparisons suggest similar results for the 2-D and 3-D cases if the 2-D section is aligned with the weakest section of the 3-D model.

Considering these uncertainties, it is evident that one single approach cannot be recommended for predicting rock mass mechanical properties. However, for the Test Case, the two approaches provided fairly similar results for the rock mass deformation modulus in the ‘good’ rock but more differences in the fracture zones. The uncertainties may have little practical engineering impact in the good rock where the repository will be sited, but will be significant in the fracture zones. It will be necessary to apply different methods for estimating the rock mass mechanical properties and then devise a procedure for making an overall judgement.

After a first set of different modelling attempts (which could be empirical and theoretical), a stage of harmonisation and amendment should follow – with the purpose of identifying and correcting errors, identifying and agreeing on non-method specific assumptions (such as geometry), and making different experts more familiar with each other's approaches.

A combined prediction should be sought for in consensus discussion involving relevant expertise at the different stages of the Site Investigation. Four main decision factors should be used in arriving at the consensus range: the overlap of the individual predictions; the confidence of the individual predictions; relevant engineering experience; and the engineering significance of differences between different predictions. This procedure was tested successfully in the Test Case work.

It must also be remembered that a primary aim of the Site Investigation is to find rock volumes suitable for repository construction. In high quality rock, differences between different methods are less significant. Even if determination of the rock mass mechanical properties cannot be made with high accuracy and precision, an uncertainty of, say, ± 15% may not have a significant impact on the value of the result in good rock conditions. The empirical and simulation approaches, if combined with reason and experience, should be able to provide a picture of where the rock mass is likely to be relatively unproblematic for underground constructions and where there is potential for difficulties.

8.2.2 State of stress

Based on the experiences gained, a stress model approach was developed as an integrated approach combining stress measurement information, geological factors, numerical modelling results, and consideration of the uncertainties involved. The approach involves different steps starting with a preliminary stress estimation, followed by steps for interpreting site specific information. If the stress pattern and structural geology of the site are complex, including major fracture zones intersecting the area, numerical analyses of the stress field are recommended.

Different numerical models (i.e. alternative geological concepts, but also different boundary conditions) can be analysed to provide possible explanations for observed stress patterns. The structural model used for modelling the state of stress should preferably be large enough to include the whole extent of the fracture zones intersecting the area, and also show the extent and orientation of the structures towards which they terminate. The orientation of fracture zones with respect to the applied stresses determines the direction of fracture zone deformation and the nature of the local stress perturbation. Using displacement boundary conditions, rather than stress boundary conditions, gave better agreement with measured data in the Test Case.

Simple models are to be preferred to complicated models, i.e. a simple linear (or stepwise constant) function should be used if there is no better model for explaining observed features. It is not recommended that non-linear curves (exponential, logarithmic or polynomials) be fitted directly to measurement data and used as 'models', if there is no mechanical explanation for the form of such curves. It is also important to note clearly within which area of the region and, even more importantly, within which depth interval a certain prediction is made. The stress models must therefore be used carefully and not for prediction deeper than the deepest boreholes.

Stress measurement results and observations from the site concerning slip directions must be used in the evaluation of the modelling. This is a most difficult step because the stress field in the models depends on loading sequence and the geometrical and strength properties of the zones. Also, the stress measurement results may be more or less affected by local geology and measurement quality problems. Therefore the different assumptions behind each model should be compared with the input information concerning rock mass and fracture zone mechanical and geometrical properties. The aim of this step is to judge which one of the possible models represents the actual stress field in the best way. Establishing the 'best estimate' of the regional stress field, based for example on results from stress databases together with evaluation of the possible influence of topography and structural geology, is an important process involving judgement.

8.3 Quality Control

The Site Investigation will be undertaken in steps, with data being produced in batches for each site. Consequently the further development of the Rock Mechanics Site Descriptive Model will be developed in a stepwise manner as well. Updates of the Rock Mechanics Description should be co-ordinated with the overall revision of the Site Descriptive Model. There should be consistency between the different discipline descriptions (e.g. the geological, rock mechanics, hydrogeological and hydrogeo-chemical descriptions). Much of the rock mechanics description builds on the geological and hydrogeological models, but the rock mechanics modelling also provides important input to the geological and hydrogeological modelling

To allow for full traceability in the modelling works, the modeller must consider control of input data, interpretation of input data and documentation of the modelling decision process. For the development of a Rock Mechanics Model the key aspects are as follows.

- Does the Modeller understand the Geological Model?
- Is the rock mechanics conceptualisation of the Site realistic?
- What kind of instinctive assumptions are made by the Modeller?
- Is the scattering significant and, is so, are the reasons for scattering in input data understood?

8.4 Recommendations for improvement of the strategy

The strategy as presented in this Report is judged sufficiently developed for guiding the practical evaluation of rock mechanics data to a rock mechanics site description. The current Report provides the foundation for the approach, but more development will be needed for application during the later stages of the Site Investigation. This strategy ought to be fully reviewed not later than the completion of the planned Initial Site Investigations (i.e. after the first two years of the SKB programme for Site Investigations).

Further development should focus on

- Enhancing the Quality Control procedures and the associated scope and content of the Protocols.
- Improving the input and output (post-processing) routines for stress modelling.
- Developing rock mechanics modelling for Design and Safety Assessment in order to create a feedback loop to the property estimation strategy.
- Preparing for possible modification to the approach during the first stages of Site Descriptive Modelling during the Site Investigations.

As the planned Site Investigations proceed and more experience is obtained and related worldwide activities progress, there should also be a mechanism for updating the rock mechanics approach. The updating of the approach should consider, not only advances in knowledge and techniques, but also the implications of the longer term use of the rock mechanics information.

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