Geological and geotechnical approach for excavation of large unlined rock cavern

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The degree of uncertainty involved in an underground storage project is relatively higher compared to other underground projects. In order to allow for a safe and economical construction of underground rock caverns, continuous updating of the geological and geotechnical model along with adjustment in construction approach is required. Investigations planned during different stages of excavation of the project assist in the identification of various geological features and ground characteristics to ensure robust design of underground structures. Investigation in each stage of the project is planned in line with the time and money available at that stage, along with level of detailing required for different design stages. In order to develop an efficient investigation scheme, it is necessary to assimilate the existing information from one stage before planning for the next stage of investigations. This article highlights the significance of various investigations carried out for one of the strategic storage site in the southern part of India.

Keywords: Geotechnical and geological investigations, cavern, tunnelling, underground excavations, rock.

THE storage of hydrocarbons in underground rock caverns is a well-known technology that has been successfully adopted in many countries. The principle of storage essentially uses groundwater pressure for confining the product within an unlined rock cavern. An underground storage scheme consists of storage caverns, connecting access tunnel, vertical shafts for inflow and outflow of oil, water curtain tunnels and associated boreholes for providing hydraulic confinement around the caverns. The first step in such projects is to locate suitable sites based on a systematic investigation involving geological, geophysical, geotechnical and geohydrological studies in the neighbourhood of oil and gas transport networks.

The construction of underground caverns requires detailed investigations to develop geological, geotechnical and hydrogeological models for each part of the underground storage system. The guiding principle for the storage of crude oil in these caverns is hydrostatic containment to make the cavern gas-tight and ensure the containment of crude vapour. This required a number of water pressure tests, groundwater level monitoring and groundwater analysis. Results from site investigations are compiled and analysed for characterization of the proposed underground rock cavern storage site.

Underground unlined rock caverns mainly require availability of competent rock and shallow groundwater table to provide sufficient pressure for hydraulic confinement of the unlined caverns. The typical size of unlined underground rock caverns is 20 m wide and 30 m high, running for a length of 1 km. Construction of such large unlined rock caverns requires detailed planning for investigations at different stages of project execution. Project execution cycle of most of the large underground works starts from detailed feasibility stage to basic engineering design (BED) stage followed by construction stage. At each execution stage during the project cycle, a set of investigations is planned to explore specific aspects below the ground using various geo-characteristics. Thus, investigations play a key role in understanding the complex behaviour of the invisible ground underneath.

Quantum of investigations required to be carried out for successful execution of any project varies for each project and depends on factors such as time, finances and project requirements. This article discusses various levels of investigation carried out during execution of a large underground project. The data obtained from these tests, and their relevance in engineering and design of underground structures are also highlighted. The project site is located in the southern part of India and was used for construction of strategic storage of crude oil.

In the first stage of the project, i.e. the detailed feasibility stage, investigations are mainly concerned with obtaining a broad characterization of the identified piece of land to get first-hand idea of subsurface conditions. Basic broad planning of the project from capacity requirements to tentative layout, structure sizing, orientation, etc. are finalized in this stage, which can be either

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updated or changed during later part of the project. Thus, at this stage, a set of investigations are carried out, viz. geophysical surveys, *in situ* stress measurement, study of regional and local geology maps and outcrops, hydrogeological test and limited core-hole drilling to assess subsurface geological conditions.

As the project execution progresses towards preparation of bid documents for inviting tenders from construction contractors, a basic engineering document is developed as part of the bid document, which provides detailed information about the project for execution. BED studies are developed to reaffirm the basic planning decisions made in the feasibility study and to establish the scope of the project based on current criteria and costs. This stage requires additional set of investigations to be carried out to develop a layout of underground storage scheme along with sizing and orientation of each structural component. Major investigations at this stage are targeted towards understanding the joint sets, specific geological features, rock mass assessment, identification of water-bearing formations and the overall hydrogeological regime.

Once the tender for the construction of the project is awarded based on bid document prepared at the BED stage, further mandatory investigations are carried out by the construction contractor as part of his requirements for carrying out detailed engineering of each component of underground storage, viz. access tunnel, water curtain tunnel, vertical shafts and storage caverns. These investigations are more inclined toward estimating strength properties of underlying soil and rock for design and modelling requirements.

The significance and details of various investigations conducted during the course of execution of the project are discussed in the following sections. The importance of the investigations is discussed through a case study, which describes complex geological condition encountered at one of the storage project sites located under a hill with a maximum elevation of 80 m amsl. The investigated area consists of a landscape which is gently sloping down into two valleys. The elevation goes from +30 m amsl in the valley to +80 m amsl in the hill slope. The caverns are approximately *D*-shaped, having an average cross-section of 20 m width and height varying from 31 m at one end gently sloping (1 in 250) to 23 m at the other end, with a horizontal roof level (elevation – 35 m amsl) maintained throughout the cavern length of 900 m. The water curtain tunnel is located 20 m above the cavern at an elevation of –15 m amsl from where horizontal and vertical boreholes are drilled to provide hydraulic confinement around the caverns.

Detailed feasibility stage

Investigations during detailed feasibility study generally cover three broad aspects, namely developing an understanding of the geological setting, gathering information for assessment of geotechnical conditions and carrying out exploratory investigations to ascertain hydrogeological regime of the site. Geological studies carried out using various local and regional maps and satellite images revealed the presence of peninsular gneissic granite intruded by amphibolites of Pre-Cambrian age at this site. The peninsular gneiss of this area is relatively complex, made up of migmatitic gneisses and granitoids, which are spread up to a depth of about 15 m. The rock type is basically granitic gneiss, with standard granite zones and locally banding (gneissic structure) with alternation of mafic bands and felsic minerals. Few dykes are also observed which are made of mafic rock, probably dolerite. Dykes are fresh in the quarries. Tectonic structures are very few and only micro-faulting was observed. The structure trend lines in the gnessic complex are predominately oriented north–south[1].

As part of the geophysical investigation, interpretation of seismic refraction survey data was carried out with special attention to finding low-velocity zone within the basement rock along the seismic lines. The seismic refraction survey data revealed four distinct layers with different values of seismic velocity contrast with depth. Seismic refraction in this stage was carried out using 24-channel analog seismographs with profile lengths covering most part of the ground where underground structures have been proposed with an average geophone spacing of 10 m. In order to generate seismic energy, shallow shot holes were dug and charged with explosives as an energy source. Seismic velocity (V_p) obtained from the seismograph identified stratified layer consisting of top lateritic soil of varying compactness comprising clay, silt and sand with a velocity of 400–800 m/s, followed by relatively soft laterite (900–1500 m/s), hard and compact laterite of highly weathered rock (1500–2500 m/s) and granite gneiss (4000–6500 m/s) at the bottom. Electrical resistivity survey was carried out using 11 profile lines and 11-point vertical sounding Schlumberger method. Resistivity values obtained in the top layer were in the range 110–7200 Ohm-m indicating overburden comprising dry residual soil followed by laterite with weathered rock (400–6800 Ohm-m) and hard massive granite gneiss at the bottom with resistivity in the range 500–10,000 Ohm-m.

Drilling of six inclined and vertical boreholes was carried out at this stage to estimate sub-surface rock mass characteristics. These holes were drilled to a length of 150 m, extending to the cavern bottom level with NX size (cored inside diameter of 47.6 mm and hole diameter of 75.7 mm). Based on the core logs it was found that the thickness of the topsoil, including lateritic soil/weathered rock, was about 20 m and rock mass below the soil formation was fresh and suitable for construction of underground oil storage caverns. However, since holes were limited to just six in number covering a large area, it was felt necessary to drill additional holes and check the cores before construction. To calculate the transmissivity (T) of rock, pumping-in and pumping-out tests were also performed in selected boreholes. From the hydro-geological test results, a hydraulic conductivity profile was prepared for each core. Transmissivity of rock mass obtained by pumping test using Jacob method was found to be 1.388×10^{-4} m^2/s.

The *in situ* state of stress in the earth's crust has been widely recognized as a basic parameter necessary in the engineering design of underground structures. These *in situ* stresses require direct measurements in field condition. *In situ* stress measurements in this underground project were carried out in vertical boreholes at the cavern level using the hydro-fracturing method. Typically, hydraulic fracturing is conducted in vertical borehole where a short segment of the hole is sealed-off using inflatable packers. In this study, stress measurements were carried out in vertical boreholes corresponding to the crown-level of the caverns and within 5 m above and below the crown level, using the technique of hydro-fracturing[2]. Relatively high horizontal stress, with a maximum value (S_H) of 7 MPa, having a direction of N160°E and minimum horizontal stress (S_h) of 3.5 MPa, which is perpendicular to S_H, were recorded at this site. Based on these values, magnitude of the horizontal *in situ* stresses (S_{cav}) that occurred along the cavern planned axis (N110°E) was calculated using eq. (1) below

$$S_{cav} = \frac{S_H + S_h}{2} + \left(\frac{(S_H - S_h)}{2} \cos(180 - 2\theta) \right), \qquad (1)$$

where θ is the angle between the cavern alignment and direction of S_H, which is 50° in this case. This value of S_{cav} (5.55 MPa) was used to calculate the effective stress ratio (k) for defining *in situ* stresses in numerical modelling of the caverns. The effective stress ratio was calculated using eqs (2) and (3) below, according to Hoek and Brown[3].

For roof of cavern

$$S_{cav(r)} = S_V (Ak - 1). \qquad (2)$$

For side wall of cavern

$$S_{cav(w)} = S_V (B - k), \qquad (3)$$

where $S_{cav(r)}$ is the maximum roof stress, $S_{cav(w)}$ the maximum wall stress, S_V the *in situ* vertical stress, and A and B are constants depending on the shape of the cavern. Table 1 shows typical values of A and B.

Basic engineering design stage

Investigations performed at this stage were designed so as to collect sufficient details beyond the feasibility stage to ensure that the project could be implemented with the design assumptions. The emphasis of this stage was towards site-specific studies which provide more details and in-depth information. Initially in-depth assessment of joint patterns was carried out based on the study of nearby quarries and satellite images available from Google Earth. Based on this information, further structural analysis of joint sets was carried out using stereonet and rosette representations to get meaningful information about joints for assessing rock mass quality. Around 199 discontinuities were measured and studied at this stage. Statistical analyses were conducted using stereonet and rosette representations (Figure 1). The predominant set was found to be N–S vertical (range ± 10° in strike and ±5° in dip) and minor set horizontal (range ±10° in all directions). Further, some E–W joints were also reported. The predominant joint set (N–S vertical) spacing was observed to be larger than or equal to 8 m, while for the minor set the spacing was larger than or equal to 5 m as recorded. Based upon the analysis of quarries and satellite images, the rock conditions were reportedly found to be good with globally massive granite gneiss environment. The assessment done in the quarries also showed overall rock mass condition equal to or higher than good class.

Four additional boreholes were drilled corresponding to the cavern, shaft and portal location. One of the holes was drilled inclined at an angle of 30° to specifically target a dyke location found earlier at the detailed feasibility stage, passing through the cavern layout. These holes were drilled to a depth of around 200 m passing across the cavern bottom from the ground surface. It was revealed from the cores that thickness of the topsoil, including lateritic soil, was about 20 m and rock mass below the soil formation was fresh and suitable for construction of an underground oil storage facility. The supplementary investigation confirmed that the overall quality of the granitic rock mass was good, as observed at the detailed feasibility stage. In addition, this investigation confirmed the presence of mafic dykes, made of dolerite, and one main dyke in particular located at the south of the site and crossed by one of the boreholes. Further, the inclined borehole also revealed a clear correlation between the dyke and tectonics as well as with hydrothermal alteration inside the surrounding granite along dyke walls. Hydrothermal alteration was also

Table 1. Shape factor constant

	Shape factor constant	
Cavern shape	A	B
D-shaped	4.0	1.5
Horse shoe	3.1	2.7
Circular	3.0	3.0
Square	1.9	1.9
Oval	5.0	2.0

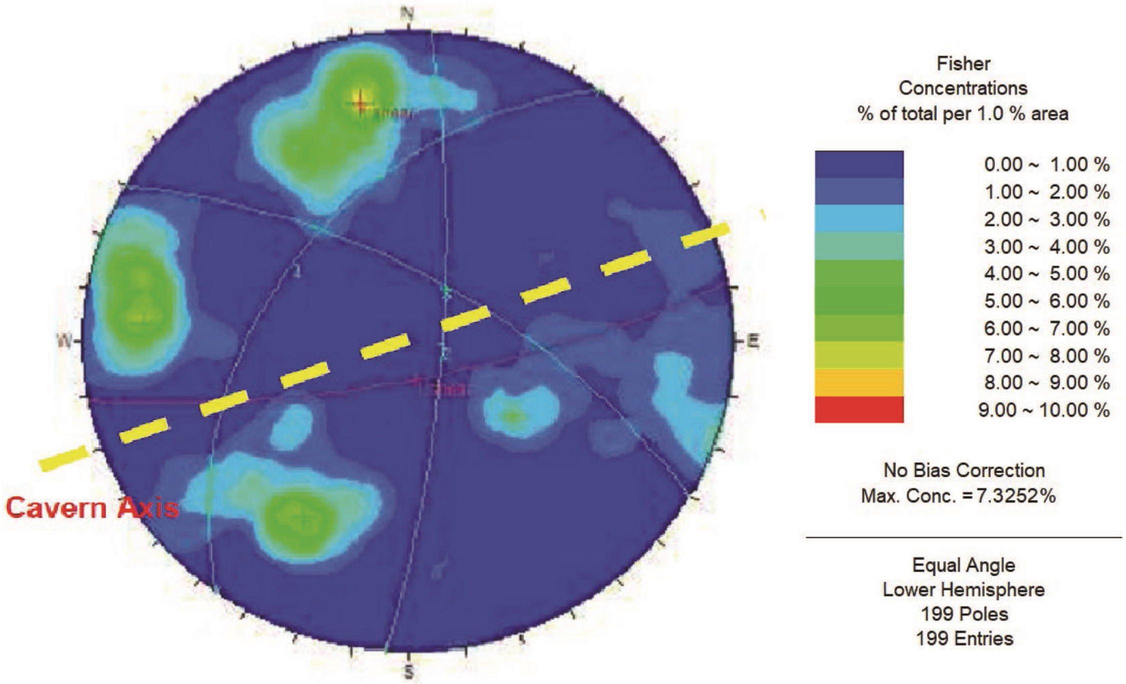

Figure 1. Contour plot in the lower hemisphere showing maximum concentration.

observed along another borehole close to the dyke which crossed it in the shallower section.

Seismic refraction survey carried out earlier in the detailed feasibility stage was further refined and carried out over a limited zone running parallel and perpendicular to the proposed cavern layout to locate different zones of soil and rock underneath the caverns. Seismic refraction survey data revealed, the presence of three distinct layers with different seismic velocities of 292–396 m/s, representing lateritic soil with loose to semi-compact overburden of varying compactness, relatively hard laterite with 1358–1880 m/s, and hard laterite of massive granite/granite gneiss at the bottom with 4000–5365 m/s. No wide low-velocity zone, which could represent a main or major fractured/faulted zone, was detected on this site. However, this does not exclude the potentiality of small fractured/faulted zones.

From the above description of the various rock types and investigations, several rock classes were distinguished based upon the Q system[4]. It was observed that about 90% of rock mass at cavern level was of good to very good quality having $Q > 10$, while only less than 10% was poor to very poor quality ($Q < 0.1$) in terms of faults, dykes and hydrothermal alteration zones. No major fault was evidenced at the site; however, one main dolerite dyke (N95°E), associated with intense hydrothermal alteration along the walls was found on the eastern and western sides of the cavern layout. Four different types of lineament (L1, L2, L3 and L4) were also located through satellite images with strike varying between N30°E and N160°E and dip angle of 85°W (Figure 2).

In order to estimate permeability of rock mass, short and long duration injection fall-off tests were carried out in boreholes at 20 m intervals up to elevation of –60 m msl, that is, cavern bottom. Based on these results, it was observed that the average permeability of rock mass was in the range 3×10^{-8} m/s at the cavern level (between –30 and –60 m msl). In some cases where close jointing was observed, higher permeability values of the order of 10^{-5} m/s were recorded. Analysis of the interference test carried out between two nearby vertical holes indicated low permeability value in the range 5×10^{-5} m/s.

Pre-construction stage

Prior to the commencement of construction activities, investigations are mandated to be undertaken by the contractor in order to focus on specific considerations such as the portal, water curtain tunnels and shafts of the underground storage facilities, and to clarify any specific aspects of site conditions with respect to the final layout of the facilities. Before initiating additional field investigations after the award of the contract, the pre-project and basic design stage engineering and design reports for the selected plan are carefully reviewed. This helps the contractor identify gaps and missing information which could be used for detailed engineering of underground structures.

Geological mapping of the area carried out as part of the mandatory studies showed different lineament sets mainly oriented along NW–SE and NE–SW directions.

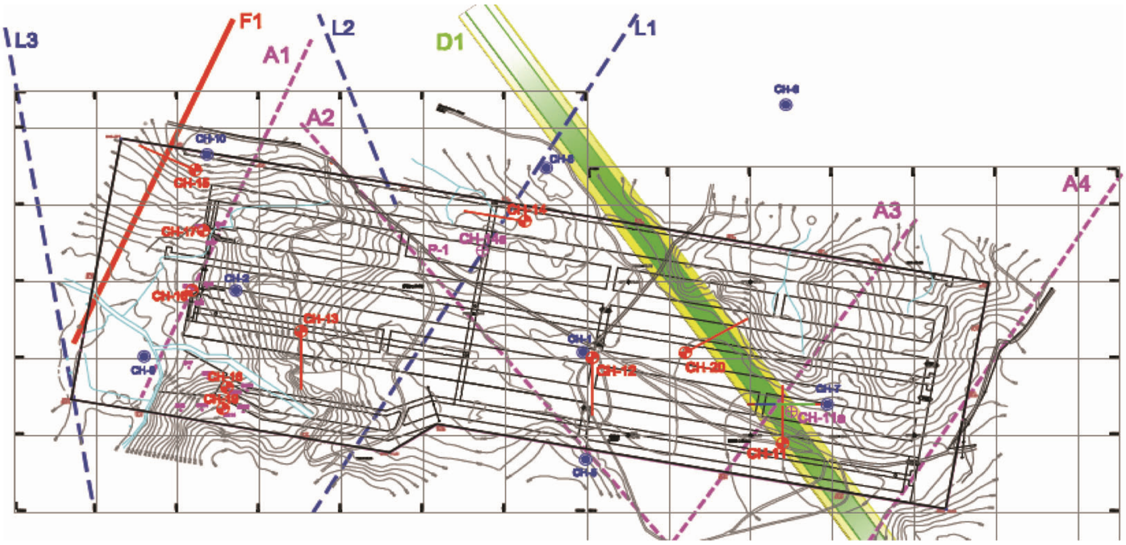

Figure 2. Layout and geological map of the underground storage system.

The results of geological structure analysis showed the presence of one main dyke (D1) and seven different lineaments (L1, L2, L3, L4, A1, A2 and A3) in and around the project area. The results were relatively consistent with the previous mapping results that showed predominance of N–S direction (N002/89).

In this investigation stage, ten core boreholes and 16 destructive boreholes were drilled for the identification of dyke and alteration borders and fault zone in the shaft, access tunnel and portal area. Four holes were drilled for dyke clarification and two holes were drilled for confirming the existence of faults. The main dolerite dyke was observed at 2–3 of the boreholes based on which its continuity in N–S direction could be established, which showed isolated presence in E–W direction during BED stage. The bedrock was mainly composed of granite gneiss which is covered with topsoil that consists of landfill, laterite, residual soil and weathered rock with thickness 8.3–19.0 m. The RQD profile was in the range 89%–100%. The result of direct shear tests indicated cohesion of intact rock in the range 2–20 MPa and internal friction angle varying between 26° and 36°. The results of strength of rock tests indicated that tensile strength was in the range 5–10 MPa, the uniaxial compressive strength in the range 50–70 MPa and Young's modulus between 35 and 50 GPa.

Descriptions from various geological maps for each type of rock mass showed similar trend while comparing with borehole data. Type I and type II indicate fresh and slightly weathered with hard rock mass, type III indicates moderately weathered with hard rock mass, type IV indicates geological lineament, while type IV is basically highly weathered and weak rock mass as fault, dyke and hydrothermal alteration zone. These were further categorized based on the Q system[4]: class I (good, $10 < Q < 40$) and class II (fair, $4 < Q < 10$) to design support requirements for caverns. For numerical modelling, Q values of 20 (based on the average excavation values) and 4 (representing the boundary of fair/poor rock mass) were considered to represent good and fair rock mass conditions. Most of the excavations were expected to fall in as Classes I and II type rock mass conditions. Classes III and IV were expected to be encountered in 15%–20% and class V in less than 10% cases.

Joint strength testing and parameter selection

The shear strength along a discontinuity in a soil or rock mass in geotechnical engineering is governed by the persistence and roughness of discontinuity, surface infill material in the discontinuity, presence and pressure of gases and fluids (e.g. water, oil), and possible cementation along the discontinuity. Different approaches are used for the determination of joint parameters. However, Barton–Bandis[5] criterion for rock joint strength and deformability is widely used (eq. (4)). The shear strength of natural rock joints is given as

$$\zeta = \sigma_n \tan\left(\phi_b + \mathrm{JRC}\log_{10}\left(\frac{\mathrm{JCS}}{\sigma_n}\right)\right), \tag{4}$$

where ϕ_b is the basic friction angle of the surface, JRC the joint roughness coefficient and JCS is the joint wall compressive strength. The values of parameters of eq. (4) can be obtained from IS 11315 (Part-4)[6]. Based on this approach, a joint friction angle of 35° and cohesion of around 0.2 MPa were estimated. However, in case of filled joints, values based on Barton and co-workers[7] recommendations are used as guidance.

Table 2. Typical geotechnical parameters for rock mass

Parameters	Values
Modulus of deformation (E_R) (GPa)	36
Cohesion (MPa)	12
Friction angle (°)	45
Hoek Brown criterion (m_b)	8.02
Hoek Brown criterion (s)	0.0205
Hoek Brown criterion (a)	0.50
Joint friction angle (°)	35
In situ stress ratio	2.4

Design requirements

The development of a geotechnical model of the project essentially consists of a comprehensive evaluation of geological and geotechnical data, review of additional site and laboratory investigations, site characterization, identification of main geological features such as dykes, rock mass and soil classification, description of *in situ* stress conditions, groundwater conditions, blast damage factor, excavation sequence, as well as support and stabilization measures, including calculations to verify the long-term stability of design, verification and modification through back analysis. Initially, rock support was designed using typical rock support chart as proposed by Barton[4]. The rock caverns were then numerically analysed with this support system to check their stability in terms of wedge stability, displacement and stresses to finalize rock support and excavation sequences[1]. Table 2 shows typical geotechnical parameter values estimated for the rock mass.

The data presented in Table 2 can be used to calculate the major and minor principal stresses using eq. (5)

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a, \quad (5)$$

where σ_1 and σ_3 are the axial (major) and confining (minor) effective principal stresses respectively. σ_{ci} is the uniaxial compressive strength (UCS) of the intact rock material. The value of UCS varies from 30 to 90 MPa for very poor rock to good rock. m_b is a reduced value (for the rock mass) of the material curve fitting constant m_i (for the intact rock). s and a are constants which depend upon the characteristics of the rock mass.

Concluding remarks

The uncertainties associated with underground construction call for continuing design during construction. A continuous adjustment of the excavation and support methods to the actual rock mass conditions contributes to safe and economical excavation. A prerequisite for successful application of such an observational approach is an appropriate basic design, based on detailed investigation results, which should incorporate means and tools to cope with difficult conditions. Interpretation of geological, geotechnical and hydrogeological data due to the complexity of the ground and interaction between ground and construction are essential for successful completion of the project. Finally, the contractual set-up has to allow for continuous optimization of the construction. Internet has made it possible to involve off-site experts at comparatively low cost in real time. All data can be made available on a server, allowing follow-up of the construction from any part of the world.

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Received 26 July 2017; revised accepted 21 March 2018

doi: 10.18520/cs/v115/i4/638-643