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**New Horizons in Rock Mechanics  
Developments and Applications**

Editors: *Abbas Majdi & Abdolhadl Ghazvinian*



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# CONTENTS

## Preface

## Organization

<b>KEYNOTE LECTURES</b> ( <i>Alphabetical order</i> )	<b>1</b>
New Directions of Rock Mechanics and Rock Engineering: Geomechanics and Geoengineering <i>Ö. Aydan</i>	3
Important Aspects of Petroleum Reservoir and Crustal Permeability and Strength at Several Kilometers Depth <i>N. R. Barton</i>	23
Reflections on New Horizons in Rock Mechanics Design: Theory, Education and Practice <i>Z. T. Bieniawski</i>	37
Slurry Fracture Injection of Petroleum and Municipal Sanitation Wastes <i>M.S. Bruno</i>	51
Geotechnical Behavior of Sedimentary Argillaceous Rocks <i>S. A. B. da Fontoura</i>	59
Recent Development and Applications of Intelligent Rock Mechanics <i>X. T. Feng</i>	73
Microwave Assisted Drilling and its Influence on Rock Breakage: A Review <i>F. Hassani, P. Radziszewski and J. Ouellet</i>	87
The Future for Rock Mechanics and the ISRM <i>J. A. Hudson</i>	105

Recent Developments on Rock Joint Roughness, and Rock Joint and Rock Mass Strength and Deformability	119
<i>P. H. S. W. Kulatilake</i>	
Prediction of Fault Zones Ahead of Tunnel Face Using 3-D Displacement Monitoring	133
<i>C. I. Lee, Y. J. Park and K. Y. Kim</i>	
Joint Factor Concept in Solving Rock Engineering Problems	147
<i>T. Ramamurthy</i>	
Observational Method in Rock Engineering	157
<i>H. Stille and M. Holmberg</i>	
Rock Failure, Wave Propagation and Tunnel Stability under Dynamic Loads	167
<i>J. Zhao, Y. X. Zhou and G. W. Ma</i>	
<b>Rock Characterization and Site Investigation</b>	<b>183</b>
Investigation of Swelling Potential of Marls (Case Study)	185
<i>R. Ajalloeian, M. Jafarkhalou, R. Yazdankhah and M. Dehghanian</i>	
Effect of Anisotropy and Confining Stress on Rock Mass Deformation Modulus at Javeh Dam Site (Iran)	191
<i>H. Aliasghari and S. Hashemi</i>	
Mixed Mode I/II Crack Growth Assessment for Dehbid Marble	197
<i>M. R. M. Aliha, M. R. Ayatollahi and B. Kharazi</i>	
Determination of Shear Fracture Resistance of Rocks	205
<i>M. R. Ayatollahi, M. R. M. Aliha and R. Pakzad</i>	
The Inference of Mechanical Properties of Rocks from Penetration Tests	213
<i>Ö. Aydan , S. Watanabe and N. Tokashiki</i>	
Confining Pressure Unloading with Axial Force Fixed Tests on Marble and its Time-Effect Property Study	221
<i>B. Wang, J. B. Zhu, A. Q. Wu and Z. M. Xiong</i>	

A Simple Practical Method to Determine the Biot Coefficient for Hydromechanical Analyses	227
<i>S. Bodaghabadi and S. J. Moosavi</i>	
Laboratory Study of Permeability in low Permeability Medium	231
<i>W. Z. Chen, J. P. Yang, G. J. Wu, X. J. Tan, S. P. Jia, Y. H. Dai, H. D. Yu</i>	
Investigation on Deformability Modulus of Asmary Formation Rock Mass, by Dilatometer Tests	239
<i>A. Dadi Givshad, H. Memarian and F. Rezaei</i>	
Swelling of Weak Rocks, Effective Parameters and Controlling methods	247
<i>R. Doostmohammadi and M. Moosavi</i>	
Interpretation of Plate Loading Test Results on major Projects	255
<i>L. Faramarzi and H. Fatahi</i>	
Determination of In-Situ Shear Strength of Rock-Concrete Contact Surface at the Abutments of a Concrete Dam	261
<i>M. Gharouni-Nik and S. Hashemi</i>	
Determination of the Failure Mode in Rock Bridge Using Neural Network Tools	267
<i>A. H. Ghazvinian, S. Setayeshi, V. Sarfarazi and S. A. Moosavi</i>	
Effect of the Back Rake Angle and Groove Geometry in Rock Cutting	275
<i>M. Ghoshouni and T. Richard</i>	
Rock Mechanics Tests for the Baixo Sabor Main Dam	285
<i>N. F. Grossmann and J. P. Neves</i>	
Comparing Different In-Situ Methods for Measuring Deformability Characteristics of Rock Masses in Roudbar Lorestan (Iran)	293
<i>S. Hashemi and M. Gharouni-Nik</i>	
Capability of a Rational Function as Rock Failure Criterion Considering Brittle-Ductile Transition	299
<i>M. Hashemi, A. H. Ghazvinian and A. Taghichian</i>	

S-wave Velocity Structure Estimated by the Microtremor Survey Method	307
<i>H. Hayashi, S. Ling and T. Kondo</i>	
Evaluation of Stress Mode in the Hydropower Station of Bakhtyari Dam, Iran	315
<i>S. Hosseinizadeh, H. Jalalifar, S. Karimi-Nasab, F. Rafia and A. M. Radman</i>	
Shear Behavior of Rock Pile Material with Oversize Particles	321
<i>H. Hosseinpour and A. Fakhimi</i>	
The Analysis on Dynamic Rheological Parameters and Influencing Factors of Soft Rock Soil	331
<i>H. Hua</i>	
Creep Characteristics of Tuff in the Vicinity of Zelve Antique Settlement in Cappadocia Region of Turkey	337
<i>T. Ito, Ö. Aydan, R. Ulusay and Ö. Kasmer</i>	
The Effect of Water Saturation on the Strength of Marbles	345
<i>S. Kahraman, O. Gunaydin and M. Fener</i>	
Acoustic Emission Signal Processing Based on Wavelet Analysis	353
<i>Y. M. Kang</i>	
Comparing the Results of Acoustic Emission Monitoring in Brazilian and Uniaxial Compression Tests	357
<i>M. Keshavarz, F. L. Pellet, K. Amini Hosseni and C. Rousseau</i>	
Derivation of Input Parameters Using for Analysis of Ground Subsidence Due to Very Shallow Longwall Mining in Tabas Coal Field of Iran	365
<i>E. Khatibi, K. Shahriar and B. Ferdowsi</i>	
Proposal of Ground Evaluation Method by Polyhedron Transparent Borehole Casing	371
<i>E. Kondo, M. Nakamura and M. Yagisawa</i>	
Application of AE for Determining In Situ Stress at UG Mine	379
<i>S. Kramadibrata, R. K. Wattimena, I. Kamil, M. A. Rai and E. Widijanto</i>	

Lithology Prediction of Fahliyan Formation Based on Rock Physics Studies in Two Wells of Two Neighbor Oil Fields, Southwest of Iran	387
<i>B. Mahbaz and H. Memarian</i>	
Development of the Strain Rockburst Test in China	393
<i>M. C. He, J. L. Miao, J. L. Feng and D. J. Li</i>	
Influence of Intact Rock Properties on TBM Penetration Rate in Karaj-Tehran Water Conveyance Tunnel	401
<i>M. Mansouri, S. R. Torabi, O. Forough and K. Goshtasbi</i>	
The Impression Creep Test as a New Method for Fast Measurement of Creep in Soft Rocks	407
<i>M. Moosavi, M. Jafari and F. S. Rassouli</i>	
Analysis of the Shear Strength of Rock and Concrete Joints Using Results of the Constant Normal Load Shear Test	415
<i>Z. A. Moradian, G. Ballivy, C. Gravel and K. Saleh</i>	
Role of the Texture Characteristics on the Strength Properties of Crystalline Rocks	423
<i>H. Nofaresti and K. S. Rao</i>	
Simultaneous Relationship Between Mechanical Properties and Index Tests of Intact Rocks	431
<i>R. Noorani and A. H. Ghazvinian</i>	
Effect of Joint Stiffness Values on Stresses of Jointed Rocks	437
<i>A. Noorzad and M. Aminpoor</i>	
Problems from Weak Rock Behavior in Operation of a Cut-and-fill Pond of a Pumped Storage Scheme	445
<i>N. Phienwej</i>	
Effect of Chlorides Solution on the Electrochemical and Physico-Chemical Properties of Western Venezuelan Shale	451
<i>C. Rabe, F. V. Artola and J. A. O. Cherrez</i>	



Laboratory Characterization of Norwegian North Sea Shale	459
<i>C. Rabe and J. A. O. Cherrez</i>	
Experimental Study on Crack Development in Rock Mass due to Strike-Slip Faulting	467
<i>A. Sato, K. Tani and M. Sawada</i>	
Geomechanical Properties along the First 10+400 KM of the Qomroud Long Tunnel Project in Iran	475
<i>A. Shafiei and M. B. Dusseault</i>	
Rock Mass Characterization at Kangir Dam Site in Iran	483
<i>A. Shafiei and M. B. Dusseault</i>	
Rock Mass Characterization along Lot No. 6 of Dez-Qomroud Tunnel project in iran	499
<i>A. Shafiei, M. B. Dusseault, H. Rahdar and S. Mesgarzadeh</i>	
Modulus Measurement with Flexible Dilatometer: Case Study of Siah Bishe Power plant	511
<i>M. R. Shahverdiloo</i>	
Proposal of a New Multiple-Step Loading Triaxial Compression Testing Method	517
<i>A. Taheri and K. Tani</i>	
Interpretation of Hydraulic Fracturing Pressure-Time Records to Evaluate In-Situ Stress Measurement Parameters	525
<i>N. Valinezhad, H. Ghasemzadeh and B. Pahlavan</i>	
Estimating the Poisson's Rate Value of the Rock Masses	533
<i>B. Vásárhelyi</i>	
Estimating Rock Mass Long-Term Strength Using In-Situ Measurement and Testing Results	537
<i>R. K. Wattimena, M. A. Rai, S. Kramadibrata, I. Arif and B. Dwinagara</i>	
A New Empirical Criterion for Prediction of the Shear Strength of Natural Infilled Rock Joints Under Constant Normal Load (CNL) Conditions	543
<i>M. Zare, R. Kakaie, S. R. Torabi, S. M. E. Jalali</i>	

<b>Ground Improvement and Rock Slope Stability</b>	<b>551</b>
Rock Slopes: Pole Counting or All-Wedge Analysis?	553
<i>A. J. Gonzalez-Garcia</i>	
Sarcheshmeh Pseudowedge Failure, A New Challenge in Expanding Western Sector of Sarcheshmeh Copper Mine, Iran	559
<i>N. Babanouri and S. Karimi-Nasab</i>	
Stress Model Generated by Freezing-Thawing Process in Rock Cracks	567
<i>M. Bost and A. Pouya</i>	
Hazard Evaluation of Rock-Fall Induced by Quarry-Blasting	575
<i>M. Coli, G. Pini and C. Cantini</i>	
Contact Detection Considering Force Direction in Discontinues Deformation Analysis	583
<i>H. Ghasemzadeh, M. A. Ramezanpoor</i>	
Influence of Load Direction on Parallel Fractured Rock Stress and Strain	591
<i>H. Ghasemzadeh, M. A. Ramezanpoor</i>	
Bishop's Simplified Method and Particle Swarm Optimization for Location the Critical Failure Surface in Rock Slope Stability Analysis	597
<i>E. Javadzadeh and R. Javadzadeh</i>	
Technology and application of reinforcing coal mine tunnel with carbon fibre reinforced plastic sheets	605
<i>X. Y. Lu and Q. Y. Ma</i>	
Effects of Geostructural Weaknesses on Flexural Toppling Failure Based on Principles of Solid and Fracture Mechanics	611
<i>A. Majdi and M. Amini</i>	
Hydraulic Investigations and Grouting Improvements Applied to Dam Sites in Brazil	619
<i>E. F. Quadros and R. A. Abrahão</i>	

Evaluation of Permeability and Groutability in Rock Mass of Ostur Dam Site	627
<i>S. M. Sadeghiyeh, M. Hashemi and R. Ajalloeian</i>	
Slope Failure Analysis in Road Cut Slope by Numerical Method	635
<i>K. Sarkar and T. N. Singh</i>	
Relationship Between Bearing Performance and Modulus in the Composite Foundation Improved by Flexible Piles	643
<i>F. Y. Tan, L. Zhao and X. Z. Wang</i>	
The Relationship Between Rheological Characteristics and Slope Stability of Siltized Intercalation of Brown Mudstone	647
<i>T. Yang, Y. F. Feng, Y. Rui, J. Liu, L. Shen and H. Yang</i>	
Stability Analysis of Excavation Unloading on the Shuibuya Spillway Slope	653
<i>L. Wang, J. Li, X. Wang, H. Deng, H. Mei and Z. Tao</i>	
Internal force Features of Anti-Slide Piles During its Operations and Analysis on the Effectiveness of Landslide Control	661
<i>D. P. Zhu, Q. L. Deng and E. C. Yan</i>	
Bedrock Slope Failures in Tri-State Region	667
<i>M. Zoghi and J. Mahar</i>	
<b>Underground Stability</b>	<b>673</b>
Some Aspects of Model Uncertainties of Block Stability Estimation	675
<i>M. Bagheri and H. Stille</i>	
The Study of Concrete Arc Pre-Supporting System Method in Twin Shallow Tunnels (Case Study: Tohid Twin Tunnels, Tehran, Iran)	683
<i>H. Chakeri and B. Ferdowsi</i>	
Characterization of Complete Radial Displacement of Tunnel Using a Horizontal Inclinometer	691
<i>S. K. Chung, D. W. Ryu, W. I. Jang and H. S. Yang</i>	

An Elastic-Viscose Solution for Estimating Displacements of Circular Tunnels in Hydrostatic Stress Field	697
<i>A. Fahimifar and F. Monshizade Tehrani</i>	
Design of Radial Active Grouted Rockbolts in Tunnel Stability with a New Theoretical Approach	703
<i>A. Fahimifar and M. Ranjbarnia</i>	
Geo-Mechanical Aspects of the Subsidence Phenomenon Due to Near Surface Excavations in Rocks Under High Horizontal Stresses	713
<i>M. Fatehi Marji, A. Dabagh and A. R. Hajibagherpoor</i>	
Influence of the Mechanical Behaviors of Weak Rocks on the Stability Analysis of Behesht Abad Water Transmission Tunnel	719
<i>M. Fatehi Marji, A. Pourzargar and M. Hashemi</i>	
Design of a Railway Tunnel	727
<i>R. K. Goel, A. Swarup, A. Garg and R. K. Dayal</i>	
Safety Analysis of the Spillway Tunnel on Granite Region with Developed Fault Fractured Zone	737
<i>K. C. Han, D. W. Ryu and S. K. Kim</i>	
Behavior of Contact Surfaces in Segmental Lining Under Dynamic Loading in Alborz Tunnels (Tehran-Shomal Freeway)	745
<i>M. Hazrati Aghchai and H. Salari-Rad</i>	
State of the Art and Current Design Approaches in Hard Rock TBM Tunnelling	753
<i>M. Herrenknecht and K. B��ppler</i>	
Stability Analysis of Headrace Tunnels in Siah-Bisheh Dam and Powerhouse Project	761
<i>H. Hassani, H. Zamani and M. Shahandeh</i>	
Mine Accidents Due to Strata Control Problems Vis-À-Vis Geotechnical Investigations	767
<i>S. Jayanthu, Y. V. Rao, V. Laxminarayana and M. Monjezi</i>	

Microseismic Assessment of Reactivation of Weak Structures During Mining for Longwall Risk Management and Production Control	777
<i>X. Luo</i>	
An Investigation into the Compressive Strength and Failure Mechanism of Mine Pillars	783
<i>H. Moomivand and V. S. Vutukuri</i>	
Progressive Failure of 2D Symmetric Roof Rock Wedges	793
<i>P. P. Nomikos and A. I. Sofianos</i>	
Response of Horizontally Stratified Square Rock Roofs	801
<i>P. P. Nomikos, A. Tzinieris and A. I. Sofianos</i>	
Prediction of Displacements in Tunnels	809
<i>N. Radončić, T. Pilgerstorfer and W. Schubert</i>	
An Evaluation of Rock Mass Classification Methods Used for Tunnel Support Design	819
<i>J. Ranasooriya and H. Nikraz</i>	
Design and Construction of Tunnels in Poor and Faulted Rock Masses	827
<i>W. Schubert</i>	
Successful Tunneling through a Very Weak Conglomerate	835
<i>A. Shafiei, M.B. Dusseault, H. Rahdar and S. Mesgarzadeh</i>	
Design of High Speed Railway Tunnel in a Large Sized Fault Zone	845
<i>H. S. Shin and S. O. Choi</i>	
Proposal of Countermeasure against Fault Rupture Damages to Mountain Tunnels	853
<i>K. Tani</i>	
Study on Relationship between Stress Fields of Deep-Buried Tunnels in a Hydraulic Power Plant	861
<i>C. H. Wang, Q. L. Guo and Y. S. Zhang</i>	

Study on Numerical Simulation of the Stability of Rocks Surrounding Gateway with Different Wide Coal-Pillars in Fully Mechanized Top-Coal Caving	869
<i>K. Yang, G. X. Xie, J. C. Chang</i>	
Stability Analysis Based On Monitoring Results, Case Study: TBM Launch Cavern Asgaran-Ghomroud	875
<i>M. Zare, H. Tavakoli</i>	
Rock Stresses and Their Influence on Tunnel Behaviour	881
<i>Y. Zhou, H. L. Ong, and J. Wong</i>	
<b>Rock Dynamics and Foundation on Rocks</b>	<b>889</b>
Three Dimensional Numerical Modeling of Stress-Dependent Permeability Considering Aperture Distribution through Fractured Rock Masses Case Study: Roudbar Dam Foundation	891
<i>J. Abbasi, M. Sharifzadeh and B. Ferdowsi</i>	
Numerical Dynamic Analysis of Seismic Effect on SiahBisheh Pump-Storage Caverns	899
<i>M. Ahmadi, M. Yazdani and A. Rahnama</i>	
The Role of Shape in Stress Distribution around Openings and its Implications for Long-Term Survivability of Civil Structures in an Earthquake Zone	907
<i>Y. Ates and Y. Askin</i>	
The Seismic Effects on the Bukit-Tingi WWII Underground Shelter by 2007 Singkarak (Solok) Earthquake	917
<i>Ö. Aydan and M. Geniş</i>	
An Experimental Study on the End Bearing Capacity of Rock-Socketed Drilled Shaft	925
<i>S. Choi, W. Park and S. Jeon</i>	
Blast Induced Pressures in Some Granitic Rocks	933
<i>M. M. Dehghan Banadaki and B. Mohanty</i>	

Experiments and Analysis of Influence on Vibratory Load to Shear Strength of Red Clay	941
<i>H. Hua and C. Yong</i>	
Application of Energy Concentration Effect in Rock Drilling Blast	947
<i>L. X. Huang</i>	
Reinforcement Force Analysis and Evaluation for Downstream Toe of High Arch Dam	953
<i>P. Lin, R. K. Wang, Q. Yang, Y. N. Zhou and W. Y. Zhou</i>	
Rock Burst Possibility Analysis Based on In-Situ Stress Index and Numerical Simulation Validation at a Underground Water-Sealed Oil Storage Cavern	959
<i>L. Liu, C. Wang, Q. Chen and J. Zhang</i>	
The Dynamic Response of Rocks during Fracturing and its Implications in Geo-Engineering and Earth Science	965
<i>Y. Ohta, Ö. Aydan and N. Tokashiki</i>	
Reliability Analysis of Rock Foundation	973
<i>L. Rouaski and S. Belkacemi</i>	
<b>Instrumentation, Monitoring and Numerical Analysis in Geomechanics</b>	<b>981</b>
An elasto-plastic damage model in multi-laminate framework	983
<i>R. Ashjari and M. Ahmadi</i>	
A Numerical Investigation of Hydraulic Fracturing Process in Oil Reservoirs Using Non-Linear Fracture Mechanics	991
<i>V. Bahrami and A. Mortazavi</i>	
Dynamic Modelling of Twin Tunnels Under Seismic Action	999
<i>A. Bagherzadeh, B. Ferdowsi and A. H. Vosogh</i>	
A Coupled Normal-Shear Damage Model for Rock Joint Elements	1007
<i>P. Bemani Yazdi and A. Pouya</i>	

Geomechanical Effects of Gas Storage in Depleted Gas Fields	1015
<i>N. Castelletto, M. Ferronato, G. Gambolati, C. Janna, I. Salce and P. Teatini</i>	
Determination of Ground Response Curve of the Supported Tunnel Considering Progressive Hardening of Shotcrete Lining	1023
<i>A. Fahimifar and A. R. Hedayat</i>	
Loy Yang Mine Ground Movement Investigation and Modelling (Case Study)	1031
<i>A. Farazmand, J. Missen and S. Newcomb</i>	
On the Boundary Element Analysis of Stresses and Strains Around Underground Excavations in Rocks	1039
<i>M. Fatehi Marji, A. Hosseinmorshedy and A. R. Shadpour</i>	
Simulation of Hydraulic Fracturing in Brittle Rocks	1047
<i>A. Golshani and T. Tran-Cong</i>	
The Exact Formulation of Joint-Element Stiffness Matrix in Element Free Galerkin Method	1055
<i>M. Hajiazizi, N. Hataf and A. Ghahramani</i>	
Towards a Three-Dimensional Version of Numerical Manifold Method	1061
<i>L. He and G.W. Ma</i>	
Stability Analysis of Right Pressure Shaft at Siah-Bishe Pumped Storage Project; Iran	1069
<i>A. Karimi and M. Moosavi</i>	
A Bonded Particle Modelling for Rock Material	1077
<i>T. Kazerani and J. Zhao</i>	
Identifying Common Area of Blocks in Face-to-Face Contact State in Main Plane Method of Contact Detection for 3D-DDA	1085
<i>S. A. R. Keneti and A. Jafari</i>	
Finite Element Study of Blast-Induced Vibration from Construction of Tunnels with Particular Emphasis on its Effect to Nearby Slopes	1093
<i>A. K. L. Kwong and J. Y. C. Lo</i>	



Modeling Initiation of Hydraulic Fractures from a Wellbore	1101
<i>A. Lakirouhani, A. Bungler and E. Detournay</i>	
Development of Reinforcement module in serial and parallel Lagrangian Analysis Program	1109
<i>Z. Li, Sh. Fu, L. Zeng and R. Dai</i>	
Three-Dimensional Damage Model for Heterogeneous Rock Failure Process and Numerical Tests	1117
<i>Z. Z. Liang, C. A. Tang, L. C. Li and Y. B. Zhang</i>	
Numerical Analysis of the Effect of Joint Geometric Parameters on Grouting Process in Jointed Rock Mass	1125
<i>A. Maadikhah and A. Mortazavi</i>	
Determination of Stress Intensity Factors for Jointed Brittle Rock Medium Using Element Free Galerkin Method	1135
<i>H. Mirzaei, R. Kakaie and B. Hassani</i>	
Simulation of Rock True Tri-Axial Compression Test Using a Multilaminate Elastoplastic Model	1143
<i>M. Nikkhah, S. A. Sadrnejad, M. Ahmadi</i>	
Numerical Analysis of Coupled Behavior of Ground Water Flow Around an Excavation Damaged Zone in Discontinuous Rock Mass	1151
<i>J. S. Park, D. W. Ryu, C. I. Lee and C. H. Ryu</i>	
Numerical Simulation of Direct Shear Test on Rock Joint Using PFC3D	1159
<i>J. W. Park and J. J. Song</i>	
Three-Dimensional Transient Thermo-Hydro-Mechanical Analysis of Underground Nuclear Repository	1167
<i>S. K. Patel and K. G. Sharma</i>	
Numerical Study of Rock and Concrete Behavior by Multi-laminate Element Modeling	1175
<i>S. A. Sadrnejad</i>	

Modeling Heterogeneity and Anisotropy of Rocks	1183
<i>S. A. Sadrnejad</i>	
Crack Propagation Modeling of Rock-Like Materials from Surface Flaw Under Uniaxial Compression	1191
<i>S. B. Tang, C. A. Tang, R. H. C. Wong, Z. Z. Liang, Y. B. Zhang, L. C. Li and T. H. Ma</i>	
Numerical Research on Zonal Disintegration of Rock Mass Around Deep Tunnel	1199
<i>Y. B. Zhang, C. A. Tang, Z. Z. Liang, Y. J. Zuo and Y. J. Zhang</i>	
Numerical Simulation on Influence of Rock Heterogeneity on Dynamic Response of Roadway in Transversely Isotropic Rock Mass	1207
<i>Y. J. Zuo, C. A. Tang and S. C. Li</i>	
<b>Earth Resources and Coupled Processes in Rock Mechanics</b>	<b>1215</b>
Effect of Water Flow Velocity in Rock Discontinuities on Safety Factor	1217
<i>M. Ahmadi and M. Islami</i>	
Study on the Range of Frost-Thaw of Surrounding Rock and the Effects of Insulation Material in Cold Regions	1221
<i>W. Z. Chen, X. J. Tan, J. P. Yang and S. Huang</i>	
Estimation of the Rock Strength While Drilling with Roller-cone Bits	1229
<i>L. F. Franca and A. Mahjoob</i>	
Magnitude of In-Situ Stresses in Sabalan Geothermal Reservoir	1237
<i>M. Haftani, B. Bohloli, M. Eliassi and B. Talebi</i>	
Investigation of Cementation Factor “M” in Reservoir Parameters Estimation “A Case Study of Sarvak Formation in One of The Iranian Hydrocarbon Fields”	1245
<i>H. Hassani and H. Hassani</i>	
Coupled Fluid-Flow and Geomechanics in Naturally Fractured Reservoirs	1253
<i>M. R. Jalali and M. B. Dusseault</i>	

Interpretation of Mechanical Integrity Tests	1263
<i>M. Karimi-Jafari, P. Berest, B. Brouard and L. Van Sambeek</i>	
Coupled Thermo-Hydro-Mechanical Analyses of Thermal Effects on Fluid Flow with 2D Finite Element Method	1271
<i>L. C. Li, C. A. Tang, S. B. Tang and G. Li</i>	
Application of Kriging as a Geostatistical method and ANNs to Permeability Analysis at Ag-chaie Dam, Iran	1279
<i>S. Nadimi, D. Javani and K. shahriar</i>	
Stochastic Finite Elements in a Complex System: Vibrating Non Stationary Fluid Flow, Mass Transport, Heat Conduction	1285
<i>S. Osmani and M. Qirko</i>	
Using a Geomechanics Study to Define the Optimum Mud Window and the Best Trajectory to be Re-Drilling the Horizontal Bor-30 Well, Borburata Field, Venezuela	1295
<i>C. Rabe, M. Prado, D. Escalona, and S. Soto</i>	
Joint Study Based on K-Means Clustering, Asmari Formation, South West Iranian Oil Fields	1303
<i>B. Tokhmechi, H. Memarian, H. Ahmadi Noubari and B. Moshiri</i>	
A Coupled Model for Brittle Porous Rocks with Stress-dependent of Permeability	1309
<i>T. Xu, C. A. Tang and L. C. Li</i>	
Research of Cracks' Propagation Rule under Rock Seepage-stress Coupling Condition	1317
<i>N. Zhuang, B. B. Shi and K. Z. Zhu</i>	
<b>Building Stones /Ornamental Stones</b>	<b>1323</b>
Sawing Problems of Poly-Mineral Carbonate Natural Stone	1325
<i>S. Kulaksiz, F. Bayram, N. E. Yaşitli and E. Yilmazkaya</i>	

<b>New Development, Special Topics and Applications in Rock Mechanics</b>	<b>1331</b>
Rock Cutting by Picks and Blades	1333
<i>H. Alehossein</i>	
Investigation of Combined Opening –Sliding Fracture Toughness Behaviour in Some Cracked Rock Samples	1339
<i>M. R. Ayatollahi and M. R. M. Aliha</i>	
Study of Deformability of Fractured Rocks with Correlated Fracture Length and Aperture	1347
<i>A. Baghbanan and L. Jing</i>	
Simulation of the Bayanlou Debris Flow through Cellular Automata Modeling	1355
<i>A. Fahimifar and S. Amirpour</i>	
Protection Against Rockfall Hazards by the Means of Rockfall Barriers (Case Study of Kamarkhani Project)	1363
<i>M. A. Faraghat and H. Zahedi</i>	
A Study of Unusual TBM Disk Cutters Wears in Karaj-Tehran Water Conveyance Tunnel	1371
<i>M. Farokhnia, K. Shahriar, M. Sharifzadeh, H. R. Tavakoli and G. H. Shamsi</i>	
Rock Mass Quality Evaluation from Muck Pile and Operational Parameters Analysis during TBM Excavation Process	1379
<i>E. Farrokh and E. Fasihi</i>	
The Effect of Surface Irregularities On Joint Closure Behavior Using New Modeling Techniques	1385
<i>A. H. Ghazvinian, Z. Y. Yang, and A. Taghichian</i>	
Direction of Principal Stresses in Sabalan Geothermal Region	1393
<i>M. Haftani, M. Eliassi, B. Bohloli And B. Talebi</i>	

Determination of Relationship between Drilling Parameters by Clustering Techniques	1401
<i>M. Hamzaban and H. Memarian</i>	
Modeling and Estimation of Rock Mass Deformation Modulus Using Geostatistical Approaches in Bakhtiary Dam, Iran	1411
<i>H. Hamzehpour, A. Parhizkar, A. M. Radman and V. Rasloui</i>	
Geotechnical Risk Based Decision-Making for Selection of Rock TBM in Difficult Ground Conditions Using Fuzzy AHP	1419
<i>J. Khademi Hamidi, K. Shahriar and H. Bejari</i>	
A Study on the Correlation Between Drilling Parameters Using Rock Model Drilling Test	1427
<i>K. Y. Kim, C. Y. Kim, K. S. Kim and H. K. Yoon</i>	
An Investigation of Hydraulic Aperture Evolution by Flow Through Test for Circular Shearing using Laboratory Experiments and Numerical Simulation	1437
<i>E. Kim, J. Rostami, C. I. Lee and Y. Y. Jeong</i>	
Weathering Mechanism and Degradation of Hallasan Trachyte in Jeju Island, Korea	1447
<i>S. B. Lee, T. F. Cho, J. H. Kim, K. S. Won and G. B. Lee</i>	
Soundless Cracking Technique and its Application in Hard-Rock Tunnel in High Gas Coal Mine	1461
<i>Q. Y. Ma and X. Y. Lu</i>	
Fuzzy Rock Quality Designation	1465
<i>A. Mahmoodi, H. Mansouri, M. A. Ebrahimi Farsangi and H. Nezamabadi</i>	
Experiences Gained from Gas and Water Inflow Toward the Tunnel, Case Study: Aspar Anticline, Kermanshah, Iran	1469
<i>H. Mirmehrabi, J. Hassanpour, M. Morsali and S. Tarigh Azali</i>	
Properties of Frozen Soil for Excavation on the Moon	1477
<i>J. Rostami, L. Gertsch and R. Gustafson</i>	

Some Advances in Rapid Tunneling Techniques in Korea <i>C. H. Ryu, B. H. Choi, J. G. Kim and H. S. Yang</i>	1487
Grout Flow in Fractured Rock Media <i>S. A. Sadrnejad</i>	1491
The Application of Digital Image Processing to Estimate the Joint Volumetric Distribution <i>V. Serajian and H. Salari-Rad</i>	1499
Design Challenges Of The Gotthard Base Tunnel Pillar Stability in the Multifunction Station Faido <i>R. Stadelmann, M. Rausch and Z. Q. Wei</i>	1507
The Stability Assessment of Natural Rock Structures in Ryukyu Limestone <i>N. Tokashiki and Ö. Aydan</i>	1515
Recommended Rock Testing Methods for Predicting TBM Performance: Focus on the CSM and Ntnu Models <i>S. Yagiz, J. Rostami and L. Ozdemir</i>	1523

## **Author Index**

## ESTIMATING ROCK MASS LONG-TERM STRENGTH USING IN-SITU MEASUREMENT AND TESTING RESULTS

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### Abstract

A method for estimating rock mass long-term strength ( $\sigma_{LT}$ ) using the results of *in situ* measurement and test has been developed. It consists of estimation of rock mass strength ( $\sigma_{cm}$ ) using Hoek-Brown criterion, determination of rock mass modulus of deformation ( $E_m$ ) through Goodman's jack test, and construction of rock mass rheological model based on displacement monitoring data, taking into account the stress changes due to stoping activities underneath the test and monitoring locations. The rheological model is used to determine the rock mass long-term modulus of deformation ( $E_{LT}$ ). It is proposed that the long-term strength of rock mass can be estimated by using the rock mass strength and deformation modulus, rock mass long term modulus, and a coefficient that depends on the rock mass characteristics.

*Keywords:* Rock mass; Hoek-Brown criterion; Long-term strength; Rheological model.

### 1. Introduction

Reliable estimate of rock mass strength is required for almost any form of analysis used for the design of underground excavations. Hoek and Brown [1] proposed a method for obtaining estimates of the strength of jointed rock masses, based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. This method was modified over the years in order to meet the needs of users who applied it to problems that were not considered when the original criterion was developed. A review of the development of the criterion and of the equations proposed at various stages in this development is given in [2].

Although it is very useful for estimating the rock mass strength, the Hoek-Brown criterion can not be used in estimation of rock mass long-term strength, for which there is no method currently applicable [3, 4, 5]. This work suggests an alternative method for estimating the long-term strength of the rock mass, in particular that in Pongkor underground gold mine, Indonesia. It combined laboratory test, *in situ* test and monitoring, and numerical and rheological modelling.

### 2. Rock and Rock Mass Strengths

#### 2.1. Intact Rock Strength

Among others, uniaxial and triaxial compression tests seem to be the most frequent tests conducted for design purposes. However, researches have revealed that the uniaxial compressive strength is not an intrinsic material property, as it depends on the specimen geometry (size and shape) and loading rates.

Researches on the geometrical effects have concluded that there is a reduction in strength with increasing sample size. Medhurst and Brown [6] have reported that for coal from Moura mine in Australia, the 'critical' sample size is about one metre, above which the strength remains constant. This argument was further extended by Hoek and Brown [2] who suggested that when dealing with large scale rock masses, the strength will reach a constant value when the size of individual rock pieces is sufficiently small in relation to the overall size of the structure being considered.

A number of studies also reported that the strength decreases as the sample slenderness increases. In addition, it has been observed over

the years that higher loading rate leads to higher uniaxial compressive strength. Bieniawski [7], for example, reported this phenomenon after conducting uniaxial compression tests with test duration ranged from 10 minutes to 5 years, as illustrated in Figure 1.

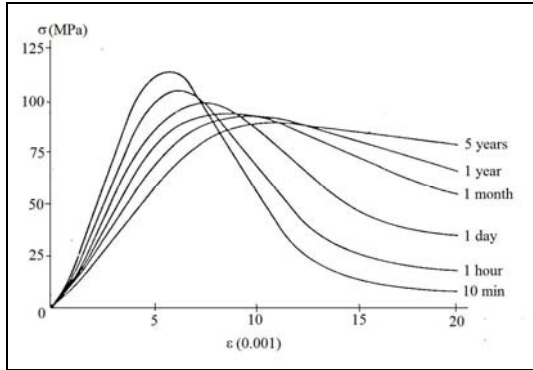


Fig. 1. Effect of test duration on uniaxial compression strength and stress-strain curve [7]

## 2.2. Rock Mass Strength

The Hoek-Brown criterion has been widely used to estimate rock mass strength. The generalised Hoek-Brown criterion for jointed rock mass is defined by [2]:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left( m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (1)$$

where  $\sigma'_1$  and  $\sigma'_3$  are the maximum and minimum effective stresses at failure respectively,  $m_b$  is the value of the Hoek-Brown constant  $m$  for the rock mass,  $s$  and  $a$  are constants which depend upon the characteristics of the rock mass, and  $\sigma_{ci}$  is the uniaxial compressive strength of the intact rock pieces.

To derive Hoek-Brown criterion, laboratory triaxial test is conducted to obtain  $m_i$ , which is the constant  $m$  for the intact rock, and  $\sigma_{ci}$ . The  $m_b$ ,  $s$ , and  $a$  are then determined by taking into account the characteristics of the rock mass and applying the following relationships [2]:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28}\right) \quad (2)$$

where GSI is the Geological Strength Index [8].

For  $GSI > 25$  (reasonable to good quality rock mass), the Hoek-Brown criterion is applicable with

$$s = \exp\left(\frac{GSI - 100}{9}\right) \quad (3)$$

and

$$a = 0 \quad (4)$$

For  $GSI < 25$  (very poor quality rock mass), the Hoek-Brown criterion applies with

$$s = 0 \quad (5)$$

and

$$a = 0.65 - \frac{GSI}{200} \quad (6)$$

Moreover, it can be said the Hoek-Brown criterion is a useful tool for relating the intact rock strength to rock mass strength. However, as the triaxial test is carried out with loading rate (0.1 MPa/s or 10 minutes of test duration) that is much higher than that experienced by the rock mass, further investigations are required to apply the criterion for estimating the long-term strength of the rock mass. Similar requirement is also needed to relate the long-term strength of intact rock to that of rock mass.

## 3. Estimating Rock Mass Long-Term Strength

### 3.1. Approach

In many cases, back analyses of underground excavations instabilities using numerical modelling have been widely used to estimate the rock mass long-term strength. In this approach the rock mass strength parameters are adjusted until the model gives a similar result with that occurs in the field. It is obvious that this approach requires an instability case which was not available in the work reported in this paper. An alternative approach was then implemented in this work as described in the followings.



Regarding the previous description of Hoek-Brown criterion, the criterion is applicable in the estimation of rock mass strength ( $\sigma_{cm}$ ), based on  $\sigma_{ci}$  and GSI. The  $\sigma_{cm}$  and rock mass modulus of deformation ( $E_m$ ) obtained from the *in situ* test can then be utilised to obtain the hypothetical rock mass failure strain ( $\epsilon_m$ ) by utilising the stress-strain curve.

Furthermore, displacement monitoring data can be used to construct the rheological model of the rock mass, which can be used to estimate the rock mass long-term modulus of deformation ( $E_{LT}$ ). Again, by consulting the stress-strain curve, the long-term strength of rock mass ( $\sigma_{LT}$ ) is likely between the stress straining the rock mass to  $\epsilon_m$  and that straining the rock mass to  $\epsilon_{LT}$ .

Figure 2 shows how this approach is conducted by utilising the stress-strain curves.

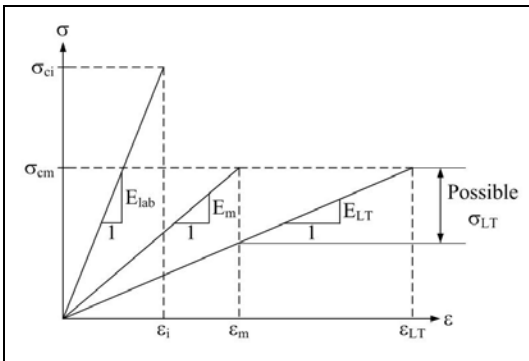


Fig. 2. Approach in estimating rock mass long-term strength

It can be seen in Figure 2, that the following relationship is applicable:

$$\sigma_{LT} = \psi \frac{\sigma_{cm}}{E_m} E_{LT} \quad (7)$$

where,

$\sigma_{LT}$  = rock mass long-term strength.

$\sigma_{cm}$  = rock mass strength.

$E_m$  = rock mass modulus of deformation.

$E_{LT}$  = rock mass long-term modulus of deformation.

$\psi$  = a coefficient.

### 3.2. Application

The above approach was implemented in the estimation of long-term strength of rock mass in cross-cut 6A of Pongkor underground gold mine, Indonesia (Figure 3).

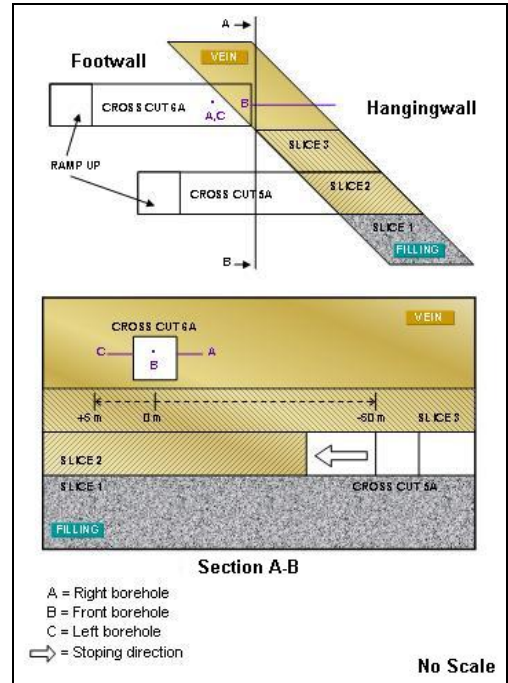


Fig. 3. Research location in Pongkor underground gold mine

#### 3.2.1. Rock mass strength

An intensive structural geology mapping has been conducted in the research location, including scan-line and window mappings and GSI of the andesitic-breccia rock mass (hangingwall and footwall) has also been calculated and an average value of 59 was obtained. As the average  $\sigma_{ci}$  of the intact rock was 88.30 MPa, application of Hoek-Brown criterion then gave a  $\sigma_{cm}$  of 22.02 MPa.

#### 3.2.2. Rock mass modulus of deformation

A number of Goodman's jack tests have been carried out and a detailed description of the tests can be found in [9]. The measured  $E_m$  of the rock mass was 6.17 GPa. This value was 47% of the

average  $E_{lab}$  and this was in accord with the review reported by Mohammad *et al.* [10].

### 3.2.3. Rock mass rheological model

Displacement monitoring using multi point borehole extensometer and convergencemeter were conducted in the research location. Figure 4 depicts the installation of the displacement monitoring devices.

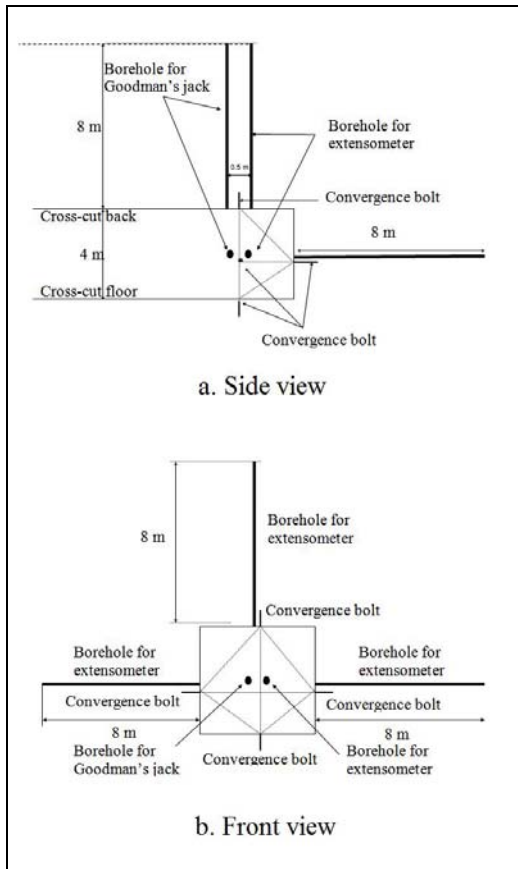


Fig. 4. Installation of displacement monitoring devices

As there were stoping activities underneath the research location, the displacement monitoring were obviously not conducted in a constant stress environment. Consequently, the stress changes had to be quantified and taken into account in the construction of the rheological model of the andesitic-breccia rock mass.

The stress changes quantification was carried out by utilising three dimensional numerical modelling. The modelling revealed that induced stress in the research location increased as the stope advancing toward the location and became constant after the stoping passed the location. Referring to this phenomenon, the appropriate time period for the construction of the rheological model was then decided, as depicted in Figure 5.

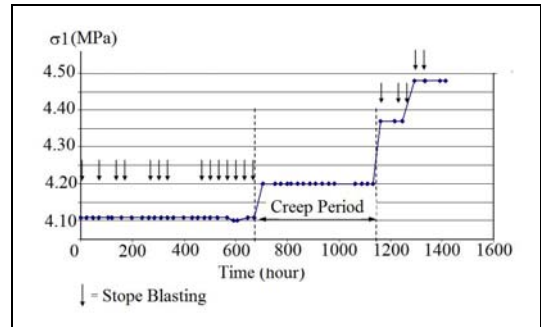


Fig. 5. An example of modelled induced stress changes

Furthermore, the Burger rheological model has been found most suitable for the Pongkor andesitic-breccia. The rheological model for the rock mass is as follows:

$$\varepsilon(t) = \frac{\sigma}{E_2} + \frac{\sigma}{E_1} - \frac{\sigma}{E_1} e^{-(E_1 t / 3\eta_1)} + \frac{\sigma}{3\eta_2} t \quad (8)$$

where all the rheological model parameters are given in Table 1.

Table 1. Rheological model parameters

Sidewall	$E_1$ GPa	$E_2$ GPa	$\eta_1$ GPa.hour	$\eta_2$ GPa.hour
Right	12.7	13.90	7.29e2	8.64e3
	0			
Left	8.92	5.39	3.88e2	7.00e3

Equation (8) can be written in the form of  $E(t)$  and by using the rheological model parameters in Table 1, the  $E(t)$  curves for rock masses in the right and left sidewalls could be constructed. The starting point of the asymptotic part of the curve was then taken as the rock mass long-term modulus of deformation ( $E_{LT}$ ), which in average was 2.53 GPa.

Using this value of  $E_{LT}$ , the estimated long-term strength of the Pongkor andesitic-breccia is likely between 9.03 MPa and 22.02 MPa, which is 10%-25% of the  $\sigma_{ci}$ .

#### 4. Discussion

The rock mass strength estimated in this work is 25% of the intact rock strength. This fact clearly shows that rock mass strength is controlled by the interlocking of rock blocks and the condition of the surfaces between these blocks, which are represented by the GSI. Extensive laboratory tests and field studies on excellent quality Lac du Bonnet granite [11] suggested that the *in situ* strength of this rock is only about 70% of that measured in the laboratory.

Rock mass modulus of deformation measured in this work is 47% of that obtained in the laboratory. Although it is in line with the review carried out by Mohammad et al. [10], further investigation of factors affected the Goodman's jack test, in particular the induced stresses, is still required.

The long-term strength of Pongkor andesitic-breccia might be only 10%-25% of the intact rock strength. Undoubtedly, if this value is taken into consideration in the design, much conservative design will be recommended and support costs will be much higher. This fact, however, could advise the mining that after some years, the rock mass strength will decrease and some support repair will be required.

#### 5. Conclusions

A method of estimating rock mass long-term strength has been developed. It contains laboratory test, *in situ* test and measurement, and numerical modelling.

The method has not been verified, as there is no rock mass failure case history that can be used to back analyse the rock mass strength. Once the failure is available, refinement of the approach used in the development of the method must be conducted.

#### Acknowledgments

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