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General Report — Session 6: Geological, Rock and Mining Engineering, Including Underground Structures and Excavations, and Subsidence of Deltas

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GEOLOGICAL, ROCK AND MINING ENGINEERING, INCLUDING UNDERGROUND STRUCTURES AND EXCAVATIONS, AND SUBSIDENCE OF DELTAS

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General Report – Sessions 6a and 6b

INTRODUCTION

This General Report summarizes the papers submitted to Session 6a “Case Histories and Failure of Geological, Rock and Mining Engineering, Including Underground Structures and Excavations, and Subsidence of Deltas” and Session 6b “Anticipation, Characterization, Design and Construction in the Geological Complexity of Mélanges, Fault Rocks, Weathered Rocks, Boulder Colluvium, Lahars, and Similar Bimrocks (Block-in-Matrix Rocks) and Rock/Soil Mixtures”

A total of 17 papers were submitted covering the broad session themes described above. Thus, the papers were classified into the following five groups:
Group 6.1: Case histories of excavations, quarries, and rock stability including 3 papers;
Group 6.2: Case histories of tunnelling, mines and underground construction including 6 papers;
Group 6.3: Case histories related to karst including 2 papers;
Group 6.4: Case histories of ground subsidence including 2 papers; and
Group 6.5: Case histories of material characterization including 4 papers.

Table 1: Distribution of the location of projects included in the papers presented in session 6.

Country	Project Location
United States of America	4
India	4
France	3
Iran	2
Russia	1
Canada	1
Italy	1
Japan	1

The geographic distribution of the case histories is listed in Table 1. Overall, seven papers were submitted from Asia, five from North America, and five from Europe.

SUMMARY OF PAPERS

Group 6.1: Case Histories of excavations, quarries and rock stability

Paper #6.01 by Lo Presti, Cravero, and Iabichino titled “An ‘unexpected’ rock failure in a limestone open pit mine” analyses the collapse of about 5000 cubic meters of rock in a limestone open pit mine in the Lucca district (Tuscany), Italy.

The rock formations of the low Serchio river valley belong to the non-metamorphic Tuscany succession, which, at the quarry location, mainly consists of the “Maiolica” limestone and the “Scaglia Ross” (i.e. varicolour claystone, with sporadic calcarenite and siliceous limestone beds) formations. The “Maiolica” limestone, which makes up the exploited ore body, gives rise to smooth fractures of a concoidal type. The local rock mass structure was characterized by highly persistent, low-angle bedding joints and three high angle (subvertical) joints.

The authors concentrate on the parametric pseudo–dynamic Newmark-type analyses which were performed using as input the accelerograms recorded at distances of about 30 m and 70 m from the blasts. The quantity of explosives charged in the blast holes during production activities were about twice those employed for re-profiling purposes. Therefore the accelerograms, which have been recorded during re-profiling blasts, were considered representative of vibration history during normal production activities without any ground motion scaling or modification.

The authors conclude that the rock failure had probably been triggered by repeated blast vibrations in concomitance with heavy rainfall levels recorded between the end of October and the beginning of November 2004 when the failure occurred.

Paper #6.05 by Zekkos, Cohen-Waeber, Medley, Hunt and Jesionek titled “Characterization of a weak rock mass and geoengineering analyses for a canyon landfill in northern California” presents a case history of the successful increase in waste capacity of a canyon landfill in northern California by steepening extensive 60-m high excavations in a weak rock mass. The landfill is underlain by the Panoche Formation consisting of a complex series of sedimentary units, thought to be dominated by turbidities.

The authors applied two independent approaches to evaluate rockmass strength allowing increased confidence in the estimates of input parameters used to evaluate the stability of excavations. Analyses were performed to evaluate the stability of the excavations against unstable wedges or planes along existing discontinuities, as well as the stability of the rockmass. In the evaluation of the rock mass, two independent approaches, one based on laboratory testing results and one based upon the Hoek and Brown approach using field observations were employed, yielding consistent results.

Characterization of the rockmass indicated a pronounced improvement in the rock structure and the condition of the discontinuities with depth, resulting in an increasing Geologic Strength Index (GSI) with depth. Subsequent analyses performed using a layered Hoek and Brown Criterion allowed further steepening of rock excavations. Comparisons were also made in the results of the analyses using a layered versus a more commonly used uniform Hoek and Brown approach. It was observed that the layered approach identified more critical, relatively shallow failure surfaces and eliminated the spurious apparently-critical deep-seated rock mass failure, generated assuming a uniform rockmass.

Paper #6.13 by Sturman, Rehwoldt and Martin titled "Support of rock cuts at Washington-Dulles International Airport" describes the geotechnical investigations, design sequence, recommended support pressures, and the results from constructing several cuts at the Washington-Dulles International Airport. Expansion at the Dulles Airport has required extensive vertical, open-cut rock excavations to depths of up to approximately 20 m (65 ft) adjacent to existing infrastructure to construct below-grade stations for the new Automated People Mover (APM) passenger transportation system.

The rockmass at Dulles Airport consist of a sedimentary sequence of rocks of the Culpeper Basin, ranging in appearance from thinly bedded to blocky. The bedding planes were identified as the controlling discontinuity; therefore, the characterization of the bedding planes were a critical issue to the rock mass characterization as they posed the greatest threat to the stability of the planned excavations. The key characterization data obtained in this study concerned discontinuity orientation and persistence, the shear strength of discontinuities, and evaluation of groundwater conditions from acoustic televiewer testing and laboratory testing of both intact cores and discontinuity samples.

Based on the pattern of the predominant discontinuities, it was concluded that bedding planes dipping into the excavation at approximately 30 degrees and intersecting near-vertical joints would present the greatest risk for rock cut failures based upon the rockmass failure mechanism of local joint- and bedding-controlled sliding. The resulting design lateral pressure necessary to support a rock face using this mechanism and the shear strength of discontinuities and intact rock was significantly lower than the initial design values. Construction-phase observations and monitoring, which included detailed field mapping, automated instrumentation monitoring, and groundwater monitoring, have verified the rock characterization and design assumptions.

<u>Group 6.2: Case Histories of tunnelling, mines and underground construction</u>

Paper #6.03a by Goel and Swarup titled "Case history of tunnelling through claystone" presents the construction challenges encountered during the excavation of a tunnel in the Himalayas. The D-shaped tunnel crossed thickly bedded, moderately soft, sparsely jointed sandstone, sheared claystones, siltstones and overburden comprising boulders/pebbles in sandy/silty matrix. The tunnel is also in the vicinity of a major structural feature, the Murree Thrust, and thus, for a considerable tunnel length the ground conditions are extremely poor.

Due to the presence of swelling minerals in claystone and weak and highly jointed rock formations with high rock cover (313 m), the tunnel experienced both swelling and squeezing ground conditions resulting in the buckling of wall supports of steel ribs, cracking of tunnel wall concrete lining at places and floor heaving up to 1.2 m. With the deformation of wall supports, the tunnel roof support also deformed. Deformations were measured even after 12-18 months since the excavation of the tunnel, and leaded to the cracking and failure of tunnel lining at places.

The support pressure and the deformation were monitored to study the performance of the support system. Numerical analysis using FLAC3D were performed to study the effectiveness of the support system. Based on the analyses, rock bolts and 40 cm thick steel fiber reinforced shotcrete (SFRS) support, were recommended to reduce the deformations to 23 cm.

Paper #6.10a by Ranadive titled "Shape optimization of tunnel by Finite Element Method" considers the ground conditions over a 300-m length of the Rayagada tunnel and performs numerical simulations for different tunnel shapes in an effort to optimize the tunnel shape so that the tangential tensile stresses are minimized and simultaneously the compressive tangential stresses are below the permissible limits. Three tunnel shapes and three ground conditions are considered. The ground conditions are variable, but generally consist of a layered profile with the tunnel typically excavated within a completely weathered schist or completely weathered gneiss or quartzite. The analyses indicate that the third case, a D-shaped tunnel with curved side walls, crown and invert, is the optimum shape from a stress point of view, even though the deflection of the crown are approximately the same for all cases.

Paper #6.11a by Nasri, Winum and Magnien titled "Rehabilitation of La Nerthe Tunnel on Paris-Marseille high-speed railway line" presents the studies performed for the rehabilitation of the La Nerthe tunnel. The tunnel is part of the Paris-Marseille railway line where it crosses the Étoile mountain range. This 4638 m long double track tunnel was built at the middle of 19th century under a maximum cover of 180 m and currently is used as part of the high-speed railway line (TGV). The masonry tunnel was constructed with hard limestone blocks at the sidewalls, and bricks at the crown.

The tunnel passes through marl, gypsum and limestone zones. For a distance of approximately 300 m the tunnel intersects a major fault and the ground conditions exhibit extensive tectonization. Hydraulic fracturing tests indicate large residual stresses of tectonic origin with a high lateral earth pressure coefficient (K_0) that could reach a value of 5.

The paper documents the damage of the tunnel lining. The tunnel exhibited pronounced deformations in shoulders, with excessive displacement at the springline, and pinching and bursting of the bricks at the crown in several zones. Finite element analyses were used to understand the causes of the observed damage and propose a reinforcement method. Based on the results of the finite element analyses, a rehabilitation system was developed that consisted of the replacement of sections of the masonry with wire mesh-reinforced concrete in alternating panels.

Paper #6.12a by Nasri and Fauvel titled "Construction of express subway line Eole in Paris" presents the case of the construction of line E of the Paris express subway network between two new stations. The paper emphasizes the construction of the tunnels and one of the two stations, the Magenta station. The Magenta station was constructed in difficult geological conditions and high water table. The construction had also to consider the presence of historical mid- 19th and 20th Century buildings at the surface. According to the traditional principles of underground construction in Paris, construction initiated with the construction of the side walls, followed by the construction of the crown and finally the invert.

The soil conditions consist primarily of sedimentary soils. The Magenta station is located on an abandoned gypsum quarry that has been exploited earlier and was filled with manmade fill, which consisted also of the foundation of the historical buildings. Dissolution of gypsum lenses and the existence of voids was a critical consideration. Microgravimetry and seismic geophysics were employed for the recognition of such voids. The construction also included the excavation of three shafts and tunnels intended to document the ground conditions, pumping tests and jet grouting in critical areas.

A 7.4 m-diameter slurry shield TBM was used for the construction of the two 1700 m long tunnels. The New Austrian Tunnelling Method (NATM) was also used for some of the galleries and vaults. Real-time monitoring was performed on a daily basis and the support system and the construction sequence was updated based on the new information. Finite Element models were also used to evaluate the measurements.

Paper #6.14a by Emeriault, Karstner, and Vanoudheusden titled "Movements induced by tunnelling with and EPB machine in overconsolidated soils: Compans Monitoring Section of Toulouse Subway Line B" presents the instrumentation results of a monitoring section installed in an under-construction section of the Toulouse subway line F in France. The tunnel is excavated by a 7.8 m diameter earth-pressure balance machine. The overburden is 12.8 m. The alignment of the tunnels goes through relatively uniform ground conditions consisting primarily of overconsolidated hard sandy clay with pockets and lenses of very dense sand, known locally as the Toulouse molasses. Overlying the molasses and extending to the surface are fill materials having a thickness of 6.8 m.

The monitoring section included five multipoint borehole extensometers with automatic data acquisition and 3 inclinometers, one along the alignment of the tunnel and two on its sides. An invar thread is also used to measure the horizontal strains (both in the longitudinal and the transverse orientations) at the ground surface, induced by the tunnelling. Two pore pressure cells are installed along the alignment and five pairs of vibrating wire strain gauges are used to measure strains in the concrete lighting segments.

The paper presents the results of the monitoring. The measurements at the surface indicate that 2 mm maximum heave instead of settlement took place during the passage of the earth-pressure balance machine. According to the authors, this is consistent with the overconsolidated nature of the material. The inclinometer along the alignment of the tunnel did not record any movement towards the tunnel face, before the TBM reached the section. The two inclinometers that were located at an offset distance from the tunnel alignment recorded horizontal movement prior to the occurrence of excavation in the section of the inclinometer. These observations are consistent also with the vibrating wire strain gauges along the concrete lining segments that indicate that the ring takes an elliptical shape with the long axis of the ellipse being close to the vertical direction. The authors note that this type of behavior was also observed for the same project in areas of similar ground conditions, excavated using compressed-air and slurry shield TBM as well as excavations using the "traditional method". The authors also provide additional recorded data for the instrumentation used.

Paper #6.17 by Taghipoor titled "Application of numerical modelling to study the efficiency of roof bolting pattern in east 1 main gate of Tabas coal mine" presents 2-D numerical modeling results using the FLAC program, implemented to assess the efficiency of the roof bolting pattern in the east 1 main gate of the Tabas Coal Mine located in the Yazd province in mid-eastern Iran. Before driving the 1250 m long roadway into sole roof bolt support, it was decided to carry out computer modeling to assess the efficiency of the roof bolt pattern during driving of the roadway and also the potential influence of face retreat.

The mine is exploiting seam C1, with a thickness of 1.8 to 2 m in E1 panel, at a gradient of 1 in 5 to 1 in 2 (11 to 26 deg.) in the initial mining area. The C1 roof contains a sequence of mudstone, siltstone/sandstone interfaces which have the potential to be water bearing. The floor has been classified as weak seat earth/mudstone underlain by stronger mudstones, siltstones and sandstones. To provide numerical modeling input parameters (rockmass properties), for the models, the RocLab computer program was used to estimate the parameters of the rockmass surrounding the gate working based on the GSI classification, GSI=RMR-5.

The output of the modeling, in the form of displacements and strains in 7.2 m extensometers, movements in 5 m long dual telltales and axial loads in roof bolts, were compared to the results of real monitoring instruments (7.2 m multipoint sonic extensometers, 5 m telltales and strain gauges bolts) installed in the gate. Good agreement of real monitoring results and the model revealed proper estimation of the rock mass properties for the modeling. The modeling showed that no significant problems would occur due to total closure of the gate during driving and retreat. At the time of the completion of this paper, the author reports that the mine has completed the E1 MG, whereas more than 1000 m of the gate road was successfully supported by the new bolt pattern with roof movements of less than 16 mm.

Group 6.3: Case Histories related to karst

Paper #6.02a by Volkov and Gera titled "Use of strengthening cementation when civil structures on karsted territories construction" describes two case histories where cement mortar was utilized to prevent development of karst-suffossion processes. The first case presents a 14-story building constructed on a site described as a complex stratification of gypsum, limestone, marl, argillite and aleurolite with a water table greater than 60 m. Soil improvement consisted of injecting 1.5 g/cm^3 cement mortar through 52 injection holes on a 5x5 grid to 5 m below the zone selected for cementation. The site's average filtration ratio decreased from 5.8 m/day before, to 0.6 m/day after cementation strengthening.

The second case presents an existing 35-m high reinforced concrete building already subjected to karstic-suffosion of the underlying gypsum bearing sandstone, limestone, marl, gypsum limestone and gypsum structure. In order to prevent further settlement, a process was implemented consisting of injecting 1.5 g/cm^3 cement mortar through 113 injection points on a 2x3 grid to depths of 12 m below the bottom of the foundation. The average filtration ratio decreased from 3.4 m/day before, to 0.003 m/day after remediation.

The authors conclude that strengthening cementation effectively stabilizes the base soils of karstic territories.

Paper #6.02b by Guisinger, Kuo, and Puckett titled "Design and construction of drilled shafts in karst environments in Florida" present a case history of the design and construction of drilled shaft foundations in karst limestone for the I-4/I-275 Downtown Interchange in Tampa, Florida. For this project a two-phase procedure was utilized by the Florida Department of Transportation (FDOT) to minimize the impact of karst environments on drilled shaft construction. In the first phase or design phase, design criteria and estimated total drilled shaft lengths were established. During the second phase or construction phase, SPT borings were performed at each drilled shaft location to set production lengths according to the design criteria established during the first phase.

Design criteria were developed from a statistical analysis of the limestone properties obtained during the design phase. The design unit skin friction of the limestone was calculated based on McVay's method; however, limestone with SPT N-values less than 25 were treated as clay and the undrained shear strength was calculated based on Terzaghi and Peck's method. From this a relationship of the Ultimate Unit Skin Friction and SPT N-value was established.

The authors compared the drilled shaft lengths from each phase and discovered that although statistically the total lengths estimated were close, 67% of the drilled shafts were lengthened during the construction phase. They conclude that the typical practice of setting production shaft lengths on a pier/bent basis is not a practical approach in karst regions, but that the case history presented demonstrates the successful implementation of a two-phase design method in karst environments.

Group 6.4: Case Histories of ground subsidence

Paper #6.08a by Marino, Gamal, and Malyala titled "Empirical correlations of longwall subsidence data for the Illinois coal basin" develops empirical correlations from longwall subsidence data for various subsidence parameters, including maximum vertical and horizontal displacements, subsidence slope and curvature and horizontal strain. Data were obtained from several mines within the Illinois Coal Basin, augmented with data from outside of the basin for parts of the analyses.

A statistical approach is used to determine expected ranges of ground movement. Through the trends observed, subsidence profiles and areal maps are generated to quantify and locate movements induced by longwall mining. Then, potential damage is assessed and precautions taken.

To evaluate the vertical displacements, the authors correlated the maximum subsidence with the coal extraction height and the ratio of the panel width to the depth of the coal seam. The subsidence factor or ratio of maximum subsidence to coal extraction height was plotted against the ratio of panel width to depth of coal measured from the ground surface.

The profile width is determined as the distance from the point of zero subsidence to maximum subsidence (SPW). SPW is calculated from multiplying the depth of the coal seam by the angle of draw and the profile development angle. The angle of draw is defined as the angle measured from a horizontal projection at the ground surface of the edge of the panel to the point of zero subsidence. For the data obtained, the angle of draw varied between 0° to 50° while the profile development angle varied between 17° and 31°.

To evaluate the range of slope and curvature ranges, the authors developed two plots. The first plot compares maximum subsidence (S_{max}) to 2(SPW)S'_{max}. The extraction ratio ranges from 50 to 87% whereas S_{max} varies from 0.5 to 5.2 ft. The second plot compares S_{max} to 4(SPW)S''_{max}. Variations of the average slope and curvature are about 3.5 to 4.5 times. Also, locations of the permanent ground deformation relative to the structure were evaluated. The correlations looked at included offset from the rib to maximum compressive curvature (OF_{cc}), offset from the rib to maxium tensile curvature (OF_{tc}), OF_{tc}-OF_{cc}, and (OF_{tc}+OF_{cc})/2 to various site conditions.

Also, horizontal displacements and strains were assessed. Horizontal strains were plotted against the maximum horizontal displacement (H_{max}) divided by SPW. Horizontal strains were found to increase directly with H_{max}/SPW.

From the range of data collected subsidence profiles are developed.

The authors conclude that a statistical approach to developing subsidence profiles is preferable to subsidence prediction methods determined by ‘best-fit’ approaches that uses averaged data.

Paper #6.07b by Ghorbanbeigi, and Naji hamodi titled “Land subsidence in Tehran district, Iran” describes the use of Interferometer Synthetic Aperture Radar (InSAR) to identify land subsidence in the Tehran and Shehriar Valleys of Iran. The region has experienced an increase in the occurrences of land subsidence from dramatic lowering of the groundwater table due to increased water demand.

The valleys have an average rainfall of 240 mm/year with an average temperature variation from 10°C to 17°C and a relative humidity of 40%. Average evaporation for the region is 2500 mm/yr. Water resources are obtained from 7 seasonal rivers, 2 permanent rivers and 3 alluvial aquifers. The aquifers are approximately 300 to 400 meters thick with an average water level drop of 40 cm/yr. Water supply is provided by 522 canals which produce 393 million m^3 of water. However, well drilling has caused most of the canals to go dry.

Through the use of InSAR along with the ROI-Pac software and digital height model the land subsidences were allocated. Accuracy was determined within 2 cm over 2 to 18 months. The land subsidence rates were determined at 70, 175 and 315 days. The pictures obtained show two profiles with 15 and 16 cm subsidence. The land subsidences are located in southwest Tehran and in the Shehriar Valley.

The authors conclude that in order to prevent further land subsidence the following actions be taken: control over groundwater production, artificially recharge the aquifer and apply water resource management techniques to protect the aquifer and minimize damage to surface and subsurface installations.

Group 6.5: Case Histories of material Characterization

Paper 6.04a by Mhaiskar, Hedge and Tata titled “Columnar Basalt – Vibrations Study and Preservations Methods at Mumbai, India” describes the characterization of columnar jointed basalt at Gilbert Hill, a volcanic plug within tuffaceous breccias. Gilbert Hill was quarried in the 1960’s and is being encroached by buildings. Quarrying has rendered the steep excavated slopes vulnerable to rock fall induced by nearby construction and development, particularly after rainfalls. The tall and slender slabs of columnar basalt are vulnerable to toppling failures, but no characterization was performed of the rock mass. Vibration studies were performed between the multi-story structures and the toe of the rock slope 10 m to 15 m away. Geophones were located between the buildings and at the toe of the slope. Vibrations were generated near the base line by an 8 tonne vibratory compactor and vibratory rock breaking equipment (jackhammer?) and peak particle velocities (PPV) identified.

For the heavy vibratory roller at the base line, the PPVs ranged between 1.6 mm/sec and 9.4 mm/sec at the baseline reducing to undetectable at the toe of the slope. For the rock breaking equipment at the base line, the PPVs ranged between 15.6 mm/sec and 75 mm/sec at the baseline reducing to undetectable at the toe of the slope. At the slope, these velocities are less than two times the referenced criteria for recommended maximum for prevention of damage to adjacent structures due to construction vibrations (50 mm/sec) and for historical buildings/monuments in poor condition (2.0 mm/sec).

Although the vibrations originating more than 5 m from the slope decayed to non-detect levels, the authors estimated that 10% of the slope will require remediation such as slope adjustment, structural support, dental equipment and drainage treatment. The authors recommend several other remediation and mitigation measures for roadway and building construction near the slope, gunite protection of the slope, drainage improvements to the slope and declaration of Gilbert Hill as a protected structure monument.

Paper 6.07a by Klein and Trimble titled "Characterization of Piedmont Residual Soil and Saprolite in Maryland" presents the case of an approximately 30 km (19 mile) highway to be constructed through the eastern Piedmont of Appalachian Mountains. It was expected that excavations would be in residual and saprolitic soils derived from weathering of parent schist, gneiss and mafic intrusive rocks. Variable weathering in Piedmont rocks produces soils greater than 30 m (50 feet) in depth with isolated blocks of relatively intact rock surrounded by soil materials and irregular bedrock pinnacles extending to the ground surface.

An investigation was performed in which 392 borings yielded SPT data, relatively undisturbed samples of drill core and about 15,000 m (42,000 feet) of seismic refraction survey lines. Laboratory testing produced over 3,000 test results for moisture content, soil classification tests, CBRs, Point Load Test and soil and rock unconfined strengths, triaxial testing with/without pore pressure measurements, and direct shear tests.

Based on the geotechnical and laboratory testing, correlations were developed for soils, intermediate soil/rock and parent bedrock conditions, the "strata" of which were discriminated by seismic velocities and SPT value. The paper explains in detail the correlations between the measured data and strength parameters for the interpreted soil, intermediate and bedrock materials (The interested reader is directed to the 18 graphical Figures and 10 summary Tables).

Considerable variation was noted in many correlations. In general, the authors concluded that the Piedmont soils and intermediate soil/rock materials within the very large project area were "…not nicely behaved' nor were they composed of "…easily predictable material properties…". . It is recognized that prediction of soil properties from existing correlations are likely to be erroneous since many of these are for soils with a sedimentary heritage. Hence within residual soils, it is preferable to perform in-situ testing as well as derive site-specific correlations such as those presented in this paper.

Paper 6.09a by Mundhe, Pandhare, Methekar and Vaijapurkar titled "Case History Compilation of Engineering Properties of Common Rocks in Maharashtra, India for Database (1982-2002)" investigates and briefly summarizes two decades of rock testing data generated by the rock mechanics laboratory of the Maharashtra Engineering Research Institute, which is commissioned to provide engineering services for state water resource projects. 50 mm diameter rock cores were drilled from 0.027 cubic meter (1 cubic foot) rock specimens, and tested for % Water Absorption, Specific Gravity/Density, and (Unconfined) Compressive Strength. Test data from over 1600 samples were related to other project attributes, location, and geological classifications as presented in Tables. Most useful of these is Table 8 showing average rock properties for rocks tested, the bulk being basalts, with a few granites and a sandstone. Overall, the rocks in the river basin area tested have similar average compressive strength of 600 kg/cm^2. The tested specimens appear to have water absorption properties that exceed permissible levels (1.5 to 2.9% vs. 1% max allowable), and Unconfined Compressive Strengths that may also be lower than required. Based on the considerable historical data the authors suggest that the permissible values of Water Absorption and Compressive strength should thus be revised.

Paper 6.01b by Nakagawa, Yamada and Tsuka titled "Characterization of the Shear-Induced Potential (SIP) in Clay and the Application to Field" presents the measurement and interpretation of the geophysical changes in overall clay electrical properties related to shear displacements. Shear -induced polarizations of clays occur due to dipolar reorganization of clay particles that have been displaced by shear deformations.

For the initial investigation of the slip induced potential (SIP), specimens of clay fault gouge were consolidated and then subjected to simple shear tests. The behaviors of the induced electrical potentials were measured using arrays of electrodes. Strains were measured at the least, intermediate and greatest principal strain axes. Positive charges accumulate on the greatest principal axial strain plane and negative charges at the least principal plane. Relationships between the induced mean potential strain and electrical potentials were plotted. For normal strain, a generally linear relationship was defined with a proportional constant of 1.91 volts per unit % strain.

In-situ tests were performed to confirm SIP at active landslides in Basuno landslide in the Nishikawa areas of Japan. About 50 electrodes were placed in holes drilled to depths of 2 m at the Basuno site and some 30 at the Nishikawa site. At some locations, electrodes were placed at different depths to observe depth-dependent electrical behavior. Spontaneous (Self) Potential Data were obtained continuously over periods of several months concurrently with ongoing slip displacements. Strong correlations were noted between rainfall events, changes in SP and slip, but the data trends were complicated.

Analysis of the in-situ data were correlated with experimental data performed using blocks of fault gouge that could be deformed in a variety of ways. The experiments showed that electrical potential varied corresponding to the sliding mass; marked spikes in SP coincided with the initiation of slip-stick style deformations. Analysis indicated that in shear, positive charges appear to accumulate in the same direction as the slip, with positive charges at the leading edge of the landslide masses and negative charges at the rear end.

It is apparent that measurement of self potential may possibly be effective as an early-warning monitoring technique given that small strains can be measured.

SUMMARY

A total of 17 papers were submitted in Session 6. The papers cover a wide range of case histories and can be broadly classified in groups including case histories of rock excavations, quarries, and rock stability (group 6.1), tunnelling, mines and underground construction (group 6.2), karst (group 6.3), ground subsidence (group 6.4) and material characterization (group 6.5). A concise summary of each paper that includes a description and major findings is presented in this general report.

Table 1. Papers submitted to this session

Paper ID	Title of Paper	Authors	Country
Group 6.1: Case Histories of excavations, quarries, mines and rock stability			
6.01a	An “Unexpected” Rock Failure in a Limestone Open Pit Mine	D. Lo Presti M. Cravero G. Iabichino	Italy
6.05a	Characterization of a Weak Rock Mass and Geoengineering Analyses of a Canyon Landfill in Northern California	D. Zekkos J. Cohen-Waeber E. Medley C. Hunt K. Jesionek	USA
6.13a	Support of Rock Cuts at Washington-Dulles International Airport	J.O. Sturman E.B. Rehwoldt (USA) C.D. Martin	Canada
Group 6.2: Case Histories of tunnelling and underground construction			
6.03a	Case History of Tunnelling Through Claystone	R.K. Goel A. Swarup	India

Paper ID	Title of Paper	Authors	Country
	Group 6.2 continued		
6.10a	Optimization of D-Shape Tunnel by Finite Element Method	M.S. Ranadive	India
6.11a	Rehabilitation of La Nerthe Tunnel on Paris-Marseille High-Speed Railway Line	V. Nasri C. Winum P. Magnien	USA France
6.12a	Construction of Express Subway Line EOLE in Paris	V. Nasri P. Fauvel	USA France
6.14a	Movements Induced by Tunnelling with an EPB Machine in Overconsolidated Soils: Compans Monitoring Section of Toulouse Subway Line B	F. Emeriault R. Kastner E. Vanoudheusden	France
6.17a	Application of Numerical Modelling to Study the Efficiency Roof Bolting Pattern in East 1 Main Gate of Tabas Coal Mine	S. Taghipoor	Iran
Group 6.3: Case Histories related to karst			
6.02a	Use of Strengthening Cementation when Construction of Residential Buildings on the Karsted Territory	F. E. Volkov L. N. Gera	Russia
6.02b	Design and Construction of Drilled Shafts in Karst Environments of Florida	A.L. Guisinger C. L. Kuo T. Puckett	USA
Group 6.4: Case Histories of ground subsidence			
6.08a	Empirical Correlations of Longwall Subsidence Date for the Illinois Coal Basin	G. G. Marino M. Gamal V. N. Malyala	USA
6.07b	Land Subsidence in Tehran District, Iran	J. Naji hamodi S. Ghorbanbeigi	Iran
Group 6.5: Case Histories of material Characterization			
6.04a	Columnar Basalt - Vibration Study and Preservation Methods at Mumbai, India	S.Y. Mhaiskar R.A. Hedge C.R. Tata	India
6.07a	Characterization of Piedmont Residual Soil and Saprolite in Maryland	E. M. Klein J. L. Trimble	USA
6.09a	Case History Compilation of Engineering Properties of Common Rocks in Maharashtra, India, for Database (1982-2002)	M.S. Mundhe V.B. Pandhare N.M. Methekar S. R. Vaijapurkar	India
6.01b	Characterization of the Shear-Induced Polarization (SIP) in Clay and the Application to Field	K. Nakagawa S. Yamada I. Tsuka	Japan