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Geotechnical Challenges and Solutions in the Construction of Underground Powerhouse with Shallow Basalt Rock Cover: A Review of Sardar Sarovar Narmada Project Case Study

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Abstract: The Sardar Sarovar Narmada Project in Gujarat, India, provides a critical case study of underground powerhouse construction, completed in 2003, under challenging geological conditions, specifically with a shallow basalt rock cover. This paper begins by detailing the assessment of the geotechnical challenges encountered during the excavation of a 58-meter-high powerhouse chamber, including the evaluation of rock mass properties and in-situ stress using flat jack and hydro-fracture tests. These assessments identified significant stress redistribution and distress in the rock mass, leading to cracks aligned parallel to the cavern's longer axis, which impacted critical structures such as pressure shafts and bus galleries. To address these challenges, a comprehensive engineering geological analysis was conducted, employing modern instruments like DEMAC gauges to monitor shear zones and crack development. Advanced analytical methods, including the 3-D Finite Element Method (FEM) and the 3-D Distinct Element Code (DEC), were utilized to assess the cavern's behavior under the identified stresses. This study underscores the critical importance of detailed geotechnical analysis and the careful selection of support systems for ensuring the stability of excavations in challenging environments like shallow basalt cover. The findings highlight the value of adaptive support systems and comprehensive geotechnical assessments in managing stress relief and deformation in similar geological conditions. Although the results are based on the specific case of the Sardar Sarovar Narmada Project, they provide valuable insights for the design and construction of underground powerhouses in jointed rock masses with shallow cover. These insights contribute to the broader geotechnical literature by offering practical guidance for similar underground construction projects. Future research should apply these lessons across a range of geological settings and rock types, refining and generalizing design practices to enhance construction strategies for underground caverns in various geotechnical contexts.

Keywords: Shallow Rock Cover, Stress Redistribution, Crack Monitoring, DEMAC Gauges, Finite Element Analysis.

1. Introduction

The underground powerhouse of the Sardar Sarovar Narmada Project was completed in 2003 and began operations shortly thereafter. The construction of underground powerhouses presents significant geotechnical challenges, particularly when the underlying rock mass exhibits jointing and shallow cover. Accurate assessment and classification of the rock mass are crucial for designing stable underground structures. The physical and engineering geological properties of rocks are influenced by its origin, diagenetic, metamorphic, and tectonic history, as well as weathering processes [1]. Key parameters such as orientation, location, persistence, water pressure, wall strength, degree of weathering, type of infilling material, and shear strength of critical discontinuities are vital for the quantitative evaluation of rock mass conditions.

Historically, various rock mass classification schemes have been developed to assist in the design and support requirements for underground excavations. [2] pioneered an empirical approach for tunnel design, while [3] utilized descriptive classification to estimate rock loads. Subsequent advancements include [4] stand-up time assessment, the Rock Quality Designation (RQD) index by [5], and the Geomechanics Classification or Rock Mass Rating (RMR) system developed by [6]. These classifications have been refined over time through extensive case histories and modifications [7–11].

In particular, the RMR system, alongside the Tunneling Quality Index (Q) proposed by [8], has been widely utilized in assessing rock mass stability and designing support systems. [12] further developed the Geological Strength Index (GSI) for very poor rock conditions, which complements the RMR and Q systems.

The Sardar Sarovar Narmada Project in Gujarat, India, provides a case study of the challenges encountered during the construction of an underground powerhouse in jointed basalt rock with shallow cover (Fig. 1). Excavation of the 58-meter-high machine hall (powerhouse cavern) revealed significant issues due to stress relief from low confining stress. This stress relief led to the formation of cracks aligned parallel to the longer axis of the cavern, particularly in the pressure shafts and bus galleries. These problems are similar to those found in very steep slopes of hard, jointed rock masses, where insufficient support leads to vertical cracks parallel to the excavation walls. [13].

Initial design supports, including 6 to 7.5-meter-long rock bolts, proved insufficient for restraining rock mass deformation. The supports were insufficient in preventing crack formation, suggesting that the rock mass did not perform as a stable structural material with the original support system. As a result, corrective actions were taken, such as the installation of longer rock bolts (12 meters) and cables (ranging from 10.5 to 32 meters) [2].

This paper aims to review the geotechnical problems encountered during the construction of the underground powerhouse at the Sardar Sarovar Narmada Project and evaluate the effectiveness of the treatment methods applied. The objectives of the study are:

1. Analyzing the adequacy of rock bolt and cable lengths in relation to stability requirements.

2. Evaluating the performance of initial support designs and comparing them with other hydroelectric projects.

3. Investigating discrepancies in horizontal stress measurements and their implications for support design.

4. Assessing the long-term stability of the cavern and the effectiveness of the roof and side wall support systems.

By addressing these objectives, the study seeks to provide valuable insights into the challenges and solutions associated with constructing underground powerhouses in similar geological conditions.

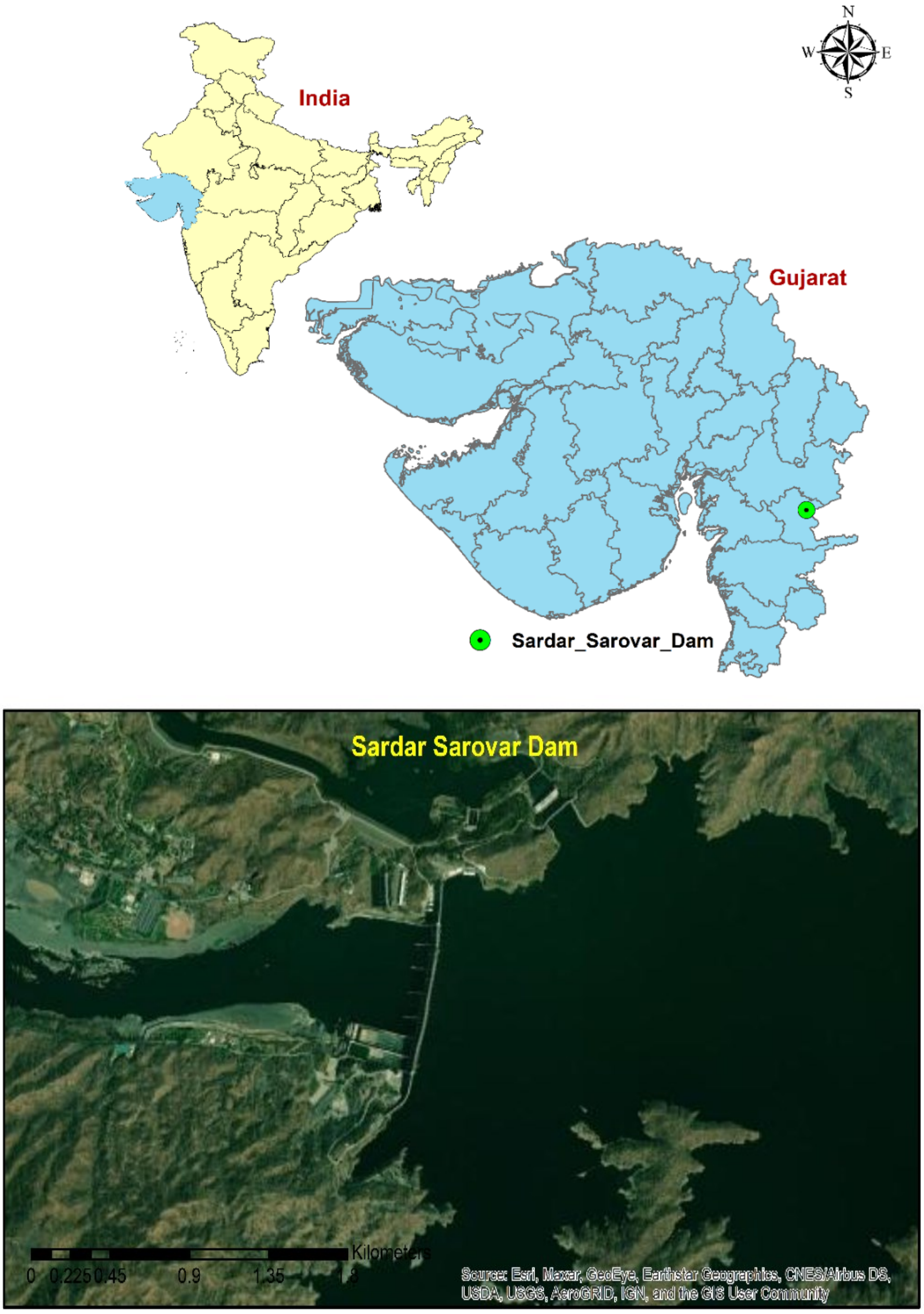

Fig. 1. Location and Layout Map of Sardar Sarovar (Narmada) Dam and Powerhouses

2. Location and Layout of Underground Powerhouse

The underground powerhouse is located 160 meters downstream from the dam on the right bank, within basaltic rock formations that have been intersected by dolerite dykes and sills. (Fig. 2). The powerhouse cavern has been strategically located between vertical and inclined dolerite dykes, which act as seepage barriers for the Rock Fill Dam-I reservoir water to the north and for the main river water to the south.

The machine hall (cavern) is aligned in a N10°E-S10°W direction, with the longer axis of the powerhouse cavern almost parallel to the direction of maximum horizontal stress (approximately N5°E). Major shear zones intersect the longer axis of the cavern. The access tunnels and draft tube tunnels are aligned across the major discontinuities, whereas the exit tunnels are aligned parallel to the Akkalbar Fault

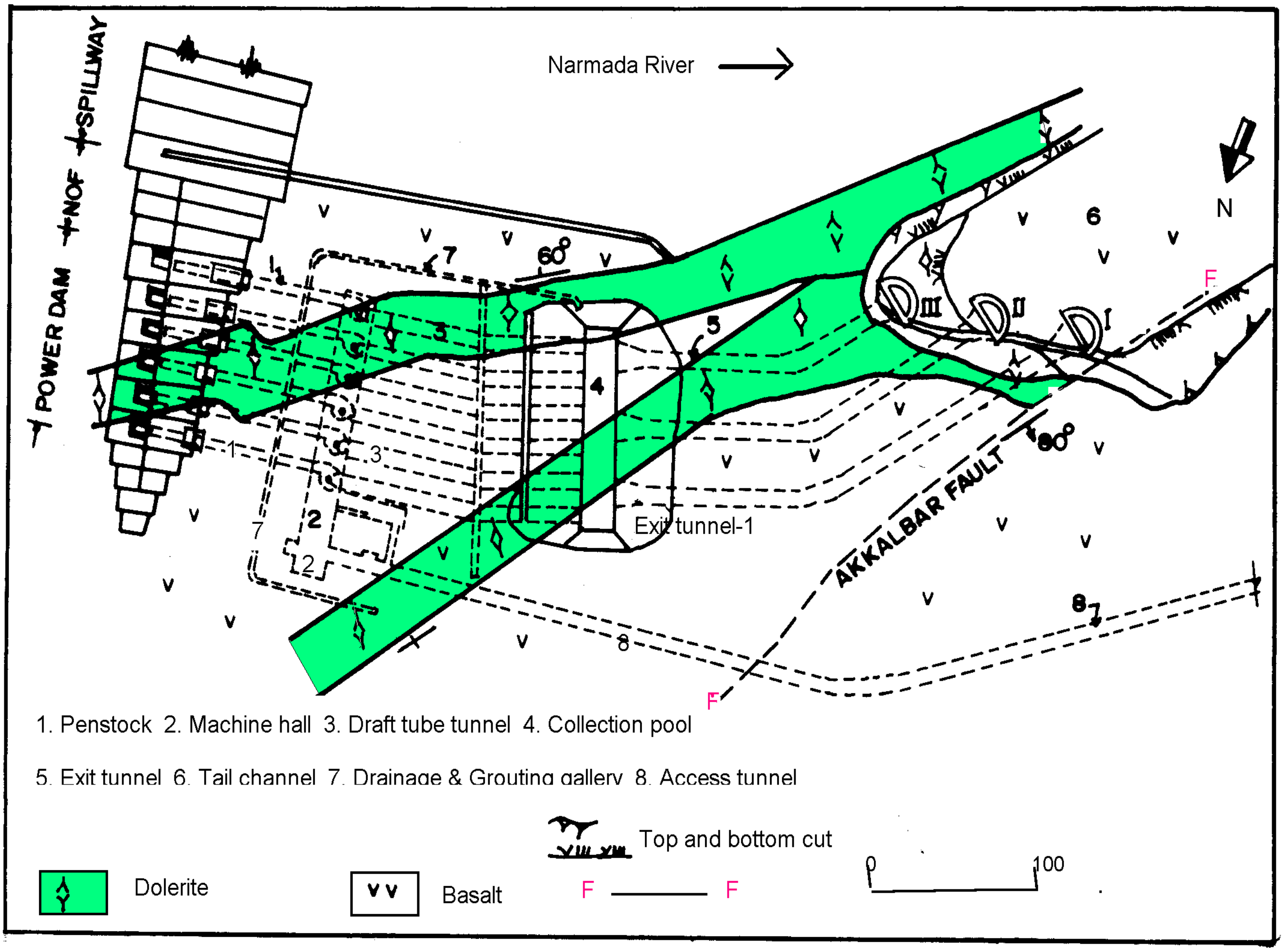

Fig. 2. Geology and Layout of Underground Powerhouse

3. Salient Features

The underground powerhouse cavern measures 23 meters in width, 58 meters in height, and 212 meters in length, with an installed capacity of 1200 MW (6 x 200 MW) (Table 1). The height of the cavern is approximately 58 meters. The crown level of the powerhouse is at Elevation 45 meters, and the bottom level is at Elevation -12.6 meters. The powerhouse is operating with a head ranging from 77.5 meters to 117.8 meters. Additional components include bus shafts and galleries, a lift well, a control room, a ventilation shaft, and a ventilation room. Given that the cavern is surrounded by water, a drainage and grouting gallery is provided around the cavern at an average elevation of 36.0 meters. The D-shaped access tunnel, which is 860 meters long (8.5 meters by 9 meters), serves as the primary access to the powerhouse. The intake arrangement consists of six inclined steel-lined penstocks with an internal diameter of 7.61 meters (9.0 meters excavated diameter). The tailrace system includes six draft tube tunnels with a finished diameter of 10 meters (10.5 meters excavated), leading to an open collection pool (surge pool) designed to accommodate surges during various operating conditions. Three exit tunnels, each with a finished diameter of 12 meters (13 meters excavated), are horseshoe-shaped and connect the collection pool to the open tail pool and the tailrace channel, which joins the main river approximately 730 meters downstream of the dam axis.

The underground powerhouse, employing Francis vertical turbines (reversible), plays a pivotal role in harnessing the river's energy, with the capability for both power generation and pumping operations. This facility is a significant engineering achievement, particularly given its location in basalt with a shallow rock cover, presenting unique challenges and opportunities.

The surface powerhouse, equipped with Kaplan turbines, complements the underground installation, contributing to the overall energy output of the project. This case study examines the engineering challenges and solutions associated with the construction of the underground powerhouse, emphasizing its significance in the context of the Sardar Sarovar Narmada Project.

Table 1. Sardar Sarovar (Narmada) Project Power Installation

Powerhouse Type	**Riverbed (underground) powerhouse**	**Canal head powerhouse**
Number of units	6	5
Rated capacity each unit	200 MW	50 MW
Installed capacity	1200 MW	250 MW
Types of Turbines	Francis vertical (Reversible)	Kaplan (Conventional surface)

4. Methodology

4.1. Monitoring of Rock Mass Movements

Monitoring engineering structures involves either visual inspections or the use of instruments to observe their condition or both. This process is crucial for assessing the response of the rock mass and, if necessary, adjusting overall designs or implementing remedial measures. Various devices are employed to monitor the movements and pressures within the rock mass or ground (Table 2).

Table 2. List of In-situ instruments as per IAEG Commission [14]

Test/ Technique	Principles of technique	Remarks
Extensometer	Length changes between the borehole mouth and one or more fixed points along the borehole are determined with transmission rods or tensioned wires. Relative movements between the borehole mouth and the fixed points are obtained.	Extensometers are used to determine movement and deformations in soil and rock mass. Length of extensometer may vary from 10 m to 200 m. Maximum number of fixed points is eight.
Earth pressure cell	A cell is embedded in the soil of a fill, cemented in the wall of a tunnel between rock and tunnel lining. Hydraulically or electrically the stress acting in the ground or tunnel lining is recorded. Earth pressure at location of the cell is obtained	Use to determine stresses in soil, rock and concrete. Used in soil fills in retaining walls, dams, in lining of tunnels, etc. These equipment's cannot be retrieved after tests.
Anchor load cell	An elastic element is fastened between the anchor plate on the surface and the anchor head. Elastic deformation of the cell can be translated to anchor tension	Used for measurement of anchor load over longer time intervals and for the determination of anchor failure load. These cells can be retrieved after use.
Glass plate	Simple device to know the movement along the cracks. Glass plate breaks easily under tension.	Used for rough visual observations.
Demac gauge	Six points each on both sides of cracks were established for observing the distance between gauges.	Used to study movement along cracks.
3-D Crack monitor	Relative displacement of two rock or concrete masses in any direction is determined as change of distance between a Sensor plate mounted on one of the masses and a Dial gauge mounted on the other rock mass. The readings are taken in the direction of X-axis, which is along the crack, Y-axis which is perpendicular to the X-axis and in the plane of the surface of the crack, Z-axis which is perpendicular to both X and Y axes and is also perpendicular to the plane of the surface of the crack.	Used to register the crack displacement readings in three directions.

4.2. Characterization of the Rock Mass

Quantitative description and assessment of the rock mass are essential for design purposes. The classification of rocks and soils should be based on the principle that the physical or engineering geological properties of a rock in its present state depend on the combined effects of its mode of origin, subsequent diagenetic, metamorphic, and tectonic history, as well as weathering processes [1]. Important parameters that can be described quantitatively include the orientation, location, persistence, water pressure, wall strength, degree of weathering, type of infilling material, and shear strength of critical discontinuities. These parameters, used in conjunction with physico-mechanical properties, are vital for accurate assessment, evaluation of the rock mass, and stability analysis.

Rock mass classification should effectively combine observations, experience, and engineering judgment to provide a quantitative assessment of rock mass conditions, support requirements, and foundation treatment. Rock mass classification schemes have been developed for over 100 years, starting with [15], who attempted to formalize an empirical approach to tunnel design for determining support requirements. [3] used descriptive classification to estimate rock loads carried by steel sets for tunnel support design. [4] proposed that the stand-up time for an unsupported span is related to the quality of the rock mass in which the span is excavated.

The Rock Quality Designation (RQD) index, developed by [5], provides a quantitative estimate of rock mass quality from drill core logs. [16] extended this by estimating RQD from surface exposures. [17] described a quantitative method for evaluating rock mass quality and selecting appropriate support based on their Rock Structure Rating (RSR) classification. [6] developed the Geomechanics Classification, also known as the Rock Mass Rating (RMR) system, which has been refined with additional case histories [7]. Bieniawski's classification is based on civil engineering project case histories.

[10,11], [18], and [19] described a Modified Rock Mass Rating system for mining. [20] and [21] also modified Bieniawski's RMR classification to produce the MBR (Modified Basic RMR) system for mining. [8] of the Norwegian Geotechnical Institute (NGI) proposed a Tunneling Quality Index (Q) to determine rock mass characteristics and tunnel support requirements based on a large number of case histories of underground excavations. [9] provided additional guidelines on rock bolt length, maximum unsupported spans, and roof support pressures to supplement earlier recommendations. [12] developed the Geological Strength Index (GSI) specifically for very poor rock (RMR <25). The RMR [6] and Q [8] classification systems are primarily used in the present study and are among the most widely used today.

4.3. Description of Rock Mass

A complete specification of a rock mass requires detailed descriptive information on the nature and spatial distribution of the materials that constitute the mass. Description is the initial step in engineering assessment of rocks and rock masses. The behavior of a rock mass is influenced by the type, spacing, orientation, and characteristics of the discontinuities present. Therefore, key parameters in describing a rock mass include the nature and geometry of discontinuities, as well as the rock mass's overall strength, deformation modulus, and secondary permeability.

A discontinuity is a surface within the rock mass that is open or potentially open under the stress levels encountered in engineering applications, due to its tensile strength being lower than that of the surrounding rock material. Discontinuities may be filled or "healed" partially or completely by minerals such as quartz, calcite, or others. Larger fissures might be sealed by the intrusion of magma. Veins may be present without healing the discontinuity or may have been broken again, forming new surfaces. Soluble fillings like gypsum can cause degradation of foundations or structures over their expected lifespan.

4.4. Geomechanics Classification

The geomechanics classification system, also known as the Rock Mass Rating (RMR) system, is used to estimate the unsupported span, stand-up time, or bridge action period, as well as the support pressures of an underground opening. It aids in selecting appropriate excavation methods and permanent support systems. Additionally, it can be used to estimate parameters such as cohesion, angle of internal friction, and elastic modulus of the rock [22]. In its modified form, RMR can also be applied to predict ground conditions for tunneling and foundation work.

In the present study, [6] Geomechanics Classification, or Rock Mass Rating (RMR) system, has been predominantly used. This classification system divides the rock mass into several structural regions, with each region being classified separately.

4.5. Estimation of s underground cavern and tunnels' supports

The principal objective in the design of underground excavation support is to help the rock mass to support itself [23]. The primary goal in designing underground excavation support is to enable the rock mass to support itself [23]. However, in most cases, additional support systems are required to stabilize the rock mass. Accurately assessing the need for roof and wall supports is essential to ensuring the safety and stability of underground tunnels and caverns.

a. Excavation support ratio and Equivalent dimension: In relating the value of the index Q to the stability and support requirements of underground excavations, [8] defined an additional parameter which they called the *Equivalent Dimension*, D_e, of the excavation. This dimension is obtained by dividing the span diameter or wall height of the excavation by a quantity called the *Excavation Support Ratio, ESR*. Hence:

D_e = Excavation span, diameter or height (m)/ Excavation Support Ratio *ESR*

The value of *ESR* is related to the intended use of the excavation and to the degree of security, which is demanded of the support system, installed to maintain the stability of the excavation. [8] suggested ESR value 1.0 for the Power stations, major road and railway tunnels, civil defense chambers, portal intersections. Thus, in the present study of underground powerhouse ESR 1 has been used.

b. Unsupported span: The maximum unsupported span can be estimated from:

Maximum span (unsupported) = 2 ESR $Q^{0.4}$

c. Support pressure: Roof and wall support pressures are estimated as detailed below:

Permanent support pressure P_{roof} (P_v) can be estimated from the equation:

P_{roof} (P_v) = (2/3) X (1/ J_r) X $(J_n)^{½}$ X $(Q)^{-1/3}$

Estimation of short-term Roof Support Pressure (pvi) after [9]:

pvi= pv/1.7

Where, pv (i.e. p_v) is ultimate roof support pressure.

Wall Rock Mass Quality (Qh): The ultimate wall rock mass quality has been estimated by multiplying Q with the wall factor (W). For different range of Q, different values of wall factor have been suggested by [8] as given below:

For Q > 10	W = 5.0
0.1 < Q <10	W = 2.5
Q < 0.1	W = 1.0

Estimation of Ultimate Wall Support Pressure (ph): The ultimate wall support pressure (ph) is estimated by the following equation:

ph = (2/3) X (1/ Jr) x $(Jn)^{1/2}$ x $(Qh)^{-1/3}$

Estimation of short-term wall support pressure (phi) can be done by following equation:

phi=ph/1.7

d. Estimation of shotcrete capacity ps: Shotcrete capacity can be determined by following equation:

ps= t σ_c/R

Where t = thickness of shotcrete, σ_c = compressive strength of shotcrete and R = radius of curvature of shotcrete layer.

e. Estimation of the bolt capacity pb: The load pb sustainable by a rock bolt is as below:

pb= Bc/a

Where

Bc = yield capacity of the bolt and

a = area of influence of bolt

The bolt yield load Bc can be estimated by following equation:

Bc= Sb/ Ab

Where

Sb = yield stress of bolt material and

Ab = cross sectional area of bolt

f. Estimation of rock bolt and cable length: [9] provided additional information on rock bolt length, maximum unsupported spans and roof support pressures to supplement the earlier support recommendations. The length L of rock bolts and cables can be estimated from the excavation width (span) B and height H for roof and sidewall and the excavation Support Ratio ESR as below:

(i) Arch roof support

Bolt length

L = (2+ 0.15B) / ESR

L = (2+ 0.15B) (ESR=1 for the powerhouse)

Cable length

L = 0.4 X B

(ii) Side wall support

Bolt length

L = (2+ 0.15H)/ ESR

L = 2+ 0.15H (ESR=1 for the powerhouse)

Cable length

L= 0.35 X H

g. Estimating In-Situ Deformation Modulus Using Rock Mass Classification

The in-situ deformation modulus (E_m) of a rock mass is an important parameter in the numerical analysis and in the assessment of deformation around underground openings and in the dam foundations. Deer's RQD approach is now seldom used for estimating in-situ deformation modulus Em [24]. Several workers have attempted to estimate its value based on the analysis of several case histories (many of which involved dam foundations) and rock mass classifications and developed following relationships:

i. [6] Bieniawski's-1978:

E_m = 2 RMR - 100

ii. [25] Serafim and Pereira-1983:

$E_m = 10^{(RMR-10)/40}$

iii. [9] Barton-1980:

$E_m = 25 \log_{10} Q$

The [9] equation provides a reasonable fit for all the observations plotted for wider range of RMR value [23].

5. Geology of the Underground Powerhouse

The Narmada Dam site is situated in a hilly terrain covered with a thin mantle of soil. The geological formation predominantly features "Aa" type Deccan basalt flows, which are underlain by sedimentary rocks known as Bagh beds and intersected by dolerite dykes, shears, and faults [13,26]. Typically, "Aa" basalt flows include a basal clinker zone, a thick middle section with columnar joints, and an upper zone of agglomerate or tuff. However, at this site, the basal clinker zone is largely absent, and the basalt flows are characterized by agglomerate or tuff at their top.

The area consists of lava flows of basalt separated by hard agglomerate and intruded by ENE-WSW trending dolerite dykes and sills. Four basalt flows have been identified, extending from ground level to 30 meters below the turbine level, i.e., between El.132 and (-)42 meters. The powerhouse is positioned between two dolerite dykes and a dolerite sill, each with a thickness of 40 to 45 meters. The dyke closest to the river dips 60°-65° towards the southeast (i.e., towards the left bank), while the second dyke near the northern end of the machine hall is nearly vertical. A significant portion of the turbo-generator units is located within the dolerite sill.

Weathered zones extend 7 to 23 meters deep in the dolerite and 0.5 to 5 meters in the basalt. These dykes act as seepage barriers between Rock fill dam (Pond) No.1, located to the north, and the Narmada River, situated to the south of the powerhouse. The rock mass permeability varies widely (0-30 lugeons), with oozing and water dripping observed at and near shear zones. Most of the exit tunnels, draft tube tunnels, and the foundations of the turbo-generator units, including the bottom sides of the machine hall, are located within the dolerite rock, which is dissected by chlorite-coated joints, shears, and slaked zones.

The cavern is excavated within rocks composed of sub-horizontal layers of amygdaloidal and porphyritic basalt, interspersed with pockets of agglomerate (Fig. 3). These formations are penetrated by dolerite dykes trending ENE-WSW, which are 25 to 30 meters thick and vary in orientation from vertical to inclined (60°-65° towards SSE). Additionally, low-dipping dolerite sills (20°-25° SE) align in a NE-SW direction. The rocks are all strong, with compressive strengths greater than 60 MPa, but they are well-jointed, with block sizes typically ranging from 1 to 2 cubic meters. The basalt flows are sub-horizontal, and there are three prominent joint sets (NNW/60°-vertical, ENE/60°-vertical, and ENE/30°-45° NW), along with some random joints.

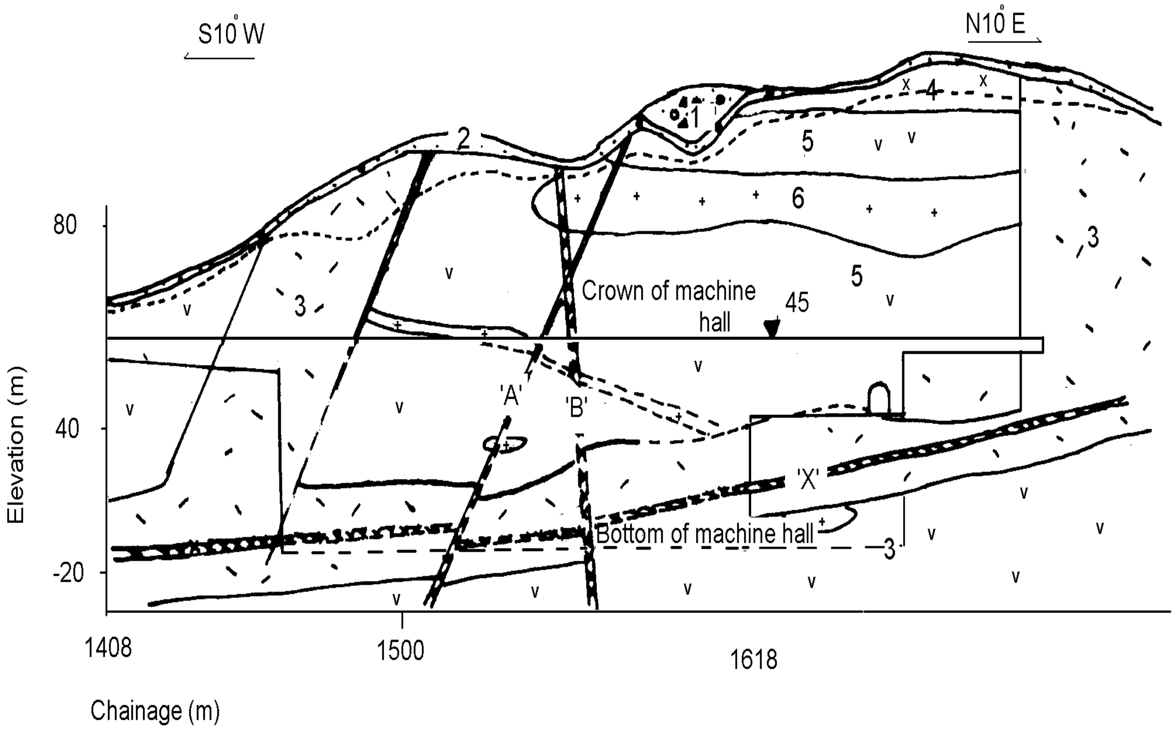

Fig. 3. Longitudinal Geological Section Powerhouse Cavern (1- Coarse Gravely soil, 2- Weathered rock mixed with soil, 3- Dolerite, 4 & 5 -Basalt Flows, 6- Agglomerate)

6. Rock Mass Characteristics of Powerhouse Cavern (Machine Hall)

The rock mass inside the underground structure is generally fresh to slightly weathered. The basalt and dolerite above the machine hall are weathered to depths of about 0.5 to 5 meters and 7 to 23 meters, respectively, from the surface. The rock mass in the powerhouse is classified as poor to good (Table 3). Major discontinuities traversing the machine hall include the sheared contact of the inclined dolerite dyke with the host rocks (basalt and dolerite sill), designated as the main shear. Other significant shear zones are:

Steeply dipping shear zones ‘A’ (N50°E-S50°W / 70°S40°E) and ‘B’ (N70°E-S70°W / 70°-80°N20°W).

Low-dipping shears ‘X’, ‘Y’, and ‘Z’ traversing the dolerite sill (see Fig. 31 & 32).

The vertical dolerite dyke and sill are dissected by chlorite-coated joints, shears, and slaked zones. The dolerite sill forms the foundation for a major part of the turbo-generator units.

In-situ tests indicate large variations in rock mass permeability, ranging from 0 to 30 lugeons. Most of the underground rock mass is free from water seepage, with oozing and water dripping occurring only in areas affected by shear zones or faults.

Table 3. Rock mass description and classification of underground powerhouse

Rock mass description	Rock mass characteristic	Barton's Q	RMR Rating (Bieniawiski 1976)	Class No.	Class Description
Jointed basalt	Three joint set plus random, rough or irregular planar joints, non-softening infilling, dry rock mass, medium stress.	9.16	60	III	Fair rock
Jointed inclined dolerite	-do-	10.00	63	II	Good rock
Jointed Vertical dolerite rock	Moderately altered dolerite, Joints infilled with chloritic material	1.5	45	III	Moderately to highly altered dolerite (slaking of the rock observed on exposure to air)
Jointed dolerite sill	Moderately to highly altered dolerite (slaking of the rock observed on exposure to air)	0.6 to 1.25	30 to 40	IV	Poor rock
Shear zone	Sandy, gravely crushed zone thick enough to prevent rock wall contact, softening or low friction clay mineral coating, dry rock mass, single shear zone containing clay or disintegrated rock.	1.25	35	IV	Poor rock

7. State of Stresses in the Rock Mass in the Powerhouse Cavern

The in-situ stress field in the powerhouse cavern was assessed using flat jack, over-coring, and hydro-fracturing methods (Table 4).

8. In-situ Stress Estimated by Hydro-fracture Tests

Hydro-fracture testing revealed that the minimum in-situ stress is vertical, attributed to the shallow rock cover. This stress is calculated by multiplying the depth below the surface by the rock's unit weight (0.026 MN/m³). The major in-situ stress is roughly 2.5 times the vertical stress and aligns parallel to the cavern's longer axis. The intermediate principal stress, which is perpendicular to the cavern axis, is approximately 1.25 times the vertical stress. With an average cover of around 45 meters over the cavern roof, the vertical stress is about 1.25 MPa, while the horizontal stress perpendicular to the cavern axis is roughly 1.5 MPa. The direction of the maximum principal horizontal stress is North ±5° [27].

Table 4. Results of in-situ stresses at underground powerhouse site

Stress	Flat jack test (MPa)	Hydro-fracture test (MPa)	Horizontal to vertical stress ratio 'k'	
			Flat jack method	Hydro-fracture method
Vertical:	1.379	1.2	-	-
Horizontal:				
1. Parallel to longer axis	1.171	3	0.85	2.5
2. Perpendicular to longer axis	-	1.5	-	1.25

It has been observed that horizontal stresses measured by flat jack tests in the exploratory drifts are different and lower than those evaluated by hydro-fracture tests. Similar discrepancies were noted at the Srisalam Dam site [2]. Further study is needed to establish any potential relationship between flat jack and hydro-fracture test results.

9. Geotechnical Problems Observed during Construction of Powerhouse

The machine hall (powerhouse cavern) is situated with a shallow rock cover, varying from 35 to 60 meters, composed of basalt flows. The machine hall and other underground structures were excavated using the heading and benching method, employing the New Austrian Tunneling Method (NATM). The fundamental principle of NATM is to utilize the rock itself as a structural material. Six cross drifts were driven from the central exploratory drifts, reaching from an elevation of 45 meters down to 39 meters. After assessing the behavior of the rock mass, the powerhouse cavern was excavated in stages to 5 meters, 9 meters, and its full width of 23 meters. The bench height varied from 2.5 to 4.0 meters.

The main geotechnical problem observed in the machine hall was the development of cracks in the shotcrete of the upstream and downstream walls, as well as inside the walls, including pressure shafts and bus galleries [2]. Minor rock falls in the crown were also observed during excavation.

a. Rock Falls in the Roof

In February 1988, a rock fall occurred between R.D. 1540 and 1556 meters, involving approximately 125 cubic meters of rock. A three-point borehole extensometer, installed at R.D. 1540 meters to monitor the behavior of the contact between agglomerate and basalt, detected a small but steady opening of the contact at a rate of 0.024 mm per month prior to the rock fall. The total opening observed from August 1984 to February 1988 was 3.03 mm [28]. Additionally, cracks ranging from 1.5 to 2 meters were noted in the upstream roof (crown) arch between shear zones ‘A’ and ‘B’. To address the issue, extra rock bolts were installed between the existing pattern bolts, and two additional layers of shotcrete were applied. Since these remedial measures were implemented, no further opening of the contact or rock falls have been observed in the treated area.

b. Formation of Cracks in the Upstream and Downstream Walls

Excavation of the roof with the pattern rock bolt support system was completed in December 1989. Excavation of the walls was completed up to Elevation 20 meters by January 1992. The crown level of the cavern is at Elevation 45 meters and the bottom level is at Elevation -12.6 meters. Further benching was carried out from the ramp along the downstream wall, approximately half the width of the cavern from the service bay end (Elevation 20.0 meters) to the riverside end of the cavern (Elevation 4.0 meters). The remaining half width along the upstream wall was completed with the pattern rock bolt support system up to Elevation -1.9 meters by June 1992. The excavation of six pressure shafts on the upstream wall was also completed.

Cracks in the upstream wall were observed between Ch. 1545 and 1585 meters below Elevation 14.0 meters. These cracks (fissures) were stitched with 4 to 6 meters long inclined criss-cross rock bolts, and an additional layer of shotcrete with wire mesh was provided in this area. The cracks propagated up to Elevation 13.5 meters and Elevation 36 meters, respectively, when bench excavation reached Elevation 10 meters. The fissures that were previously treated and covered with shotcrete reappeared when bench excavation reached Elevation 1.9 meters, along with additional peripheral fissures around pressure shafts 2, 3, and 5. Cracks were also observed on the downstream wall between Elevation 39 meters (spring level) and Elevation 9.0 meters, extending inside the bus galleries. Popping of shotcrete between Chainage (Ch.) 1505 meters and Ch. 1520 meters and between Elevation 9.0 meters and Elevation 27.0 meters along shear zone ‘A’ was noted.

c. Nature of Cracks in the Upstream and Downstream Walls

The cracks (fissures) developed in the pressure shafts and bus galleries are aligned parallel to the longer axis of the machine hall. These cracks do not follow geological discontinuities. Sub-horizontal to low-dipping cracks developed in the downstream wall in an en echelon pattern parallel to the excavated profile of the ramp. A few cracks were also observed near major shear zones ‘A’ and ‘B’. These cracks extended into the rock mass, creating openings in the shotcrete.

10. Results and Discussions

Table 5. Result of 3-D FEM and 3-DEC analysis

Displacement/ Stresses/ FOS	3-D FEM analysis	3-DEC analysis
Displacement	The maximum displacement on the wall was 7.4 mm without dam loading and 7.6 mm with dam loading. Displacement of upstream wall was less in comparison to downstream wall without dam loading while displacements of both the walls became almost equal with dam loading.	Rock mass movement is continuous in the continuum analysis. The horizontal displacement contours, compared to continuum analysis, are not symmetric about the longitudinal axis of cavern and the continuity of contours is disturbed at the shear zones. The contour shows the movement of wall towards the cavern. The shear movements primarily along shear zone "A" and "B" are higher near the excavation face and reduce inside the rock mass. The shear movements are more pronounced on the upstream wall relatively at higher elevations i.e. El.35, 30 and 25m whereas on the downstream wall more movement is at lower elevations i.e. El.20 and 15m. Displacement of the walls as observed (at El. 20m): 1.During continuum analysis 1.6 to 1.8 cm with ramp and 1.9 to 2.2cm without ramp. 2. During discontinuous analysis 2.75 cm with ramp and 3.25 cm without ramp.
Stresses:		
Major principal stress	The major principal stress 11.5 MPa (115 kg/cm^2) occurred at El. 6.4 m at section through pressure shaft.	The major principal stresses are oriented along the longitudinal axis of the cavern. Major principal stress contours show that the stress concentration area lies at the junctions of bus galleries and pressure shafts with powerhouse cavern.
Minor principal stress	A tensile minor stress zone developed at sections through rock pillar between pressure shafts at upstream wall but stresses in the downstream wall were all compressive. The maximum depth of the tensile zone is 6.5 m. The direction of the minor principal stress is nearly right angle to the wall.	The minimum principal stress contours show the tensile regions at the junction between the bus galleries and pressure shafts with cavern. Higher magnitude of tensile stress observed up to 10 m distance in the pressure shafts, 12 m to 20 m in the bus galleries. The rock mass in the bus gallery-3 is affected most where shear zone A is forming wedge with northern side of the gallery wall where ramp support is almost negligible.
Factor of safety (FOS)	The dam loading has little influence on the factor of safety contours. The maximum depth of the 1.0 FOS contour is 10 m in the upstream wall and 6.5 m in the downstream wall. The factor of safety of the pillars in between Draft Tube Tunnels is sufficient but between the pressure shafts it is less than the 1.5. In the area affected by cracks safety factor contour of 1.5 extends up to 25 m in depth.	The Contour of FOS of 1.5 is about 16 to 17 m away from the cavern wall face going inside the rock mass. The maximum displacement was observed at a distance of 20m from the face in and around bus gallery-3.

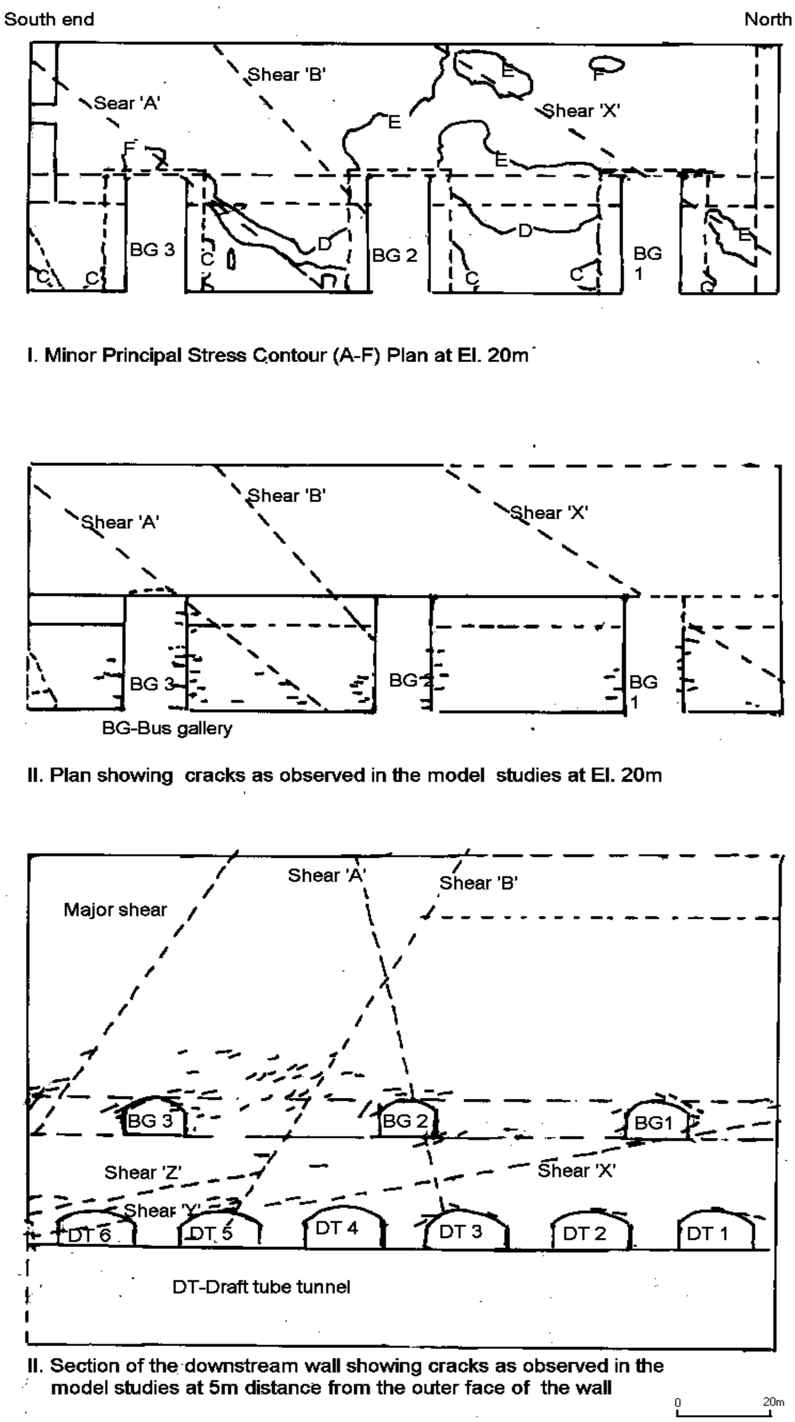

Fig. 4. Plot of 3-DEC Discontinuum Analyses Showing Model Results for the Downstream Wall of the Underground Powerhouse Cavern Prior to Removal of Ramp (DT - Draft Tube; BG - Bus Gallery, C,D,E-Contours)

10.1. Three-Dimensional Numerical (FEM and DEC) Analysis of the Powerhouse Cavern

Back analyses using 3-D FEM and 3-DEC were performed following the formation of cracks in the machine hall walls to evaluate the past and future behavior of the underground powerhouse cavern (Table 5) (Fig.4).

10.2. Stability Analysis Based on the Geological Features and Stresses

The geological stability analysis was conducted to identify the causes of crack development in the machine hall.

a. Wedge Failure Analysis: Wedge failures are commonly observed in jointed rock masses at relatively shallow depths, typically involving wedges that either fall from the roof or slide out from the side walls of excavations [29].The major shear zones ‘A’ and ‘B’ crossing the machine hall create stable wedges in the upstream wall, as the intersection of these shear zones plunges approximately 22° toward the northeast that is inside the upstream wall. Wedge sliding can only occur along the line of intersection of two planar discontinuities if it is daylighted in the open space (that is towards free face) [33]. The orientation of these major shear zones is such that they diverge in the downstream wall, which prevents any wedging issues (Fig. 5).

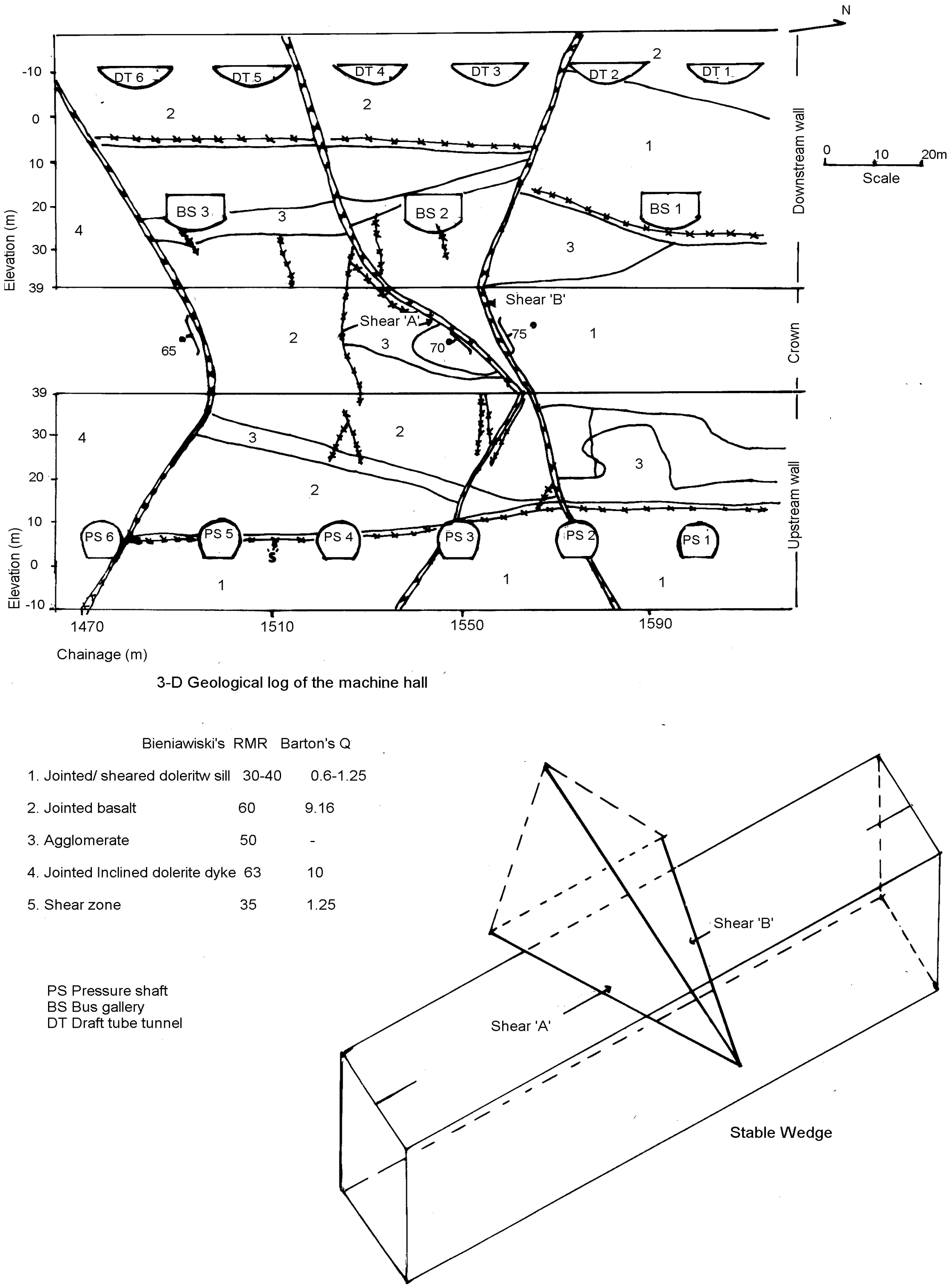

Fig. 5. Disposition of major Shear Zones and Rock mass characteristics of Machine Hall

Sliding wedges have formed at the lower part of the downstream wall in the dolerite sill with the intersection of joints J1 and J3. These wedges have a 25° plunge towards S34°E (i.e., towards the free face) [33]. Similar minor rock wedges have formed with the intersection of joints J1 and low-dipping shears 'X', 'Y', and 'Z'. These wedges were stabilized by installing 12-meter-long rock bolts during progressive excavation.

b. Plane Failure Analysis: Plane failure happens when the sliding plane is parallel or nearly parallel (within about ±20°) to the slope face [23]. In the machine hall, the major shear zones and joints are oriented at angles greater than 30° to the cavern's longer axis, thereby minimizing the risk of plane failure.

c. In-Situ Stresses: The in-situ horizontal stresses in the machine hall, which are perpendicular to the cavern's longer axis, are relatively low (1.5 MPa). Given that the average compressive strength of the surrounding rock exceeds 60 MPa, the likelihood of cracks developing due to these in-situ stresses is minimal.

10.3. Initial Support Designed for the Powerhouse Cavern

The initial support system for the cavern included a pattern of rock bolts and two layers of 38 mm thick shotcrete, with a layer of welded wire mesh in between. The roof supports consisted of 25 mm diameter, 6-meter-long tensioned rock bolts, spaced 1.75 meters apart (center to center), preloaded to 14 tonnes, along with two layers of shotcrete and wire mesh. The wall supports featured similar tensioned rock bolts, 25 mm in diameter, 6 meters long, and spaced 2.5 meters apart, also with two layers of shotcrete and wire mesh. For the middle third of the wall height (Elevation 13 to 33 meters), additional 7.5-meter-long rock bolts were installed, reducing the spacing to 1.52 meters center to center. These support measures were designed by the Central Water Commission, New Delhi [30].

10.4. Review of Rock Support Design and Performance

A review of design supports using various approaches (empirical methods by [31], [32], [23], [9]) and comparisons with rock bolt and cable lengths used in other hydroelectric projects worldwide indicated that the 6-meter-long rock bolts used in the arch of the Sardar Sarovar cavern fall within the acceptable range. Therefore, they were deemed sufficient for providing permanent arch support. However, none of these approaches, except Barton's method, provide criteria for estimating support pressure. The available roof support capacity of 1.19 kg/cm² in jointed basalt and shear zones, and 1.06 kg/cm² in jointed dolerite, against the estimated ultimate roof support pressures of 0.88 kg/cm² in shear zones, 0.73 kg/cm² in jointed basalt, and 0.69 kg/cm² in jointed dolerite, indicate that the roof support is adequate [28]. Performance monitoring over fourteen years has confirmed the stability of the cavern roof.

For side wall support, similar methods and plots for a 58-meter-high cavern suggest average rock bolt lengths of 10-11 meters and cable lengths of 20 meters. The 6- to 7.5-meter-long rock bolts installed in the side walls of the Sardar Sarovar powerhouse cavern proved to be too short [28,33], resulting in inadequate restraint and the formation of cracks in both the upstream and downstream walls, including the pressure shafts and bus galleries.

10.5. Mechanism of Rock Mass Behavior and Crack Development

The major problem associated with excavating this powerhouse cavern in a jointed rock mass with shallow cover is stress relief due to low confining stress. This situation is similar to those encountered when excavating very steep slopes in hard, jointed rock mass. The stress relief caused by removing the 'cut' can induce movement and/or failure in the rock mass. The vertical cavern walls can be viewed as steep rock slopes, and without adequate support during excavation, deformation and cracking can occur. In the absence of adequate support, vertical tension cracks, which are common in steep rock slopes, can develop parallel to the walls. The symmetrical pattern of cracks parallel to the longer axis of the cavern in the pressure shafts and bus galleries suggests that these cracks developed due to tensile stresses acting on inadequately supported rock mass.

Increased cracking in the shotcrete on both the walls (upstream and downstream) was observed during the installation of additional supports. This cracking was likely a manifestation of gradual adjustment of the loosened rock mass due to earlier inadequate support. It is expected that the rate of deformation in the rock mass will become negligible after placing concrete in the turbo-generator foundations and installing adequate wall supports. Numerical modeling suggests that changes in stresses induced by raising the reservoir level (from the current level of Elevation 80 meters to the full reservoir level of Elevation 140 meters) will be minimal.

10.6. Remedial Measures Adopted to Stabilize the Rock Mass in the Machine Hall

Remedial support in the upstream wall consisted of cables ranging from 10.5 to 32 meters in length, each with an 80-ton capacity. These cables were tensioned to 50 tons and fully grouted. Additionally, 12-meter-long, 32 mm diameter rock bolts, tensioned to 20 tons, were installed at various locations. In the downstream wall, numerous 12-meter-long, 32 mm diameter rock bolts were installed, pre-tensioned to 20 tons before grouting. Additionally, several 25-meter-long cables with a 50-ton capacity were installed and tensioned to 5 tons prior to grouting. For the remaining excavation in the lower section of the cavern, 12-meter-long tensioned rock bolts were used. To stabilize the loosened rock mass, low-pressure grouting was carried out in both the upstream and downstream walls.

11. Conclusions

The construction of the 58-meter-high machine hall at the Sardar Sarovar Narmada Project presented substantial geotechnical challenges due to the shallow cover and jointed basalt rock. The low confining stress encountered during excavation led to significant stress relief, causing cracks aligned parallel to the longer axis of the cavern, particularly affecting the pressure shafts and bus galleries. These challenges are similar to those encountered when excavating very steep slopes in hard, jointed rock masses, where insufficient support often leads to the formation of vertical cracks parallel to the excavation walls. A review of the initial support design revealed that the 6 to 7.5-meter-long rock bolts used were inadequate for controlling rock mass deformation. This insufficiency resulted in persistent cracks and highlighted that the initial support system was not sufficient to stabilize the rock mass effectively. As a result, corrective measures were taken, including the installation of longer rock bolts (12 meters) and cables (ranging from 10.5 to 32 meters), to resolve these issues.

The findings from this study offer valuable insights for designing and constructing underground powerhouses in similar geological conditions. They underscore the necessity of selecting appropriate support systems tailored to the specific geotechnical challenges of the site. For projects involving jointed rock masses with shallow cover, it is crucial to ensure that the support systems are robust enough to manage stress relief and deformation effectively. The study highlights the importance of thorough planning and the potential advantages of adopting adaptable and enhanced support strategies based on the unique conditions of each project.

However, this study has limitations as it is focused on a specific case with unique geological conditions. Future research should broaden this scope by including additional case studies across different geological settings, encompassing a variety of rock types and depths. This approach would help refine support design practices and provide more generalizable insights for similar geotechnical challenges in underground construction.

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