ROCK MASS MODELING
IN POLYAXIAL STRESS STATE

by

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submitted

in fulfilment of the requirements of the degree of

DOCTOR OF PHILOSOPHY

to the

INDIAN INSTITUTE OF TECHNOLOGY DELHI

HAUZ KHAS, NEW DELHI-110016, INDIA

July 2004

Rock deformation

Dedicated

to

Shruti, Rishabh and Kriti

CERTIFICATE

This is to certify that the thesis entitled "**Rock Mass Modeling in Polyaxial Stress State**" being submitted by Mr. Rajendra Prasad Tiwari to the Indian Institute of Technology Delhi for the award of the degree of **DOCTOR OF PHILOSOPHY** is a record of the bonafide research work carried out by him. Mr. Tiwari has worked under my guidance for the submission of this thesis, which to my knowledge has reached the requisite standard.

The thesis or any part thereof has not been presented or submitted to any other University or Institute for any degree or diploma.

KSRao
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ACKNOWLEDGEMENTS

The research work presented in this thesis was carried out from July 2001 to July 2004 at the research group of rock mechanics, Department of Civil Engineering, Indian Institute of technology Delhi, India. During this period, many people have contributed to this work; to all of them the author would like to express his utmost gratitude.

First of all, the author wishes to express his sincere gratitude and indebtedness to his supervisor Dr. K. S. Rao of the Department of Civil Engineering I.I.T. Delhi for his noble guidance, ever-extending cooperation and enthusiastic support and specially, for his trust and interest in this work. It is because of his great care and tender treatment that this thesis could be brought to the present form within stipulated time frame.

The author is grateful to the authorities of Engineering College Rewa and Principal Secretary Technical Education and Personnel Training, Government of Madhya Pradesh for deputing him under the Quality Improvement Programme (QIP) to pursue this doctoral programme. Thanks are due to Government of India, for providing the author an opportunity to work under the QIP scheme. The author is thankful to his colleagues Dr. Sunil Kumar, Mr. D. K. Jain, Mr. Ravi Nagaich, Dr. S. K. Jain, Dr. A. K. Jain, Mr. P. Patel and other faculties at Engineering College Rewa for sharing his duties during his absence from institute.

The author is thankful to Professor T. Ramamurthy, Director, Ang Ron Geotech Pvt. Ltd. New Delhi for his valuable suggestions during testing programme. The author also expresses his sincere gratitude to Professor K. G. Sharma, Dr. K. K. Gupta, Dr. G. Benipal for their fruitful technical advises in analytical part of thesis. The author expresses his special thanks to Professor Bhawani Singh and Dr. Mahendra Singh at I.I.T. Roorkee for their fruitful discussions and suggestions during this research programme. The author is thankful to the faculty members of the Geotechnical Section, Civil Engineering Department, I.I.T. Delhi for the extended cooperation whenever needed during the programme. The author is grateful to authorities of Central Water Commission, New Delhi for permitting him to carryout numerical modeling part of research work at their computational laboratory. Special thanks to Dr. A. K. Dhawan, Director, Central Soil and Materials Research Station, Ministry of Water Resources, Government of India to provide financial support for fabrication of True Triaxial System (TTS). The facility extended by Mr. Jattinder Singh, Managing Director, Hydraulic Engineering Instrument Company, New Delhi during fabrication of TTS is duly acknowledged.

The author expresses his deep appreciation and sincere thanks to his fellow researchers Mrs. Neelima Satyam, Mr. Sandeep Trivedi, Mr. Chandan Kumar, Mr. Nityanand Singh Maurya, Mr. B.K. Singh and Mr. D. A. Hindoliya for their fruitful discussions and cooperations through out this programme. Thanks are due to the staff of the Geotechnical Engineering, Geology and Computational Laboratories especially Dr. M. M. Rao, Mr. D.S. Gossain, Mr. Lal Singh, Mr. Kailash, D. Biswas and others. Thanks are also to staff of Civil Engineering workshop and I.D.D. Center, I.I.T. Delhi for their cooperation in fabrication of Cutting system. The SEM and XRD facilities extended by Textile and Chemistry Departments for characterization of samples is greatly acknowledged. The help in preparation of tracings and drawings by Mr. N.L. Arora and Mr. Sunil Aneron is appreciated.

The author is always indebted to his parents, brothers and In-laws for their affection and support extended during difficult time of work. The author owes his dearest thanks to his lovely daughters Shruti, Kriti and son Rishabh for their love, patience and support during study. The acknowledgement will be remaining incomplete without expressing feelings towards the devotion and sacrifices made by his wife Manju Tiwari. She was always the main source of motivation and encouragement during whole tenure of research.

Rajendra Prasad Tiwari
2001RCE005

ABSTRACT

One of the major problems confronting designers of engineering structures in rock is that of estimating the strength and modulus of the rock mass in pre and post failure zones. The rock mass usually made up of an interlocking matrix of discrete blocks and failure of such rock mass is due to sliding along joint or rotation of block element. The stress conditions near ground surface are usually uniaxial i.e. σ_2 and σ_3 are negligible or zero whereas rock slopes and deep underground structures are always subjected to high confining pressures. In case of rock slopes, at the surface and in tunnel openings near the face, only σ_1 and σ_2 exist and the third axial, σ_3 stress is negligible or zero, so biaxial stress condition prevails. Further, at deep depths, rock slopes, foundations and tunnel openings or caverns experience all the three sets of stresses and triaxial or polyaxial condition is prominent.

The strength and deformational responses of rock mass can be estimated through numerical modeling, in-situ and laboratory large scale testing and physical model studies. The numerical methods need experimental results for their validity before they can be used and the insitu and laboratory methods are practically very difficult and time consuming. So, physical model studies are always preferred in such situations.

Few modest failure criteria are available in literature, which include the influence of joint geometry and intermediate principal stress. Most basic procedure to describe the deformational behaviour is stress strain curves. The available empirical and constitutive models in rock mechanics literatures considering the joint configuration and true triaxial stress state are not very large.

The studies on post failure response of rocks are sofar limited to few intact and fractured rocks only. The laboratory physical modeling needs testing equipments but most of the triaxial testing equipments available in literature are suitable only for soils and smaller size specimens of intact rock and as such their use is limited for testing rock mass in multiaxial stress conditions.

In the present study a high capacity (100 ton) true triaxial equipment was designed and fabricated with facility to apply, maintain and control the stress independently on large size prismatic faces of rock mass specimen during testing. The single joint model rock specimens were tested in polyaxial stress states by varying joint inclination and σ_2/σ_3 ratios. The models of jointed block mass having three orthogonal joint sets were also tested in the same stress states with different σ_2/σ_3 ratios. The joint geometry and axes of σ_2 and σ_3 were changed to account the influence of joint configuration and rotation of horizontal stresses. Based on detailed testing, the influences of intermediate principal stress on modes of failure and full stress-strain response in pre and post peak regions for block mass specimens were also investigated.

Based on extensive test results, a suitable failure criterion for rock mass was suggested in triaxial stress state as below:

$$\sigma_1 = \sigma_{cj} + B\sigma_3^a \qquad (1)$$

where, σ_3 is confining stress and B and a are joint geometry and material parameters respectively. The notations of parameters B and a can be replaced by B_m and a_m in case of rock mass and B_j and a_j for single joint rock. The σ_{cj} is the uniaxial compressive strength of jointed rock mass specimen. The σ_{cj} can be calculated, initially by carrying out a joint mapping at site and further using either RMR or Joint factor, J_f approaches.

The following equations are developed for the evaluation of parameters B_m and a_m and B_j and a_j

$$B_m = 5.94\left[\cos\left(\frac{2\pi}{9}-\theta\right)\right]^{-0.51} \tag{2}$$

$$a_m = 0.55\exp\left[0.41\cos\left(\frac{2\pi}{9}-\theta\right)\right] \tag{3}$$

$$B_j = 8.18\left[\cos\left(\frac{2\pi}{9}-\theta\right)\right]^{1.05} \tag{4}$$

$$a_j = 0.38\exp\left[0.49\cos\left(\frac{2\pi}{9}-\theta\right)\right] \tag{5}$$

where, θ is dip inclination of critical joint set in rock mass. The strength criterion developed iin polyaxial stress state is as below:

$$\tau_{oct} = D\sigma_{ci}\sigma_{oct}^{e} \tag{6}$$

where, τ_{oct} is the octahedral shear stress i.e. $[(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2]^{1/2}/3$ at the point when σ_1 attains its peak value $\sigma_{1,peak}$ and σ_{oct} is the mean normal stress acting on failure plane and is equal to $(\sigma_1+\sigma_2+\sigma_3)/3$. The D and e are joint material parameters and they are function of joint geometry. D and e systematically vary with joint configuration for both rock mass and single joint rock specimens. The notations of parameters D and e can be replaced by D_m and e_m in case of rock mass and D_j and e_j for single joint rock respectively. The σ_{ci} is the uniaxial compressive strength of model material. The criteria have been derived based on the assumption that joints are smooth (ϕ_j =36.8°) and the extent of interlocking level (s = 0.5) is medium.

The expression for criterion parameters is suggested as following:

$$D_m = 0.08\left[\cos\left(\frac{2\pi}{9}-\theta\right)\right]^{-0.8} \tag{7}$$

$$e_m = 0.59\exp\left[0.40\cos\left(\frac{2\pi}{9}-\theta\right)\right] \tag{8}$$

$$D_j = 0.09\left[\cos\left(\frac{2\pi}{9}-\theta\right)\right]^{-0.58} \tag{9}$$

$$e_j = 0.46\exp\left[0.58\cos\left(\frac{2\pi}{9}-\theta\right)\right] \tag{10}$$

In this study constitutive and empirical expressions are suggested for evaluation of deformation modulus in single joint rock and rock mass. A constitutive model was formulated for prediction of deformation behaviour of rock mass as under:

$$\frac{\Delta\varepsilon}{\Delta\sigma_1} = \frac{1}{E_j} = \frac{(1-2\nu K)}{kP_a\left(\frac{\sigma_3}{P_a}\right)^n} + \frac{1}{2}\sum_{i=1}^{N}\frac{\cos^2\theta_i}{S_i k_{n(i)}}[(1+K)+(1-K)\cos 2\theta_i] + \frac{\cos\theta_i \sin\theta_i}{S_i k_{s(i)}}[(1-K)\sin 2\theta_i] \tag{11}$$

where, K = ratio of principal stresses $=\frac{\sigma_3}{\Delta\sigma_1}$, ν is Poisson's ratio of intact material, E_i is initial modulus of elasticity of intact rock, P_a is atmospheric pressure equal to 0.101 MPa. k, n are parameters and can be determined from logarithmic plot of E_i/P_a against σ_3/P_a, S_i is spacing of joints in i^{th} set, σ_1 and σ_3 are major and minor principal stress, $k_{s(i)}$, $k_{n(i)}$ are shear and normal joint stiffness of joints in i^{th} set.

The expression to predict deformation modulus, E_{tm50} is derived as in Eqn. (12), which incorporates Janbu coefficients for rock mass, n_m K_m, intact rock strength, σ_{ci}, atmospheric pressure, P_a, confining stress, σ_3 and joint geometry parameters P, q.

$$E_{tm50} = \frac{PK_m P_a \left(\frac{\sigma_3}{P_a}\right)^{n_m}}{\left(\frac{\sigma_3}{\sigma_{ci}}\right)^q} \tag{12}$$

The expressions to predict Janbu's coefficients K_m, n_m and joint geometry parameters, P, q were derived as below:

$$\frac{K_i}{K_m} = 9.86\exp[2.16\cos(\theta_{min} - \theta)] \tag{13}$$

$$\frac{n_i}{n_m} = 0.29\exp[-0.97\cos(\theta_{min} - \theta)] \tag{14}$$

$$P = \exp[-0.019\theta] \tag{15}$$

$$q = 0.12\exp[0.018\theta] \tag{16}$$

The expressions for the prediction of modulus in UCS conditions are available in literature using joint factor concept for different failure patterns. The empirical expression for modulus of large block mass under triaxial compression is developed which incorporates strength, σ_{cj} and modulus, E_j of rock mass in UCS and confining stress, σ_3.

$$\frac{E_j(\sigma_3 = 0)}{E_j(\sigma_3)} = 1 - 0.93\exp\left[-0.087\left(\frac{\sigma_{cj}}{\sigma_3}\right)\right] \tag{17}$$

It was concluded from comparison of all three methods that the constitutive and Janbu's coefficients approaches are predicting modulus close to the experimental results. The choice of using a method for estimating deformation modulus is solely depends upon site conditions and availability of input parameters in the field.

Based on the extensive polyaxial test results, the expression of E_j in true-triaxial or polyaxial stress states ($\sigma_2 > \sigma_3$) was also suggested as below:

$$\frac{E_j(\sigma_2 > \sigma_3)}{E_j(\sigma_2 = \sigma_3)} = 1 + T\left(\frac{\sigma_2}{\sigma_3} - 1\right)^r \quad (18)$$

T and r are material and joint inclination parameters and vary with dip inclination, θ of joint set-I as given in Table 1.

Table 1 Parameters T and r at Different dip Inclination, θ

θ (°)	β (°)	T	r	r^2
0	90	0.31	0.78	0.59
20	70	0.07	2.06	0.87
40	50	0.98	0.55	0.91
60	30	1.62	0.9	0.82
80	10	0.29	0.67	0.62
90	0	0.1	1.05	0.62

The post failure studies on models are also conducted and the results show the strain softening, strain hardening and plasticity behaviour in rock mass depending upon joint configuration and surrounding stress state. The expressions for estimation of post peak modulus are being formulated which are very important in longwall mining and designing of mine pillars. Lastly, the post peak behaviour of rock mass is summarized in a zonation table indicating different zones viz. strain softening, strain hardening and plastic.

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