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Volume 6 Number 1
Radioactive Waste Disposal August 1999
The Daini Shibisan Tunnel is one of the 13 tunnels being constructed for the Japanese high-speed train, Kyusyu Shinkansen, on the island of Kyusyu south of the Japanese mainland. These tunnels will significantly improve the communication between the cities of Kagoshima and Kumamoto.

The length of the Daini Shibisan tunnel is 3.4 kilometres and the total length for all the 13 tunnels is 69 kilometres. The cross-sectional area of the tunnels is 73 m². When constructing the Daini Shibisan tunnel complicated ground conditions were encountered. Weathered sandstone was encountered as well as shale and clay. On top of this large water inflows occurred which resulted in that the holes drilled for rockbolts collapsed almost immediately.

In these difficult conditions Swellex rockbolts provides the solution. Cement grouted rebars were tried but they do not function well in wet conditions. Instead, 14 pieces of 3 meter long Midi Swellex are installed as pattern bolt.

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Rocha Medal

A Bronze Medal and cash prize has been awarded annually since 1982 by the ISRM to honour the memory of Past President Manuel Rocha and to recognize outstanding young researchers in the field of Rock Mechanics.

The award shall be for an outstanding doctoral thesis in rock mechanics or rock engineering. The thesis must have qualified the candidate for a doctorate or the equivalent. To be considered for the award, a candidate must be nominated within two years of the date of the official doctoral degree certificate. The nomination should be submitted to the appropriate ISRM Regional Vice-President by registered letter, and may be presented by the nominee, the nominee’s National Group or some other person or organization acquainted with the nominee’s work. The nomination should include the following supporting information:

◆ A one page curriculum vitae, including the name, nationality, place and date of birth of the nominee; also position, address, telephone and fax numbers;
◆ A thesis summary in one of the official languages of the Society, preferably English, of about 5,000 words, detailed enough to convey the full impact of the thesis, and accompanied by selected tables and figures, with headings and captions also in English;
◆ One copy of the complete thesis and one copy of the doctoral degree certificate;
◆ A letter of copyright release, allowing the ISRM to make copies for review and selection purposes only.

Nominations for the 2001 Rocha Medal must be received by 31 December 1999.

Supplementary details of the selection procedure, conferring the award, etc., are provided in ISRM By-Law No. 7, found on pages 33-34 of the ISRM Directory for 1996. National Groups and Corresponding Members will be officially reminded by the Secretariat as the deadline approaches, but are encouraged to consider possible nominees and to recommend names to the appropriate ISRM Regional Vice-President as early as possible.
Report from the President

ISRM Membership

Though the field of rock mechanics has itself been growing, for some time now the membership of ISRM has been gradually decreasing. It is my opinion that the decline in ISRM membership results from the lack of interest from practicing engineers to participate in our activities. The majority of our members, at present, are academics working in universities and research institutes, and we tend to discuss our research on fundamental principles of rock mechanics, new theories and such when we meet at Symposia and Conferences. This research-oriented atmosphere is, I have come to believe, not so pertinent for practicing engineers and in turn discourages them from joining the ISRM.

I am not positing that research is unnecessary. Indeed, it is the lifeblood for many of our careers and further leads to the advancement of technology, and thus serves industry as well. In fact, many computer programs containing sophisticated capabilities for data analysis and modeling have become commercially available thanks to years of research and development. Still, many of the practicing engineers well know that the real behavior of rock structures like tunnels, caverns and slopes often differs from that predicted by computer modeling. This discrepancy between modeling reality and reality itself is quite common in rock engineering, and so many practicing engineers often fail to pay attention to recent developments in our field. This failure to keep abreast of recent developments is coupled with the tendency of practicing engineers to rely on the trial and error methods garnered from personal experience and empirical approaches. Thus, practicing engineers are more and more refraining from joining our academically oriented Society.

Considering this dilemma I have come to the conclusion that ISRM should pay more attention to the practical aspects of rock mechanics in order to appeal to engineers and thereby increase its membership. We should encourage our members to observe the real behavior of rock masses in the field, and not simply fancy modeling programs from a laboratory. Many young researchers seem to be overly impressed with attractive computers that churn out impressive looking data, but they simply lack field experience. It seems too obvious to point out, but nothing substitutes for real experience and real data. On the other hand, as I mentioned before, many engineers fail to keep up with the latest theories available in their work, and so our Society should offer them a forum in which they can find better solutions for their projects.

Given all this, then, the ideal solution would be an ISRM where researchers and engineers could exchange both ideas and experiences with colleagues in their respective fields and with “the other side.” As I believe, we are stressing fundamental research too much at the present. I would urge more current

Opening session of the International Symposium on Clean Coal Initiatives in New Delhi

With Prof. B. Singh, Dr. A.K. Dube and their colleagues at the University of Roorkee, India

members to focus on the applicability of their research when compiling papers and reports for ISRM conferences and symposia. This shift will, in the end, lead to a greater ISRM, not only in membership, but also in the contribution to the rock engineering community.

Symposium on Clean Coal Initiatives

I was invited by Dr. T.N. Singh, Director of the Central Mining Research Institute (Council of Scientific and Industrial Research) to visit the main office of the Institute in Dhanbad, India in January, where I learned about their rock mechanics research activities.
Central Mining Research Institute, to visit the Roorkee campus, where I had a chance to discuss a variety of rock mechanics issues with his colleagues.

In Roorkee, I also visited the University of Roorkee, where I met Prof. B. Singh and his colleagues in the Department of Civil Engineering, and we discussed our common interest in rock engineering. I gave a lecture entitled “Observational Methods in Tunneling Practices.”

**Vail Rocks ‘99**

The 37th US Rock Mechanics Symposium was held in Vail, Colorado in June. This was the first U.S. Rock Mechanics Symposium organized by the American Rock Mechanics Association (ARMA) and the ARMA Foundation. It was co-sponsored by ISRM as an ISRM Regional Symposium with the theme “Rock Mechanics for Industry.” More than 300 participants from 22 countries attended. I felt very honored to deliver a short speech on behalf of the ISRM at the Opening Ceremony. The details of this Symposium will be reported later.

—Shunsuke Sakurai

**Enjoying a congenial moment at Vail Rocks ’99. From the left:** Miklos Salamon, Charles Fairhurst, Agi Salamon, Margaret Fairhurst, Jeff Whyatt, and Bill Pariseau.
Candidates for the ISRM Board 1999–2003

ISRM Vice-President for Africa

Riza Güner Gürtunca  
(nominated by the ISRM NG S. Africa)

I would like to concentrate my efforts on the following two issues if I am elected as the ISRM Vice President for Africa. First, I have begun to realize that the quality of the international rock mechanics symposiums has started to drop over the past five years. All the delegates attending the conferences and symposiums spend a considerable amount of time and money exchanging knowledge and meeting people. In terms of meeting people I think that these conferences still meet expectations, however, if we do not improve the quality the attendance might drop. Furthermore, I have noticed very few people from industry attend rock engineering conferences and this does not help to transfer knowledge to those who could implement the new knowledge and technology.

The organization of these conferences also needs attention in terms of accepting good quality papers together with good quality presentations and I believe that ISRM should re-emphasize the guidelines on holding rock mechanic conferences and give thought to monitoring the quality.

Second, I would like to continue our efforts to promote and expand ISRM activities and membership to various countries on the African continent. Dr. Nielen van der Merwe started this initiative four years ago and I would like to continue to support his initiative by making countries such as Zimbabwe, Zambia Ghana and Egypt more active in the field of rock mechanics.

Riza Güner Gürtunca was born in 1952, in Turkey. He obtained his B.Sc. in Mining Engineering at the Middle East Technical Univ., Ankara, Turkey; his M.Sc. in Mining Engineering at Ege Univ., Bornova (Izmir) Turkey; and his Ph.D. in Rock Mechanics at the Univ. of New South Wales, Kensington NSW, Australia.

Following a production experience in coal mining, he worked as a Rock Mechanics Officer in the Randfontein Estates gold mine, S. Africa. In 1986, he joined CSIR Mining Technology (then known as COMRO), Auckland Park, S. Africa, where he currently is the Director.

Dr. Gürtunca is a member of the ISRM through the South African National Group on Rock Mechanics (SANGORM); currently, he is the President of SANGORM. He is also a council member of the South African Institute of Mining and Metallurgy (SAIMM).

Dr. Gürtunca is the author or co-author of about 45 papers and reference reports. In 1992 he was awarded a silver medal by the SAIMM for one of his publications, and in 1994 he received the CSIR Outstanding Achiever Award. He is regarded as an expert in backfilling in the South African gold mining industry.

ISRM Vice-President for Asia

Satochi Hibino (nominated by the ISRM NG Japan)

Since the first ISRM Congress held in Lisbon in 1966, rock engineering has been progressing with the rapid development in theories, testing, measuring and analysis methods. Although many rock mass phenomena have become explainable recently, a lot of actual mechanisms have not been unveiled yet. The main reasons are due to the difficulties inherent in rock mass such as discontinuity, non-linearity, and uneven distribution of geo-stress, etc. In recent years, increasing environmental concerns have demanded more reliable understanding of rock mass. In this regard, ISRM has the greater responsibility to meet this expectation.

As rock mass properties or issues are either local or regional, the application of rock engineering tends to be influenced by the cultures and customs of the regions hence regional or national symposia are deemed important to promote rock engineering.

In the Asian region the first Asian Rock Mechanics Symposium, ARMS, was held at Seoul, Korea in 1997. The second ARMS will be held in 2001 in Beijing, China to be followed by the third ARMS in Japan in 2004. Moreover, the International Symposium on Rock Stress was held at Kumamoto, Japan in 1997. In addition, the regional symposium took place at Taipei, Taiwan in 1998 and subsequently the second Japan-Korea Joint Symposium on Rock Engineering will be in 1999 at Fukuoka, Japan.

I fully understand and support the significance of regional and national symposia as a means to advance the development of rock engineering globally. Therefore I will pledge my utmost effort in promoting such symposia in Asia.
Satochi Hibino was born in 1938 in Japan. He obtained his B.Sc. in Mining Engineering at Kyoto University in 1962; his M.Sc. in Mining Engineering at the Graduate School, Kyoto University, in 1964; and his Ph.D. in Engineering also at Kyoto University in 1973, with the thesis “Progressive Relaxation and Behavior of Rock Masses during the Excavation of Large Scale Underground Caverns.” Since 1964 he has worked as Research Engineer at the Central Research Inst. of the Electric Power Industry (CRIEPI), Abiko, Japan, where he was successively Head of the Underground Structures Section, the Foundation Engineering Section, and the Geotechnology Section. Currently, he is the Vice President of CRIEPI. Since 1985 he has also been Lecturer and Visiting Prof. at the Tokyo Institute of Technology, and in 1977 and 1993 at Yamaguchi University, Ube, Japan. In 1994 he held the post of Lecturer at Tottori University, Tottori, Japan. In 1987 he was a Consultant for Sinotech Engineering Consultants, Inc., Taipei, China R, and since 1993 he has been an Adviser to the Japanese Ministry of International Trade and Industry.

Prof. Hibino is a member of ISRM through the Japanese Committee for ISRM; he was a member of the Steering Committee of the 8th International Congress on Rock Mechanics, and several other international rock mechanics events, and currently, he is the Vice President of the Japanese Committee for ISRM. He is also a member of the Japan Society of Civil Engineers (JSCE) (from 1991–94, the Chairman of the Subcommittee of Underground Space Utilization, and Chairman of the Committee of Underground Space Research) from 1994–97, the Japanese Geotechnical Society (JGS) (a Councilor from 1992–96), the Mining and Materials Processing Inst. of Japan (MMPIJ), and the Japan Electric Power Civil Engineering Association (JEPOC).

Prof. Hibino is the author or co-author of 259 papers and books. He has received the Takahashi prize of the JEPOC. His main topics of research are: Rock Behavior around Large Scale Caverns, Compressed Air Energy Storage, Underground Siting of Nuclear Power Stations, and Hot Dry Rock.

**Chung-In Lee (nominated by the ISRM NG Korea R)**

Asia is a vast region with a large population. It consists of many different countries of their own economic levels. But, most of them, especially Japan, Korea, Taiwan, and coastal China, confront the same “environmental” problems which occur in the process of industrialization, urbanization, and overpopulation. These facts create a keen interest in the utilization of underground space. In those countries, the applications of rock mechanics and rock engineering are highly required.

Construction of infrastructures like high-speed railways, highways, dams, hydro and nuclear power plants, nuclear waste repositories, and underground storage facilities often involve a large cross-sectional excavation and excavation in the vicinity of pre-existing structures, which causes “safety” issues from design to maintenance. Rock mechanics and rock engineering have thrived in this regard, an environmental and safety issue.

Oil producing countries and other countries that have rich mineral resources including Turkey, India, Iran, China, and Vietnam, are also interested in exchanging the technologies of rock mechanics for economic and safe operation in exploitation. But, there are countries in Asia that have not yet organized their ISRM national group. Therefore, it is of great importance to help them to participate in symposia, conferences, and workshops to encourage the exchange and transfer of technology in rock mechanics and rock engineering.

To meet the above needs, the Korean Society for Rock Mechanics has successfully hosted the first Asian Rock Mechanics Symposium as a regional symposium sponsored by ISRM in October 1997. The theme of the Symposium was “Environmental and Safety Concerns in Underground Construction.” The second symposium is scheduled to be held in China in 2001. The Asian Regional Symposium should be continuously held in the future. Joint symposia and workshops of a practical size incorporated with a few countries at a time will be efficient considering the economic benefits. The Japan-Korea Joint Symposium on Rock Engineering held in Seoul in 1996 and in Fukuoka in 1999 are good examples.

In conclusion, I propose the following goals for the period 1999–2003:

1. To help the ISRM supported Asian Regional Rock Mechanics Symposium, begun in 1997, to continue, which will enhance the opportunities of technical exchange and transfer among many countries.
2. To promote joint symposia and workshops among Asian National Groups and sister societies.
3. To help organize ISRM supported International Symposium on Rock Mechanics in Korea under the theme of Environmental and Safety Issues.
4. To help organize more national groups in Asian countries.

Chung-In Lee was born in 1941 in Korea. He obtained his B.Sc. in Mining Engineering at Seoul National Univ., Seoul, Korea R, in 1963; his M.Sc. in Mining Engineering also at Seoul National Univ., in 1965; and his Ph.D. in Rock Mechanics at Tohoku Univ., Sendai, Japan, in 1974.

From 1965–69 he worked as Research Engineer at the Korean Tungsten Mining Co. Ltd, Korea R, and from 1969–75 as Lecturer and Assistant Prof. at Chung-Ang Univ., Korea R. Since 1975 he has been Assistant, Associate Prof., and Prof. at Seoul National
What is the common thread?

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Univ., where he is also, since 1997, the Director of the Research Institute of Energy and Resources, and, since 1998 the SK Telecom Research Chair Prof. From 1982–83 he was Visiting Prof at the Tohoku Univ. and since 1997 Adjunct Prof. at Northeastern Univ., Shenyang CHINA PR. Since 1993 he is also Director of the Korean Mining Promotion Co., and from 1994–97 he was Director of the Underground Energy Storage and Rock Engineering Center.

Prof. Chung-In Lee is a member of the ISRM, through the Korean Society for Rock Mechanics (KSRM); he has been the President of KSRM for the term of office 1983–85, and a co-chairman of the Organizing Committee of the First Asian Rock Mechanics Symposium. He is also a member of the Korean Geotechnical Society (since 1994 its Vice-President, and also the Vice-President of the Organizing Committee of the 11th Asian Regional Conference of the ISRMGE), the Korean Inst. of Mineral and Energy Resources Engineers (President, 1995–96) and the National Academy of Engineering of Korea.

Prof. Lee has been internationally active as an editor or co-editor, organizer of conferences or symposia, and keynote lecturer. He is the author or co-author of more than 150 papers; and has supervised over 15 Ph.D. and 50 M.Sc. theses.

His academic interests include: Underground Fluid Flow, Coupling Effects, Elasto-Plastic Deformation and Failure of Rocks under Different Loading Conditions; and In-Situ Stress Measurement. Professionally, he has participated in projects concerning: Underground Oil and Gas Storage; Pumped Storage; Highway and Railway Tunnels; Blasting Vibration Control; Slope Stability; Geotechnical Site Investigations.

ISRM Vice-President for Australasia

Chris Michael Haberfield
(nominated by the ISRM NG Australia)

The expertise of the geotechnical professional must encompass soil mechanics, rock mechanics and engineering geology. Despite the obvious overlap in these disciplines, they have remained separated for too long. It is time to break down the official barriers that currently exist and to strengthen collaboration, especially amongst the technical activities of the international societies. I will actively lobby for the formation of joint commissions (including ISRM, ISSMGE and IAEG) in areas such as education, classification, slope stability, underground excavation, in-situ testing and laboratory testing.

I am particularly concerned with the narrow focus of many universities when it comes to the teaching of geomechanics. Geomechanics should not concentrate just on rock mechanics or soil mechanics or engineering geology. It is important that they are taught together, with a unified approach. I would like to work with a joint commission on education to pursue this approach.

Of concern also is the limited involvement of our young professionals in the activities of both the AGS and the ISRM. They have much to offer, and we need to involve them more in our activities. We therefore need to be proactive in increasing their involvement.

Chris Michael Haberfield was born in 1958 in Australia. He obtained his B.Sc. at the Univ. of Sydney, Sydney NSW Australia in 1979; his M.Sc. in Civil Engineering with first class honors also at the Univ. of Sydney in 1981; and his Ph.D. at Monash Univ., Clayton Vic., Australia in 1988.

From 1981–83 he worked as Consulting Geotechnical Engineer at Coffey and Partners Pty Ltd, Sydney NSW Australia, and from 1983–96 as Senior Tutor, Lecturer, and Senior Lecturer at the Dept of Civil Engineering of the Monash Univ. Currently he is Associate Prof., Director of Research, and Deputy Head of the Dept of Civil Engineering at Monash Univ., and since 1998 a Consultant at Golder Associates Pty. Ltd. in Melbourne.

Prof. Haberfield is a member of the ISRM through the Australian Geomechanics Society (AGS). He was the Deputy Chairman and Treasurer of the National Committee of the AGS from 1996–97, and Chairman of this National Committee from 1998–99. He is also a member of the ISSMGE, the Institution of Engineers, Australia (IEAust). He is a member of the Organizing Committee for the GeoEng 2000 Confer-
ence, Chairman of the Conference Technical Committee, and was the mentor for the Organizing Committee of the third ANZ Young Geotechnical Engineers Conference, in 1998.

Prof. Haberfield has delivered several keynote lectures and panel reports in Australia and abroad; and is the author or coauthor of more than 80 papers.

Currently, he is involved in the following projects: Performance of Weak Rock Masses; Shear Resistance of Rock Joints; Bored Piles in Rocks; Wellbore Stability; and Slope Stability Analyses for Soil and Rock Slopes.

**ISRM Vice-President for Europe**

**Pekka Särkkä (nominated by the ISRM NG Finland)**

The largest challenge of the ISRM is to create a broader bridge between academic research and the applications in the field. In academic research the habit often is to simplify the problem so far that it can be solved scientifically. This gives satisfaction and merits to the researcher, but does not always help the practicing engineer at the tunnel entrance or in the open pit. On the other hand, it is not uncommon among practitioners to find even a clear aversion to science.

This can lead to segregation of these two groups, and weaken the whole rock mechanics unity. There will be less financing of pure science without any practical applications. As well there will be no progress in the planning and implementation of rock engineering projects, if there will not be any new ideas to be tested and approved for general use.

Another challenge is increasing internationalization. The enlargement of the European Union makes both the projects and manpower in Europe moveable. This increases the requirements in education and demands that rock mechanics engineers act professionally in the most different geological areas. More and more systematic international education is needed, and courses should be made as comparable as possible.

The increasing amount of information needs better ways to be handled. The Internet is one of the possibilities for rapid transit of new ideas and thoughts. It diminishes the world by taking away the time lag of publishing and mailing. On the other hand, there is no discrimination on what can be sent. The different possibilities in the sound use of the Internet should further be studied and tested.

Pekka Särkkä was born in 1945 in Finland. He obtained his M.Sc. in Applied Geophysics at the Helsinki Univ. of Technology (HUT), Espoo Finland in 1970; and his Ph.D. in Rock Engineering also at HUT in 1978.

From 1970–72 he worked as Rock Mechanics Engineer at the Oulokumpu mines, Finland, and from 1972–78 as Senior Assistant at HUT. In 1979 he acted there as Prof. of Excavation Engineering, and in 1980 as Docent in Rock Mechanics. From 1980–84 he was Senior Research Fellow at the Academy of Finland. In 1985 he returned to HUT as Supervising Assistant in Rock Engineering. From 1989–94 he worked as Chief Specialist in Rock Cavern Engineering at the Gas Div. of Neste Oy, Finland, and from 1994–97 as Managing Director of the Contecco Co. R&D and consulting company. Since 1997 he has been Prof. and Head of the Laboratory of Rock Engineering at HUT.

Prof. Pekka Särkkä is a member of the ISRM, through the Finnish Rock Mechanics Association (SK); he has been the Secretary of the SK from 1980–84, and the Chairman of this Association, from 1985–90 and also since 1995. He is also a member of the ITA and the IAEG, the Society of Mining Engineers, the German Society of Miners and Metallurgists, the Geological Society of Finland, the Association of Mining and Metallurgical Engineers in Finland (of which he has been the Secretary of the Mining Section from 1982–84), and the Nordic Association for Rock Blasting Technique.

Prof. Särkkä has been involved in the European Mining Course (EMC) and the Training for Under-
ground Environment (TRUE) programs; he is the author or co-author of about 140 papers and three patents; and has served on the examination boards for 15 dissertations.

He has done numerical modeling of stoping areas and pillars, carried out 3-D stress measurements (and put together the Finnish part of the World Stress Map), designed mines in vein-type deposits in South America, and worked on the alteration of stone facade plates, and the deformation of paper machine granite rolls. Lately, he has researched ways to store natural gas in hard rocks, and participated in the implementation of five underground LPG storages.

ISRM Vice-President for North America

Alfredo Sánchez Gómez
(nominated by the ISRM NG Mexico)

The formation of the ISRM Council at the end of this century allows all participants the opportunity to promote the diffusion and development of rock mechanics in an environment that has been characterized in the past few years by the existence of economic blocks that have had a sustainable development compared to others that have fallen in a recess which has made research and new projects develop at different speeds.

Stricter ecological standards and low prices on raw materials have also taken their toll on investments in the mining and oil sectors, areas that have traditionally demanded the development of new technologies and designs that rationalize their high production investments.

The consolidation of new conditions of economic competition and new work schemes, predicts an important challenge developing new infrastructure and maintenance projects, as well as economic support and refurbishing existing ones in years to come.

In this sense, the task of communication and promotion of the ISRM becomes fundamental, the human factor being one of the most important assets—developing research, discussion forums and symposiums that allow the technology for sustainable development of that our civilization requires.

The North American region is formed by three countries with very different interests and development and whose economic integration has given important results and led the search to intensive scientific and technological interchange mechanisms.

Alfredo Sánchez Gómez was born in 1960 in Mexico. He obtained his B.Sc. with honors in Civil Engineering at the National Polytechnic Inst. (IPN), México DF Mexico in 1983; and his M.Sc. in Soil Mechanics at the National Autonomous University of Mexico (UNAM).

He has worked as a Professor in the Department of Civil Engineering at Iberoamerican University in Mexico, and a Guest Professor, teaching Tunnel Construction, at the UNAM. Currently he is General Manager of Civil Engineering at ICA Ingeniería.

Prof. Alfredo Sánchez is a member of the ISRM through the Mexican Rock Mechanics Society (SMMR). He was President of the SMRR from 1996-98. He was the Director Secretary of the Mexican Association of Tunnels and Underground Excavations 1995-96, and from 1990-92 the Chairman of the Mexican Soil Mechanics Society. He is also a member of the Mexican Seismic Engineering Society.

Prof. Sánchez is the author or co-author of several papers on design parameters of tunnels, slurry walls, and underground excavations for subway lines and other structures; he has been involved in the foundation design and construction procedures of several tunnels and bridges in Panama and Honduras, as well as in the geotechnical project and other associated jobs of a 9.2 km long aqueduct in México.

Currently, he is involved in the geotechnical project and construction of the subways in Mexico and Monterrey, in areas such as tunneling, construction procedure performance, and open cut stability.

ISRM Vice-President for South America

Gianfranco Perri Aprile
(nominated by the ISRM NG Venezuela)

1. To be active in meetings and other initiatives taken by the Board, to lead the point of view of the South American societies and to permanently keep the South American societies informed about these Board decisions, policies, suggestions and initiatives.

2. To keep permanently active the flow of information toward the Board about the activities and the necessities of all the different South American societies, transmitting verbal and all written information received through the visits that I will personally give to all of the South American societies.

3. To ask for the Board's institutional support for the regional, scientific, technical and cultural activities of each South American society and the region.

4. To have relations of exchange and cooperation with the rest of the regions of the ISRM represented on the Board.

Gianfranco Perri Aprile was born in 1951, in Brindisi, Italy. He obtained his Ph.D. cum laude in Mining Engineering at the Politecnico di Torino.

From 1974-75 he was Professor of Geomechanics at the Politecnico di Torino, and from 1975-78 Professor of Geotechnique and Rock Mechanics at the Escuela Superior Politécnica del Litoral de Guayaquil, Ecuador Prof. of Geostatistics at the Univ. de
Guayaquil, and Prof. of Geotechnique for Roadways at the Univ. Católica de Guayaquil. Since 1978 he is Professor of Geotechnical Design of Tunnels and Slopes at the Univ. Central de Venezuela in Caracas, where he has also been Chief of the Mining Engineering Department for six years.

Prof. Perri is a member of the ISRM through the Venezuelan Society for Geotechnique (SVG). He was President of the SVG (then called the Venezuelan Society for Soil Mechanics and Foundation Engineering) from 1991-92.

Prof. Perri is the Director and a Founding Partner of the firm Geomecânica Ing. Consultores in Caracas since 1980. He is the designer for the tunnel of Line 3 of the Las Adjuntas-Los Teques Line of the Metro de Caracas. Currently, he is the geotechnical designer of the Caracas-Tuy Medio railway line tunnels, and responsible for the geotechnical aspects of the civil works of this 40 km railroad; he is also a geotechnical consultant for the Venezuelan Metro de Valencia, and Metro de Maracaibo projects, and the geotechnical designer of Line 4 of the Metro de Caracas.

Eurípides do Amaral Vargas Júnior (nominated by the ISRM NG Brazil)

The Brazilian Association of Soil Mechanics and Foundation Engineering (ABMS), through its rock mechanics committee (CBMR) has put forth my name as a candidate for the ISRM Vice Presidency for South America. After a little hesitation, I finally agreed to be a candidate for that post. The world of rock mechanics in Brazil is not unfamiliar to me, having served for two terms as President of the Brazilian Committee in Rock Mechanics (CBMR). On the academic side, since after finishing my Ph.D. at Imperial College, London in 1983 I have been dedicated to research and teaching in rock mechanics, both at the Department of Civil Engineering of Catholic University and the Department of Geology of Federal University, both in Rio de Janeiro. I have also served as advisor to a considerable number of M.Sc. and Ph.D. students at both institutions. These former students are now professors in universities in Brazil or busy working as Geotechnical Engineers. I note that amongst the students that I have had and have at the moment, a number of them came from South American countries with whom Brazil has established educational links in graduate courses. Through the years I have also had contact with the industry mainly through research projects, and to a lesser extent through consulting activities. Rock mechanics in Brazil, and I believe the same trend goes for other South American countries, is shifting its emphasis from Civil Engineering activities to mining and petroleum related activities. If elected Vice President, I plan to strive to find ways inside ABMS, the Brazilian Association of Soil Mechanics, to establish stronger contacts with mining and petroleum professionals working in rock mechanics. In particular, I have already established contacts with the representatives of the Society of Petroleum Engineers (SPE) in Brazil with that objective. Petrobras, the Brazilian National Oil company, has its headquarters and research center (CENPES) located in Rio de Janeiro. Also, CENPES and Petrobras as a whole have strong links with other South American oil companies. It is known that in other South American countries such as Argentina, Venezuela, Peru, Colombia and Bolivia, petroleum activities are very important. ISRM should, in my view, work hard to attract petroleum related professionals in South America. I plan to work in that direction, using my research links with CENPES and Petrobras. At the University I plan to continue my activities attracting motivated students to rock mechanics work, organizing special courses, bringing foreign visitors, and trying to extend such activities to other South American countries. Finally, being elected Vice President for South America, I plan to establish strong communication channels with the local rock mechanics communities in the various countries in order to best represent their interests.

Eurípides do Amaral Vargas Júnior was born in 1948 in Pompeia SP, Brazil. He obtained his B.Sc. in Civil Engineering at the School of Engineering of São Carlos, Univ. of São Paulo, São Carlos SP, Brazil in 1972; his M.Sc. in Geotechnical Engineering from the Department of Civil Engineering, Catholic Univ. of Rio de Janeiro in 1975, with the thesis "Study and Applications of a Simple Electro Analogue to the Solution of Practical Problems of Flow in Porous Media"; his M.Sc. in Engineering Rock Mechanics at the Royal School of Mines, Imperial College, London Univ. in 1978, with the thesis "A Simple Electrical Analogue Model and the Boundary Integral Equation Method (BIFEM): A Critical Evaluation of Their Applicability to the Study of Water Flow in Porous Media"; and his Ph.D. in Rock Mechanics also at the Royal School of Mines, in 1982, with the thesis "Development and Application of Numerical Models for the Behavior of Fractured Rock Masses."

Since 1991 he has been Associate Prof. of Civil Engineering, Catholic Univ. of Rio de Janeiro (PUC Rio), and since 1988 Visiting Prof. at the Dept. of Geology, Federal Univ. of Rio de Janeiro (UFRJ).

Prof. Eurípides Vargas is a member of the ISRM through the Brazilian Committee of Rock Mechanics (CBMR), and the Brazilian Association of Soil Mechanics and Foundation Engineering (ABMS) (ISRM NG Brazil). He has been the President of the CBMR for two terms. He is also a member of the ISSMGE and the ITA, as well as of the Brazilian Association of Engineering Geology (ABGE), the Brazilian

--- ISRM Board Candidates, continued on page 73 ---
Guest Editorial

In rock mechanics and rock engineering we are often faced with difficult problems, but few can compare with the difficulties of modeling the thermo-hydro-mechanico-chemical processes associated with a radioactive waste repository—and then obtaining permission to excavate and operate the repository. In fact, the process of designing a repository to contain spent nuclear fuel and other radioactive waste is unique in rock engineering terms for a combination of four main reasons:

- unlike the location of a civil engineering project or the location of an orebody for mining, there is wider flexibility (in principle) in choosing the location of the repository, i.e., the project location may not be pre-determined;
- in comparison to the functional objectives of other rock engineering schemes, the objective of the repository is that very little should happen, with only an acceptably low level of radionuclide migration;
- the design life of the scheme, being in the order of tens of thousands of years and up to a million years, is well beyond that of any other rock engineering project; and
- a high degree of design confidence is required because there is little opportunity to observe how the facility performs in the long-term, nor can any necessary remedial action be easily implemented.

Because of the long design life resulting from the half-lives of the radioactive materials, the repository cannot be designed by precedent practice. So, there has to be a systematic consideration of all thermal, hydrological, mechanical and chemical effects that could prejudice the integrity of the repository and its man-made and natural barriers in the short and long terms. Then an adequate model for the radionuclide migration can be developed.

The rock mechanics and rock engineering subjects form a crucial component of the overall repository design studies. Many countries have conducted rock mechanics research to support their national programs, often involving considerable amounts of money; the cost of the characterization program at the potential repository site at Yucca Mountain in the USA had already reached $2.5 billion in 1996. Table 1 has been generated from a worldwide review, edited by Witherspoon*, of international activities in this field.

This special issue contains reviews of some of the work that has been conducted in four countries.

- Finland: an article concentrating on the site investigation program for four repository candidate sites.
- Sweden: an article on the interim storage facility, repository siting, the Åspö Hard Rock Laboratory and the DECOVALEX program.
- United Kingdom: an article on the lessons learned during the last 20 years of rock mechanics research funded by the Department of the Environment and Nirex.

Table 1. Types of anticipated repository host rock for high level radioactive waste disposal in different countries

<table>
<thead>
<tr>
<th>Country</th>
<th>Rock Type</th>
<th>Progress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Belarus</td>
<td>Clay, salt</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>Belgium</td>
<td>Clay</td>
<td>URL**, since 1980</td>
</tr>
<tr>
<td>Bulgaria</td>
<td>Granite, marl</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>Canada</td>
<td>Granite</td>
<td>URL, since 1980</td>
</tr>
<tr>
<td>China</td>
<td>Granite</td>
<td>Initial area screening</td>
</tr>
<tr>
<td>Czech</td>
<td>Granite</td>
<td>Initial assessment</td>
</tr>
<tr>
<td>Finland</td>
<td>Granite</td>
<td>3 candidate sites</td>
</tr>
<tr>
<td>France</td>
<td>To be determined</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>Germany</td>
<td>Salt</td>
<td>Specific sites identified</td>
</tr>
<tr>
<td>Hungary</td>
<td>Claystone</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>India</td>
<td>Granite</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>Indonesia</td>
<td>Basalt</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>Japan</td>
<td>To be determined</td>
<td>Mine experiments</td>
</tr>
<tr>
<td>Netherlands</td>
<td>Salt</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>Poland</td>
<td>To be determined</td>
<td>Initial assessment</td>
</tr>
<tr>
<td>Slovakia</td>
<td>To be determined</td>
<td>Initial assessment</td>
</tr>
<tr>
<td>Spain</td>
<td>To be determined</td>
<td>Initial assessment</td>
</tr>
<tr>
<td>Sweden</td>
<td>Granite</td>
<td>URL, since 1990</td>
</tr>
<tr>
<td>Switzerland</td>
<td>Clay, granite</td>
<td>URL, since 1992</td>
</tr>
<tr>
<td>Ukraine</td>
<td>Granite, salt</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>United Kingdom</td>
<td>Volcanics</td>
<td>Initial evaluation</td>
</tr>
<tr>
<td>United States</td>
<td>Tuff</td>
<td>URL + Viability assessment</td>
</tr>
</tbody>
</table>

**Underground Research Laboratory

— Guest Editorial, continued on page 80
Rock Engineering and Technical Design of the Finnish High Level Nuclear Waste Repository

By Reijo Riekkola* and Jukka-Pekka Salo**

In accordance with Finnish Government guidelines, Posiva Oy is running a site investigation program to examine the suitability of four candidate sites to host a high-level nuclear waste repository. The final selection of the host site will take place in 2000, allowing the start of construction of the repository in the 2010s. A research and development program has been run concurrently, with the aim of developing the technology for encapsulation and disposal. The performance analysis of spent fuel disposal will be submitted to the authorities by the end of 2000, together with the updated technical plans. Repository layout and technical design will be adapted to prevailing rock conditions at the site. Suitable areas for construction of the repository are currently being identified at each site.

Introduction
Electricity is an important source of energy in Finland, and its share in total energy consumption continues to rise. Nearly 30 percent of the electricity consumed in Finland is produced by nuclear power and the rest is mainly generated by hydro, coal, oil and peat. The first commercial nuclear reactor in Finland began operations in 1977. Electricity is now generated by four reactors at two sites. Fortum Power and Heat Oy operates the nuclear power plant units in Lovisa and Teollisuuden Voima Oy (IVO) in Olkiluoto.

The principles of nuclear waste management have been prescribed in the Nuclear Energy Act and Statute which came into effect in 1988 and define obligations, license proceedings and liability provisions. In 1994, the Act was amended to state that all nuclear waste generated in Finland shall be disposed of within the Finnish borders, and that nuclear waste from abroad shall not be imported into Finland. The power companies are responsible for the safe implementation of nuclear waste management and for all costs of waste management.

The Council of State must make a decision in principle regarding the final repository of spent nuclear fuel. A precondition for a favorable decision shall be the supporting statement on the safety of the final disposal, to be issued by the Finnish Center for Radiation and Nuclear Safety, and the approval of the municipality where the final repository will be built. The decision made by the Council of State shall then be adopted by Parliament. Further state licenses are needed to construct and operate the repository.

Nuclear power produces two types of radioactive waste: low- and medium-level operating waste and high-level spent fuel. Furthermore, decommissioning of the power plants produces radioactive decommissioning waste.

Low- and medium-level radioactive waste is temporarily stored at the power plants and at their interim storages, usually in solid form and then packed. Later it will be disposed of in facilities excavated in the bedrock at the sites. This kind of final disposal facility for operating waste—the VIJ Repository—began operation in 1992 at Olkiluoto in Eurajoki and at Hästholmen in Lovisa in 1997. Later, these facilities will be enlarged to accommodate the decommissioning waste.

Spent nuclear fuel will be accumulated at the Lovisa and Olkiluoto power plants during the 4C year operation of the power plant units. Spent fuel assemblies removed from the nuclear reactors are first transferred into water basins in the reactor building to cool down. After a few years of cooling they will be transferred to the interim storages for spent fuel situated at the power plant sites. The spent fuel assemblies will be further stored in these pools of water before their disposal into the bedrock, which will begin in 2020. During storage the radioactivity of the spent fuel will diminish, which will facilitate the final disposal operations.

The estimated total amount of spent fuel and the number of waste canisters corresponding to a 40 year operation time for the power plants is about 2,600 tU and 1,400 canisters respectively. If operation times were extended up to 60 years, the corresponding amount of spent fuel would be about 4,000 tU with 2,200 canisters. If two new nuclear power plants were to be constructed, the amount of spent fuel corresponding to 60 years of operation time for all these power plants would be about 9,000 tU. This figure represents what is believed to be the maximum likely volume of spent fuel.

Site Selection
The search for the final disposal site was begun by nationwide screening research in 1983 (see Figure 1). In 1987 more precise bedrock investigations were started at five localities in different parts of Finland. In 1993 Olkiluoto at Eurajoki, Kivett at Äänekoski and Romuvaara at Kuhmo were selected for more detailed investigation. Studies at Hästholmen in

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Figure 1 (left). Development of the site selection and characterization program from 1983 onwards. This program was preceded by a general survey of the bedrock of Finland to examine the possibilities for final disposal of spent fuel.

Lovisa were started in 1997 on the basis of conclusions from the preliminary study results. Investigations at Eurajoki, Äänekoski, Kuurno and Lovisa (Figure 2) will continue until the selection of the final disposal site is made at the end of 2000. Then an investigation shaft or tunnel will be constructed deep in the bedrock at the selected site.

The bedrock area suitable for final disposal must be geologically stable and without large fracture zones. In addition it should not have any potential for mining or other similar activity which could lead to excavating the bedrock in the area of the final disposal facility. Fracturing and hydraulic conductivity as well as movements of the groundwater in the bedrock at the investigation sites are to be ascertained through geological research because only the groundwater would transfer radioactive substances into the environment. Data is acquired by doing extensive ground surface mapping and measurements as well as by boring numerous deep investigation holes. Migmatic gneisses with cordierite, sillimanite and garnet porphyroblasts are the most abundant rocks at the Olkiluoto investigation site. The bedrock of the Kivetty investigation site consists almost entirely of plutonic rocks, por-

Figure 2. Site investigation areas in Finland
phyritic granodiorite and porphyritic or equigranular granite being the most common. The most abundant rock type in the eastern and western parts of the Romuvaara investigation site is banded and migmatitic tonalite gneiss, while in the central part of the area leucotonalite gneiss is dominant. The bedrock of the Hästholmen site consists of rapakivi granites.

Together with the geological research, an environmental impact assessment (EIA) will be carried out at all investigation sites. The EIA will evaluate economic, social and environmental impacts of final disposal at each possible site.

**Technical Description of the Repository**

Alternative repository designs for the disposal of spent nuclear fuel were assessed by Autio et al. (1996). The basic KBS-3 type repository was recommended for further development. The KBS-3 design was evaluated to be robust and flexible. In addition, the transfer, emplacement and sealing operations were considered technically uncomplicated.

The basic concept for the disposal of spent fuel from the nuclear power plants run by TVO and Fortum Power and Heat Oy is based on the emplacement of encapsulated spent fuel in crystalline bedrock at a depth of 400–700 m (Figure 3). The selection of the final depth for the repository will be based on site-specific properties and conditions in the bedrock. The total excavated volume of the repository, assuming the operation of existing reactors for 40 years, is estimated to be about 400,000 m$^3$.

The design of the basic concept is based on the use of the cast insert canister. The diameter of the canister for BWR fuel from TVO is 1,052 mm and the length is 4,800 mm. The canister length for the VVER-440 type PWR fuel from Fortum Power and Heat Oy is 3,600 mm, the diameter of both canisters being the same.

The spent fuel canisters are emplaced in vertical holes which are excavated in the floor of the deposition tunnels. The diameter of the deposition holes is 1,752 mm and the spacing of the holes is based on site-specific thermal properties of the rock mass. Spacing of the tunnels is 25 m and the current design intent spacing of the deposition holes is 7.5–8 m. The depth of the deposition holes is 7.8 m for the TVO type canisters and 6.6 m for Fortum Power and Heat Oy type canisters. The deposition tunnels will subsequently be backfilled with crushed rock and bentonite.

An encapsulation plant will be constructed above the tunnels on the ground surface where the spent fuel rod assemblies will be packaged in canisters made of copper and nodular cast iron. Access from the surface to the repository will be provided by three vertical shafts: a work shaft, a personnel shaft and a canister shaft. An inclined tunnel can also be used to provide access to the repository.

**Principles of Implementation**

Construction work will begin with the excavation of the investigation shaft or tunnel using the drill and blast technique. This may be followed by the excavation of additional investigation tunnels at the repository level for characterizing the rock mass in situ. Raise boring may be employed in the excavation of other shafts. It is proposed that the investigation shaft will be modified later to be used as the work shaft. Construction of the repository is planned to start at the beginning of the 2010s and will include the remaining shafts, the rest of the central tunnel and the first part of the deposition tunnels, as well as other tunnels and surface facilities.

The operational phase is planned to start in 2020. The reference concept is based on an approach in...
which deposition tunnels are excavated stepwise during the operational phase, simultaneously with the installation of the waste canisters. The simultaneous excavation and installation of canisters appears to have advantages over the alternative approach, in which the whole repository is excavated before starting the operational phase. Figure 4 illustrates the steps from excavation to backfilling that are needed for simultaneous excavation and operation.

The central tunnel, deposition tunnels and other facilities will be excavated by a carefully controlled drill and blast technique. A smooth blasting technique will be employed and the blast design adjusted to the prevailing rock conditions and excavation disturbance constraints. The most advantageous method for the excavation of deposition holes is considered to be boring based on rotary crushing and vacuum dry suction of the crushed rock (Figure 5), a technique demonstrated in the research tunnel at Olkiluoto (Autio & Kirkkomäki 1996). After the final disposal of the last canisters the encapsulation plant will be decommissioned, the remaining empty tunnels will be filled with bentonite and crushed rock and the access shafts will be sealed.

**Rock Mechanical Properties and Analyses**

Knowledge of rock mechanical properties is of great importance when planning a deep repository. The rock strength, in-situ state of stress and the stress-strain behavior of rock affect the choice of repository depth, as well as the shape and orientation of the deposition tunnels and disposal holes. The thermal properties of the rock must be taken into consideration, too, because tempera-
tures are expected to increase in the immediate vicinity of the repository due to the radiogenic heat produced by the spent fuel.

The strength properties of the different rock types have been studied by carrying out preliminary loading tests in the field with more detailed tests taking place in the laboratory. As an example, Table 1 presents the mean values for the uniaxial compressive strength ($\sigma_{uc}$), tensile strength ($\sigma_t$), Young's modulus ($E$) and Poisson's ratio ($\nu$) of the rock types at Romuvaara (Matikainen & Simonen 1992, Kulla 1994, Tolppanen et al. 1995, Johansson & Autio 1995, Heikkila & Hakala 1998). The majority of the tests took place on tonalite gneiss which is the dominant rock type at Romuvaara.

In-situ stresses have been measured in deep boreholes both by hydraulic fracturing and also by overcoring methods. The results show that the stresses increase with depth. As an example the maximum horizontal stress ($\sigma_h$) at Romuvaara has a value of about 28–30 MPa at a depth of 500 m (Figure 6). The maximum principal stress is sub-horizontal with an average dip of nine degrees and is oriented about southeast–northwest over the depth interval 250–450 m, rotating to lie approximately east–west at greater depths.

The strength of the rock and the in-situ stress field are important factors when considering a suitable range of depths for the repository. So far, estimates of geotechnical behavior have been based on empirical methods, and the Q-system, as well as on numerical analyses which have been used in several phases of the project. The latest rock mechanical tests and site investigations provide input parameters for numerical 3DEC analyses that are currently being made for each site. An example of a 3DEC model that was used for rock mechanical calculations in the investigation phase in 1992–96 is shown in Figure 7 (Tolppanen et al. 1995).

**Preliminary Layouts of the Repository**

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Uniaxial compressive strength $\sigma_{uc}$ (MPa)</th>
<th>Tensile strength $\sigma_t$ (MPa)</th>
<th>Young's modulus $E$ (GPa)</th>
<th>Poisson's ratio $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tonalite gneiss</td>
<td>197.4 (51.4)</td>
<td>11.0 (3.0)</td>
<td>72.7 (7.0)</td>
<td>0.26 (0.06)</td>
</tr>
<tr>
<td></td>
<td>n=26</td>
<td>n=25</td>
<td>n=40</td>
<td>n=40</td>
</tr>
<tr>
<td>Leucotonalite gneiss</td>
<td>214.1 (41.6)</td>
<td>75.9 (4.3)</td>
<td>69.9 (6.0)</td>
<td>0.28 (0.03)</td>
</tr>
<tr>
<td></td>
<td>n=6</td>
<td>n=6</td>
<td>n=6</td>
<td>n=6</td>
</tr>
<tr>
<td>Mica gneiss</td>
<td>99.4 (11.1)</td>
<td>49.9 (10.0)</td>
<td>69.9 (6.0)</td>
<td>0.29 (0.05)</td>
</tr>
<tr>
<td></td>
<td>n=6</td>
<td>n=6</td>
<td>n=6</td>
<td>n=6</td>
</tr>
<tr>
<td>Granodiorite</td>
<td>171.6 (2.7)</td>
<td>70.0 (0.1)</td>
<td>70.0 (0.1)</td>
<td>0.3 (-)</td>
</tr>
<tr>
<td></td>
<td>n=2</td>
<td>n=2</td>
<td>n=2</td>
<td>n=2</td>
</tr>
<tr>
<td>Metadiabase</td>
<td>239.9 (101.7)</td>
<td>100.5 (8.7)</td>
<td>100.5 (8.7)</td>
<td>0.28 (0.01)</td>
</tr>
<tr>
<td></td>
<td>n=3</td>
<td>n=3</td>
<td>n=3</td>
<td>n=3</td>
</tr>
</tbody>
</table>

Table 1. Strength and deformation properties of the rock types at Romuvaara. Values presented are arithmetical means, standard deviation is given in brackets, $n=$number of samples
Figure 8. Illustrative example of the area required by deposition tunnels at Romuvuara

orientation of the fracture sets and by the location of the fracture zones. The orientation selected is likely to be a compromise between the stress orientation and the various relevant properties of the rock mass.

The depth of the repository is determined mainly by the maximum horizontal stress, the strength of the rock, the location of the fracture zones and the groundwater chemistry, as well as by long-term safety aspects.

Based on the classification of the bedrock, an illustrative example of the area needed for the construction of deposition tunnels for 2,200 canisters, corresponding to 4,000 tU, is shown in Figure 8. The area shown has been adapted to the volumes of rock that have been recognized as being potentially suitable for constructing the repository. It is located at a depth of 500 m in the central part of the borehole investigation area and extends some hundreds of meters outside the area covered by the existing boreholes. This example is based on the assumption that the construction of deposition tunnels will take place exclusively in tonalite gneiss, leaving a minimum distance of 50 m between the deposition tunnels and

Figure 7. 3DEC model of deposition tunnel and deposition hole. Location of points of maximum compressive stress and dimension of the model (Tolppanen et al. 1995)
the water-bearing structures, R3, R9, R13, R14, R21, R22 and R31.

Still, there are uncertainties concerning the exact locations, dimensions and properties of bedrock structures. The repository layout area shown in Figure 8 should be considered to be only preliminary and illustrative. The details of the repository layout and the locations of the deposition holes will be determined using an observational method process during construction, such that the repository layout will be refined and adjusted to the local structure of the bedrock as new data on rock properties become available.

Information on in-situ conditions will be obtained by investigations during excavation of the investigation shaft, during excavation of the central tunnel, in the construction phase and during excavation of the deposition tunnels and boring of the deposition holes in the operational phase. Finally, the suitability of a potential location for a deposition hole may be assessed on the basis of the characterization and water inflow measurements in a small diameter pilot borehole, which may be drilled down the centre of each deposition hole.

Figure 9 illustrates a possible surface development for the Romuvaara site, and includes structures such as the encapsulation plant, storage areas for rock spoils, etc. The locations of the encapsulation plant and office buildings, as well as the work shaft buildings, will have a close relationship with the repository design, because it is intended that they are to be located above the three shafts.

Acknowledgements

Posiva Oy is acknowledged for support and permission to use this material.

References


By Ove Stephansson

The Swedish Nuclear Fuel and Waste Management Company (SKB) is responsible for ensuring that Sweden's radioactive waste from nuclear power, medical care, industry and research is disposed of in a safe manner. Today SKB has a transportation system, a final repository for radioactive operational waste (SFR) and interim storage for spent nuclear fuel (CLAB). The repository for operational waste is located underground at the nuclear power station of Forsmark, about 300 km north of Stockholm. The central interim storage facility for spent nuclear fuel is located in an underground rock chamber adjacent to the nuclear power station at Oskarshamn, in southeastern Sweden. An additional rock chamber of CLAB is under construction. A deep repository and an encapsulation plant for the spent fuel remain to be built.

After approximately 40 years of interim storage, the radioactivity and heat output of the spent fuel will have decreased by about 90 percent. Then it is time for encapsulation of the fuel elements in copper canisters with inner steel containers. Today SKB has built a Canister Laboratory in Oskarshamn for development of encapsulation technology and training of personnel for the encapsulation plant that will be built adjacent to CLAB. After encapsulation, the canisters will be transported to the deep repository and embedded into high compacted bentonite in deposition holes at the floor of tunnels about 500 m down in the granite bedrock. The Swedish system for handling radioactive waste is shown in Figure 1.

Nuclear Waste Management in Sweden

In Sweden more than 90 percent of radioactive waste is generated by nuclear power plants, and the remainder by research facilities, industry, and medical care. Questions related to the handling, storage and final disposal of nuclear waste have been studied in Sweden since the early 1970s. The primary responsibility lies with the owners of the reactors to guarantee safe and efficient management of the radioactive waste. For financing the handling of waste a fee is levied on nuclear electricity production.

Today Sweden has twelve reactors at four different sites (Figure 2) which generate about 10,000 MW, corresponding to about 50 percent of all electricity produced. In 1980 the Swedish Parliament decided, based on the outcome of a referendum, that no more reactors should be built in Sweden and that the ones already existing should be taken out of operation by the year 2010 at the latest. In 1998 the Swedish government decided to shut down one of the two reactors at Barsebäck.

Low-level waste can be handled and stored in simple packages, transported by ship to the Forsmark site (Figure 2) and deposited in the final repository for radioactive operational waste (SFR). Intermediate-level waste must be radiation-shielded for safe handling during seaborne transport to Forsmark and then deposited in underground silos at SFR. The high-level waste and spent fuel require both radiation shielding and cooling during the sea-borne transportation to the central interim storage facility for spent nuclear fuel (CLAB) at Oskarshamn (Figure 2). The total amount of radioactive waste from the Swedish nuclear power program is estimated to be 7,800 tons of spent fuel and 230,000 m³ of low- and intermediate-level waste; about 110,000 m³ of that represents the decommissioning waste. CLAB has 2,700 tons in storage and SFR has 23,000 m³ (as of 1996).

SKB is also responsible for the comprehensive research and development work necessary to provide all facilities required for the management of spent fuel and radioactive waste. State review and assessment of the activities of SKB are carried out by a number of governmental agencies issuing directives for the work of SKB. Every three years SKB has to present their state-of-the-art and future plans for research, development and demonstration (RD&D). The most recent RD&D program was presented to the government in 1998 (SKB, 1998a,b). The program is reviewed by national and international experts, and the Swedish Nuclear Power Inspectorate (SKI) and National Institute of Radiation Protection (SSI) collect and evaluate review comments, judge the content of the program and report to the Minister of Environment.

Central Interim Storage Facility for Spent Nuclear Fuel—CLAB

CLAB is situated on the Simpevarp Peninsula near the Oskarshamn nuclear power station (Figure 2). Construction started in 1980 and the storage facility began operations in 1985. The storage capacity of the existing facility is 5,000 tons of uranium corresponding to about 20,000 BWR assemblies plus 2,500 PWR assemblies of fuel. CLAB consists of a receiving section at ground level, where the casks containing the spent fuel are received and the fuel is unloaded under water. The fuel assemblies are lifted out and placed in a storage canister which is lowered to the underground chamber, 25–30 meters below.
ground level. The rock chamber, which is 110 m long, 27 m high and has a span of 21 m, contains five concrete storage pools. CLAB will be full by around 2004, which means that more space is needed to accommodate all of the fuel from the Swedish nuclear power program. For this reason, an extension of the storage capacity by means of an additional rock chamber with storage pools for an additional 3,000 tons of spent fuel elements is needed. The blasting of the new rock chamber, 40 m away from the existing one, started in January 1999.

After 30–40 years of storage in CLAB, the spent fuel will be encapsulated in canisters. Each canister is about 5 m long and 1 m in diameter and the wall is planned to consist of 5 cm copper. The fuel is placed in an inner steel container; the total weight of each canister will be 25–30 tons. SKB has built and operates a canister laboratory in Oskarshamn. The laboratory will serve as a center for development of encapsulation technology and training of personnel for the encapsulation plant that will be built as an addition to CLAB.

Figure 3 illustrates the present configuration of the CLAB facility, with the receiving building on top of the existing rock chamber and shaft. The new rock chamber is shown under construction and the planned encapsulation plant will be located on the ground surface above the rock chambers.

The rock mass characterization of the underground chamber for the concrete pools of CLAB—phase 1 was done using conventional methods existing in the mid-1970s. Geological mapping, seismic refraction survey for detection of major discontinuities, diamond drilling followed by core mapping, hydraulic pump tests and rock stress measurements by coring were conducted.

The rock mass at the CLAB site is rather complex and consists of a mixture of granites, metavolcanics and greenstones. The rock chamber is located in a fairly large rock plinth surrounded by steeply dipping major discontinuities with high water conductivity.

The design of the chamber was based on the experience gained from constructing a large number of rock caverns for oil and gas storage in hard Swedish rocks. The 110 m long chamber was excavated in three steps; one roof (tunnel) excavated by horizontal drilling and blasting and two benches by vertical drilling and blasting. The roof was reinforced with systematic rock bolting and shotcrete.
and the walls were spot bolted and partly shotcreted. An extensive monitoring program with extensometers and sliding micrometer instruments was installed to control the long-term stability of the cavern (Röhoff et al., 1983). Of special interest to the rock mechanics community is the long-term monitoring of rock displacements in the roof and walls of the chamber and displacements due to warming of the rock chamber.

The spent fuel is loaded in special storage canisters and the canisters are placed vertically in predetermined positions in the concrete pools. The spent fuel generates heat and is cooled by circulating pure water. The average water temperature in the pools is +36°C under normal conditions and the total cooling capacity for all the pools is 8.5 MW. Warm water in the pools increases the air temperature and warms up the surrounding rock mass. Displacements of the roof and walls of the chamber and the convergence of the chamber walls have been recorded from the time of installation in the beginning of 1980 till now.

A common phenomenon of many chambers in hard rock is the reduction of leakage with time due to precipitation of minerals and mineral hydroxides in the existing joint system. At the start of the operation of CLAB in 1985 the groundwater leakage into the facility was in average 60 l/min. Ten years later the leakage is reduced to about 40 l/min with minor fluctuations over the year. The monitoring of displacements and water leakage, with an estimated inflow of 40 l/min, will continue and be more frequent during the construction of the rock chamber for CLAB—phase 2, starting in 1999.

**Construction and control of CLAB—Phase 2**

During excavation of CLAB—Phase 1, preparations were made for an expansion of the facility. A second rock chamber and a small part of the communication tunnel between the two chambers and the entrance to the access tunnel to the new chamber were blasted. The new CLAB—phase 2 chamber now under construction is located only 40 m away from the first chamber, at the same depth and with identical dimensions (Figure 3).

The Nuclear Power Inspectorate in Sweden (SKI) is responsible for the supervision of safety at all nuclear installations, including CLAB. SKI has demanded rigorous planning and safety control for construction of the new rock chamber and, in particular, the blasting procedure. Blasting will be performed at a distance of 40–100 m from 2,700 tons of the spent nuclear fuel from the Swedish nuclear power program! SKB has met the requirements for safety control and is performing extensive investigations and measurements of the existing rock chamber during the excavation of the new chamber.

Of special concern is the risk of the fall of a block from the roof of the existing chamber due to the vibrations from the blasting of the new chamber. The block has to exceed the retaining support of a systematic rock bolting of 3 m long grouted rebars with a central distance of 2 m, two layers of shotcrete (about 15 cm thick) and a corrugated steel plate roof. This event, though not very likely, is a risk and has been studied in detail.

The maximum blasting-induced vibrations allowed at roof, walls and installations of the existing rock chamber is 33 mm/sec. The system of vibration instruments gives a warning when velocities exceed 75 percent of the maximum value. Any displacement of the storage pools exceeding 2 mm will cause action and the blasting procedure will be redesigned. Test blasting has been performed in the access tunnel to the chamber (not shown in Figure 3) and in boreholes and the blast runs for the tunnel blasting and chamber blasting have been designed accordingly.

**Caverns For Radioactive Operational Waste—SFR**

The final repository for radioactive operational waste, called SFR, is located near the Forsmark nuclear power plant (Figure 2), at the Baltic Sea, about 300 km north of Stockholm. The waste from the reactor operation, which is low- and intermediate-level waste and short-lived (<500 as), consists, for example, of ion exchange resin, components and protective clothing contaminated with
radioactive substances. SFR started operation in 1988 and has a current capacity of 60,000 m³ of waste of which about one third was stored by 1998.

The repository is located in granitic rock at a depth of about 50 m beneath the sea bed. The depth of the sea at the site is about 5 m. With the present glacial rebound from the latest glaciation of about 0.5 mm/annum in the Forsmark area, the sea bottom at the repository will reach above sea level in about 1000 years, i.e., when the waste is no longer harmful to the biosphere.

SFR consists of five rock chambers (one silo and four vaults) of different design according to the type of waste to be emplaced. Two entrance tunnels, each 1,000 m long, lead from the coast out to the repository area. The silo—30 m in diameter and 50 m in height—contains most of the radioactive substances. It has been provided with several barriers which make it the most interesting construction at SFR. The barriers consist of the waste package itself and a concrete backfill, the silo's wall of concrete, a thick layer of sand bentonite mixture to seal against water flow and, finally, the grouted rock mass as the immediate surrounding. The waste is placed by remote control in the vertical shafts of the silo and later backfilled with concrete (Figure 4).

An extensive rock mass characterization was made for the design and construction of the facility, (Stille and Fredriksson, 1988). Of particular concern were the expected magnitudes and orientation of the rock stresses in the roof of the silo. Prior to excavation of the silo, overcoring method rock stress measurements were performed. From the three boreholes drilled for stress measurements the typical scatter in magnitude and orientation of stresses for superficial hard rocks in Fennoscandia were obtained (Stephansson, 1997).

Excavation of the silo was performed by drilling and blasting against a central shaft and muck loading from a transport tunnel entering the bottom of the silo. Grouting and rock reinforcement with grouted rebars and shotcrete, and an automatic monitoring system were installed in the roof of the silo. From the start of the operation of the facility in 1988 until now, very small displacements in the roof have been recorded by the extensometers and the arch of the silo remains stable.

**Figure 4. Silos for intermediate-level waste at the operational waste storage facility, SFR**

**Siting of a Repository**

It has now been 25 years since a coordinated Swedish nuclear waste program began to take shape.Important steps forward were the development of the different phases of the so-called KBS concept of deep geological disposal. This system has, in principle, been maintained over the years. Several different barriers are supposed to combine and complement each other to provide the best possible protection and long-term safety (Figure 5). If radioactive substances should for any reason escape from the canister, the buffer and the rock will retard their transport to the surface and the biosphere. This will allow time for the radioactivity to decay so that safety is not jeopardized.

SKB began siting of a deep repository in 1992. At that time an extensive body of information on the bedrock in different parts of the country was already available. Regional studies were performed in about 10 counties. Six communities have been selected for feasibility studies of which two have been eliminated in the municipalities of Mala and Storuman in northern Sweden. Selection of at least two sites for site investigations will take place during 2001. Test drilling should be able to start in 2002 (SKB, 1998a). The site investigation will take 4–8 years and detail characterization and construction an additional 6–10 years. The initial operation of a deep repository for about 10 percent of the total amount of spent fuel and waste is expected around 2012 (SKB, 1998a).

SKB is now developing the technology for deep geological disposal by the following main activities:

- testing canisters at the Canister Laboratory at Oskarshamn;
- developing technology for handling and depositing canisters, backfilling deposition tunnels and retrieving deposited canisters at Åspö Hard Rock Laboratory; and
• completing a safety assessment study (SR'97) during 1999.

Site investigation and site evaluation
The choice of areas for site investigations in Sweden is made in close consultation with public authorities, municipalities and other concerned parties within the framework of an established environmental impact assessment (EIA) process. The site investigation conducted by SKB is divided into two stages: initial and complete site investigation (SKB, 1998a). Site investigations with drilling should be able to start in 2002. Results of the investigations will be compiled in a geoscienctific description of the site. This description will contain discipline-specific models that are strongly coupled to each other (Figure 6).

• minimize the risk for future intrusions and alternative use, such as mines.

To meet these criteria for isolation function, the important parameters to determine during geoscienctific site investigation have been studied by Andersson et al. (1998). Mechanical stability is one of the fundamental safety features of the bedrock and entails that the performance of the buffer and the canister should not be compromised by movements in the rock mass. Displacements of existing discontinuities and initiation and propagation of new fractures may alter the groundwater flow around the repository so that its retention capacity is impaired.

The input to the rock mechanical model comes from the geological and thermal models plus the features, events, and processes that have been identified from scenarios. Information from rock mechanical investigations also forms the input to the model. The modeling itself has to consider the constitutive relations of the rock mass, the input parameters and the modeling tools (codes).

It can be concluded that rock mechanical questions and modeling have so far been given scant treatment in published safety assessment documents by SKB, and that even though general knowledge is available, there is limited experience as to which rock mechanical parameters are important for long-term safety (Andersson...
et al., 1998). SKB is therefore performing a study about the strategies to be applied in handling the rock mechanical issues.

SKB has performed a series of thermo-mechanical modeling of the far-field stability of a repository subjected to a heat load corresponding to about 10 percent of the total amount of waste from the Swedish program. Quantitative rock mechanical modeling of the influence of glaciation on the deformation and stability of the rock mass at the depth of a future repository was conducted by Rosengren and Stephansson (1993). The rock mass is described as being composed of discrete blocks bounded by discontinuities and the geological situation of the SKB test site at Finnsjön was simulated in a series of UDEC models. Loading from a future ice sheet with a thickness of 3 km was simulated and the vault stability was analyzed.

**Instruments and methods for site investigation**

Many of the methods and instruments used in the Swedish nuclear waste program are so specialized and advanced that they are not available on the general market. SKB owns their own instruments for site investigation and hires consultants and firms to organize storage and service for them. SKB has also been a major organization for developing new and advanced methods and techniques for geological site characterization.

Regarding rock stress and rock stress measurements SKB has access to both overcoring with the Swedish State Power Board method, that allows overcoring measurements at depths down to 1,000 m, and the hydraulic fracturing measurement method with the multi-hole system. The hydraulic fracturing equipment at the Hydropower Company of the Swedish State Power Board, described by Amadei and Stephansson (1997), has been modernized and a new 1,000 m long umbilical hose has been installed.

**Åspö Hard Rock Laboratory**

In 1996, SKB decided to construct the Åspö Hard Rock Laboratory to provide an opportunity for research, development and demonstration in realistic and undisturbed bedrock down to the depth of a planned future repository in Sweden. The work at Åspö started 20 years after SKB initiated the research work at the Strippa Mine. The aim of the laboratory work at Åspö HRL is to:

- develop and test site characterization methods;
- verify models describing function of natural and engineered barriers;
- develop, test and demonstrate repository technology;
- provide experience and train personnel for future work in a deep repository; and
- inform outsiders regarding technology and methods for the deep repository.

Information about the location, site investigation, construction and present research activities are presented in Åspö Hard Rock Laboratory (1996, 1998) and SKB (1998a,b,c).

Geoscientific investigations on Åspö and nearby islands began in 1986 and were completed in 1996. At the same time the construction phase was completed. The operation phase of Åspö HRL began in
1995 and is expected to continue for 15–20 years when the first stage of a deep repository in Sweden is expected to be in operation. Research at Åspö HRL is carried out via contracts with universities, institutes, consultants and industrial companies and other Swedish and foreign researchers. The research activity is mostly conducted in project form where one or several international organizations participate. Agreements on participation and cooperation presently exist with ten international organizations. Figure 7 shows the name and location of the ongoing experiments within the Åspö HRL. Of special interest to the rock mechanics community are the completed ZEDEX experiment on the excavation disturbed zone (Emsley et al., 1997) and the Prototype Repository (SKB, 1998b).

The ZEDEX experiment was performed in two parallel tunnels, located 420 m below the ground surface 25 m apart. These were excavated by drill and blast (DrB) and by using a tunnel boring machine (TBM). The initial condition of the rock mass was characterized by tunnel mapping and core logging and in-situ seismic and radar measurements from boreholes (Figure 8).

No evidence of damage from any of the excavation techniques was recorded in the far-field, more than 2 m from the perimeter of the drifts. In the near-field, there was little evidence of damage around the TBM drift, except within 2–3 cm of the drift perimeter. The extent of induced fractures around the DrB drift reached a depth of about 80 cm in the tunnel floor and about 30 cm in the wall drift (Emsley et al., 1997). The extent of the FDZ has also been verified by dye penetration tests in the floor of the DrB drift and the wall of the TBM drift (Åspö HRL, 1998).

**Prototype Repository**

The Prototype Repository Project (Figure 9) is focused on testing and demonstrating the function of the SKB disposal system and its components under realistic conditions on a full scale. It also seeks to compare results with models and assumptions. The objectives are to develop, test and demonstrate appropriate engineering standards, quality criteria and quality systems for the repository method. The evolution of the Prototype Repository will be followed over a long period of time, possibly up to 20 years (SKB, 1998b). The purpose is to gain experience over a long operation period, which can be referred to in the application for licensing a deep repository.

**Governmental Research & Development Activities**

The Swedish Nuclear Power Inspectorate (SKI) is responsible for supervising the safety of nuclear waste management. In order to maintain and develop national and international knowledge in the field of nuclear waste management and to train SKI personnel for review work and inspection of the work conducted by SKB and international nuclear waste handling companies, SKI has initiated several international projects like INTRACOIN, HYDRO-

![Figure 9. Planned layout of the Prototype Repository at Åspö HRL. After SKB, 1998b.](image)


The main tasks of HYDROCOIN were development of geohydrological modeling and code inter-comparison. The objectives of INTRAVAL and GEOVAL were validation of groundwater flow models and their application to radioactive waste isolation. In 1992, SKI took the initiative to start the DECOVALEX project (acronym for DEvelopment of COupled models and their VALIDation against EXPERiments in nuclear waste isolation), (Stephansson, 1996).

**DECOVALEX**

The DECOVALEX project is an international effort to develop mathematical models, numerical methods and computer codes for coupled thermo-hydro-mechanical (THM) processes in fractured rocks and buffer materials for geological isolation for spent nuclear fuel and radioactive waste and validate them against laboratory and field experiments. DECOVALEX I consisted of nine funding organizations and 15 research teams. The activities were organized around the numerical simulation of three benchmark tests (BMT1–3) and six test cases (TC1–6) by international research teams using different mathematical models and computers. The BMTs were hypothetically defined specifically to represent the some or all of the coupled processes of a repository in fractured rocks as either near- or far-field problems. The TCs were previous or ongoing laboratory or field experiments, (Stephansson et al., 1996).

The major scientific findings from studies of all BMT and TC problems in the DECOVALEX I project were that heat conduction in jointed rock masses can be simulated with high accuracy, but displace-
ments and stresses less accurately. Groundwater flow still causes a problem and gives divergent results. Good progress was made and the scientific achievements and lessons learned were transferred to the next phase of the DECOVALEX project.

The International DECOVALEX II project started in 1995 and ended in 1998 and was supported by 11 funding organizations and 11 research teams. The project has undertaken a numerical study of Nirex's Rock Characterization Facility (RCF) proposed shaft excavation at Sellafield, UK as Task 1, and a numerical study of PNG's in-situ THM experiments in the Kamaishi mine, Japan as Task 2.

In addition, the project has undertaken a review of the state of the art of the constitutive relations of rock joints as Task 3 and has presented a report on the current understanding of the coupled THM processes related to design and performance of radioactive waste repositories as Task 4. Results of the initial phases of Task 1 and 2 have been published by Jing et al. (1998a,b) and results of coupled THM issues related to repository design and performance by Stephansson et al. (1999).

The DECOVALEX III co-operative research project is in preparation and will start in 1999 and end in 2002. The project will use the full-scale engineering barrier experiment FEBEX in the Grimsel test site, Switzerland, as one test case (Task 1) and the large-scale heater test at Yucca Mountain, Nevada as the second test case (Task 2). In addition, the project will undertake modeling of a series of benchmark tests related to performance assessment. Treatment of THM issues in safety and performance assessment will be studied in Task 4 of DECOVALEX III.

SITE-94 Project

The aim of the SITE-94 research project conducted by the Swedish Nuclear Power Inspectorate, SKI, was to test and present a methodology for performance assessment for a deep repository of spent nuclear fuel at a real site in Swedish bedrock (SKI, 1997). The main objective of the project was to determine how site-specific data should be assimilated into the performance assessment process and to evaluate how uncertainties inherent in site characterization will influence performance assessment results. Other important elements of SITE-94 were the development of a practical and defensible methodology for defining, constructing and analyzing scenarios. The project used data from the rock mass characterization for the Åspö Hard Rock Laboratory.

The rock mechanic studies conducted within the SITE-94 project dealt with the impact of the stress and deformations in fractured rocks on the safety of a hypothetical repository, due to heating from the spent fuel and mechanical loading of glaciation and deglaciation cycles (Hauck et al., 1995). The computer code 3DEC was used and the computed results were verified by comparison with measured in-situ stresses in two boreholes in the Åspö area, with fairly good agreement. The numerical results indicated that closure is the major mode of fracture deformation due to heating and glaciation.

A numerical study of fracture propagation and fracture coalescence in the immediate vicinity of a tunnel and deposition hole was conducted by Shen and Stephansson (1996).

Conclusions

The Swedish Nuclear Fuel and Waste Management Co (SKB) is responsible for the management and disposal of Sweden's radioactive waste. Most of the waste comes from the 12 nuclear power plants, but some waste also comes from hospitals, industry and research institutions. Today Sweden has the following facilities for handling nuclear waste and spent fuel:

- a transport system, Swap;
- a repository for operational waste, SFR;
- an interim storage for spent nuclear fuel, CLAB;
- an underground hard rock laboratory, Åspö; and
- a canister laboratory, Oskarsnäs.

SKB began situating of a deep repository for spent fuel in 1992 and plans to start site investigations at two sites in 2002. The initial operation of a deep repository is expected around 2012. Rock mechanics work related to situating of a deep repository is limited at present, but work is expected to increase once the site investigations start. The present rock mechanics research and development within the Swedish waste program is concentrated at the Åspö Hard Rock Laboratory. The Prototype Repository project at Åspö contains a number of interesting rock mechanics research topics.

The Swedish Nuclear Power Inspectorate, SKI, has initiated and conducted several international research projects related to radioactive waste disposal over the years. The DECOVALEX project is the most recent, with its effort to develop mathematical models, numerical methods and computer codes for coupled thermo-hydro-mechanical processes for radioactive waste disposal.

References


Lessons Learned from 20 Years of UK Rock Mechanics Research for Radioactive Waste Disposal

By John A. Hudson

Although UK rock mechanics research has been mainly oriented toward mining and civil engineering, in the two periods 1979–86 and 1991–97 there have been studies directed specifically toward information for underground radioactive waste disposal. An overview of the 1979–86 Department of the Environment (DoE) work is presented and the more recent 1991–97 Nirex work associated with the Sellafield site in west Cumbria is described. The latter work included a major site investigation, repository shaft and cavern analyses, coupled hydro-mechanical system models, continuum and discontinuum predictions of shaft stability, and the ideas of prediction protocols and back-analysis discriminators. The lessons learned from both programs and the ways ahead are summarized in ten recommendations at the end. Following a period of uncertainty in the UK program since 1997, the 23 March 1999 report of the House of Lords Select Committee on Science and Technology has now recommended that a new commission be established to develop and oversee future strategy for radioactive waste disposal and that a new company be formed to design and construct the repository and dispose of the waste—and that the Government should act without delay.

Introduction and Historical Overview

In the UK, rock mechanics work in the 1950s and 1960s supported the internal coal mining industry and the overseas metallurgical mining industry. In the 1970s, 80s and 90s, rock mechanics research for mining continued but was extended to civil engineering, providing information and techniques for projects such as the Dinorwig hydro-electric scheme and later the Channel Tunnel. Thus, rock mechanics research for mining and civil activities has been a continuing thrust over the years—by both academia and industry. Three seminal rock mechanics books were produced in the 1970s and 80s: Hock and Bray (1977); Hock and Brown (1980); and Brady and Brown (1985).

However, rock mechanics research for radioactive waste disposal occurred during two specific periods (see Table 1):

- 1979–86, when work was funded by the Department of the Environment; and
- 1991–97, when work was funded by UK Nirex Ltd.

The 1979–86 generic research was organized by the Building Research Establishment on behalf of the Department of the Environment. Although some of the research work was conducted in-house, 21 research contracts on rock mechanics and related subjects were let to industrial and academic organizations. The 1991–97 work relates to the investigations by Nirex at the Sellafield site in west Cumbria and related issues. Most of this work, which included drilling and studying 29 deep boreholes and the associated core, was also contracted out to industrial and academic organizations.

The UK has produced radioactive wastes since the 1950s. These wastes are classified by the Parliamentary Office of Science and Technology (1997) as:

- **Very Low Level Waste** (VLLW), which can be treated as non-radioactive waste.
- **Low Level Waste** (LLW), which requires no shielding; it is placed in drums in containers with a cementitious grout, and is buried in a shallow site at Drigg in Cumbria. By 1995, 850,000 m³ of LLW had been deposited at Drigg.
- **Intermediate Level Waste** (ILW), which has been near to the active materials in a nuclear reactor and requires shielding and special handling. About 75,000 m³ of ILW currently exists.
- **High Level Waste** (HLW), which is produced when spent fuel is reprocessed and generates significant heat. About 700 m³ of HLW currently exists. There is a relatively low volume of HLW, but it contains about 75 percent of all the radioactivity present in the irradiated fuel.

There has been discussion about these classifications in terms of the hazards, isotope half-lives, harmfulness of the wastes, and the link between the classification and the waste management approach.

### Table 1: Rock mechanics research in the UK

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The first major milestone toward disposal was the 1976 report by the Royal Commission on Environmental Pollution (the Flowers Report) which recommended that a national disposal facility should be built. The Radioactive Waste Management Advisory Committee (RWMAC) was formed in 1978 to provide advice to the Government. Also, because the Flowers Report recommended that the strategy for waste disposal should rest with the Secretary of State for the Environment, the generic DoE rock mechanics research program was undertaken in the period 1979-1986, and coordinated by the Building Research Establishment. This program is reviewed in the next section.

Another main recommendation of the 1976 Flowers Report was that a nuclear waste disposal corporation should be established. This resulted in the formation of NIREX in 1982, initially with no legal status. However, in 1985 as UK Nirex Ltd., Nirex became responsible for finding and developing a site for ILW and H.W. A difficult period followed involving a series of proposed disposal sites: an anhydrite mine at Billingham in Cleveland, an abandoned clay pit at Balspaw, Killingholme on Humberside, Fulbeck in Lincolnshire, and Bradwell in Essex. Following much public opposition, it was decided to use a deep underground site. In 1989, geological investigations at Dowra in Caithness and Sellafield in west Cumbria were approved. In 1991, Nirex concentrated its efforts on the Sellafield site—resulting in the second period of rock mechanics research from 1991–1997. This period ended on 17 March 1997 when Nirex did not obtain planning permission to construct an underground Rock Characterization Facility (RCF) at the Sellafield site.

This decision was unexpected and a period of uncertainty followed while the report of the House of Lords Select Committee on Science and Technology was awaited. This report has just been published (23 March 1999) and it recommends that the underground disposal of all the UK’s radioactive waste should be pursued as a matter of urgency (see The Way Ahead for the UK).


**Background to the research program**

The first major milestone towards disposal was the 1976 report by the Royal Commission on Environmental Pollution. This was followed in 1977 by a Government White Paper, “Nuclear Power and the Environment,” giving the responsibility for nuclear waste management policy to the Secretary of State for the Environment, together with the Secretaries of State for Scotland and Wales. In 1980, scientific responsibility for geotechnical aspects of the research program was given to the Building Research Establishment (BRE) of the DoE. A program of internal research plus 21 external contracts was developed and managed by the author. At that time, land disposal in clay and seabed disposal were also being considered, so research work was also conducted in these areas, but most of the work was related to rock mechanics and rock engineering. The 1979–1986 DoE program is summarized in Rock Engineering Consultants (1986) and Cooling and Hudson (1987).

The internal BRE work involved experiments in granite at the Troon experimental mine in Cornwall to study rock fracture characterization, seismic tomography for rock mass characterization, and hydraulic fracturing for rock stress determination (Cooling and Hudson, 1987, Hudson and Cooling, 1988, Cooling et al., 1988). The results of the in-situ rock stress determinations in the Cornish granite resulting from this program are shown in Figure 1.

![Figure 1. Orientation of the principal rock stresses in the Cornish granite (BRE program, Cooling et al., 1988)](image-url)
Figure 2. Location of the Sellafield site (from Nirex, 1993)

The suite of 21 research contracts let by DoE through BRE covered the following subjects:

- Repository schemes for high-level radioactive waste disposal
- The disposal of high-level radioactive wastes into crystalline rock formations
- Geological factors pertaining to the efficacy of H.W disposal in the UK
- The properties of discontinuous rock masses
- Simulation of fluid flow in fractured rock

6. A seismic transmission tomography technique for rock quality evaluation
- The seismic investigation of rock properties at the Carwynnen test mine
- A preliminary investigation of vertical crustal movements in the UK

- The state of stress in British rocks
- Determination of near surface in-situ stresses
- The measurement of in-situ stress in near surface environments
- Measuring in-situ stress in deep boreholes
- Numerical modeling of water flow in rock masses
- Computer programs for the numerical modeling of water flow in rock masses
- Some rock mass assessment procedures for discontinuous crystalline rock
- A critical review of the data requirements for fluid flow through fractured rock
- Field implementation and testing needs for a high level waste repository
- Effect of underground construction on the geohydrology of proximate rocks

Figure 3. WSW-ENE section through the Sellafield site (from information supplied by Nirex)
Rapid advance tunneling with special reference to its implications for repository excavation
Geotechnical site assessment methodology I
Geotechnical site assessment methodology II

These contracts were let to government laboratories, consulting civil and geotechnical engineering firms, and universities.

**Summary of the DoE rock mechanics research results**
Apart from the "project specific" conclusions associated with each project, there were three overall conclusions arising from the rock mechanics work:
- There is a need for a coupled approach
- Consideration of the long-term effects is required
- A multi-phased approach to site investigation should be adopted.

The studies indicated that not only should each of the subjects be studied within the context of the total system but that the related mathematical models should be coupled. The consideration of the long-term effects refers to the fact that site investigation is a snapshot in time but the rock properties can change with time. For example, the conductance of a rock fracture can change by several orders of magnitude as a result of changes to the water flow rate, changes to the rock stress components and changes induced by proximate excavation. The requirement for a multi-phased approach to site investigation results from the previous two conclusions and because the required rock mechanics information is different for the different stages of site selection, repository design and excavation, emplacement, sealing and post-closure. It was also noted that an underground geotechnical testing facility operating over a period of some years is essential.

**Site Investigation at Sellafield & Associated Studies**

**Background**
The initial years of Nirex in the 1980s were occupied with site identification for LLW and ILW—in the face of much public opposition to the sites considered. In 1989 Nirex concentrated on geological investigations at two sites in Dounreay in Caithness and Sellafield in west Cumbria. Work at Dounreay concentrated on the rock characteristics and hydrogeology. However, from 1991, Nirex concentrated its efforts on the Sellafield site because, although the geology and hydrogeology of the two sites were similar, most radioactive waste arises at Sellafield. This work led to the proposal for an under-
ground Rock Characterization Facility (RCF)—which did not go ahead because Nirex did not obtain planning permission to construct the facility. Nevertheless, a large amount of state-of-the-art site investigation involving 29 boreholes had been conducted and a variety of lessons were learned (summarized in Lessons Learned at the end of this article).

In this section, the site investigation at Sellafield is described and the approach to coupled models, and especially the hydro-mechanical coupling, is discussed. The rock mechanics predictions for the first RCF shaft and the development of protocols are presented on page 36. The lessons learned from the 1991–1997 Sellafield site investigation and modeling work, together with the prognosis for the next research phase, are discussed in Lessons Learned and the Way Ahead on page 38.

**Geological and hydrogeological setting at Sellafield**

The Sellafield area occurs in a transitional structural zone between the western margin of the Lake District Massif of basement rocks and the adjacent, mainly offshore area of the East Irish Sea sedimentary basin (Figure 2).

Major structural elements include east-northeasterly trending faults such as the Seascale/Gosforth Fault Zone and faults trending between north and northwest, such as the Lake District Boundary and Fleming Hall Fault Zones (Figure 3).

These faults influenced sedimentary basin development, so that the Fleming Hall Fault Zone locally forms the eastern edge of the Carboniferous Limestone, and the Lake District Boundary Fault Zone marks the eastern extent of the East Irish Sea Basin. Thus the upland area of the Lake District Massif forms a basement inlier of lower Palaeozoic igneous (mainly volcanic) and metamorphic rocks, with an unconformable cover sequence of carboniferous and Permian-Triassic sedimentary rocks that dip away from the inlier. The structural framework and geological evolution of the Sellafield area are described in Nirex Report 524 (1993) and Michele (1996).

The potential repository host rock at Sellafield comprises basement rocks of the Ordovician Borrowdale Volcanic Group (BVG). These volcanic rocks form a sequence dominated by welded ignimbrite sheets which have a proven thickness in excess of 1 kilometer. At the location of the shafts for the originally proposed RCF, the top surface of the BVG is at a depth of approximately 520 m below ground level, occurring beneath the immediately overlying Brockram, a 70 m thick sedimentary breccia of Permian age. The Brockram is in turn overlain and overlapped by early Triassic sandstones of the Sherwood Sandstone Group (SSG). This group consists of three formations, the lowest of which is the St. Bees
Sandstone Formation overlain in turn by the Calder and Ormskirk Formations (Barnes et al., 1994). In the vicinity of the proposed RCE shafts, the basal 80 m of the St. Bees Sandstone Formation is called the North Head Member, differentiated from the overlying sandstone by an increase in the frequency of thin beds of claystone and siltstone, interbedded with sandstone.

The Sellafield Site is at the interface between a groundwater system driven by basinal processes operating in the East Irish Sea Basin, and groundwater in sedimentary and basement rocks topographically driven by the upland area to the east. There are three key components to the hydrogeological system, differentiated by different processes and factors which influence groundwater flow. These three “hydrogeological regimes” (Figure 4) are:

- the “Irish Sea Basin regime,” where flow is driven by basin processes which vary over long periods of time. Most of the groundwater in this regime is basin derived brine;
- the “Coastal Plain regime,” where flow is topographically driven and boundaries are strongly determined by the position of the coast and the vertical extent of the Sherwood Sandstone Group. Groundwater in this regime is fresh and meteoric in origin; and
- the “Hills and Basement regime,” where flow is largely topographically driven, but at depth other processes such as density variations and the interface with the Irish Sea Basin regime are also important. The generally low groundwater flow within this regime results in groundwaters which are mixtures of fresher meteoric waters and other groundwaters of a saline nature which are slow to flush out of the system.

These three regimes are described in more detail by Black and Brightman (1996).

Fractures
The rocks in the Borrowdale Volcanic Group (BVG) have a relatively high compressive strength (most of the samples tested gave values between 75 and 250 MPa), so the geotechnical site investigation work concentrated on the fractures and the parameters comprising the Q rock mass classification index (NGI, 1994a). An example of the BVG core is shown in Figure 5a, with the characterization of the fractures in the BVG shown in Figure 5b for example information. A Q:RMR comparison for a 500 m length of BVG core is shown in Figure 6. At least 8 km of core were logged by the Norwegian Geotechnical Institute (NGI) for rock mechanics purposes.

In situ rock stress
The in-situ rock stress was measured by GeoScience and reported in Nirex (1997). The stress was measured/estimated from density measurements, overcoring, hydraulic fracturing, borehole breakout and seismic fault plane solutions. An illustration of the use of the travel time image of the borehole televiwer is shown in Figure 7.

A histogram of the orientation data is shown in Figure 8 and a plot of the stress components vs. depth is shown in Figure 9.

In a significantly fractured rock mass like the BVG, we might expect wide variations in the magnitudes and orientations of the in situ rock stress. This is expected because fractures alter the local stress distribution (Hyett et al., 1986; Sugawara, 1997). One of the surprising results in the Sellafield site investigation was the consistency of the stress data. There was little evidence of local stress field rotations or decoupling of the stress field at lithological boundaries. The conclusion must be that the rock stresses were sufficient to ensure that the fractures remained effec-
tively closed—so that the stress could be transmitted mainly through the fractures, thus avoiding major local stress variations.

This result is significant in terms of direct rock mechanics modeling and coupled hydro-mechanical modeling: the consistency of the stress data provides an additional level of confidence in the model assumptions.

The stress components at depth can be encapsulated in the formula: \( \sigma_1 = 0.025D + 0.27 \); \( \sigma_{\text{imin}} = 0.020D - 0.32 \); and \( \sigma_{\text{imax}} = 0.031D + 1.89 \) where the stress values are in MPa and the depth, \( D \), is in m. At a depth of 650 m for example, this gives the values \( \sigma_1 = 16.5 \text{ MPa}, \sigma_{\text{imin}} = 12.7 \text{ MPa}, \) and \( \sigma_{\text{imax}} = 22.0 \text{ MPa} \) which, in terms of depth below Ordnance Datum, converts to \( \sigma_1 = 18.6 \text{ MPa}, \sigma_{\text{imin}} = 14.3 \text{ MPa}, \) and \( \sigma_{\text{imax}} = 24.7 \text{ MPa} \).

**Initial Shaft and Cavern Analyses for the Sellafield Site**

**Distinct element modeling studies of the shaft and caverns**

Preliminary distinct element modeling studies of the stresses, displacements and joint apertures resulting from a 4.25 m diameter ventilation shaft excavation through the BVG to 600 m depth were conducted (NGI, 1992). These studies indicated that the engineering disturbance associated with excavation caused deformation and joint shearing from a half to one shaft diameter distance into the rock mass.

Utilizing updated site investigation data, NGI (1994b) undertook a series of sensitivity studies to investigate the effects of key cavern design variables for the BVG rocks using the 2-D UDEC-BB and the 3-D 3DEC distinct element codes.

The results indicated that, based on Excavation Disturbed Zone (EDZ) considerations, the optimal orientation for the cavern was with the cavern axis perpendicular to the maximum horizontal principal stress direction. This is interesting because an alternative view might be to minimize the stress concentrations by orienting the caverns parallel to the maximum horizontal principal stress. The results highlight the importance of a coupled hydro-mechanical approach to the studies.

**Heterogeneity of rock mass properties**

An important aspect of the site investigation work is consideration of the rock mass properties and their variation—in the context of input parameters for numerical modeling and division of the rock mass into structural units. Work conducted by the British

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![Figure 8. Azimuth of the maximum principal horizontal rock stresses at the Sellafield site (from GeoScience and Nirex, 1997)](image)

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Geological Survey (1996) was directed at characterizing the 3-D rock mass variability, estimating rock mass properties from seismic data and geostatistical techniques, and developing visualization models. The studies were aimed at both the rock mechanics and hydrogeological properties and covered properly the magnitude of fracture frequency prediction, development of rock quality and hydrogeological indices, fractal methods for upscaling and seismic attribute analysis, in addition to the geostatistical characterization and spatial variability visualization models. This work provides the basis for practical and full characterization of rock masses in the future.

The Nirex Digital Geoscience Database (NDGD)

The site investigation at Sellafield was extensive and complex. For the information to be useable in a coherent way, it was necessary to "...store in a compatible digital format on a unified database, accessible to the authorized users, all data relevant to the geological and hydrogeological modeling undertaken in support of the safety assessment." Nirex invested considerable time and effort in the development and maintenance of a computerized information system—the Nirex Digital Geoscience Database (NDGD). A Relational Database Management System (RDBMS) and a Geographical Information System (GIS) operating within a client/server framework provide the NDGD software platform (Nirex, 1996).

There are many advantages to such a system and a digital database feeding into the visualization and modeling software will be a key component of any future studies in the UK.

Survey of caverns worldwide

A detailed precedent experience survey of worldwide caverns was conducted by Geo-Engineering to establish the range of depths and dimensions of existing underground caverns. This survey provided information on existing caverns which can be used to demonstrate the technical viability of given depth-span parameters.

Studies of the excavation disturbed zone (EDZ)

Further studies of the EDZ were conducted by Sir Alexander Gibb and partners in association with Geo-Engineering and GeoScience. These studies were aimed at the relations between the EDZ, gross water movement and frequency of transmissive features as functions of depth, orientation and location. This work provides a useful foundation for studying the interaction between the variables and how general trends can be established.

Coupled Hydro-Mechanical Modeling

For a correct understanding of both the rock mass response to excavation and the rock mass permeability in the excavation region, it is essential to use a coupled hydro-mechanical model. However, the approaches to mechanical modeling and hydrogeological modeling are different. Moreover, there are difficulties in relating the mechanics and the hydrogeology: the water pressure affects the rock stress and the rock stress affects the fracture conductance—but in different ways. Accordingly, work was undertaken to study how a combined model might
be developed, the variables involved (Table 2), how the content of numerical codes could be presented, and how an EDZ systems model could be developed (Rock Engineering Consultants, 1997a).

A main conclusion of this work was that it is important to consider the mechanisms linking the hydrogeological and geotechnical areas. It became evident that study of either the hydrogeological system alone or the geotechnical system alone is insufficient. In other words, analyzing any sub-system alone will not produce correct answers for the overall site responses.

Additionally, a method for auditing the analysis capability of numerical codes was developed. It was clear that all the codes studied will give different results for a problem covering all aspects of the ten-variable system used to characterize the codes. This has important consequences for a validation exercise. We can predict already that no code will give the actual fully-coupled site response, although some codes may provide a sufficient engineering approximation. It is crucial, therefore, to develop prediction protocols that are not geared to evaluate a code by its specific prediction of the response of one particular variable—but to provide diagnostic assessment of each code’s ability to predict the whole fully-coupled response adequately. It is only after such a successful overall validation that one would feel confident that the essence of the fully-coupled system had been captured in the numerical code.

RCF Shaft Predictions: Continuum and Discontinuum Approaches

Motivation for continuum and discontinuum modeling of shaft excavation

As explained earlier, in common with other countries investigating potential sites for deep geological disposal, Nirex proposed to build an underground rock laboratory, or Rock Characterization Facility (RCF), to obtain a more detailed picture of the geology and hydrogeology of the Sellafield area. Assessments of the safety of a potential repository at the site published as Nirex Report S/95/012, (Nirex, 1995a), helped to identify some remaining issues that need to be addressed in the RCF before Nirex could take a decision on whether to propose a repository at the site (although in the event planning permission to construct the RCF was refused on 17 March 1997). These issues related to aspects of:

- groundwater flow and radionuclide transport;
- natural and induced changes to the geological barrier; and
- design and construction of the repository.

These issues were to be addressed by testing models related to the geology, hydrogeology and geol-

![Figure 10. Rock mass Young's moduli values used for the continuum predictions (from GeoScience, 1997)](image)

Table 2. Variables that can be used in hydro-mechanical coupled models

<table>
<thead>
<tr>
<th>Hydrogeological Parameters</th>
<th>Common Hydrogeological and Geotechnical Parameters</th>
<th>Geotechnical Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Discontinuity channeling</td>
<td>15. Discontinuity orientation</td>
<td>30. Rock deform/strength</td>
</tr>
<tr>
<td>3. Rock mass compressibility</td>
<td>17. Discontinuity intensity</td>
<td>32. Discontinuity shear stiffnesses</td>
</tr>
<tr>
<td>5. Fluid viscosity (dynamic)</td>
<td>19. Initial hydraulic head</td>
<td>34. In-situ stress</td>
</tr>
<tr>
<td>8. Hydraulic conductivity</td>
<td>22. Repository orientation</td>
<td></td>
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<tr>
<td>10. Fluid recharge boundary</td>
<td>24. Induced stress</td>
<td></td>
</tr>
<tr>
<td>11. Temperature</td>
<td>25. Excavated space state</td>
<td></td>
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<tr>
<td>12. Dispersion</td>
<td>26. EDZ geometry</td>
<td></td>
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<tr>
<td>13. Diffusion</td>
<td>27. Fluid pressure distribution</td>
<td></td>
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<td></td>
<td>29. Subsequent access sealing</td>
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</tbody>
</table>

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technical behavior of the site with the geological models underpinning the hydrogeological and geotechnical models. The geotechnical models, which were intended to include a range of elastic and discontinuum models, would simulate the rock mass response to excavation, and provide an assessment of the response of the rock mass induced by the excavation of shafts and underground roadways.

The main elements of the planned RCF were a series of horizontal galleries at a depth between -650 and -900 m aOD, accessed by two vertical shafts located at the centre of the facility (the North and South Shafts). The location of the South Shaft would have been coincident with that of a Nirex deep borehole (Borehole RCF3), which is located about 100 m south of Longlands Farm, near Gosforth in West Cumbria. The shafts would have been sunk through approximately 520 m of sedimentary rocks into underlying basement rocks of the Borrowdale Volcanic Group. The Borrowdale Volcanic Group of rocks is the potential host rock for the deep waste repository and also for the main galleries of the proposed RCF. Thus, entry to the RCF would have been by shafts penetrating through different rock formations (Sherwood Sandstone Group, a Permian breccia called the Brockram, and the Borrowdale Volcanic Group of rocks).

Two types of models were to have been tested: gross models which represented the overall conditions within the RCF; and sector-specific models which represented the conditions in a specific sector. The predictions relating to the gross continuum and discontinuum models for the shaft are summarized here.

Predictions of the gross rock mass response were based on two approaches:
- assuming the rock mass behaves as an elastic continuum; and
- assuming the rock mass behaves as a discontinuum.

These gross geotechnical models were designed to simulate the performance of the rock mass during shaft excavation over the full depth of the shafts. In particular, the gross models were to identify trends in response that could be measured during shaft sinking.

The objectives of the geotechnical gross models were:
- to determine whether the rock mass responded to shaft sinking in an essentially elastic manner, as a discontinuum, or as a combination of both;
- to identify where more detailed modeling is required for the sector specific models.

Thus, the modeling program had been developed to determine if the rock behaves during shaft sinking primarily as an elastic continuum or as a non-elastic discontinuum, and whether this behavior varies throughout the shafts. The models were described as gross because it was the trends of response that were of interest. The trends would be interpreted by developing models over the following units: St. Bees Sandstone, North Llead Member, Brockram (elastic only), Faulted Longlands Farm Member, Altered Longlands Farm Member, and Bulk Longlands Farm Member.

**Continuum modeling**

The gross elastic predictions for the shaft were developed using the analytical elastic solution for the stresses and displacements around a circular hole and by numerical modeling of the 3-D elastic effects of the incremental shaft excavation (GeoScience, 1997). The rock stress values were available using the formulae noted on page 33. The elastic modulus of the rock was estimated from wireline log-derived dynamic values in the Nirex Digital Geoscience Database and scaled for compatibility with static values obtained from borehole core, as illustrated in Figure 10.

The predicted radial displacements are shown in Figure 11. This plot is interesting because the displacements do not linearly increase with depth: they are effectively constant with depth, apart from local irregularities. The stress components increase linearly with depth and so does the modulus—with the two effects cancelling to produce an effectively constant displacement of between 2–3 mm in the upper section of the shaft and 1–2 mm in the lower section.

The conclusions of the continuum modeling work were that the predicted displacements would form a two-cycle sinusoid around the shaft with the
maximum and minimum displacements coinciding with the directions of the maximum and minimum horizontal principal stresses, with a total range of displacements of 1–5 mm. Interesting studies and conclusions were made concerning the optimal arrangement of displacement measuring pins to identify the form of the actual displacements. This is important for distinguishing the continuum or discontinuum nature of the rock response and for establishing “back analysis discriminators,” which will be discussed later.

Discontinuum modeling

The gross discontinuum predictions for the shaft were developed considering 3-D fracture frequency, block fall-out in the shaft sidewalls, and numerical modeling of stresses and displacements, which included the effects of fractures (Rock Engineering Consultants, 1997b). A feature of the work was that the predictions for each subject were made by two independent methods for fracture frequency and potential rock block instability. For the stresses and displacements, a sensitivity study approach was adopted. An example of the fracture occurrence was shown in Figure 5. For the first two types of analysis for the gross discontinuum predictions, i.e., the fracture frequency predictions and the potential rock block instability predictions, the analyses were conducted with:

- the fractures represented by sets; and
- the fractures as individually present.

This not only provided a useful comparison providing confidence in the predictions, it also provided confidence for using the fracture data sets for the stress and displacements modeling.

Predictions were made of the fracture frequency and RQD in all directions, plus how the fractures would appear on the shaft walls—which can be considered as portions of vertical planes in all directions. An example plot of the 3-D fracture frequency for the Bulk Longlands Farm Unit (for the fractures in sets) is shown in Figure 12a. The plot in Figure 12b shows the fracture frequency in all horizontal directions (i.e., the frequency around the perimeter of the net in Figure 12a, indicating that the approximation of isotropy in the continuum predictions is acceptable in the engineering framework.

From this work, it was evident that the fracture frequency is not a single fixed value for a formation: its value will depend on the direction of the line along which it is being measured. The RQD is not a fixed value either: its value depends on the direction of the line being considered through the rock mass. Predictions were made concerning on which sides of the shaft the most numbers of fractures would be visible in each of the geotechnical units.

For the gross potential block instability predictions, methods were required to assess the kinematic feasibility of the rock blocks to be potentially released by the shaft excavation. These were the use of a. instability overlays on stereographic projection, b. the program UNWEDGE, and c. the use of a computer program to identify blocks and establish whether they are mechanically unstable using the original individual fracture data. The results are shown in Figures 13 and 14.

Note that for the UNWEDGE analysis in Figure 13 there are only certain shaft wall orientations at which potentially unstable blocks can occur, because the fractures are considered in four specific sets of parallel fractures. However, for the general analysis in Figure 14, a potentially unstable block can occur at any orientation depending on the fractures sampled. It was found that, for the fracture data used, the UNWEDGE program provided a good indication of shaftwall instability despite the fact that all the fractures had been assumed to belong to only four sets.

The stresses and displacements were estimated using the UDEC distinct element code, which explicitly includes the fractures. A series of sensitivity studies was conducted in order to assess the effect of different parameters, i.e., the modulus and fracture parameters, especially fracture persistence. The 2-D
analyses provided a good indication of the trends due to the sub-horizontal and sub-vertical nature of the discontinuities. Wide variations in the local stresses in the vicinity of the shaft were found due to the block-jointed nature of the modeled medium. If the rock has few fractures, the stresses will have a greater overall consistency. If the rock has many fractures, the individual values of stress (stress is defined as a property at a point) at specific points cannot be predicted—because of the local idiosyncrasies of the fracture geometry. The stress values are thus best obtained as average values from the companion elastic continuum solutions.

Wide variations in the displacements were also found but these were more consistent than the stress values, especially in terms of the shaft wall displacements which represent an integrated effect of the block-jointed rock mass behavior. Displacements of up to 30 mm could be experienced, depending on the local fracture conditions. The discontinuum displacements were not symmetrical about the directions of maximum and minimum principal stress—providing a basis for back analysis discrimination of whether a continuum or discontinuum solution is more appropriate once the measured values had been obtained.

**Back analysis discriminators**

It is important to anticipate the need to compare the predictions with the data measured during and after shaft excavation. How can we decide whether the predictions have been accurate and which model has produced the best predictions? This leads to the need for comparison protocols and back analysis discriminators. The most important continuum-discontinuum discriminator is the symmetry-asymmetry of the shaft wall displacements about the 160°–340° horizontal major principal stress axis at a given level. If the displacements are reasonably symmetrical about this axis at a given shaft level, it means that:

- The displacements are not highly variable and hence not dominated by block geometry, and
- The displacements are dominated by the continuum nature of the medium.

Conversely, if the displacements do not form a simple pattern and are not symmetrical about the 160°–340° horizontal major principal stress axis, the continuum model is not appropriate and a discontinuum model has to be used.

The subject of comparison protocols and back analysis discriminators is one that needs to be developed further because the basic principles of the subject have not been developed fully.

**Lessons Learned and the Way Ahead for the UK**

**Lessons learned**

The lessons learned from the UK rock mechanics studies supporting radioactive waste disposal during the DoE work in 1979–86 and the Nirex work in 1991–97 are summarized as follows:

A multi-phased approach to the rock mechanics should be adopted—as a fundamental approach method and in harmony with the multi-phased approach to site selection, design, excavation and closure, with the attendant phased performance and safety assessments. The disposal of radioactive waste is a complex subject and the repository design cannot be conducted in one phase. A phased approach is required to establish how to solve the problem and to develop the solution by a
process of increasing knowledge. In each phase, confidence should be established in the approaches used through validation of the modeling—where validation means ensuring that the modeling does indeed represent the rock reality.

**There is a need for a coupled approach.** Disposal of radioactive waste involves consideration of many subjects. Disposal should be considered as a complete system, and not as a series of subjects in isolation. It is difficult to adopt a coupled approach because researchers tend to be trained in specific areas and numerical codes tend to be focused in specific areas, as illustrated in the previous section discussing coupled hydro-mechanical modeling. Also, the links between the subjects have not all been established. Work is advancing in this area with initiatives such as the thermo-hydro-mechanical couplings being studied by the international DECOVALEX program (Stephansson et al., 1995), reviews such as the thermo-hydro-mechano-chemical one conducted for the US program, and the use, by several countries, of underground research laboratories in which many of the couplings are automatically included when the in situ rock mass response is studied.

The distinction between the rock mechanics aspects of repository design and construction for the direct civil engineering and the rock mechanics aspects of the repository and host rock for the performance and safety assessments needs to be clarified. This has never been clear—but is essential "top of the flowchart" information required to plan and execute the work. In a hard rock such as that at Sellafield, there is no rock mechanics problem in excavating and supporting caverns at around 650 m depth. There is, however, a modeling and design problem if the work has to include considerations of minimizing the Excavation Disturbed Zone (EDZ) when the EDZ is not defined, understanding the interactions between the rock mechanics and hydrogeology, designing the repository to minimize radionuclide migration given the multi-barrier concept, and predicting precisely what will happen in 100,000 AD and beyond.

**Time dependent effects should be studied.**

The intended design life of the repository is much longer than for other civil and mining projects, being measured in thousands of years, yet most of the rock mechanics work to date has not incorporated the effects of time. The site investigation tests have not included any explicit time dependent parameters, nor do the numerical codes model any time-dependent mechanisms. The rock mechanics analyses are based to a large extent on the theory of elasticity which has no time component. In particular, the hydro-mechanical couplings are likely to be significantly time dependent.

**The work should be technically audited on a continuous basis with the information available to everyone,** so that the work is optimally managed, is transparent, and the reasons for decisions are traceable with a full audit trail developed. This is not yet normal for other rock engineering civil and mining projects but is critical for the development of a radioactive waste repository—because of the complexity of the system and the need to generate confidence in both geoscientists and the public. Currently, much has to be taken on trust which, in an adversarial framework, does not lead to a robust approach.

**Modeling and site work should be supported by a digital database.** There should be a coherent supporting database of all the data, both for scientific reasons and to justify decisions made. It is an essential feature of technical auditing that suitable evidence is required. The concepts behind the Nirex Digital Geoscience Database (NDGD) were excellent and should be pursued in the next phase of studies.

**There should be greater emphasis on protocols,** i.e., the agreed procedures for site investigation, modeling, making predictions, site measurements in boreholes or an underground laboratory, deciding if the predictions are adequate, back analysis procedures, etc. Without protocol restraints, the work can arbitrarily travel in unnecessary directions and, more importantly, not be sufficiently coordinated to achieve the objective in hand. The protocols are not intended to inhibit innovation but to ensure that the work is completed satisfactorily.

**There should be a greater degree of international cooperation.** Although in the early days of rock mechanics it was possible to conduct useful research without reference to any overseas work, this is now not possible in rock mechanics, and certainly not in radioactive waste disposal. The conferences and journals are international. The Internet exists for rapid international communication. There are international nuclear overseeing agencies. Many countries have radioactive waste disposal programs. Some have underground research laboratories. There are international programs such as the DECOVALEX initiative to assess coupled models. There are international surveys such as Witherspoon's worldwide review (1996). International cooperation should not be something which will, hopefully, be included in the program: it should be a dominant conduit through which a significant proportion, if not the majority, of the information is obtained.

**There should be much more emphasis on communicating the rock mechanics information to other geoscientists and to the public.** Although many of the techniques used are well accepted and the numerical codes invoked are state of the art, with the advantage of hindsight, this may not be clear to other scientists and engineers, nor to the public. There should be a method of communicating to everyone the rock mechanics mechanisms involved, the associated variables and parameters, how these are modeled using computers, and how the models have been tested, etc. The scientific work
Figure 14. Sample polar plots of the factor of safety of tetrahedral blocks forming on all sides of the shaft and sliding on two fracture planes—for 20,000 samplings of three fractures from the fracture data set (from Rock Engineering Consultants, 1997b)

should be paralleled by a major communication effort.

**Workshops need to be held to resolve outstanding issues.** Many rock mechanics issues are not resolved: examples are the method of establishing the connectivity of rock fractures, the method of dealing with effective stresses in fractured rock masses; an adequate method of characterizing fractures for a combined hydro-mechanical model; how to incorporate time dependent effects into models. In an adversarial framework, these and other unresolved issues are major vulnerabilities in the design case presented. However, in engineering there will always be unknowns; the engineer has to find methods of dealing with uncertainty. Thus, workshops are required so that consensus statements can be established on the unresolved issues pertinent to repository design. Technical auditing is made according to evidence, known criteria, and the current scientific framework. If the issues are unresolved, the technical auditors, and certainly the critics, may well conclude that key information is missing. If consensus statements are available, at least an agreed procedure has been established for coping with the uncertainties, which the technical auditing can use as evidence. An important point, highlighted by Michie (1998), is that a pragmatic approach is required: the site has to be adequate, but it does not have to be the perfect site. Similarly, the supporting analysis only has to be adequate; it does not have to be perfect.

**The way ahead for the UK**
The first period of rock mechanics research, funded by the Department of the Environment in 1979–86, was intended to increase the UK’s strategic geological knowledge in order to support future work and to evaluate proposals put forward. The second period, funded by Nirex in 1991–97, was intended to investigate and characterize the Sellafield site using boreholes, with the further intention of developing a Rock Characterization Facility (RCF).

The end of the second period of research was caused by the Cumbria County Council’s 1994 refusal of Nirex’s planning permission application for the RCF, and the subsequent Secretary of State’s 17 March 1997 rejection of Nirex’s appeal. Despite the rejection, the Secretary of State did reaffirm the previous Government policy of constructing a deep repository for radioactive waste as soon as reasonably practicable once a suitable site is found. However, a period of uncertainty and Nirex downsizing followed during 1997–99 while the report of the House of Lords Select Committee on Science and Technology with recommendations for future action was being prepared. Interim measures of interest have occurred. The Geological Society of London in association with the British Geological Survey held a two-day meeting to establish a series of consensus statements that could be made about radioactive waste disposal from the geological perspective. The Royal Academy of Engineering held a meeting to update members on the issues involved. Preparations are being made for a National Consensus Conference on radioactive waste disposal. This is a mechanism for formally involving the public and obtaining views. It is a forum at which a citizens’ panel, of 16-20 people selected from the general public, questions experts and witnesses on the topic, assesses the responses, discusses the issues raised, and reports its conclusions at a press conference. The idea is based on a model of technology assessment which originated in the USA during the 1960s. Fifteen such consensus conferences have been held in Denmark but there has only been one previous conference in the UK—in 1994 on plant biotechnology.

The House of Lords report “Management of Nuclear Waste” was made available on 23 March
The Committee responsible for the report consisted of 11 members supported by a specialist advisor. The report outlines the background to the subject and explores the issues involved, having taken evidence from many witnesses. It was noted that, with the 1997 rejection of the Nirex RCF planning application, the UK was left with no practical plan for radioactive waste disposal. However, phased disposal in a deep repository is feasible and desirable, and public acceptance is required. The Committee recommends that a new body, a “Nuclear Waste Management Commission” should be established to develop a comprehensive strategy with public consultation, and that a “Radioactive Waste Disposal Company” should design, construct, operate and close the repository. The last recommendation in the report is that “We recommend that the Government acts without delay.”

Thus, the UK is probably about to embark on another phase of radioactive waste disposal work which will involve a third phase of rock mechanics studies, although it will take some years before the organizations are formed and the statutory powers are established.

Conclusions

The rock mechanics work supporting the disposal of radioactive waste in the UK has been reviewed. The first phase of work funded by the DoE was strategic and resulted in a considerable increase in knowledge, plus recommendations for a coupled approach, consideration of long-term effects, and adoption of a multi-phased approach to site investigation. The second phase of work funded by Nirex was mainly tactical and directed at improving knowledge of the Sellafield site and preparing for a rock characterization facility. The site investigation involved, inter alia, 29 deep boreholes, development of a structural atlas and a digital database. Again, there was a considerable jump in knowledge and the development of techniques for rock mass characterization and analysis.

The lessons learned in both phases were encapsulated in the ten recommendations for a future program listed in Lessons Learned. These relate to a multi-phased approach to the work, the need for a coupled approach, the purpose of the rock mechanics work, time dependency, technical auditing, databasing, protocols, international co-operation, public perception, and workshops.

The future of the UK program has been uncertain since March 1997 when the Nirex appeal against the adverse underground laboratory planning decision was rejected. However, the March 1999 House of Lords report on the management of nuclear waste recommends that the Government should act without delay in the formation of a new commission to develop and oversee strategy and a new company to design and build a repository and dispose of the waste. If these recommendations are taken up, the UK will enter the next phase of work—although it may take some years for the organizations to be established.

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Dr. John Cosgrove provided advice on geological interpretation and Dr. John Harrison produced the plots in Figures 12 and 14. Dr. Yong Jiao carried out much of the Rock Engineering Consultants work and Dr. Lanru Jing assisted with some of the work.

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A Potential High-Level Nuclear Waste Repository at Yucca Mountain, Nevada, USA

By William J. Boyle, United States Department of Energy and Robin N. Datta, URS Greiner Woodward Clyde

The following paper is based upon a general report given at the 4th North American Rock Mechanics Symposium, held in Cancun, Mexico in June 1998. It has been updated with some diagrams from the recently published Viability Assessment that is available for viewing, downloading, and ordering at www.ym.gov.

Introduction

The papers in the session focused on two on-going in-situ thermal tests in the Exploratory Studies Facility (ESF) at Yucca Mountain, Nevada, USA. These are the Single Heater Test (SHT) and the Drift Scale Test (DST). The papers represented a snapshot in time, but the general report and the presentations of the papers discussed more recent results, obtained since the papers were submitted. In order to provide a framework for understanding why the thermal tests are being conducted and how the results will be used, an overview of the Yucca Mountain Project is provided in the general report, with particular emphasis on the role of rock mechanics and thermal-mechanical processes in the Yucca Mountain Project. Specifically, the general report describes how thermal testing and numerical modeling of heat-driven coupled processes in the rock are being used in assessing the ability of Yucca Mountain to successfully function as a repository for spent nuclear fuel and high level nuclear waste.

General

Yucca Mountain, located approximately 135 kilometers northwest of Las Vegas, Nevada (Figure 1), is being characterized by the United States Department of Energy (DOE) to determine the suitability of the site to host a geologic repository for the permanent disposal of spent nuclear fuel and high level nuclear waste. Part of a series of north-trending ridges of the Basin and Range physiographic province, Yucca Mountain is as much as 1500 meters above sea level at its crest. The ridges are relatively less faulted structural blocks, bounded by typical basin and range normal faults. The subsurface facilities for the proposed repository will be located in a relatively undisturbed block of Topopah Spring Tuff (Figure 2).

Lying in the southern part of the Great Basin physiographic province of North America, Yucca Mountain is underlain by 1000 to 1500 meters of tertiary volcanic tuff, formed from the ash of eruptions occurring between 8 and 16 million years ago. The volcanic tuffs are generally bedded, separated into beds that are non-welded to densely welded. In addition, some are devitrified and others are vitric. The proposed repository horizon is within a sequence of beds, up to 350 meters thick, of moderately to densely welded devitrified tuff known as the Topopah Spring Tuff of the Paintbrush Group. The subunits of the Topopah Spring Tuff are based primarily on the abundance of lithophysae, cavities with dimensions on the order of millimeters to thousands of millimeters, formed by gases in the cooling ash flow. The presence or absence of the lithophysae can have a pronounced effect on the mechanical and hydrologic properties and response. The Topopah Spring Tuff is divided into four subunits: the upper and lower lithophysae, and the middle and lower non-lithophysae. The bulk of the proposed repository would be located in the lower lithophysae and middle non-lithophysae subunits. The in situ thermal tests described in the session are all in the middle non-lithophysae unit.

The Yucca Mountain area is semi-arid to arid with low rainfall (averaging about 17 cm annually) and high evaporation. The water table is deep, approximately 800 meters below the land surface. The potential repository would be approximately 200 to 350 meters deep in the unsaturated zone, more than 300 meters above the water table.

The aridity and consequent small amount of water infiltration, are central to the reasons Yucca Mountain is being considered for a nuclear waste repository. The small amounts of water mean: a) little to no water may reach the waste containers; b) corrosion will be slow, and consequently long waste package lifetimes will allow the waste to be substantially contained for thousands of years; c) there will be a slow rate of release of radionuclides from the waste form and package; and d) there will be a reduction in the concentration of nuclides as they are transported through and interact with the engineered and natural barriers.

Repository Performance Assessment and Roles of Process Models

The DOE and NRC decisions about suitability and licensing will be based on evaluations of the expected performance of the repository and the site. It is useful to explain certain terms to facilitate the discussion of repository performance. In the temporal domain, repository performance is considered in
two periods, the pre-closure period and the post-closure period. The pre-closure period is when the repository subsurface facilities are operational or accessible. It ends with the sealing and closure of the repository. The design for the pre-closure period will permit waste retrieval for at least 50 years after initial emplacement, but will also allow access over several hundred years if desired. During the pre-closure period, worker safety (including radiation safety) and stability of underground openings are the prime goals of repository performance. The post-closure period extends many thousands of years from the end of the pre-closure period and is closely connected with the performance measures or criteria, by which repository performance will be assessed. The length of the post-closure period is dependent on regulations, which are in the process of being revised, but may be 10,000 years.

In the spatial domain, repository performance can be considered in simplified terms of the near-field, and the far-field (Figure 3). The near-field extends a relatively short distance into the rock around the repository subsurface facility. The thermal, mechanical, hydrological and geochemical processes in the near-field rock are intensified relative to those in the far-field, mainly due to the decay heat from the emplaced spent fuel. These processes substantially affect the response of both the engineered barrier system and the natural barrier system.

The engineered barrier system (EBS) is situated within the natural barrier system (NBS) as shown in Figure 3. The EBS consists of the container and other retarding material holding the waste as well as any and all other man-made material placed around and over the containers. The boundary of the EBS is at the wall of the underground opening of the repository. The NBS, consisting of the unsaturated and saturated rocks underlying the controlled area, extends from the EBS to the accessible environment. The EBS together with the NBS makes the whole repository system. The function or process of assessing the performance of the whole repository system is referred to as total system performance assessment (TSPA).

Figure 1. Location of Yucca Mountain Project
Figure 2. Plan view of proposed repository outline
Figure 3. Simplified cross section of potential repository system

There are numerous components or aspects of the total repository system, and probably as many or more physical phenomena which are considered to be occurring during the pre- and post-closure periods. All these processes acting in/on one or more components of the system are to be understood and modeled in the TSPA, and the outcomes of the modeling are to be progressively and appropriately synthesized, as illustrated in the hierarchical pyramid in Figure 4. This involves a series of abstraction steps leading to the total system model, which is exercised to evaluate the performance of the total repository and site.

Figure 4. Performance assessment abstraction and information flow pyramid

Understanding and quantitatively describing the processes specific to the Yucca Mountain site, comes from using the scientific and engineering data, which are characteristics or properties of the various components of the system, with appropriate conceptual models. All four papers in the session were related to developing the two lowest layers of the pyramid. The papers describe tests, measurements, and analyses that constitute parts of the bottom layer, feeding into the conceptual and process models.

The TSPA modeling representation of the total system and its component subsystem models of the top of the pyramid in Figure 4 are shown in more detail in Figure 5. In order to perform the TSPA, quantified understandings are required of the models down through the process and conceptual models, and must be based on the data as shown in Figure 4. Quantified understanding of the next level of phenomena or processes lead to the component models illustrated in Figure 5, which have an impact on the performance of the EBS and/or the NBS, the two subsystems. For example, quantified understanding of the elemental process of heat conduction in the lower lithophysal unit of the Topopah Spring Tuff is the thermal conductivity of most of the emplacement horizon rock. Knowledge of this thermal conductivity is needed for exercising the thermal hydrology models. Or, quantified understanding of the process of fluid flow through the unsaturated zone (UZ) rocks in YM under ambient and elevated temperature conditions leads to the understanding of the bulk permeabilities of those rocks. Knowledge of the bulk per-
meabilities of those rocks is needed for exercising the UZ flow model.

A more explicit linking of the models and computer codes, and some of their parameters, shown in Figure 6, constitute the core of TSPA, by which the total system and subsystem models are linked to the Yucca Mountain site and the potential repository system.

As shown in Figures 5 and 6, some of the processes interact with other processes. Such interaction can be referred to by the term “coupled processes.” All the papers of the session describe efforts to gain an understanding of and quantify such processes and their coupling.

Another term germane to any discussion of repository performance is performance confirmation monitoring. Given the uncertainties inherent in the site, its performance, and TSPA, it will be prudent to monitor the repository to determine whether the repository facility is performing as designed during the construction and development period as well as the operational period. Performance confirmation is the reality check for performance assessment. The tests described in the session are the beginnings of this monitoring, provided the repository is built.

Rock Mechanics and Thermal-Mechanical Processes in Pre-Closure Performance
During the pre-closure period, stability and integrity of the underground openings are a key concern. The main reasons are worker safety and the regulatory requirement to maintain the ability to retrieve the waste from the emplacement drifts during this period. Rock mechanics and thermal-mechanical analyses are used, both in characterizing the site and in designing the facility. Such analyses will continue during construction and operation of the facility.

During the pre-closure period, the performance confirmation monitoring, analysis of the resulting data, refining process models and making improved predictions of the performance of the repository will be ongoing activities.

Some of the computerized models employed to perform rock mechanics and thermal-mechanical analyses for the Yucca Mountain repository are FLAC, UDEC, DDA, ANSYS, JAC and a compliant joint model developed by Sandia National Laboratories. The response of Yucca Mountain to the excavation of the ESF tunnel is being systematically monitored by measuring the movement of the rock around the openings and the convergence (or otherwise) of the drift walls. These monitoring data provide a valuable opportunity to understand the baseline, pre-heating mechanical processes occurring in the rock under ambient temperature conditions and serve as the bases for the models used to predict the monitored data. Although not described in any of the papers at the session, the deformations measured to date are all within expectations, considering the material properties, existing fractures, and rock mass quality.
Figure 6. TSPA code configuration

Thermal Testing to Understand Nearfield Coupled Processes

Adequately understanding the heat-driven near-field coupled processes is essential to the development of a number of the models shown in Figures 3 and 6, and therefore essential to understanding the performance of the proposed repository. The Yucca Mountain Project has a strategy for thermal testing to gain this understanding. The strategy is to conduct a series of tests that generally progress from smaller to larger, shorter duration to longer, and simpler to more complex. All four of the papers in the session are part of the strategy and described efforts to gain better understanding of one or more of these processes. The paper by Brodsky and Barker describes laboratory tests conducted to measure specific coupled properties under controlled conditions at laboratory scales of volume and time. The papers by Sobolik et al. and Ballard deal with a larger, longer, less constrained in-situ test—the Single Heater Test (SHT). Although the test conditions were not as controlled as those in the laboratory tests, analysis of the results provides conclusions about specific coupled processes. The final paper describes the Drift Scale Test (DST), the largest, longest, most complex in-situ test to be conducted at Yucca Mountain. As is the case with the SHT, the DST is not as controlled as a lab test, but will be used with all other tests and data to gain an understanding of coupled processes.

Results To Date

All the session papers detail their results. At a high level, the results can be summarized as follows: For the SHT geometry and size, heat transfer by conduction alone explains the temperature distribution fairly well, especially for subboiling temperatures. The effect of hydrology is small and not significant for predicting the temperature distribution. Figure 7 compares the temperatures measured by one sensor to the temperatures predicted for that location, taking into account two different hydrologic conditions. As can be seen, during the heating phase, the low permeability model predicts the measured tem-

Figure 7. SHT results, comparison of measured and two predicted cases
peratures better at below boiling levels, while the high permeability model makes a better prediction of the measured temperatures in the above boiling regime. The differences in temperatures predicted by the high and low permeability models are small, indicating as mentioned before that the effect of hydrology on temperature distribution in the SHT is small. An effective continuum model (ECM) was used to simulate the thermo-hydrologic process in the Single Heater Test employing both the TOUGH2 and NUFT codes. The predicted temperatures made by the thermo-hydrologic model were input for analyzing the mechanical response of the SHT. The predicted displacements were compared with the measured ones. Although interpretation of the mechanical response of the SHT was clouded by numerous fractures in the test block, the analysis did yield measured values of rock mass thermal expansion which were less than the laboratory measured values by a factor of two to three. Additionally, in the SHT, the pore water of the rock mobilized by the heat condensed in one of the instrument holes. Isotopic and chemical analyses of samples of this water led to new insights into the chemical processes that may take place in the repository near-field due to heat. Details of the test results and analysis are in the papers by Sobolik et al. and Ballard et al.

A description of the Drift Scale Test is given in the paper by Finley et al. Early results from the Drift Scale Test confirm the expectation that because of the larger scale and its geometry, hydrology will have more effect on the temperature distribution. Figure 8 shows temperatures measured by a number of sensors for the first 420 days of heating. Temperature steadily rises to approximately 95°C and stays at that level for many days as the pore water in the rock is mobilized by the heat and evaporates. The applied heat is largely used in the boiling process, resulting in leveling of the temperature plot. Once all the mobilized water is evaporated, temperatures rise again.

The measured convergence at the approximate middle of the concrete-lined section (farthest 12.5 meter) of the drift and the temperature are plotted in Figure 9 against elapsed days since start of heating. The plot shows that the horizontal convergence is, so far, substantially larger than the vertical one. This is primarily due to the additional thermal pulse from the wing heaters on either side of the drift at approximately midheight.

**Summary**

The Yucca Mountain Project has a structured approach for assessing the performance of the proposed repository, TSRA. An integral part of the TSRA is gaining an understanding of heat-driven coupled thermal, mechanical, hydrologic, and chemical processes. The Project developed a strategy for thermal testing and is carrying out that strategy. The results of some tests that are part of the strategy are described in papers presented at the Fourth North American Rock Mechanics Symposium.
A New Test Procedure for the Determination of the Block Punch Index and its Possible Uses in Rock Engineering

By Resat Uluşay and Candan Gökceoğlu

Abstract

Measurement of rock strength requires testing which must be undertaken on test specimens of particular sizes in order to fulfill testing standards. Often, the coring process breaks up the weaker core pieces, and they are too small to be used in either uniaxial compressive or point load tests. The Block Punch Index (BPI) test which requires only small, flat disc specimens has been developed in the last decade. In this study, a device was constructed and a method for size correction in BPI test was developed. Analysis of the results indicated that the BPI is sensitive to specimen size. It is also noted that corrected BPI values leads to insignificant errors in assessing uniaxial compressive strength when compared to those obtained from point load testing. Possible uses of the BPI in rock engineering were also suggested.

Introduction

Most rock engineering investigations require the determination of the physical properties of the rock strata. Typically, samples of the rock are cored and the larger pieces are evaluated using the uniaxial compressive strength (UCS) or point load test. However, there are some shortcomings associated with these conventional tests. Where rock cores are only divided into small discs, due to the presence of thin bedding or schistosity planes, the core length may be too short to allow preparation of the specimen long enough even for the point load index test. To overcome this limitation, the possibility of using relatively short samples for a rock strength test has always been attractive.

Mazanti and Sowers (1965) and Stacey (1980) suggested using a simple apparatus to measure direct shear strength when the test specimen is a thin plate of rock. The work of these authors has been extended at Delft University of Technology by Tasseelaar (1982; Schirer, 1988) and Schirer (1988), and has led to the development of the Block Punch Index (BPI) test. They obtained high correlations between UCS, Brazilian tensile strength and BPI values. In these studies, core specimens about 10 mm in thickness and 40 mm in diameter were tested and the size effect of the specimen was not taken into consideration. To the author's knowledge, published material suggesting a size correction factor to be used in the BPI test is not currently available, except studies performed by Uluşay and Gökceoğlu (1997, 1998). On the other hand, how a BPI value obtained from a specimen with a weakness plane which makes an angle with punching direction can be converted to the highest standard BPI value is another question which should be answered for the improvement of the BPI test. The main purposes of this present study are to develop a method for obtaining a size correction factor which could be used to compensate for the influence of size in the BPI test and to consider its possible uses in rock engineering.

Materials and Apparatus

Block samples of relatively fresh igneous, volcanic, sedimentary and metamorphic rocks were collected from 25 different locations in Turkey to perform BPI and UCS tests on air dried specimens. The specimen thickness ranged from 5 to 15 mm for the BPI test. While the range in core diameters, for BPI and UCS tests covered the core sizes between BX (42 mm) and NX (54 mm) commonly used sizes in geotechnical investigations. Attention was paid to obtaining the specimens required for both tests from the same core. A total of 1,858 rock discs (of these 154 are oriented for the evaluation of anisotropy and prepared from marl), and 350 cylindrical specimens with height to diameter ratios of 2.5 to 3.0 were prepared for BPI and UCS tests, respectively. The UCS test was performed in accordance with the suggestions outlined by ISRM (1981) using a servo-controlled stiff testing machine. There are no published standards for construction of the apparatus for a BPI test, and since this test apparatus is not commercially avail-

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able, it was designed and fabricated in-house. The end result of the design and fabrication process was a unit consisting of three parts of hardened steel (Rockwell hardness 40). The disc specimen is placed on two parts from the base support and then clamped from two ends by means of steel bars, which are screwed down (Figure 1). The third part, a rectangular rigid punch block (Figure 1), passes between two blocks with a clearance of approximately 0.25 mm and applies the load onto the specimen. The apparatus is designed to fit into a point-load testing frame (Figure 2).

**Test Procedure**

To begin a specific test, both the diameter and the thickness of the specimen, on which the faces are as parallel and free of irregularities as possible, are measured at several different locations using a caliper. The readings of each of the measured values are later averaged for use in subsequent calculations. The specimen is then centered on the lower platen and clamped, then the punching block is threaded down until it is nearly touching the specimen. The load is then gradually applied by the point-load device at a constant rate that will cause a failure within 10–60 s. Fracturing is thus forced to take place along two parallel planes on which the normal stress is considered to be zero while the tensile stresses caused by bending are reduced to a minimum. After a failure, the specimen is broken into three parts as illustrated in Figure 1. The test should be rejected as invalid if the parallel fracture planes are either absent or not fully developed (Figure 3).

The BPI (in MPa) of the rock disc is determined by using the following equation, which divides the failure load (F; kN) by the area (A; m²) through which the shearing takes place.

\[ BPI = 10^{-3} \frac{F}{A} \left( \frac{A=4t}{r^2 - 95.1} \right)^{0.5} \times 10^6 \]  \hspace{1cm} (1)

where \( t \) and \( r \) are the thickness and the radius of a disc specimen (in millimeters), respectively.

**Size Correction in BPI Test**

To evaluate the size effect, the diameters and thicknesses of each rock type were plotted versus the accompanying BPI value and curves were fitted to the plotted points. A typical example illustrated in Figure 4a for a mudstone indicates the combined influence of the specimen diameter (D) and thickness (t) in the BPI test. This variability of the BPI (t,D) clearly demonstrates the importance of the size effect in this test.

The BPI test results obtained from 1,744 non-oriented specimens (i.e., tests normal to the plane of weakness in stratified rocks and in other rock types) indicated that the number of invalid tests was a negligible amount (1–10 percent) for the specimens with thicknesses ranging between 5 and 10 mm. It shows a distinct tendency to increase if the thickness exceeds 10 mm (Figure 5). These findings indicate that 10 mm seems to be a reference or standard thickness in the BPI test. NX (54 mm) or NQ (47.6 mm) size cores are commonly extracted both in site investigation drilling and laboratory coring. A 50 mm diameter lies between NX and NQ sizes and is commonly used as a standard diameter in strength tests, such as point load testing. Considering these facts a core diameter of 50 mm was chosen as a reference diameter.

For the establishment of the thickness correction factor, failure load (F) versus specimen thickness plots of each test set consisting of a number of discs in different thickness (t) and diameter (D) were drawn and the relationship between F and t for each sample was determined by regression analysis with various functions. The best fit with the experimental data was obtained using the linear function. The load (F_{10,d}) corresponding to the failure of a specimen with standard thickness was determined for different diameters for each sample separately (Figure 6). A thickness correction factor, \( K_t \), given by the following expression is suggested.

\[ K_t = \frac{F_{10,d}}{F_{t,d}} \]  \hspace{1cm} (2)

where \( F_{t,d} \) is the load required for the failure of a specimen of any diameter and any thickness. The thickness correction factor then was correlated with the specimen thickness for various diameters used in this study by regression analysis. A generalized \( K_t \)–t relationship involv-
ing all the diameters employed (Figure 6) was expressed by the following equation
\( r = 0.89 \).

\[ K_t = 14.7 t^{-1.167} \]  \hspace{1cm} (3)

The failure loads determined for the cores in different diameters were initially corrected for thickness using Equation 3 and then the failure load (\( F_{10,50} \); equivalent load) corresponding to the failure of a specimen having a diameter of 50 mm and a thickness of 10 mm was determined for each set separately (Figure 6). The following expression is proposed for the diameter correction factor, \( K_d \):

\[ K_d = \frac{F_{10,50}}{F_{10,d}} \]  \hspace{1cm} (4)

The results of the regression analysis carried out considering \( K_d \) and diameter data pairs indicated the following equation (\( r = 0.88 \)):

\begin{align*}
\text{Figure 3. Specimens after testing (V: valid test, IV: invalid test)} \\
\text{Figure 4. Variation of uncorrected BPI(a) and corrected BPI(b) with specimen thickness and diameter for a mudstone} \\
\text{Figure 5. Histograms showing the ratios between valid and invalid BPI tests for various specimen thicknesses}
\end{align*}

\[ K_d = 211.5 D^{-1.3687} \]  \hspace{1cm} (5)

Combining both the thickness (Equation 2) and the diameter (Equation 4) correction factors, the following expression is obtained.

\[ F_{10,50} = K_d K_t F_{10,d} \]  \hspace{1cm} (6)

Dividing both sides of Equation 6 by the failure area of a standard disc specimen and inserting the values found for \( K_t \) (Equation 3) and \( K_d \) (Equation 5) yields the following expression suggested for the calculation of the corrected BPI where \( t \) and \( D \) in Equation 7 are measured in millimeters.

\[ \text{BPI}_c = 3376 D^{-1.3687} t^{-1.1672} F_{10,d} \text{ (MPa)} \]  \hspace{1cm} (7)

Figure 4b, for a selected rock type (mudstone), indicates that the differences in the BPI values are eliminated after size correction.

**Possible Uses of BPI in Rock Engineering**

**BPI as a Means of Assessing UCS**

The statistical analyses, carried out to examine the relation between the size-corrected BPI and UCS, indicated the equation given in Figure 7 with a statistically significant correlation of 0.94 at an 85 percent confidence level. It is customary to convert point load index to an equivalent UCS by multiplying by a factor, \( k \). However, no investigator was able to approach the same \( k \) value. A wide scatter of values of \( k \) ranging from 13 to 50 is apparent (Norbury, 1986). The rock mechanics community now agrees that the factor \( k \) of 24 suggested by Bieniawski (1975) cannot be universally applied and therefore, the point load should be used as an index. Similarly, the validity of the conversion factor of 5.5 for the UCS-BPI\(_c\) relationship found in this study was checked for each rock separately. Figure 8 indicated that the strength conversion factors range between 4.6 and 6.2 with a mean value of 5.39 which is in very close agreement with that found in the study.
Figure 6. A flow chart summarizing the methodology of this study (Figure 7). In general it is suggested that the strength conversion factor of 5.5 can be used to predict UCS values of rocks. Assuming a mean UCS/BPIc ratio of 5.5 leads to errors of up to 16 percent in estimations of the UCS from BPIc. This may be sufficiently accurate for using BPI as an index for intact rock strength in rock mass classifications.

The Use of the BPI in Rock Mass Classification
Because of a high correlation between BPIc and UCS, and very little scattering of data points, it was considered that the corrected block punch index (BPIc) which represents the UCS can be introduced into rock mass classification systems as an alternative input parameter, especially for weak rocks where obtaining a standard specimen is rather difficult. Considering the strength conversion factor of 5.5 found in this study, a combined chart (Figure 9) showing the variation of the ratings both for block punch index and UCS to be used for the RMR (Bieniawski, 1989) and M-RMR (Unal, 1996) a classification scheme was suggested.

BPI as a Means of Strength Anisotropy
Transversely isotropic rock strength values are obtained when failure initiates normal and parallel to the planes of transverse anisotropy, i.e., bedding, cleavage etc. The ratio between these values may be regarded as the maximum strength anisotropy of the rock as considered in uniaxial compressive test and point load test. The diagram shown in Figure 10a presents a plot of BPIc as a function of the angle between loading direction and bedding for the fami-
nated mahl tested in this study. It is evident from the figure that the greatest strengths were found for the specimens drilled normal to the weakness planes. These results are similar to those obtained by Stacey (1980). Thus, it seems possible that the BPI test can also be used to determine the strength anisotropy. Block punch testing along weakness planes may lead to variable results. Therefore, in any case, these may not be correlated with the compressive strength measurement in the same direction. This led the authors of this study preferring to measure only the strength in the strongest direction, the intact strength, and to consider the weakness planes separately as structural features. But the question, in this case, is that if the testing is carried out on cores from boreholes inclined at any angle to the weakness planes, how can the BPI be converted to strength in the strongest direction (BPI\textsubscript{90}). Therefore, a strength anisotropy transformation factor of K\textsubscript{a} was suggested to transform the BPI obtained from any direction (BPI\textsubscript{\alpha}), to that of the strongest direction (BPI\textsubscript{90}).

\[
K_a = \frac{BPI_{\alpha}}{BPI_{90}}
\]  

(8)

Values of K\textsubscript{a} have been plotted on Figure 10b as a function of the angle, \(\alpha\), between the loading direction and weakness plane. Each point on the curve represents the average value determined in five tests on the marl. The equations for K\textsubscript{a} and BPI\textsubscript{90} are given in Figure 10b.

**Conclusions**

- It was experimentally shown that the size correction in block punch testing cannot be ignored. Considering a reference diameter and thickness of 50 mm and 10 mm, the size correction factors suggested in the study for thickness and diameter, could be applied to the specimens tested.
- The BPI test is particularly useful for laminated weak rocks divided into discs from which suitable specimens for UCS and/or point load test can not be obtained. The suggested method can offer a quicker, cheaper and easier means to estimate strength. This advantage of the test makes the BPI as an alternative input parameter for intact rock strength in rock mass classification.
- Because the BPI tests lead to insignificant errors in determining UCS when compared to those obtained from point load testing, the BPI test could be more preferable to point load index in estimation of UCS.
- The strength anisotropy can be assessed by the BPI test. The BPI test seems to offer the opportunity to determine the strength in the strongest direction from the core samples with weakness planes at any inclination using the strength anisotropy transformation factor.
- The authors believe that the use of equivalent disc diameter and thickness will provide a rational approach for the standardization of the test in the future. It is recommended that correction factors be determined for several types of rock such that the degree of variance in correction factors between rock types could be evaluated. Future studies on the stress distribution within the rock discs under BPI test by means of numerical methods seem to be useful.

**References**

Phenomenon of Dilatancy in Fragmented Rocks

By Sergei Tsirel

Changes in the volume of fragmented rocks are of great importance for dynamic processes (whether natural or induced) in a rock massif. Mechanisms of fragmented rocks volume changes versus their initial volume are manifold: in cases near the surface the main factors constitute raveling and tension, in the case of deeper layers the main factor is compression, tension and dilatancy. The latter can result in the most essential volume changes as these concern not the rock blocks compressibility but the differences in their packing (in the broad sense of the word).

Of late, different researchers have paid great attention to the phenomenon of dilatancy. Three dilatancy processes have been investigated most often: dilatancy loosening because of the formation of microcracks in an otherwise undisturbed medium [1,2]; density changes in a model medium built of balls and cubes [3,4]; a rise in the massif volume during its slip along a single rough crack [5,6]. The least investigated is the kind of dilatancy first registered by O. Reynolds, namely, changes in the density of the granular (fragmented) medium consisting of irregular form elements when subjected to shear. The present paper is an attempt to fill that blank.

To determine the influence of particle size distribution on dilatancy velocity it is necessary to consider the case of a simple shear of fragmented but otherwise undisturbed rock (Figure 1). Under minor shears particles are in engagement and dilatancy loosening is very much similar to that observed in rocks fragmenting under a nonuniform loading.

Data from [1] were used in the discussion of that stage of loosening. When blocks start disengaging, gaps appear. First only very small fragments can get into them, then when gaps grow in size they admit more important fragments, etc. Under important shears all kinds of fractions (with the exception of the smallest and biggest) gradually fill the gaps between the largest fragments, at the same time creating gaps for the smallest fragments. The size of gaps is determined by the shear angle \( \xi \), particle size distribution \( F(x) \), distribution of angles \( \alpha \), which control fragment movement under shear. The process is represented schematically in Figure 2.

Computation of the dilatancy loosening process is done on the basis of the method for determining the package density of granular and fragmented materials [7]. As shown in [7], the main values controlling density are the index of fragmentation uniformity

\[
n = \left( \frac{\int_{x_{\text{min}}}^{x_{\text{max}}} \chi \left( \frac{\partial F}{\partial x} \right)^2 \, dx}{\int_{x_{\text{min}}}^{x_{\text{max}}} \chi \, dx} \right)^{1/2}
\]

(for the blocks of same size \( n \rightarrow \infty \), as fragmentation grows \( n \) becomes smaller); quality of fraction mixture \( 0 < p < 1 \) (for fragmented rock \( p \), on the average, is 0.7-0.75); the degree of loosening of mixture of similar size fragments \( K_p^0 = V/V_0 \) (in the case of a totally loosened medium consisting of rough fragments, \( K_p^0 \) constitutes 1.8-2.1 depending on fragment shapes). In that particular case \( K_p^0 \) is substituted for by series of values \( 1 \leq K_p^{\alpha} \leq K_p^0 \) with masses of different \( K_p^{\alpha} \) values being mixed (Figure 2 shows

Figure 1. Volume changes in the course of fragmented rock shear

Figure 2. Scheme of dilatancy loosening
the case of squares when $\alpha_i = \pi / 4$.

Specimens of dilatancy curves calculated by this method for the real particle size distribution of fragmented rocks are shown in Figure 3. The run of the curve $n(\xi)$ in the case of a uniform distribution of shear movement angles $\alpha$, depends essentially on the content of small fractions. If it is not too high, the curve has a portion of the volume drastic growth when $\xi \approx 0.02$–0.1, but if shear grows, the dilatancy velocity $A$ diminishes noticeably and then continues to decrease gradually. If the content of small fractions is very high, then the changes of $A$ acquire a smoother but, at the same time, more complicated character. The difference occurs because the smallest fragments can penetrate even into very narrow slots which were formed at the early stages of the shear. Consequently, if the quantity of such fragments is important, it hinders the volume growth. The character of the run of the curve $n(\xi)$ depends on the $n$ value. If $n$ is rather great (there are few medium size fragments), then, at certain stages of the shear, there occurs a deficiency of fragments capable of getting into accessible slots. Consequently, the volume deformation increases substantially. There is another factor which determines dilatancy loosening, namely, the predominant direction of cracks and shear angles $\alpha$ (Figure 3b). When great values of $\alpha$ prevail, the dilatancy curve is close to dilatancy curves characteristic of rock masses with a little content of small fractions. A predominance of small values of $\alpha$ influences the dilatancy curve in much the same manner as the high content of small fractions.

The above method of dilatancy curves computation makes it possible to establish the mechanism of loosening in rock blasting. According to V.V. Rzhhevskii [9] fragmented rocks in the overburden are divided depending on the degree of viscosity into three categories: I. Fragmented granular rocks (forming distinct high walls, $K_p = 1.4$–1.65); II. “bound-granulated” fragmented rocks (not forming distinct high walls, $K_p = 1.15$–1.3); III. “bound-fragmented” rocks (high walls are steep, $K_p = 1.03$–1.05, more seldom 1.05–1.1). The most obvious mechanisms of fragmented granular rock formation, approximating $K_p$ values of drastically loosened mass, are raveling, scattering, tension (collision of layers or formation of a dome, see Figure 4). Rocks belonging to category I or III are found deep in the overburden, forming under conventional parameters of blasting the major part of the rock mass, are not liable to either raveling or scattering or tension. Our computations show that the mechanism of deep layers loosening must consist in dilatancy (great shear deformations are due to a sharp decrease of shift as the distance to the surface grows).

This hypothesis is reinforced by the comparison of theoretical curves with the data of E.G. Baranov and I.A. Tangidev [8], which were obtained in the course of simultaneous measurements of $\varepsilon$ and shifts during blasting in open cast collieries. The data [8] gives grounds to expect that both types of dilatancy curves are observed in practice (Figure 3).

The second example of the use of the suggested method applies to granular medium deformation. According to the method, processes of granular mass compaction are divided into four types: fragment movement to “good” places ($p$ grows); fragment position “bettered” ($K_p$ decrease); compression of these fragments ($V$ decreases); or fracturing of fragments ($n$ decreases). The impact of each of these processes is determined by the general relation $K_p(n, K_p, p, n)$ which is illustrated in Figure 5.

During shaking or at the early stages of vibro-compaction (as long as vibroseparation is of minor importance) the density change is controlled, first and foremost, by the value of $K_p$ and $p$. If the size of fragments is nearly the same ($n$ is great), then the mixture quality does not much influence the density, and $K_p$ by means of the “trial-and-error” method is nearing its $K_p$ stable value. The loosening (compaction) effect is governed by the difference between $K_p$ and $K_p^0$. The $K_p$ value arrived at is not stable. Under strong vibrations loosening can once more become greater (decrease). When vibration is effectuated simultaneously with compression, a disengagement of fragments (if the material consists of particles similar in size) takes place in

![Figure 3. Specimens of dilatancy curves: a. the equiprobable distribution of shear angles; b. the occurrence of dominant angles; the points shows experimental data [8] (56)](image-url)
the narrow time interval (quick slumps). On the other hand, if the material consists of fragments heterogeneous in size ($n$ is small), then slumps have a smoother character. If there is no compression, then, for the medium with a small value of $n$, the major factor is the quality of mixture. Such materials require greater amplitudes of movement; at the same time their compaction is more resistant to further vibrations than materials with a homogeneous particle size distribution.

Dilatancy leading to $K_p^0$ changes and fragmentation plays a decisive role in compaction under a nonuniform compression. The degree to which dilatancy and fragmentation influence compaction depend, first of all, on the ratio of $\sigma_2$ to $\sigma_1$, friction of fragment surfaces and their strength. As large fragments are characterized by greater roughness and lesser strength, it follows that when the fragment size decreases so does the contribution of fragmentation whereas the role of dilatancy increases. In granular media with a widely varying particle size distribution the role of average size particles belongs to the conventional size of the rigid skeleton particles $x_n$. When the quality of mixture is poor, $x_n$ in fact, represents some range of sizes. In a good quality mixture ($p = 1$) the $x_n$ value is roughly equal to the minimum size rigid skeletons particles and can, approximately, be computed by the formula:

$$x_n = \frac{d_p(n+1)}{n(5n-1)^{1/(n+0.4)}}$$

A tentative evaluation, taking into consideration the strength scale dependence to

$$\frac{\Delta d/d_0}{\Delta d/d_0} = \sqrt{x_n}$$

where

$$\Delta d = d_p - d,$$

means a change of average size particles as a result of fragmentation. The influence of $x_n$ on fracture can be taken as a change of the

Figure 4. Main mechanisms of complete loosening: ravelling (a), scattering (b), tension (c, d).

Figure 5. Influence of different factors on fractured rock volume. a) an example of $K_p K_p^0$ relation, $p$ values are shown near the curves; b) influence of $x_n$ on the mechanism of fractured rock compaction.
of all, to the appearance and growth of microcracks. When blocks are smallish and \( n \) values are small too, shear loads lead to block movement, which are accompanied by a dilatancy volume growth. So when \( n \) values are small, then the \( x_n \) values are much smaller than the value of \( d_n \) and the appearance of large cracks in blocks is improbable. Besides, the slip of blocks in relation to each other results in a surface disturbance (small fragments break away from block surfaces \([8,11]\), which results in an additional decrease of the \( n \) value. The process enhances the packing density (because of the swing from one dilatancy curve to another) and, to a considerable degree, hinders the volume growth. Later on the movement is essentially reduced to narrow strips of slip with minimum \( n \) values, whereas in the remaining parts of the fractured zone favorable conditions for "doctoring" are created.

The most complicated case is observed when both average block size and the index of fragmentation uniformity (\( n > 1-1.5 \)) are relatively great. In fractured rocks of this type block movements and slips are possible. However, these are followed by an important growth of specific volume and strengths, which prevents their occurrence. In the fractured zone block fragmentation (not only superficial but volumetric as well, i.e., blocks divide into parts comparable in size \([11]\)) is observed. The cause of it is that \( x_n \) and \( d_n \) values are near in the order of their magnitude. Simultaneously, considerable cracks are formed with the ensuing sharp release of strength and seismic phenomena. Alternating loads can result in fractured rock consolidation but at the cost of \( K_p^0 \) decrease by the trial-and-error method. However, such consolidation is unstable and in the course of the following movements which have a more or less stable sense the process of dilatancy loosening and fragmentation resumes. The further course of the process depends on the relation between the intensity of disturbances in the zone itself and the surrounding area. With disturbances (mainly on the surface) prevailing in the zone, the values of \( d_n \) and \( n \) gradually decrease which, eventually, leads to the rock consolidation and the disappearance of important disturbances. In the opposite case, cracks occur and seismic phenomena continue, with centers of the most intensive processes moving to adjoining areas.

References
Pulse-induced Fault Slip: Model and Application to the 1995 Hanshin (Kobe) Earthquake

By K. Uenishi* & H.P. Rossmanith

Introduction

The classical approach to fault rupture mechanisms associated with earthquakes (e.g., Scheidegger 1982) assumes that rupture initiates at one point on the fault and spreads across the entire fault zone until arrest of the rupture occurs. However, Heaton (1990) showed, based on seismic inversions, that local rise times for fault slip are much shorter than would be expected by the classical approach. As a model for the short rise time observed, he suggested that self-healing pulses of slip occur in earthquake ruptures (Turcotte 1997).

From the results obtained by laboratory frictional experiments, Brune et al. (Brune et al. 1993; Anoshehpoor & Brune 1994) propose fundamentally the same fault rupture mechanism. Their idea is that dynamic rupture on a fault is triggered by a pulse with a separational section propagating along the fault, or, in other words, by ripples which propagate along the fault and slip occurs while the compressive stress is reduced (Mora & Place 1994). It is suggested that the excitation of Rayleigh waves on a rupture surface lead to pulses of separation (Turcotte 1997).

These mechanisms, however, have not been confirmed in a conclusive manner; mathematically, rupture pulses with interface (fault) separation were studied by Comminou and Dundurs (1977, 1978a,b). They showed that a pulse involving separation can stably propagate along an interface when two elastic media are compressed and simultaneously sheared, though the validity of their solution was questioned by Freund (1978) from an energy point of view (Anoshehpoor & Brune 1994). Numerically, Mora and Place (1994) showed that self-healing pulses can be sustained if interface surface roughness is present. Andrews and Ben-Zion (1997) conducted 2D numerical simulations of dynamic rupture along a planar material interface governed by simple friction, showing self-sustaining propagation of slip pulse and spontaneous break up of the propagating pulse to a number of smaller pulses.

In this contribution, the basic mechanisms of dynamic fault rupture (instability) induced by a Rayleigh (R-) pulse will be investigated (Uenishi 1997; Uenishi et al. 1997, 1998). First, the results of laboratory model experiments utilizing dynamic photoelasticity will be presented. Second, the problem will be numerically simulated using the SWIFD finite difference simulator (Rossmanith & Uenishi 1996). Finally, the results obtained by the model investigation will be applied to explain the damage concentration caused by the 1995 Hanshin (Kobe), Japan, earthquake.

Experimental Investigations

The experimental model is schematically shown in Figure 1. It consists of two plates of Araldite B (a transparent, birefringent, homogeneous, and isotropic polymeric material) which are statically compressed. The upper surface of the plate 2 is given a very blunt double wedge cut such that only the central section of the surface would initially be in contact with plate 1. The wedge-type gap introduces a contact singularity which is utilized for contact trace purposes. It has very little bearing on the phenomenon to be investigated. Contact region is not glued and during the interaction process the contact area can recede or be increased depending on the relative position of the R-pulse with respect to the contact region. The dynamic disturbance is generated in plate 1 by detonating a small amount of explosive. A Czerny-Schardin type multiple spark gap camera is used to record a sequence of isochromatic fringe patterns (contours of maximum in-plane shear stress) of the dynamic interaction process.

Figure 2 shows two “snapshots” of isochromatic fringe patterns of the dynamic wave interaction process. The R-pulse propagates in plate 1 from left to right and interacts with the contact region. The time scale t indicates the time elapsed from the instance of maximum stress amplification at the lhs.

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Figure 1. The experimental model for Rayleigh pulse interaction investigation (all lengths in mm)
Figure 2. Experimentally obtained snapshots of isochromatic fringe patterns at times: (a) t=0μs; and (b) t=53μs

in Figure 2(b), partial wave energy transmission occurs across the interface into the lower plate 2.
Due to the separational movements of surface particles in the trailing part of the R-pulse, the corresponding fringes in the lower plate 2 are missing (interface separation).

Numerical Simulations

The dynamic R-pulse interaction with a contact region between dissimilar materials is numerically investigated by using the SWIFT finite difference wave propagation simulator (Rosmanith & Uenishi 1996). Figure 3 shows a numerically generated isochromatic fringe pattern pertaining to the case where the incident R-pulse speed ($c_{R1}$) lies between the shear (S-) and longitudinal (P-) wave speeds of the lower material 2, ($c_{S2}$) ≪ ($c_{P1}$) ≪ ($c_{P2}$). Since the slip pulse propagates at the Rayleigh wave speed of the acoustically harder material 1, ($c_{R1}$), the pulse energy is transferred along the contact region at transonic speed with respect to the acoustically softer material 2. A shear-type (S-) head wave is generated which propagates from the contact region into material 2.

Comparison of Figures 2(b) and 3 indicates the influence of the acoustic impedance mismatch of the two contacting materials on wave transmission and reflection at the interface.

Case Example

On 17 January 1995, at 5:46 a.m. local time, an earthquake of moment magnitude 6.9 struck the region of Kobe and Osaka (Hanshin region) in the west-central part of mainland Japan. Seismic inversion (Wald 1996) indicates that the rupture was initiated at a shallow depth on a fault system running through the city of Kobe and propagated bilaterally: along the Suna/Suwayama faults toward the city of Kobe and along the Nijima fault [Figure 4(a)].

Strong ground motion lasted for some 20 seconds and caused considerable damage within a radius of 100 km from the epicenter, but most severely affected were Kobe and its neighboring cities (EQF International 1995).
One of the puzzling phenomena observed in Kobe (Kawase 1996; Suzuki et al. 1996) is the emergence of the "belt" of the most severely damaged zone [the Japan Meteorological Agency Intensity 7 zone indicated in dark gray in Figure 4(a)]. This belt, about 20 km long, was found close, but not parallel, to the Suma/Suwayama faults. Damage due to liquefaction was hardly observed inside this belt, and it is suggested that the damage was directly due to the seismic waves.

As the Suma/Suwayama faults dip steeply, nearly 90°, and a near-source, S11 directivity pulse from strike slip faulting was recorded (Toki et al. 1995) in the city of Kobe, a 2D model including a plane normal to the fault plane is considered as appropriate for an approximate first order analysis of the rupture mechanisms of the Hanshin earthquake.

In Kobe [lower lhs corner in Figure 4(a)], a region of relatively large slip (seismological asperity) was found (Wald 1996). Seismological asperities are of practical importance in earthquake hazard analysis because the failure of asperities radiates most of the high-frequency seismic energy into the far-field. In the model study, it has been shown in Figures 2(b) and 3 that a relatively large amount of pulse energy is radiated into the far-field in the form of hulk waves from a slipping contact region. This observation suggests that, when scaled up, the contact region in the model can be considered as an asperity on a geological fault.

In Figure 3, where an interface is located between dissimilar materials \( \{c_{S2} \leq c_{R1} \leq c_{P2}\} \), an S-head wave is observed. Head waves generated at fault zones have been theoretically predicted for planar material interfaces (e.g., Ben-Zion & Aki 1990) and they have been observed along several sections of the San Andreas fault (McNally & McEvilly 1977; Ben-Zion & Malin 1991; Andrews & Ben-Zion 1997), along small segments in the Mojave, eastern California, shear zone (Hough et al. 1994; Andrews & Ben-Zion 1997). Also in central Kobe, arrival of a concentrated shear disturbance as well as the resulting large (particle) velocity were recorded (Wald 1996). This indicates that the rupture-induced shear wave in Kobe was of a head wave type and the numerical model \( \{c_{S2} \leq c_{R1} \leq c_{P2}\} \) can be used to analyze the influence of the rupture-induced waves. Material 1 in the model corresponds to the acoustically harder region in the Rokko Mountains, and material 2 fits the acoustically softer region including the "damage belt."

The maximum values of velocity and acceleration experienced during dynamic interaction at each point in the materials are of practical importance and, hence, die peak particle velocity (PPV) and the peak particle acceleration (PPA) have been selected as the design parameters in many applications such as blasting in mines and quarries, and in engineering seismology. Figure 4 shows the contours of high PPV [Figure 4(b)] and PPA [Figure 4(c)] obtained by the numerical simulation \( \{c_{S2} < c_{R1} < c_{P2}\} \). It is interesting to note that the high PPV (or PPA) region is found in a narrow band, similar to the shape of the "damage belt" in Kobe [Figure 4(a)]. The angle between the fault (interface) and the high PPV (PPA) region is influenced by the S-head wave generated during the dynamic interaction (Figure 3). This result indicates that a simple two-dimensional model may be able to provide the information about the earthquake rupture and the ensuing dynamic wave phenomena, although for a more sophisticated study a three-dimensional analysis which includes local geological as well as topographical effects on the real fault rupture process is required.

Conclusions
The purpose of this study was to obtain an improved understanding of the pulse-induced instability of a fault between similar and dissimilar materials. The
experimental and numerical model investigations have given a clear insight into the basic mechanisms of die Rayleigh (R)-pulse-induced instability of a statically pre-stressed fault: fault interface separation (instability) can be induced by a R-pulse, and the observed dynamic stress amplification and reduction at the interface are caused by the push and pull normal particle motion associated with a R-pulse. The SWFD finite difference simulator has shown that pulse interaction patterns are controlled by the acoustic impedance mismatch of the contacting materials and head waves can be generated at and propagated from a contact region between dissimilar materials.

As a case example, damage concentration mechanism associated with the 1995 Hanshin (Kobe) earthquake has been studied using the 2D numerical model. It has been suggested that the rupture-induced shear wave was a head wave which propagated from a rupturing asperity (contact region) located between dissimilar materials. The numerical simulation has shown that the regions of high peak particle velocity (PPV) and peak particle acceleration (PPA) form a narrow band, which is similar to the shape of the "belt" of the most severely damaged zone in Kobe.

If an interface pulse with a separational zone can propagate along a fault under the high compressive stress that is expected at depth in the Earth, such a pulse may be a solution to the long standing paradoxes associated with earthquakes: the origin of short rise times in earthquake slip; dynamic contributions to spatio-temporal slip complexities; the anomalous P-wave radiation; the heat flow paradox; and the low static shear stress levels on geological faults.

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References


Man-Made Rock Structures in Cappadocia, Turkey and Their Implications in Rock Mechanics and Rock Engineering

By Omer Aydan¹, Resat Ulusay², Erdogan Yüzer³ and Mustafa Erdogan³

Introduction

Turkey, a bridge between the Asian and European continents, is richly endowed with a cultural mosaic of various civilizations that have inhabited this land since the Paleolithic Age. One of the seven sites, which are included in the World Heritage List, is the Cappadocia Region. Cappadocia area extends over 5000 km² within the triangle of Kayseri-Aksaray-Nigde in Central Anatolia of Turkey (Figure 1). This area forms a high plateau and is covered by almost horizontally layered acidic volcanic tuffs and lavas from Erçiyes, Melendiz and Hasandag volcanoes several hundred meters thick.

Easy carving and thermal isolation properties of these soft tuffs have been the main reasons for the extensive multi-purpose underground settlements in this region from past to present. There are 23 known large-scale underground cities in which thousands of people lived in the Cappadocia Region in historical times (Figure 2). It is also interesting to note that in Göreme, the volcanic tuff layers eroded by the water and wind action created the strange landscape of towering cones named “Fairy Chimneys” (Figure 3), and the valleys in which impressive hundreds of dwellings, catacombs and rock-cut churches were carved. The self-supporting structures in these soft rocks, 90 m below the surface, consisted of 8-10 floors with large ceremonial halls, storage areas with amazing ventilation systems. The underground openings with different sizes and shapes excavated in this area were also being used today for lodging, housing, and the storage of foods and fruits in a limited extent compared to past usage.

In this article, geography, climate and geological characteristics of the Cappadocia Region are first briefly explained. Then, the geomechanical characteristics of the tuffs and history of rock settlements are described. In addition, a critical overview is made on rock mechanical aspects of historical and modern rock structures, and their implications in rock mechanics and rock engineering from a viewpoint of long-term performance of the structures. Finally, some possible international collaborative research topics on rock structures are pointed out.

Geography and Climate of the Cappadocia Region

Cappadocia is situated in Central Anatolia area of Turkey. Its altitude ranges between 1,300 m and 1,400 m, forming a high plateau. It has a triangular shape and is surrounded by the Erçiyes (3,917 m), Melendiz (2,935 m) and Hasandag (3,254 m) volcanoes. The Kızılırmak river (the longest river in Turkey) flows through the northern part of the area and makes a turn near Avanos. This river with its sub-branches was the principal architect of the spectacular morphological features in the Cappadocia Region. The drainage system developed within the tuffs is generally dendritic with local structurally controlled trellis patterns. Another important physiographic feature of the area is the fairy chimneys located along the valleys.

In Cappadocia a continental climate dominates the region. The winters are cold and severe with moderate snowfall which may remain on the ground for long periods. In spring, rainfall becomes dominant. During summer, it is hot and dry. Evaluation of the meteorological records provided by the Nevşehir meteorological station indicates that the highest temperatures occur in July and August. During the period between 1960 and 1990, the highest precipitation has occurred during winter and spring, and the lowest during July and August. The average humidity is low in summer, and the summer is generally dry throughout the region.

On the basis of the meteorological data collected by Topal (1995) from Nevşehir station, 68, 38 and 62 freezing-thawing cycles were recorded during 1990, 1991 and 1993, respectively. These cycles were obtained from the temperature variations around 0°C.

The region is poorly vegetated. Vegetation is confined to valleys. Since the area is barren, most of the rainfall and snow melts form surface runoff which promotes active erosion.

Geology of the Site

The Cappadocia Region is generally underlain by volcanic rocks from the Neogene-Quaternary period belonging to the Cappadocian Volcanic Province (CVIP) (Toprak et al., 1994). It extends as a belt in a NW-SE direction with a long axis of about 300 km (Figure 1). The related volcanic activity is generally considered to occur as a consequence of plate convergence and continental collision occurring between African-Arabian and Eurasian plates in the

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Several individual volcanic complexes have been identified within the CVP (Figure 1). They correspond to the major eruptive centers in the province and form huge topographic masses in the area. Most of them are polygenetic volcanoes. Others are in the form of either a dome or a caldera.

Figure 1. Location map of the Cappadocia Region and simplified geological map of the Cappadocian Volcanic Province (after Toprak et al., 1994)
1. Karacadag volcanics
2. Küllüdagi volcanics
3. Keçikalesi volcanics
4. Hasandağ volcanics
5. Keçiboyduran volc.
6. Melendiz volcanics
7. Tepeköy volcanics
8. Çinarlı volcanics
9. Göllüdagi volcanics
10. Kızılçıng volcanics
11. Aşığıl volcanics
12. Hüushimağ volcanics
13. Tekerdağ volcanics
14. Seksenveren volcanics
15. Hamurcu volcanics
16. Erkilet volcanics
17. Erçiyes volcanics
18. Koçdağ volcanics
19. Develişdag volcanics
Basement rocks, the Yeşilhisar and Ürgüp formations and Quaternary deposits are the main units observed in and around the area. The basement rocks consist of ophiolite (gabbro and pyroxenite) and plutonic rocks (syenite and monzonite) (Temel, 1992). However, Batum (1978) and Seymen (1981) also indicated the presence of metamorphic rocks within the Mesozoic basement rocks. The Yeşilhisar formation is a coarsely bedded fluvial deposit consisting of red mudstone, sandstone, conglomerate alternations. The formation includes granite, quartzite, chert, marble, limestone and ophiolitic fragments which are rounded and 20–30 cm.

The Ürgüp formation, unconformably overlying the Yeşilhisar formation, has widespread exposures in the area, and the underground structures and the fairy chimneys are located in this formation. It comprises continental (fluvial to lacustrine) sediments interbedded with several pyroclastic deposits. The formation is nearly horizontal and constitutes a 1,100–1,200 m high plateau near Ürgüp. Its age is Late Miocene (Tortonian)–Pliocene, as determined by the radiometric dating of pyroclastic rocks (Temel, 1992). Because of the very marked lithological variability both vertically and horizontally within the formation, the formation is divided into 12 members in the region (Temel, 1992). These are, from older to younger: Cökek, Kavak, Zelve, Sarımaden, Damsa, Cemilköy, Tahir, Gördeles, Sofular, Topuzdag, Kızılkaya and Kılsladag. A generalized vertical section of the Ürgüp formation which gives brief geological information about each member is depicted in Figure 4. The Kavak member, covering an area of at least 2,600 km², is the most com-
The tuffs are generally white, gray and pink and clastic in character, alternating from fine grained to coarse types with enclosed large and small lumps of pumice and obsidian, in some places interbedded with welded tuffs and ignimbrites in different induration and in others places with clay and marly clay beds.

Two fault systems cut across the volcanic province (Toprak and Gönçüoğlu, 1993). The first system, trending dominantly in a N-S to NW-SE direction, is the Tuzgöl-Ecemis fault system (see Figure 1). The faults within this system are still active and played a role in the recent deformation of the Anatolian block. The second fault system strikes NE-SW, almost parallel to the long axis of the province. Although some of these faults are still active, most of them were buried beneath the later eruptions of the CVP (Toprak et al., 1994).

Seismicity of Cappadocia

The Cappadocia region is bounded by two active strike-slip faults—Tuzgöl and Ecemis. The region is seismically less active compared with other regions of Turkey (Aydın, 1997). The largest earthquake, with a magnitude of 5.2, occurred in February 1940 near Erzýeys Volcano during the instrumental period (Eyidogán et al., 1991). Thirty seven people were killed.

Material and Rock Mass Characteristics of the Tuffs

Several investigations in the Cappadocia Region were performed for the purposes of geological and geomorphological interests, deterioration and conservation, and environmental and rock mechanics (Doyuran, 1971; Erguvanlı and Yüzer, 1977; Erguvanlı et al., 1988; Erdogan, 1986, 1989; Yılmazer, 1993; Toprak, 1985; Binal, 1996). Most of these experimental studies were concerned with the preservation of man-made historical structures in the Cappadocia region. The purpose of the experimental study by Erdogan (1986, 1988) was an exception among the previous experimental studies. It was concerned with the suitability of tuffs as a construction material in the Nevşehir-Urgup area.

Recently, an international joint research project between Japan and Turkey was initiated in mid-1997 in Cappadocia. The project, entitled “Environmental Study on Underground City Derinkuyu, Turkey” (MONBUS10-Res. Project No: 09044154), is supported by the Japanese Ministry of Education. This ongoing project is being led by the first author of this article; the other authors constitute the Turkish side. In 1997 the Cappadocia area, particularly the underground cities, was visited for investigations and Schmidt hammer tests on the walls of these openings, continuous measurements of temperature and humidity were carried out. In addition, a laboratory testing program involving various rock mechanics tests on the samples obtained from Urgup and Avanos (Ozkonak Underground City) to determine the physical and short- and long-term mechanical properties of the tuffs has begun. The process on the available data and site visits for the assessment of long-term performance of the underground openings will be continued in 1998 and 1999.

The values of the uniaxial compressive strength indicate that the tuffs are moderately weak to very weak. Their strength sharply decreases when they are saturated. This is an important factor in why they can be easily carved. They have high deformability. The engineering properties of the Kayak tuffs, covering a large area in the region, do not show significant changes in the vertical and horizontal directions. The rebound number ranges between 10 to 30 with an average of 15 and the effect of depth is almost non-existent. Lower values were obtained when rock surface was wet. Figure 5 shows the relation between the rebound number and uniaxial compressive strength for Cappadocia tuffs.
compressive strength of Cappadocian tuffs. From this empirical relation the in-situ strength of tuffs is expected to be between 3 to 6 MPa. Based on the slake durability index, the Kavak tuffs have low-to-medium durability after two test cycles. In the recent study the number of cycles was increased to four and the slake durability index values decreased to 30-40 which indicate low durability (Ulusay et al., 1997). These results reveal that the tuffs are highly slakable and prone to atmospheric conditions. It is also noted that the freezing-thawing cycle is effective in weakening mechanical properties of the tuffs (Erdo?an, 1991; Topal, 1995; Binal, 1996; Ulusay et al., 1997).

Although the bedding planes are nearly horizontal, they are not distinct everywhere in the area. Therefore, joints are the major discontinuities within the tuffs. In order to estimate the rock mass quality of the tuffs, the rock mass rating scheme proposed by Bieniawski (1989) was used. The authors measured necessary rock mass parameters on the limited number of selected outcrops in the Kavak tuff. Since no consideration is made on a particular engineering application, the effect of orientation of the discontinuities and any adjustment are discarded. Thus the basic RMR value is obtained as 58–72, corresponding to fair to good rock. This preliminary estimation shows a good agreement with the range of basic RMR values (49–77) determined by Topal (1995) on the same tuffs using a number of detailed scanline surveys.

**History of Rock Settlements**

The rock settlements in the Cappadocia area can be divided into two main categories: cliff or semi-under-
Semi-underground settlements
Cliff settlements can mostly be seen along valleys and hills with steep cliffs in Nevşehir, İhlara, Zelve, Göreme and Selime. Dwellings, in hollowed-out rocks, are found around the main hills and steep valleys (Figure 6). These kinds of settlements lie mainly in and around the citadel. It is possible to use the passages or the steps carved inside the rocks or carved on the rocks, respectively, to go from one place to another. The people who lived in the citadel area until recent times were forced to leave these places due to a great increase in population and erosion of the dwellings.

In the Ionoclastic Period (725–845 A.D.) the Cappadocia Region gradually became the most important religious center in Anatolia. The Göreme Valley was chosen as a center for monastic education; monks were trained as missionaries in order to disseminate Christianity. Chapels, basilicas, churches with one or several domes, and dwellings up to six stories high, were carved in the Cappadocia tuffs. As well as being an important Christian settlement and religious center between the 9th and 13th centuries, the rock settlements of Zelve were also home to the first seminaries for priests (Gülyaz and Yenipazarlı, 1996). They were inhabited until 1952.

Another place for rock settlements is in the İhlara (Peristima) Valley, near Aksaray, where the tuffs are 14 km long. The numerous dwellings, churches and monasteries on both sides of the 170 m high canyon are connected to each other by tunnels as in the underground cities. In this valley there are 4,535 caves and 105 churches and monasteries.

Underground Cities
There are 23 known large-scale underground cities in the Cappadocia region. The underground cities were connected by hidden passages to the houses in the region. Hundreds of rooms in the underground cities were connected to each other with long passages and labyrinth-like tunnels. The corridors were made long, low and narrow to restrict the movement of intruders. There are milestones between the floors to separate the various areas. They can be opened only from the inside and are 1–1.5 m high and 30–35 cm wide.

Inside the underground cities shafts (usually connected with the lowest floor of the underground cities) were used for both ventilation and communication. They were also used as wells. Although some researchers claim that the underground settlements were connected to each other with tunnels, no con-

Figure 8. Temperature variation at various levels in Derinkuyu Underground City
clusive evidence to support this idea has been found yet (Gülyaz and Yenipazarlı, 1996).

The important underground settlements are Derinkuyu, Kaymaklı, Mazi, Tatların and Özkonağ. The underground cities of Kaymaklı and Derinkuyu are investigated in more detail than others. Therefore, in this article, a brief summary of these two cities and the Özkonağ Underground City is presented.

Kaymaklı Underground City is 19 km away from Nevşehir on the Nevşehir-Nigde road. This huge underground city is said to have accommodated 60,000 people. Today small halls and rooms with tunnel-like passages between them, large kitchens, large polished pits used as water cistern and wine vats, storage rooms, little chapels and ventilation chimneys can be seen. Only four floors are open to the public. According to the measurements in June 1977, the inside temperature was 12°C. on the first and second floors and 10°C. on the third and fourth floors when the temperature outside was 27°C. (Erguvanlı and Yüzer, 1977).

Figure 9. A wall painting showing a town in the foreground and an erupting volcano in the background, probably Hasandag (3268 m), found at Çatalhöyük (Neolithic, 6th millennium B.C., Museum of Anatolian Civilizations
reported that in June 1977, when the outside temperature was 27°C, the inside temperature of the Derinkuyu Underground City was 12°C, 5°C, and 7°C on the first, sixth and eighth floor, respectively. Figure 8 shows one of our recent measurements in Derinkuyu underground city on May 29, 1998. As seen from this figure, temperature variations underground are small compared with those on the surface. The amplitude of variation decreases as the depth increases.

**Possible Motivations for Underground Space Utilization in the Cappadocia Region**

The question is why and when the inhabitants of Cappadocia area excavated rock settlements. Possible answers to this question would probably be similar to those of the present time except waste disposal. The excavations of semi-underground settlements or underground cities are generally associated with the settlement of Christian communities escaping from the raids of Romans first and Arabs later. The reasoning is very simple and implies that the excavation dates for these settlements should be 1,400–1,800 years ago. However, there is strong archaeological evidence that Anatolia was also inhabited in the Paleolithic Age. It is most likely that the first inhabitants of Anatolia lived in natural caves whenever they were available. It is known that the first mining activities in Anatolia started at least 9,000 years ago in the form of open-pit mining (Kaptan, 1992). As a natural consequence, we can easily presume that they should have been able to produce utensils which can be also used as excavation tools. It is also known that underground mines were in operation 5,000 years ago, also located in Celaller Village (Nigde) in modern Cappadocia (Kaptan, 1992). These facts simply imply that the earliest inhabitants of Anatolia must have been able to excavate rock structures, particularly in soft rocks such as those found in the Cappadocia area much earlier than presumed.

The underground structures constructed by Hittites, who came to Anatolia through Caucus, are known and they are more than 5,000 years old. A Persian historian visiting the Cappadocia area 2,600 years ago when Persia invaded the area reported the existence of underground cities. The area was wealthy and was subjected to numerous raids from Macedonians, Greeks in the
west, Persians in the east, and Assyrians in the south. As a natural consequence, people realized the importance of building underground rock shelters against these raids in a relatively flat region. The first and second floors of underground cities are built irregularly while the floors below those levels are built in a regular manner. This implies that the upper floors must belong to earlier times and the new floors were built in later periods as a result of the settlement of larger Christian communities in the area.

The area is surrounded by the Erçıyê, Melendiz and Hasandag volcanoes which are known for rhyolitic eruptions. These volcanoes should have been erupting with pyroclastic flows. The danger of such hot ash flows is very well known as experienced in Fugen-dake in Japan, and Mt. St. Helens in the USA in modern times. The volcanic activity of Hasandag was depicted on a wall in the Çatalhöyük settlement 8,500-9,000 years ago (Figure 9). The Persian historian also mentioned that Erçıyê, Melendiz and Hasandag were holy fire mountains. Persians were known as worshippers of fire during that time. This should simply imply that these mountains must have been active at least 2,600 years ago. In addition, the Phrygian historian Strabon mentions burning marshes in Sultan Sazligi near the Erçıyê volcano which may be easily interpreted as the result of gases seeping through fractures in the ground. It is very likely that natural disasters caused by these volcanoes could also force people to build rock settlements and result in living in underground settlements for thousands of years.

The area is very hot in summer and very cold in winter. Temperature variations in underground caverns, as shown in Figure 8, are much less than outdoor temperature variations. Underground space is warmer during the winter and cooler in the summer which provides a very comfortable living environment as well as saving energy. It is very natural to expect that people would select underground space in such a harsh climatic environment. This could be a very important reason why the people of Cappadocia chose to live in rock settlements.

Modern Rock Structures

Even today, underground openings of different sizes and shapes in the Cappadocia Region are being used for housing, storing foods (especially fruits), mushroom plantations, pigeon holes for natural fertilizer and ceramic factories in a limited extent compared to the past use. The inside temperature of the underground storage spaces is controlled through air ventilation. The number of modern hotels hollowed out in the tuffs has considerably increased in recent years, particularly in the town of Ürgüp (Figure 10). Food preservation, particularly citrus fruits brought from the Mediterranean Region of Turkey, and a mushroom plantation are more important among the above-mentioned present uses. Such storage is common in the vicinity of Ürgüp, Ortahisas and Avanos. The storage is built up as one floor, supported by prismatic pillars in 1 m × 1 m dimensions (Figure 11) and/or cut-and-cover structures.

Implications of Rock Structures in Rock Mechanics and Rock Engineering

In this century cities are growing faster than in the past. This urbanization indicates that the population density in cities will reach high numbers, thus the use of underground openings will become more important in the near future.

The environmental and anthropological factors of the Cappadocia Region have been the main reasons for extensive subsurface settlement and multi-purpose use in the past and present. These factors, as also stated by Erguvanli and Yüzer (1977), can be categorized into six groups:

a. Severe daily and seasonal changes of temperature in the region,
b. Thermal isolation properties of the rock units covering the area,
c. Self-supporting behavior and construction opportunities in these rocks,
d. Easily carved, particularly soft pumice tuffs,
e. Provide hiding places and camouflage to provide a defensive advantage and safety against enemy attacks,
f. Superior resistance and protection against natural disasters due to earthquake and/or volcanic eruptions.

Today, the above-mentioned factors, except item e., increase the importance of the tuffs in the Cappadocia Region from the view point of rock engi-
neering. Preservation of the existing openings, stability and the future use of new underground openings in the tuffs are the most important rock mechanics aspects.

During a 1997 site visit to some of the large underground cities located in the Cappadocia Region, by the authors of this article observed some evidence of instability problems, especially near the entrances due to weathering and the structural discontinuities of the tuffs. It is also noted that fractures and slabbing occurred in roofs and pillars, which are indicators of high stresses. It is clear that underground openings suffer from the fundamental problems of material decay due to weathering and long-term loading.

Deformations of underground openings may continue for a considerable period of time. One of the reasons for this time-dependent behavior may result from swelling minerals in rock. Nevertheless, the deformation due to swelling probably accounts for a small fraction of the total deformation (Aydan et al. 1993). The second reason, which is probably the most significant one, is the degradation of deformability and strength characteristics of rocks with time (Bieniawski, 1970; Sakurai, 1978; Akagi et al., 1984; Aydan et al., 1993). Furthermore, cyclic thermal loading and seismic loading could be another cause of rock degradation and instability in the long term. Due to the above-mentioned possible rock engineering problems based on the observations in some of the underground cities visited in the Cappadocia area, the investigation of the long-term performance of these openings to develop appropriate remedial measures for preservation can be considered an important research topic. It should be also noted that these structures have a wide range of construction age, which may provide very valuable case histories for predicting the long-term performance of man-made underground structures for important rock engineering projects in modern times.

The other problem which threatens the cliff settlement areas and the natural monuments in the Cappadocia Region is the rockfalls and rock slides from the castles and from the high rock cliffs. Sets of discontinuities as well as yielding due to high stress concentrations create huge rock blocks which exhibit sliding or toppling tendencies (Figure 12). Assessment of the occurrence of such blocks and their sliding mechanisms with investigation of necessary remedial measures for environmental protection seems to be another scientific study which can be performed in the area.

In conclusion, it should be noted that the underground cities in the Cappadocia Region are still at an early stage of investigation and preliminary analysis of data, because of the lack of detailed rock engineering studies. For this reason, it would be desirable to conduct scientific expeditions, possibly international in scope, in order to appreciate the scientific value of the old and recent underground openings and suggest recommendations for their long-term performance and conservation, and for future use of the new underground openings in similar rock environments.

Acknowledgements

This work is a part of an ongoing joint research program focusing environmental study on Underground City Derinkuyu, Central Turkey (MONBUS) Res. Project No: 09044154). It was initiated by Prof. Dr. M. Tsujimoto (formerly with Nagoya University). Financial support was provided by the Japanese Ministry of Education. The authors would like to thank Prof. M. Tsujimoto for initiating this research project and gratefully acknowledge the support provided by the Japanese Ministry of Education (MONBUS). The authors also would like to express their indebted appreciation to Professor Shunsuke Sakurai, the President of the ISRM, for his kind invitation to contribute this article with information on an up-to-date topic.

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Association of Sanitary Engineering (ABES), and the Brazilian Association of Underground Waters (ABAS).

Prof. Vargas has delivered 14 courses, general reports, keynote lectures, panel interventions, seminars, and short courses, in Brazil, Canada, Costa Rica, France, and Portugal; is the author or co-author of 92 papers; and has supervised 13 Ph.D. and 36 M.Sc. theses.


--- ISRM Board Candidates, continued from page 9 ---

--- Man-Made Structures, continued from page 72 ---

struction, La Rochelle-France, 167–177.


--- News from the Secretariat ---

ISRM Annual Meetings

The Board, Council and Commissions will meet at the Palais des Congres in Paris on August 23 and 26 (Board), and 24 (Council and Commissions), in conjunction with the 9th ISRM International Congress.

Besides other matters of interest for the Rock Mechanics community, new Vice-Presidents will be elected for the term 1999-2003.


The ISRM is going to publish a fully-revised Directory immediately after the Paris Congress, when the election of the new Vice-Presidents will take place, in order to have their names and addresses duly added.

The Secretariat is thus urging all the National Groups that have not yet complied with the Secretariat’s request of supplying it with updated List of Members, to do it as soon as possible and to strictly follow the recommendations given for the Format (EXCEL) in the letters sent to them.

Attention is called to the great importance of including fax and e-mail numbers, which are presently the most used and effective means of communication.


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National Group News

IAEG BOARD

The International Association of Engineering Geology has elected a new Board for the term 1999-2003. The new President is Prof. Wang Sijing from China.

Prof. Paul Marinos has sent a kind letter recognizing the excellent collaboration that has been established between IAEG and ISRM. The ISRM Secretariat, sure that it interprets the feeling of all the members, wishes a great success to the new Board and thanks Prof. Paul Marinos for his role on the strengthening of links between our societies.

Belgium

The Board of the Belgian National Group has now the following members: President: Dr. A. Vervoort; Vice-Presidents: Dr. R. Charlier and Dr. G. Simon; Secretary: Dr. J. P. Tshibangu Karsh.

Belgian National Group Activities in 1998

The first priority for the Belgian National Group (BVRM-GBMR) is to bring together, on a regular basis, professionals active in rock mechanics. To this effect, several activities are organized each year. To stimulate interdisciplinary contacts, these activities are also open to non-members. Most of the people from outside the rock mechanics fraternity who are interested in the activities of the BVRM-GBMR are working in the domains of soil mechanics, engineering geology or earth sciences in general. In 1998, two visits and two presentations were organized.

The first activity in 1998 was a visit to the shaft sinking at the nuclear research site in Mol (29 January 1998). In the early 1980s an underground laboratory was excavated at a depth of about 225 m inside a thick soft clay layer. This laboratory was served by a single vertical shaft only. The mining inspectorate would not authorize an extension to the underground facilities unless a second exit was provided. The laboratory forms part of an international cooperation (in the past, the HADES project; currently the PRACLAY project). One of the future research topics is to investigate in situ the effect of a heat source on the clay behavior.

The new shaft has an inside diameter of 3 meters. To excavate the shaft through weak and water containing material, the freezing technique was applied. During the visit, specific problems were discussed related to the long-term stability of the shaft at the contact zone sand-clay, the restriction to make the shaft watertight and the monitoring program to measure deformation and stresses. In Figure 1 a view into the shaft is presented.

In May 1998, T. You from Géostock, France was invited to give a presentation on the design and the geotechnical monitoring of an underground LPG storage in Lavéra in the south of France (area of Marseille). In the 1970s, a first underground storage cav-

Figure 1: View into the new shaft at the nuclear research site in Mol

er was created for propane, while in the 1980s a second was excavated for butane. More to the north of these two, a third storage facility has been excavated in the nineties for LPG. The latter was discussed in detail. The total storage capacity is 800,000 m³, divided over four caverns with a height of 14 m and a width of 10 m. The host rock is limestone. To seal the caverns against leakage a "water curtain" is created above and around the caverns, in which the water pressure is raised, so that a continuous water flow is initiated towards the caverns.

On 22 September 1998, a session on rock mass behavior in old room and pillar workings was organized. A presentation was given on the design criteria developed in South Africa for coal room and pillar workings and a visit was organized to old underground limestone quarries. During the visit pillars in different stages of failure were observed. In Figure 2, a view on a pillar with a fractured and failed skin is presented. To determine the stability of the pillars and the entire area, an extensive monitoring campaign is undertaken.

Figure 2: View of a limestone pillar in old workings

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The last activity of 1998 was organized on 13 October 1998. Six presentations were given during half a day on the deformation of limestone under tectonic and geological conditions. It was jointly organized by the Belgian national group and an ad-hoc consortium of Belgian and French universities, that have been working together on this topic over the last 10 years. It was worthwhile to bring together researchers with a background of rock mechanics in civil or mining engineering and researchers who are oriented towards geology. Both are looking at similar problems, although sometimes at another scale. The research focuses on the differences in limestone properties that are observed around a fault in comparison to the rest of the limestone formation. The deformation of the matrix material, the change in petrophysical characteristics, geochemical and physico-chemical changes, and the reconstruction of the geo-environment were some of the topics presented.

**China**

The Chinese National Group has elected the following board for the term of the next four years: President, Prof. Wang Sijing; and Secretary-General, Prof. Yang Zhifa.

**France**

Scientific Golden Jubilee of Pierre Habib

On May 19, 1998 seven French societies had a joint meeting in the Senate Palace in Paris to celebrate the scientific jubilee of Pierre Habib. Opening the colloquium, Academician Jean Salençon recalled the curriculum of Pierre Habib under a four word title Ingénieur homme de science. Graduated from Ecole Polytechnique, he worked in the Laboratory for Buildings and Public Works, submitting a thesis on soil mechanics, and performing the first Young’s modulus in-situ measurements in rock mechanics. Back in Ecole Polytechnique, he created, with the late Professor Jean Mandel the Laboratory for Solid Mechanics, becoming its Head in 1973. Now succeeded by Pierre Berest, he is Chairman of the Board of G3S (Group for Studies on Underground Storage Structures).

Pierre Habib has chaired three French societies, Rock mechanics, Soil mechanics, and Rheology, and is an active member of four more, Engineering geology, Large dams, Tunneling, and Paraseismic engineering. In 1967 he launched CEMRE, the French national group on Rock Mechanics. At the international level, he managed the ISSMFE conference in Paris in 1961, and was President of ISRM from 1974 to 1979.

Among other points of interest, he focused on grain behavior, localization of rupture along surfaces, and scale effects. He contributed about 235 papers and some books, the latest in 1997, Génie Géotechnique (Geotechnical Engineering).

After the Jean Salençon speech, Pierre Habib addressed two short topics, one about the cohesion brought to sand by a low water content, a theory for sand pâtes and castles built by children on beaches, the other one about settlements above shallow tunnels. Settlements arise from five mechanisms, four of which can be tackled at 2D, the fifth one involving extrusion of the face. At a time of so many sophisticated numerical models, such rough demonstrations were refreshing.

Ten more presentations were given by first rank engineers and researchers, Andrew Schofield on the Mohr-Coulomb error, Marc Panet and Pietro Lunardi on tunneling, Pierre Berest and Mehdi Ghoreychi on storage caverns, Jean Louis Auriult and Minh Phong Luong on heat transfer and thermography, Jean Francois Corte and Alain Pecker on centrifuge studies on failure mechanisms, Francois Schlosser and Patrick de Buhan on soil reinforcement and compatibility of deformations, and Pierre Londe on assessment of foundation safety (the classical safety factor is meaningless and dangerous, instead the relative weight of parameters leads to better safety criteria).

All twelve lectures are published by Ecole Polytechnique, in 150 pages, under the title Mécanique et Géotechnique.

**News from the French Committee for Rock Mechanics**

The traditional autumn meeting of the French Committee for Rock Mechanics was held in Grenoble on 29 October 1998. This meeting, which was organized by the Soil Solid Structures (3S) laboratory and the Firigm, focused on the theme of Geomechanics for Alpine Tunnels. More than one hundred persons participated in this meeting.

Many tunnel projects for high speed rail links across the Alps are currently under study to improve communication between Switzerland, Austria, France and Italy. The aim of this meeting was to create a forum for an exchange of views and information for contractors, consulting engineers and researchers from laboratories interested in the subject.

The day-long meeting was divided into two parts, be inning with short visits to laboratories with special experimental devices such as a full 3D direct shear box and a high
capacity press with triaxial cells.

The conference proper began with a welcoming address by Prof. Pierre Berest, president of the French Committee for Rock Mechanics, and by Frédéric Pellet representing the organizing committee.

An introductory lecture was given by Marc Panet on the history and future prospects of tunneling through the Alps.

The scientific presentation program was divided into two parts. Professor Pierre Antoine from the University of Grenoble and Pierre Emilien from the University of Montpellier chaired the first part of the morning, which was devoted to the geological aspects and the initial conditions of the Alpine chain.

During this session, Patrick Lacombe from Alpetunnel described the geological aspects relating to a project for a high-speed rail link between Lyons and Turin under the Mont d'Ambin. He described the new geological and hydrogeological survey program involving a very long (about 3,000 metres) horizontal borehole requiring the use of petroleum exploration techniques. He also gave details of the measurement techniques used such as hydraulic testing of pre-existing fractures.

Then, Felix Amberg, consulting engineer from Switzerland, gave a detailed presentation of the Gotthard tunnel. He stressed the structural aspects of the geology, including recognition of heavily fractured areas where high pressure water was to be expected. This sector, known as Mora, will not give rise to any problem as its existence had been foreseen.

Dr. Max John from IFI consulting engineers in Innsbruck, Austria, presented the new Brenner railway. This is an international main line involving three countries which forms a link between the high-speed railway lines of Germany in the north and Italy in the south. The scheme for passenger trains with a design speed of 250 km/h includes a continuous 55 km-long base tunnel from Innsbruck to Francofeste, a twin-track tunnel with a distance between track centre lines of 4.7 m and a clear area of 83 m².

Dr. Denis Fahre from Urtigia described the measurements of the initial stress field in the Mont d'Ambin region by means of hydraulic testing of pre-existing fractures. These measurements were compared with the results of a 3D numerical simulation.

The afternoon session was devoted to stability problems and numerical modeling. Professor Marc Boulot from Laboratoire 3S chaired this session.

Prof. Giovanni Barla from Politecnico di Torino gave a lecture on continuum and discontinuum modeling for design analysis of tunnels. His lecture was focused on tunnels design by advanced computing methods. These methods are a powerful means of analyzing rock structures, in terms of the assessment of intrinsic tunnel stability and tunnel performance. The equivalent continuum and discontinuum approaches in the design analysis of deep tunnels were discussed. An example of a numerical modeling study relating to a TBM tunneling project in quartzitic micaceous in overstressed conditions was then described in detail.

Professor Descoedres from the Swiss Federal Institute of Technology in Lausanne presented a new approach for characterizing the mechanical behavior of cataclastic rocks. This lecture was illustrated by experience gained on the famous Cleuson Dixence hydroelectric project.

Dr. Axel-Pierre Bois from Simec (France) and Daniel Collomb from Bonnard & Gardel consulting engineers (Switzerland) presented a design method for the excavation of tunnels crossing faulted zones. They presented an application of this method for the Loetschberg tunnel in Switzerland.

The presentation given by François Laigle from EDF-CNEN in France focused on the modeling computation carried out for the CERN-HIC project. The complex geometry of caverns requires a full 3D approach.

The final presentation was made by Pergorgio Grassi from Geodata in Turin who presented the use of numerical modeling for the design of large underground excavations, with experience of the Borzoli cavern in NW Italy being given as an example.

The last part of the meeting was give over to a round-table discussion on the following question: “What is the future of the large Alpine tunnel?” Professor Descoedres explained the situation in Switzerland. He said that the two major tunnels (Loetschberg and Gotthard) will be constructed in the future, subject to budget approval being given.

The French situation was described by Marc Panet and Michel Marcou representing the inter-governmental group for studies. No decision has been made to go ahead with the construction.

The day ended at 6 p.m. with a cocktail party.

For more information, contact Frédéric Pellet—Laboratoire 3S. Tel: (33) 476 82 70 23; email: Frederic.Pellet@hmg.inpg.fr, http://www.3S.hmg.inpg.fr

Italy

Beginning with the January 1998 issue, the Italian Geotechnical Journal (IGI) entered a new period, after more than thirty years of life. The board of AIGI (the Italian Geotechnical Society) felt that it was the right time for the Journal to be known and well recognized internationally.

“The main purpose is to contribute to a more extended distribution in Europe and worldwide of the results of both the research activities and technological advances in the field of geotechnical
engineering in Italy. As this is achieved, the Italian reader will be offered a wider view of geotechnical engineering, as more technical papers from abroad will be made available than in the past.

Professor Roberto Nova from the Politecnico di Milano is the new Editor of the Journal. Dr. Eng. Ulrich Hegg and Piero Semenelli will cooperate with him as Vice Editors. There will be an Advisory Board with members from abroad and Italy. This board will help in reviewing contributions and in keeping with the same high quality of national journals which are distributed worldwide.

In addition to the usual papers, some technical notes will also be published. These will be briefs dealing with applications, descriptions of engineering works, news from work sites and recent technological developments. It is hoped that each issue will include, together with four papers, two such briefs. As described in the "Notes for Contributors," papers and technical notes can be written in either English or Italian.

With this issue Professor Carlo Viggiani leaves the editorship of the Journal, and joins the Editorial Board. The warmest thanks are due to him for his continuous efforts as formal editor of the Journal. The present editorial change was indeed favored by him and is now taking place under the auspices of the President and Board. We rely on the cooperation of those who care for the Italian Geotechnical Society and the Italian Geotechnical Journal."

— Roberto Nova, Editor of IGJ and Giovanni Barla, President of AGI

The Italian Geotechnical Journal, published by the Associazione Geotecnica Italiana

Zambia

The ZARMs Executive Committee elected for the term of office 1998-1999 has the following members: Chairman, Mr. W. Claey; Vice-Chairman, Mr. W. Moono; Secretary/Treasurer, Mr. M. Wondrad; Excom Member, Mr. M. Broome; and Excom Member, Mr. C. M. Mkandawire.

Coming Events

A clash of dates can hurt your meeting. Please, carefully check the pre-booked dates in the ISRM News Journal or in this Events Calendar, and notify the ISRM News Journal Editor of the ISRM Secretariat, as soon as you know your conference dates, theme, and venue. Apply early to the ISRM Secretariat for ISRM sponsorship, to help publicize your meeting. National, Regional, and International symposia, and the 4-yearly ISRM Congress, impose different requirements for ISRM recognition, languages, etc. Details are given in "bylaws.htm" ISRM By-Laws 4 and 5, reprinted in the annual ISRM Directory and available from the ISRM Home Page.


1999 November 8-12, Bangkok THAILAND — AIT 40th Anniversary Civil and Environmental Engineering Conference. Themes include Geotechnical and Geo-Environmental Engineering. Dr. A.S. Balamurugan, School of Civil Engineering, All. PO. Box, 1, Bangkok, 12111, Kamphaeng Phet, THAILAND. TEL: 66 2 516-2126; FAX: 66 2 516-2126.


1999 December 1-3, SINGAPORE — Field Measurements in Geomechanics '99 (FMGM-99), an international symposium, organized by the National Univ. of Singapore, Ass. Prof. Barry Tan Siew Ann, Dept. of Civil Engineering, National Univ. of Singapore, Singapore, SINGAPORE. TEL: 65/7422728 or 7791635(FAX): E-MAIL: cuvers@nus.edu.sg.

natural resources management). KS: Abstracts (1 p.; 3 copies); 1999 1115; Papers - 2000 02 15; Final papers: 2000 05 30. An exhibition is foreseen. On August 15-18, several 1-day short courses on Environmental drilling and sampling, Environmental instrumentation, and Environmental monitoring will be held. (To: percent before 2000 06 15.) 5th International Symposium on Environmental Geotechnology and Sustainable Development, Deputado de Engenheiros de Pesquisadores, Paula de Espanha da Uni. Federal de Minas Gerais, Av. Do Doutor Cao, S. 220, 31010-600 Belo Horizonte, MG, BRAZIL. TLP 55/31/2328/1724 or 232779 (Fax); EM: cossafge@ufmg.br. 2000 12-14 September, Prague CZECH REPUBLIC — Fifth International Symposium and Exhibition on Environmental Contamination in Central and Eastern Europe, sponsored by Florida State University and the US Department of Energy. Workshops, special topic sessions, concurrent technical sessions, International environmental symposium and exhibition. Check the website at www.papers2000.hse.edu or contact: Prague, Florida State University, 205 East Paul Dirac Drive, Tallahassee, FL 32310-3900 USA 850-645-2721 Fax 850-574-6701. 2000 20-21 September, Singapore — 3rd International Conference on Ground Improvement Techniques, Call for papers: Please contact Dr. John S Y Tan, CF-Premier Pte. Ltd. 150 Orchard Road 807-1 Orchard Plaza, Singapore 238841 Tel: (65) 738-2922 Fax: (65) 326-3250 Email: cipremier@sgnet.com.sg 2000, October 10-13 Hanover GERMANY — Engineering Geology and Environmental Planning, a symposium, organized by the IAGS GERMANY and the RIGB, sponsored by the IAGS. 2000 October 15-16, Bologna ITALY — EuroGeo 2000, 2nd European Geosciences Conference & Exhibition, organized by the Italian Chapter of the IGS (AGI-IGS), and sponsored by the IGS. In conjunction with the International Fair for Building Technologies and Construction (IMC 2000) (October 15-18). On October 15, technical visits shall take place. The technical program as well as the accompanying persons is being prepared. Post-conference tours are foreseen. Secretariat of the EuroGeo 2000, AGI-IGS, Piazza Bologna, 22, 1-40122 Roma, ITALY. TLP: 39/6/44209724 (Fax). 2000 October, Singapore SINGAPORE — 3rd International Conference on Ground Improvement Techniques, & John S Y Tan, Conference Director, CF-Premier Pte Ltd, 150 Orchard Road 807-1 Orchard Plaza, Singapore 238841, SINGAPORE. TLP: 65/3733292 or 235550 (Fax); EM: cipremier@singnet.com.sg. 2000, Belo Horizonte MG BRAZIL — 5th International Symposium on Environmental Geotechnology and Sustainable Development. 2000 November 19, Melbourne Vic. AUSTRALIA — International Symposium on Scour of Foundations, sponsored by the ISSMGE TC 33 (on Scour of Foundations). Topics: The scour problem, Predicting scour depths, Scour countermeasures, Case histories. J. Louis Baladi, Dept of CIVIL Engineering, Texas A&M Uni., College Station, TX 77843-3100, USA. TLP: 1-409/8435795 or 8436554 (Fax); EM: JMB4@tamu.ee. 2000 November 19-24, Melbourne Vic. AUSTRALIA — GeoEng 2000, an international congress, sponsored by the ISSMGE, the ISRM, and the IAGS as well as by the International Geotechnical Society (IGS), the IAEA, and the International Association of Hydrologists (IAH). Themes: Geotechnical earthquake engineering; Underground works; Stability of natural and excavated slopes; Environmental geotechnics; Ground improvement and ground support. TS: Call for abstracts; 1999 February; Abstracts: 1999101; Acceptance: 19990701; Papers: 19991101. Return of papers: 20000301; Final papers: 20000701. Poster sessions are foreseen. The GeoFair 2000, a wide and varied exhibition of equipment, software, instruments, and machines for geotechnical and geological education, investigation, design, and construction, will be held. Site visits are being prepared. On November 19, a pre-conference tour shall take place. The 2000 ISSMGE Board, Council, and Commission Meetings will be held in conjunction with this Conference. Secretariat: GeoEng2000, c/o IERGS. P/Shak Punjab 15050, INDIA; TLP: 091/4062029 or 4062093 (Fax); EM: geocentric@hcl.com.au. Mr. Max E. Ehrman, Chairperson, GeoEng 2000 Organizing Committee, c/o Geotek Associates P/L, 39 Burwood Road, Hawthorn, Vic. 3122, AUSTRALIA. TLP: 03/9812981 or 9814290 (Fax); EM: maxe@geotek.com.au. 2000, Rio de Janeiro RJ BRAZIL — 5th International Symposium on Environmental Geotechnology 2000, UK — International Young Geotechnical Engineering Conference, sponsored by the ISSNME, and organized by the RGS (ISRM NG UK). 2000, Belo Horizonte MG BRAZIL — 5th International Symposium on Environmental Geotechnology and Sustainable Development. 2000, GERMANY — Pore Pressure, Scale Effect, and the Deformation of Rocks, the 2000 Euroconfer, supported by the European Commission. 2001 July-August, Ekaterinburg RUSSIA — Engineering Geotechnical Problems of Urbanized Territories, an international conference, sponsored by the IAEG. 2001 August 27-31, Istanbul TURKEY — XVth ISSMGE International Conference on Soil Mechanics & Geotechnical Engineering (XV ICSMGE), Prof. Engin Engin, Chairman. XV ICSMGE Organizing Committee; Fac. of Civil Engineering, Istanbul Technical University, TR-80626 Ayaazaga, Istanbul, TURKEY. TLP: 90/212/285747 or 285658 (Fax). 2001, August, Helsinki FINLAND — Aggregate 2001, Environment and Economy, sponsored by the IAEG. 2001 September 7-8, Lausanne SWITZERLAND — Modelling in Engineering Geology, a symposium, organized by the Swiss National Committee of the IAEG, and the Lab. of Geology of the Federal Institute of Technology (EPFL), Lausanne, and sponsored by the IAEG. Topics: I. Theoretical background on coupled processes (flow, thermal, rheo- mechanical, mechanical); II. 3D geological modelling, including engineering structures; III. Treatment of uncertainties in models; IV. Applied modelling of flow in geological media, i.e. groundwater, interacting with civil engineering works; V. Applied modelling of contaminant migration underground. Lab. of Geol. EPFL, CH-1015 Lausanne, TLP 41/244962505, 41/24496200 (Fax), or 605289 (Fax). EM: bacri@geolgene.epfl.ch. 2001, Milan ITALY — 27th ITA Annual Meeting, to be held in conjunction with the World Tunnel Congress 2001. E1: Italian; French; only at the General Assembly. Mr Claude Berenger, ITA/ITAIS Secre- tariat, 109, Av. Salvador Allende, F-69000 Brie, FRANCE. TLP: 33/78/20495 or 26049 (Fax); TEL: 39/7/70 01 ELIOT. 2001, Milan ITALY — World Tunnel Congress 2001, organized jointly by the ITA and SWITZERLAND, sponsored by the ITA. Theme: Progress in Tunnelling after 2000. E1: English, French. — Guest Editorial, continued from page 10 • United States: an article on the Yucca Mountain site, performance assessment, and thermal testing. We hope that these contributions will provide some indication of the complexity of the subject and how work is progressing in these four countries. To what the appetite, there are also many indications of subjects that need further development. Four further contributions are included. The subjects are the Block Punch Index, dilatancy in fragmented rocks, pulse-induced fault slip, and man-made structures in Turkey. Although these further contributions were not submitted specifically for this special issue, in addition to their own contexts they are all potentially relevant to rock mechanics research for radioactive waste disposal. — John A. Hudson, former ISRM Vice President-at-Large jainrockeng.co.uk.
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