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Evaluation of In-situ Stresses in Rock Mass: Challenges and Applications in Hydropower Development

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ABSTRACT

All the rocks are under stress and strain because of geologic and tectonical history. During excavation, the stress distribution takes place and stress state may change dramatically. Understanding the state of stress in rock mass is required for the design of structures in or on the rock mass. Large stresses may cause rock bursts, squeezing or deformation/convergence of roof and walls of underground structures or subsidence on ground surface. Failure in underground caverns or tunnels may be termed as structurally or stress controlled. Structurally controlled failures occur at shallow depth involving falling of wedges at the crown or sliding out of the side walls, whereas stress controlled failures dominate at depth where the rock mass is weak, relatively homogeneous and isotropic. In-situ stresses are used to optimize the shape, size, orientation and rock supporting systems for underground tunnels and caverns. The importance of stresses in hydropower development and limitation of stress measurement techniques have been elaborated in this paper. Five case histories have been cited.

Keywords: Tectonic stresses; Hydraulic fracturing tests; Overcoring; Vertical stress; Horizontal stress; Pressure tunnels; Steel liner.

1. INTRODUCTION

In-situ stresses in rock influence the stability of underground structure to a great extent. Most types of failures in underground structures are related to the prevailing in-situ stress. Stress is a point property and varies with location. It is a tenser quantity and contains magnitude as well as direction in three-dimensional space. Even though a number of direct and indirect techniques are available, accurate determination of primary in-situ stress field is still not possible. At the most, it can be estimated to a certain degree of accuracy depending on the type of method used. Amadei and Stephansson (1997) based on the experiences apprehended that '*rock stresses cannot be determined accurately due to complexities in rock and rock masses. At best, and in good to very good rock conditions where the rock is essentially linearly elastic, homogeneous and continuous, and between well-defined geological boundaries, rock stresses can be determined with an error of ± 10-20% for their magnitude and an error of ± 10-20^0 for their orientation. On the other hand, in poor (weathered, weak, soft and heavily fractured) quality rocks, the measurement of rock stresses is extremely difficult. In such rocks the success rate of stress measurements is usually low*'. Practical experience of author while dealing with the subject showed similar difficulties. Experiences with hydraulic fracturing tests indicated variation in test results even in close proximity. The magnitude of stresses from two tests conducted in the same drillhole and within 2-3 m distance may vary by 20-30%. In such circumstances, defining the stresses by a unique value is a big challenge. This paper describes the challenges, issues and implications pertaining to measurement of in-situ stresses in rock and its applications in design of structures for hydropower development.

2. THEORIES

At any point inside the rock mass, weight of overlying rock mass is the major source of in-situ stresses. Apart from overlying rock mass, there are numerous factors that affect the magnitude and direction of the in-situ stress field. Stress is a second rank tensor in terms of continuum mechanics and can be described by nine components (Eq. 1).

$$S = \begin{vmatrix} S_{11} & S_{12} & S_{13} \\ S_{21} & S_{22} & S_{23} \\ S_{31} & S_{32} & S_{33} \end{vmatrix} \tag{1}$$

In equilibrium conditions $S_{12}=S_{21}$, $S_{13}= S_{31}$ and $S_{23} = S_{32}$. Therefore, for complete information of in-situ stresses, one must estimate three magnitudes and their orientations with reference to co-ordinates system.

While dealing with projects near the earth's surface, measurements of in-situ stresses are absolutely necessary because stresses near the surface of earth are affected by a number of non-tectonic processes. The World Stress Map (WSM) project carried out under the auspices of the International Lithosphere Project (Zoback, 1992) was completed in July 1992, considering the effect of tectonic stresses based on the stress measurements at a depth of 100 m or more, involving over 30 scientists from 18 countries and the same has been updated several times. Stress Maps based on the world stress map Database Release 2016 is available online currently (Heidbach et al., 2016; Heidbach et al., 2018).

The vertical stress at any element in rock can be estimated from the relationship, $\sigma_v = \gamma z$, where γ is average unit weight of overlying rock and z is the depth of element from surface. Measurements of vertical stress at various mining and civil engineering sites around the world confirm that this relationship is valid although, as illustrated in Fig. 1 (Brown and Hoek, 1978), but there is a significant amount of scatter in the measurements particularly at shallow depth.

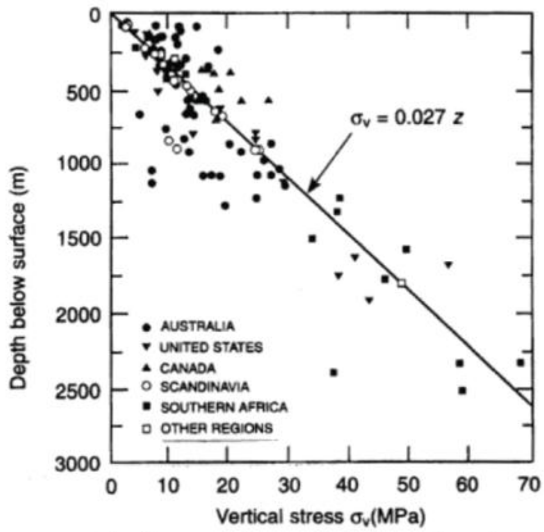

Fig. 1 - Vertical stress measurements from mining and civil engineering projects around the world. (After Brown and Hoek, 1978)

Earlier, hydrostatic stress field was assumed in earth's crust similar to liquids (Heim 1878). The theory was replaced by lithostatic stress field theory with isotropic state assumption which means:

$$\sigma_H = \sigma_h = \sigma_v = P_c \tag{2}$$

where σ_H and σ_h are maximum and minimum horizontal stresses, σ_v vertical stress and P_c the confining pressure.

In the absence of stress measuring techniques, lithostatic stress theory was used for the design of underground structures. Though vertical stress may be estimated based on overburden, but it is difficult to estimate the horizontal stress. Lithostatic stress theory was later replaced by uniaxial strain model (Terzaghi and Richart, 1952). The concept assumes the rock to be confined at depth and horizontal expansion is restricted. Ideally, the principal strains $\varepsilon_1 \neq 0$, $\varepsilon_2 = \varepsilon_3 = 0$, makes the horizontal stress a function of the weight of the overburden and the Poisson's ratio only. The maximum horizontal stress to vertical stress ratio (σ_H/σ_v) is denoted by the letter k:

$$\sigma_H = \sigma_h = \rho g z \left[\frac{v}{1-v}\right] = k\sigma_v \tag{3}$$

Where ρ is integrated rock density of the overburden, g is gravitational acceleration, ν is the Poisson's ratio, Z is depth at the point of interest, σ_v is the vertical stress.

As illustrated below, major principal stress is vertical stress. For ν = 0.25, the stresses in horizontal direction derived from Equation (3) shall be as follows:

$$\sigma_H = \sigma_h = 0.33\ \sigma_v \tag{4}$$

Horizontal stresses were observed to be higher than the vertical stress due to overburden. All the orogenic and tectonic events with large horizontal movements in the earth's crust were connected with continental drift hypothesis (Wegener, 1915). Hast (1958) found that horizontal stresses often exceeds by 1.3 to 1.5 times the vertical stress and in extreme cases it goes upto 8 times. Dewey (1972), Mckenzie and Sclater (1973), Courtillot and Vink (1983) further corroborated and verified the plate tectonics hypothesis by contributing substantial evidence about the driving forces between continents. Actual measurements of stresses suggested that 'k' is very high at shallow depth and it decreases with depth (Brown and Hoek, 1978; Herget, 1988). Near surface, stresses are affected by vertical stress relief, topography and fracturing.

An elasto- plastic thermal stress model considering the curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle was developed by Sheorey (1994). For estimating the horizontal to vertical stress ratio (k), the following equation was proposed:

$$k = 0.25 + 7E_h \left[0.001 + \frac{1}{z}\right] \tag{5}$$

Where z is depth below surface in m, E_h (GPa) is the average deformation modulus of the upper part of earth's crust measured in horizontal direction.

Whyatt (2001) attempted back calculation of load sources in naturally varying stress field in rock based on the stress measurements.

3. MEASUREMENT AND EVALUATION TECHNIQUES

Direct and indirect methods of evaluation of in-situ stresses do exist. Hydraulic methods include hydraulic fracturing (HF), sleeve fracturing and hydraulic tests on pre-existing fractures (HTPF) whereas relief methods may be categorised as surface relief methods, undercoring, overcoring, relief of large rock volumes (bored raise, under excavation techniques). Flat/curved jack is another method used for the determination of in-situ stresses and this method can be used for determination of deformation modulus of rock too. But, this method is out of use now due to its limitations. The other methods include strain recovery, borehole breakdown. Indirect method based on acoustic emission theory and the Kaiser Effect, can also be used for estimating the in situ stress field in rock masses. Of these, relief and hydraulic methods are widely used for the assessment of in-situ stresses. All the methods have their own limitations as these are based on certain assumptions.

Hydraulic fracturing is the only direct method available which can be used for measurement of stresses at great depths prior to excavation. Different methods of interpretation exist for the evaluation of stresses using the data from hydraulic fracturing tests. These methods have their own limitations. In hydraulic fracturing tests, shut-in pressure is interpreted from the plots of pressure-time and flow rates. Suggested method for hydraulic fracturing (ISRM, 2007), contains certain assumptions briefly described below:

- There is no theoretical limit to depth of measurement provided a stable borehole can assess the zone of interest and the rock mass is elastic and brittle.
- Classical interpretation of hydraulic fracturing test is possible only if borehole axis is parallel to one of the principal stresses.
- Principal stress directions are derived from the fracture delineation on the borehole wall under the assumption that fracture attitude persists away from the hole.
- Rock mass is linearly elastic, homogeneous and isotropic

Further assumptions are involved in hydraulic tests on pre-existing fractures (HTPF) in additions to above. There are 3-4 methods of interpretation listed in ISRM suggested method and it has been advocated to use more than one method for obtaining the critical shut-in pressure. In deep drillholes (> 2 km), the fracture initiation may not be possible with the capacity of the available equipment (Franquet et al., 2011; Mishra, 2016). The created fracture is likely to span multiple formations bearing different stresses, thus only providing average estimate over these formations (Bérard et al., 2019). Development of micro hydraulic fracturing technique with short section of drillhole and small injection rates may minimise these problems (Haimson, 1993; Haimson and Cornet, 2003).

Due to practical restrictions, relief methods are limited to shallow depths wherein stresses are influenced by a number of natural and unnatural factors. For example, excavation techniques may alter the primary stress field. Ideally, the stresses have to be measured beyond the zone of influence and in most of the time, it is not feasible due to limitations of the testing and drilling equipment. It is difficult to demarcate the zone of influence too. The vibrations caused by drilling may also alter the state of stress.

Due to economy and convenience, acoustic emission method is widely used to estimate the rock stress particularly in mining and oil fields (Kang et al., 2018). Lehtonen et al. (2012) described the limitations of Kaiser effect and its suitability for estimation of stresses and concluded that method can be successful if it is supported by key geological and other stress measurement information.

4. DESIGN OF UNDERGROUND CAVERNS

In-situ stresses play a vital role in design of underground tunnels and caverns. The magnitude of stress is used to design the rock supports whereas the direction of horizontal stress is used for recommending favourable orientation of the cavern for safe and economic construction. Unfavourable direction may result in large deformations or convergences leading to excavation and stability related problems. Therefore, measurement of stresses is extremely important for safe and economic design of any tunnelling project. Dev et al. (2016) demonstrated the application of stresses in finalising the orientation of underground caverns. The following points needs to be accounted for while designing any underground tunnel or cavern:

- Ideally, long axis of the cavern should be oriented perpendicular to the minimum horizontal stress direction. This will lead to minimum stresses acting on the walls of the cavern resulting in lesser deformations/convergence, thereby needing lesser rock supports.
- To minimise possibilities of potential wedges, long axis of the cavern may be aligned perpendicular to the strike of bedding plane. Wedge analysis can be adopted to suggest favourable orientation with minimum wedge formation. In jointed rock mass, cavern can be reoriented or relocated to avoid wedge formations particularly in the crown section.
- At times, the orientation based on the (determined) major principal stress direction and the wedge analysis of the structural discontinuities are at obtuse angle or even at right angle. In such a situation, it becomes very difficult to finalise the best orientation of the cavern. In case of disagreement, the following criteria needs are suggested:
  - In case the stresses are not of very high order, orientation based on discontinuities may be preferred.
  - In case of high stresses, orientation should be finalized in the direction of maximum horizontal stress.
  - Numerical modelling tools can be used to arrive at balance between the two criteria.
- The shape of the cavern should be selected to avoid stress concentrations. For example, a slight curvature in walls may be helpful in reducing the wall convergence considerably. Optimization of curvature in crown to create arching effect helps in distribution of loads on larger area and providing stability.
- Case histories of bursting of pressure tunnels have been reported in the literature. Such failures are related to internal water pressure and in-situ stresses. Therefore, minimum stress at any point along the pressure tunnel alignment should always be greater than the internal water pressure to avoid hydraulic fracturing. Hence, while locating the pressure tunnels, vertical as well as lateral rock cover are needed to be adequate.

Norwegian criteria developed for preliminary design of unlined pressure tunnels is shown in Fig. 2 and given in Equations 6 and 7:

$$h > \frac{H \times \gamma_w}{\gamma_r \times Cos\alpha} \qquad (6)$$

$$L > \frac{H \times \gamma_w}{\gamma_r \times Cos\beta} \qquad (7)$$

Where, h is vertical depth of the point of study (m), L is the shortest distance between the surface and the point of study (m), α is the inclination of the pressure tunnel or shaft, β is the average available inclination of the valley side, H is the static water head (m), Υ_r is is the specific weight of rock and Υ_w is the specific weight of water.

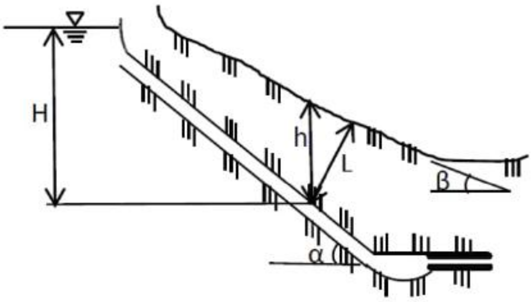

Fig. 2 - Norwegian criteria for minimum rock cover

Berg-Christensen and Dannevig (1971) modified the above equation incorporating factor of safety (FS) as given in Equation 8. Factor of safety of 1.1 is suggested.

$$L \geq \frac{FS \times H \times \gamma_w}{\gamma_r \times \cos\beta} \tag{8}$$

Norwegian criterion considers topography only and it does not take into account the rock mass geological conditions, water conditions etc. Hence, Brekke and Ripley (1987) proposed criteria for providing steel liner in terms of minimum stress (σ_3) as expressed in Eq. 9.

$$\sigma_3 > FS \times H \times \gamma_w \tag{9}$$

5. APPLICATION OF STRESS MEASUREMENTS

5.1 Pressure Tunnel Design

Pressure tunnels need to be designed for internal water pressure with a condition that minimum external stress due to rock load vertically as well as laterally is always greater than internal water pressure. The design process involves sharing of water pressure jointly by the lining system and the adjacent rock. Selection of concrete (plain/reinforced) or steel lining depends on the internal water pressure and the competency of adjacent rock. Major part of the water pressure is borne by the lining system and adjacent rock bears a part of it. Hence, knowledge of in-situ stresses and modulus of rock is important in deciding the choice and design of lining system. In case of high stresses or incompetent rock or both, steel liners are provided. The thickness of the lining depends on the pressure to be borne by it.

Case Study 1: In one of the hydropower projects in the Himalaya in India, steel liners with reduced diameter was recommended in 780 m length of head race tunnel in a zone where it crosses a drain with 9 m rock cover only. Dynamic internal water pressure was of the order of 2.74 to 2.88 MPa. Stress measurements in horizontal and vertical drill-holes were performed to evaluate the minimum stress for optimizing the length of steel liner. The steel lining was restricted in the section where minimum stress (σ_3) was less than the internal water pressure (dynamic). It helped in saving of material, time, cost, human efforts; thus economizing the design process. Through optimization, length of steel liner was reduced to 710 m in length (Fig. 3), thus avoiding 70 m length of steel liner. Additionally, 140.4 m long bye pass adit proposed for erection of steel liners could also be averted.

Case Study 2: Failure of a water conveyance tunnel for the hydropower development in India is an example of tunnel bursting (Sharma, 1994). The water conveyance tunnel was designed for internal water pressure of 1.4 MPa. Part of tunnel was provided with steel (60 m) and RCC lining (70 m) between surge shaft and penstocks whereas plain concrete lining was provided in the remaining length. During pressure testing of the tunnel while commissioning of the project, RCC section of the tunnel cracked open and stream of water gushed out of the sloping hill mass. Maximum pressure in the tunnel was reported as 0.7 MPa. The cracking was observed in entire section of RCC lined tunnel and these cracks extended upto 30 m in plain concrete lining also. On investigation, a sub-vertical joint in the rock was seen through the crack in the concrete lining. As remedial measure, based on the concept of minimum rock cover and competency of rock mass, the steel lining was extended to a length of 240 m beyond. The failure of pressure tunnel demonstrated the need for stress measurements for locating the pressure tunnel.

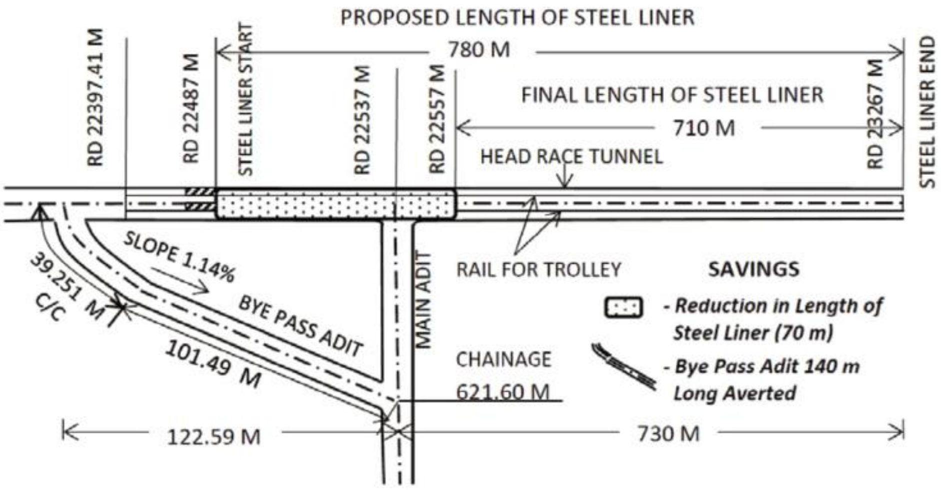

Fig. 3 - Optimization of steel liner in head race tunnel

5.2 Design of Grout Pressure

Weak and jointed rock masses require strengthening by grouting the surrounding mass. In no case, the maximum grout pressure should be greater than minimum principal stress. In jointed rocks, the excessive grout pressure may result in opening of the existing joints and may create new fractures too. Hence, the grout pressure should always be less than the reopening pressure of the rock joints. For safety, the grout pressure should be restricted to plug the open joints for improving the deformation modulus rather than damaging it further.

Cast Study 3: In a 260 m high earth and rockfill dam in India, consolidation and curtain grouting in dam foundation was resorted to improve the stress-deformation characteristics of weak phyllites rock mass. Hydraulic fracturing technique played an important role in finalisation of maximum grout pressure. The design grout pressure was of the order of 0.2 to 0.4 MPa for consolidation grouting and 1.0 to 1.2 MPa for curtain grouting. Re-opening pressure of the existing joints was determined using hydraulic fracturing method. Using hydraulic fracturing equipment, minimum reopening pressure of rock joints was found to be 1.73 MPa with an average value of 2.38 MPa. The recommended design grout pressure for consolidation and curtain grouting being less than the minimum value of reopening pressure of rock joints was adjudged to be safe and thus adopted.

5.3 Design of Caverns

Criteria based on rock mass discontinuities and in-situ stresses are used in geometric design and orientation of the underground caverns. Sometimes, not always, stresses are influenced by local geological factors too. As stress is a point property, data from nearby areas or projects cannot be used for the design of any underground structure. It has to be measured to be sure. The in-situ stress data is necessary for the geometric as well as engineering design of the underground caverns.

Case Study 4: A special phenomenon was observed in one of the hydro power projects wherein in-situ stress measurements were carried out using hydraulic fracturing tests in the proposed desilting chamber and power house caverns approximately 10 km apart. The magnitude as well as direction of the stresses changed drastically. The maximum and minimum horizontal stresses in proposed desilting chamber cavern were of the order of 6.52 MPa and 5.02 MPa with intermediate vertical stress of 4.28 MPa (estimated), whereas in the power house complex, the corresponding values were found to be 4.02 MPa and 2.99 MPa, but vertical stress estimated from overburden was 5.17 MPa. Hence, vertical stress was more than the horizontal stresses in power house complex. Stress ratio, k (σ_H/ σ_v) was found to be 1.52 in desilting chambers and it was 0.78 in power house complex. The variation of 57^0 (N49^0W to N17^0E) was also observed in interpreted maximum horizontal stress direction at both sites. The main reason of variation in magnitude and direction of stresses within close proximity was attributed to the fact that continuous rise in hill slope was observed at the desilting chambers site whereas apart from flowing river on the one side, a water stream was observed on the downstream side of the hill in which power house cavern is situated. Thus, making it an isolated ridge and the phenomenon of stress release was noticed. Such experiences advocate the necessity of actual measurements rather than estimating.

5.4 Design of Steel Lining

Stress measurement also helps in design of steel liners for pressure shafts. RCC or steel lining may be required in certain sections of pressure tunnels with low rock covers. The knowledge and variation of stresses in the zone under consideration shall help in optimization of tunnel/shaft lining. In the case of high head pressure shafts, the thickness of steel lining can be varied with increase in static head along its alignment.

Cast Study 5: The stress measurements were carried out in pressure shafts for a project in Bhutan with static water head of 860 m to optimize the thickness of steel liners. Two pressure shafts of more than a kilometre in length at an inclination of 52^0 and 55^0 with horizontal are part of the water conveyance system. Stress measurements were carried out at various elevations along the alignment of pressure shafts and the results were used to optimize the thickness of steel liners. Variable thickness of steel liner and stiffeners was adopted in different sections. Variation in maximum horizontal stress along the pressure shaft alignment is shown in Fig. 4. As the elevation decreases, the rock cover above the measured location increases and thus the maximum horizontal stress also increases (Fig. 4).

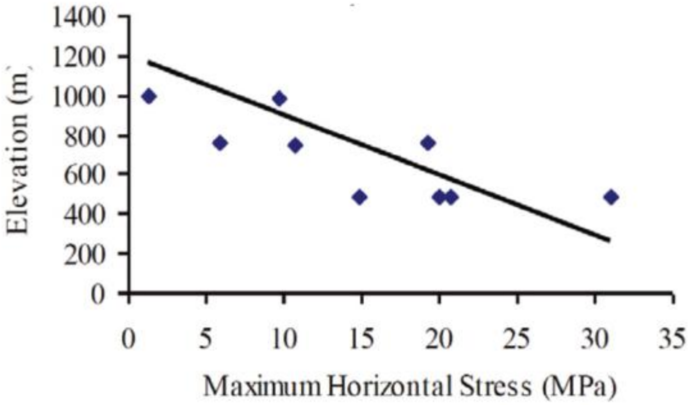

Fig. 4 - Variation in In-situ stresses along the pressure shaft alignment

Steel lining with 30, 34 and 38 mm thick steel plates conforming to ASTM-A-537 Cl-II in the upper part of the shafts (EL 1259 m to EL 1056 m) were provided whereas plates of 38 mm and 40 mm thickness conforming to ASTM-A-517 Grade F were provided in the lower part of the shafts (EL 1056 m and below upto EL 499 m) and the penstocks. Stiffeners conforming to the same material were also provided to restrict the thickness of lining to 40 mm. The sickle plates and ring girders of bifurcations were fabricated from 96 mm and 118 mm thick plates.

6. CONCLUSIONS

In-situ stress varies with the location and its measurement is absolutely necessary for planning and designing any structure within a rock mass.

Weight of overlying rock and locked in stresses of tectonic origin are the major causes of in-situ stress. Stresses are affected by local topographical features also. Near surface and at shallow depths, stress ratio (k) is high and it reduces at greater depths. Vertical stress relief of rock mass may be attributed to high stress ratio near surface and at shallow depth.

The stresses have great influence on the stability of any underground structure, but it cannot be accurately assessed using the available measuring techniques. It can be inferred to certain degree of accuracy. Moreover, indirect methods do not take into account the influence of all the factors responsible for stress state. Therefore, determination of in-situ stresses becomes absolutely necessary for economic and safe design of hydraulic structures, as proved by 5 case histories.

References

Amadei, Bernard and Stephansson, O. (1997). Rock Stress and its Measurements, Chapman and Hall, London.

ASTM A 517/A 517M. Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered.

ASTM A 537/A 537M. Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Manganese-Silicon Steel.

Bérard, Thomas; Chugunov, Nikita; Desroches, Jean and Prioul, Romain (2019). Feasibility and Design of Hydraulic Fracturing Stress Tests Using a Quantitative Risk Assessment and Control Approach, Petrophysics, Vol. 60, No. 1 (February 2019), 19, pp.113 -135.

Berg-Christensen, J. and Dannevig, N.T. (1971). Engineering Geological Considerations Concerning the Unlined Pressure Shaft at the Mauranger Power Project, Unpublished report, Geoteam A/S, Oslo, Norway.

Bergh-Christensen, J. and Dannevig, N.T. (1971). Engineering Geological Evaluations of the Unlined Pressure Shaft at the Mauranger hydropower Plant Technical Report, Geoteam A/S, Oslo, Norway.

Brekke, T.L. and Ripley, B.D. (1987). Design Guidelines for Pressure Tunnels and Shafts, EPRI, Research Project 1745-17, AP-5273, Final Report, 160 pages.

Brown, E.T. and Hoek, E. (1978). Trends in Relationships between Measured Rock In Situ Stresses and Depth, Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., 15, pp.211-215.

Courtillot V. and G.E. Vink (1983). How Continents break-up, Sci. Am., 249, pp. 42-49.

Dewey, J.F. (1972). Plate Tectonics, Scientific American, 226, pp. 56-68.

Franquet, J., Gaz, I., Ibragimov, F., Seryakov, A., and Huthwaite, J. (2011). Caprock Integrity Testing Through Openhole Micro- Fracturing in Caspian Sea Sub-Salt Formations, Paper 2011-062 presented at the 10th Offshore Mediterranean Conference and Exhibition, Ravenna, Italy, 23–25 March.

Haimson, B.C. (1993). The Hydraulic Fracturing Method of Stress Measurement: Theory and Practice, Chapter 14, in Hudson, J., Editor, Comprehensive Rock Engineering, 3, 395–412, Pergamon Press. DOI: 10.1016/B978-0-08-042066-0.50021-5.

Haimson, B.C. and Cornet, F.H. (2003). ISRM Suggested Methods for Rock Stress Estimation - Part 3: Hydraulic Fracturing (HF) and/or Hydraulic Testing of Pre-existing Fractures (HTPF). Int. J Rock Mech. Min. Sci. 40:1011–1020.

Hast, N. (1958). The measurement of Rock Pressure in Mines, Sver. Geol. Under. Ser. C. 52, pp.1-152.

Heidbach, O., Rajabi, M.; Cui, X.; Fuchs, K.; Müller, B.; Reinecker, J.; Reiter,K.; Tingay, M.; Wenzel, F.; Xie, F.; Ziegler, M. O.; Zoback, M.L. and Zoback, M. D. (2018). The World Stress Map Database Release 2016: Crustal stress pattern across scales, Tectonophysics, 744, 484-498, doi:10.1016/j.tecto.2018.07.007

Heidbach, Oliver; Rajabi, Mojtaba; Reiter, Karsten; Ziegler, Moritz (2016). World Stress Map Database Release 2016. GFZ Data Services, doi:10.5880/WSM.2016.001

Heim, A. (1878). Untersuchungen über den Mechanismus der Gebirgsbildung, 2 vols, Basel.

Herget, G. (1988). Stresses in Rock, Rotterdam: Balkema.

ISRM (2007). The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006, International Society for Rock Mechanics, Edited by R. Ulusay and J.A. Hudson.

Kang, Zhao; Gu, Shuijie; Yan, Yajing; Zhou, Keping; Qiang Li and Zhu, Shengtang (2018). A Simple and Accurate Interpretation Method of In Situ Stress Measurement Based on Rock Kaiser Effect and Its Application, Geofluids, Volume 2018, Article ID 9463439, 13 pages, https://doi.org/10.1155/2018/9463439.

Lehtonen, A., Cosgrove, J.W., Hudson, J.A. and Johansson, E. (2012). An Examination of In-situ Rock Stress Estimation using the Kaiser Effect, J. of Engineering geology, ISSN 0013-7952, DOI:10.1016/j.enggeo.2011.09.012.

Mckenzie, D.P. and Sclater, J.C. (1973). The Evolution of the Indian Ocean, Sci. Amer. 228(5): 63-72.

Mishra, V.K., Guzman, R., Rylance, M., Carvajal, E.I., Castiblanco, I., Cañas, J., Garcia, G., Dumont, H., Rubio, N., Kayo, A. and Alatrach, S. (2016). Wireline Microfracturing: Ultrahigh-Value Stress Measurements at Ultrahigh Pressures in an Ultradeep Lower Tertiary Play, Paper OTC-27091 presented at the Offshore Technology Conference, Houston, Texas, USA, 2–5 May. DOI: 10.4043/27091-MS.

Sharma, V.M. (1994). Cases of Tunnel Failures in Two Hydro-Electric Projects, XIII ICSMFE, 1994, New Delhi, India.

Sheorey, P.R. (1994). A Theory for In-situ Stress in Isotropic and Transversely Isotropic Rock, Rock Mech. Min. Sci. and Geotech. Abstr., 1994, 31(1), pp. 23-34.

Terzaghi, K. and Richart, F.E. (1952). Stresses in rock about cavities, Geotechnique 3, 57-90.

Wegener, Alfred (1915). The Origin of Continents and Oceans, Book published in 1915.

Whaytt, J.K. (2001). A Method for Modeling Variation of In Situ Stress Related to Lithology, DC Rocks 2001, The 38th U.S. Symposium on Rock Mechanics (USRMS), 7-10 July, Washington, D.C.

Zoback, M.L. (1992). First and Second Order Patterns of Stress in the Lithosphere: The World Stress Map Project, Journal of Geophysical Research 97, 11703-11728.