A Navratna Company

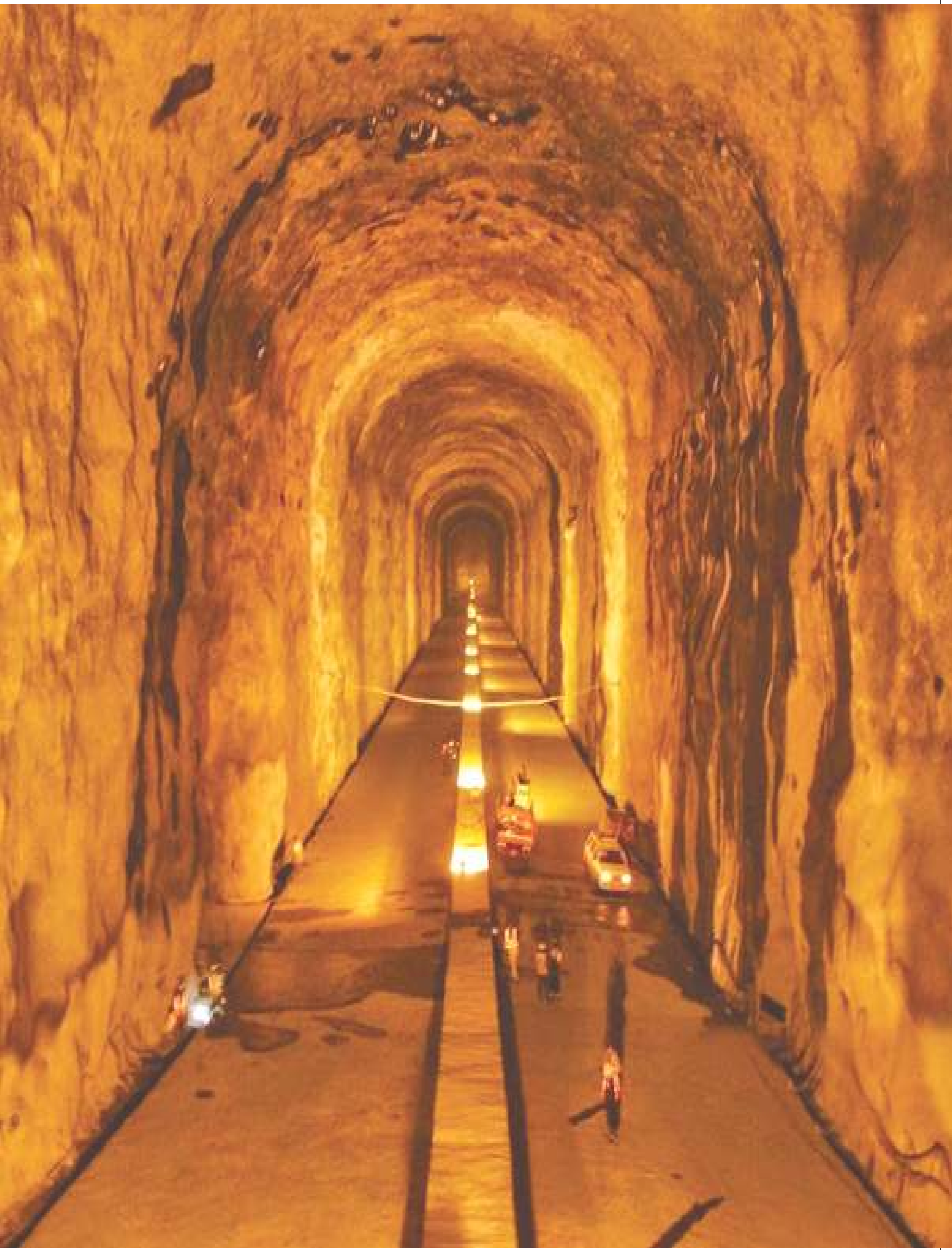
Underground
Storage Technologies

Edited by
Dr. A. Nanda, Dr. R. Rath & Dr. A. Usmani

ISBN No. 978-93-5254-383-0

Editors:
Dr. Atul Nanda , Dr. Ranjit Rath, Dr. Altaf Usmani

Published by:
Engineers India Ltd., New Delhi - 110066



Message

From Chairman & Managing Director
Engineers India Limited

Human beings are endowed with a natural spirit of creativity and innovation. At the very outset, we wish to take this opportunity to commend the indomitable spirit, passion and perseverance of the Engineers and other team members involved in probably one of the most enterprising initiatives in the Indian Oil & Gas industry. The completion of the Phase-I of the Underground Caverns brings about a culmination of a journey full of travails, knowhow, engineering and construction marvel. This book is a true compendium of the knowledge acquired in the process, to hopefully, enable the future generations to benefit out of this strategic knowledge for the country.

Strange are the ways of nature. It appears that there is a strange balancing, act which providence seems to have crafted out for humanity as a challenge. On the one hand, there are countries which are endowed with natural resources in plenty, to not only meet their captive requirements but to address the wholesome requirements of other nations. On the contrary, there are a set of developing nations including India, which have been deprived of the priceless fossil fuels in abundance, to meet the urge and the craving demands of an ambitious and restless populace. It is this very fact, that has possibly provided crude oil the status of becoming one of the most political and game changing commodity across the globe. It is now universally recognized that crude oil has the power to bring about cataclysmic changes across the globe, based on its fluctuating prices, demand-supply scenario and speculative propensities.

A nation, such as India urging to move ahead to join the league of developed and powerful nations needs energy in bounty. However, given the strategic and speculative intent behind the crude oil prices, it is equally important to shield the country against the vagaries of massive volatility and cyclic upheavals in crude oil prices. It is against this backdrop that the Government of India decided in January 2006 to yield, to the tremendous background work carried out by EIL earlier, in developing the perspective for setting up strategic storages for crude oil, to secure the country against vicissitudes of cycles. It was, as if it were, the culmination of a dream, which EIL began to see in early 1999.

Through the creation of ISPRL, the steps were taken by the Government to set the ball rolling for setting up the underground storages. EIL was in 2006 formally entrusted with the task of implementation of three Rock Caverns to the tune of 5.33 MMT storage capacity at Vishakhapatnam, Mangalore and Padur. The Vizag Cavern is already commissioned and crude has been taken in while Mangalore and Padur are in advanced stages of completion.

Given the fact, that it was a first time endeavor, there have been immense learnings out of this initiative. To start with, it is a matter of privilege and pride to acknowledge that on a technology, which to start with, was exclusive prerogative of foreign knowhow, EIL today, can claim with pride of having absorbed this unique know-how, which is hugely technology driven. The country can now entrust its faith in the acquired knowledge by EIL, for all the

Message

future phases of setting up Rock Caverns in the country. The journey certainly has been exciting and tumultuous. At every step of this enterprise, there has been learning, as the endeavor is geology driven and its predictability in conceptualization, can therefore, never be a certainty. It is always a prognostication and a discovering exercise. EIL in the process, from time to time, has adapted itself, to learn big in this unique exercise.

As the refining capacity in the country continues to increase, the Underground Caverns are sure to emerge as an inevitable necessity. The scarcity of land will create more and more avenues for Underground storages. As a matter of fact, the process has already begun with the Vizag Refinery taking a cue to obviate the shortage of space by utilizing the Underground Cavern as a crude buffer.

This book is the culmination of huge amount of knowledge acquired in the process by some extraordinary engineers whose devotion, passion and perseverance has been exemplary. The spirit to create, innovate and lead through example has been the single motto, which perhaps these engineers have taken upon themselves, to create a unique asset for the country. The book dwells on the entire technology backdrop, to set the course for future, in this interesting but intricate technology domain. The fact, that these brilliant engineers have decided to put together a compendium of knowledge itself, is a great acknowledgement of their humility in sharing this unique knowledge acquired by them over the years. Let us join in applauding this yeomen's effort and consolidation of this knowledge, as probably, one of the greatest contributions to engineering marvels, that EIL has been engaged with.

We also would like to thank the ISPRL team for guiding, coaxing and encouraging their respective teams and providing outstanding leadership to make this endeavor possible. Without the unstinted support of ISPRL, such a mammoth exercise would perhaps have not been realized as well as it has been.

On behalf of authors of this huge compendium, we would like to take this opportunity to dedicate this book to the spirit of knowledge and creation and the humility of sharing, to allow it to grow to excel for all times to come!

Sanjay Gupta

Chairman & Managing Director

Engineers India Limited

Message

From CEO & Managing Director ISPRL

Delighted to see that Dr Nanda, and his team have managed to put together a book on underground storage technologies which includes the learning's from the rock caverns storages that ISPRL, has constructed. Right from the time, I was involved in executing SALPG's LPG cavern storage facility at Visakhapatnam, I had this burning desire to enlighten the oil sector about the advantages of the technology used. This remarkable technology was used for the first time in India, for hydrocarbon storage, in the SALPG project.

Kudos to Dr Nanda, Dr Ranjit Rath, Dr. Altaf Usmani and all the team members, for taking this initiative, on their own, and for disseminating the knowledge gained, in this field. In my opinion, the oil industry should be seriously looking at this technology, for more storage facilities, because of the high-level of safety it offers, in today's security environment and also for the very low land requirements compared to aboveground storages.

When the ISPRL projects were commenced, Ministry of Petroleum and Natural Gas, wished that Engineers India Limited, an organisation under this Ministry, becomes a repository of the knowledge gained from the building of the strategic reserves. This book is a proof of the knowledge gained. We are proud, that the Padur project, which is the largest project of ISPRL was executed without the assistance of a full time Foreign Back Up Consultant and was managed by a team of Indian engineers and geoscientists' from ISPRL and EIL.

The ISPRL projects were gigantic in size. They required underground excavation rates, which were never attempted in the country. The projects involved deployment of large number of equipment and were to be executed with very tight time schedules. Thus they presented challenges in terms of safety, not only due to the large movement of men and equipment in constrained spaces, but also due to the geological uncertainties.

I am glad that, the EIL team has covered all the major aspects of underground rock caverns. I thank the team from EIL for also throwing light on Underground concrete tanks and underground solution mined salt caverns. These technologies have not been used in the country so far. We however look forward to their use, in the future reserves, which are required to ensure the energy security of this great nation.

Rajan K Pillai

CEO & Managing Director

Indian Strategic Petroleum Reserves Limited

TABLE OF CONTENTS

A Navratna Company

PREFACE

Underground storage of crude oil is more secure, safe, economical and environmental friendly than conventional above ground storage. From a energy security perspective, most of the developed countries have stockpiles of crude oil in different types of underground storage facilities. In India, underground storage is a new concept and the first underground rock cavern for storage of crude oil has recently been commissioned at Vishakhapatnam and the two other projects namely Mangalore and Padur are likely to be commissioned by 2015-16.

A compendium of knowhow and experiences that was gained during the implementation of large underground rock caverns for storage of crude oil in our country as well as the various feasibility studies carried out for future storage programmes, this book is an attempt to collect, collate and comprehend the underground storage technologies with specific focus on the rock cavern storage alternatives.

The adopted pattern of this book is an lucid effort to make readers aware about the requirements, concept and technical outlines of large underground unlined rock caverns created for storage of crude oil and the entailing key aspects of geological setting, hydrogeological regime and geotechnical characterizations that are essential for such large and complex projects. The comprehensive narrative account provide an insight into different factors that contribute to the overall planning and execution of such subsurface projects, including guidelines for construction and brief details about the above ground process aspects. An attempt has been made to present select case studies along with photographs taken through the entire execution period so as to make it interesting and create a connect with the readers. In addition to the detailed description of various aspects of unlined rock caverns, an introductory description of the other advance technologies of underground storage alternatives such as underground salt caverns and concrete tanks has also been presented in this compilation.

We are optimistic about the compilation and hope that the presented narrative in this edited volume will give the readers not only an introduction to the exciting and challenging field of underground storages, but will also aid in the planning, design, construction and operation of underground storage technologies in our country.

The editors would like to thank all the authors for sparing their valuable time and considerable efforts put in towards compilation of this volume. We would also like to thank each and everyone who has contributed for the successful execution of underground storage projects from Engineers India Limited (EIL), New Delhi; Indian Strategic Petroleum Reserves Limited (ISPRL), New Delhi; SWECO Interntional, Stockholm; GEOSTOCK, Paris; Hindustan Construction Corporation (HCC), Mumbai; GEOCONSULT, Singapore and New Delhi; SK Engineering & Construction, Seoul; KCT, New Delhi; DAEWOO Engineering, Seoul; TV Engineering Consultants, Johannesburg; and DEEP Underground Engineering, Bremer.

This preface will remain incomplete without giving a citation to Shri Suresh Malkani, formerly with EIL, without whose relentless and unstinting effort; these projects would not have got a kick start in India under the aegis of Ministry of Petroleum and Natural Gas, Government of India with the Phase I storage program commencing at three locations namely Vishakhapatnam, Mangalore and Padur. Subsequently, the Govt. of India has now initiated the activities for the Phase II storage program.

Having been associated with these coveted projects and been involved in successful completion of the underground excavation works followed by precommissioning and commissioning activities, Ideation of such a book happened over a discussion on the complexities and uncertaintities that accompany these kind of large subsurface projects. In this context, The editors would like to thank the senior leadership and management of ISPRL and specifically the support and

kind consent offered by Sh. Rajan K Pillai, MD & CEO, ISPRL and Sh. HPS Ahuja, Dy. CEO, ISPRL. The constant encouragement offered by the senior leadership & management of EIL and specifically by our C&MD Sh. Sanjay Gupta is acknowledged with thanks.

We also wish to place on record our thanks to every other stakeholders, partners and team members who have been associated with these prestigious projects and without whose support and hardwork these projects would not have taken the shape towards a greater resilient energy security mandate of India. The reviewers, who have meticulously read through the entire manuscript and offered their valuable comments and feedbacks, are also thankfully acknowledged.

A subject of such technological ingenuity can not be considered complete in all respect and at EIL we are committed to these pursuits with passion and sustained focus. Should you, the readers, wish to share any valuable feedback, observations and insights; we will be glad to receive them with all humility at r.rath@eil.co.in.

Dr. A. Nanda, Dr. R. Rath and Dr. A. Usmani
Editors

New Delhi
October, 2015

PART -I

Underground Rock Caverns

A Navratna Company

INTRODUCTION

R. Rath & A. Nanda

CHAPTER - 1

1.1 Energy Security

Oil consuming economies, in particular Indian economy is becoming increasingly vulnerable to oil and gas supply disruptions in the coming decades due to growing reliance on specific regions for energy supply, political instability associated with such regions, diminished market buffers for offsetting supply losses and operating inventory and continued limited responsiveness (price elasticity) of oil demand in the short run. Political and economic damages from oil supply interruptions could be limited through emergency preparedness and response measures, both long term and short term in nature.

The long term measures include the following:

- Diversification of oil import sources
- Augmented Exploration & Production Efforts
- Enhanced Oil Recovery
- Improving efficiency
- Removing market impediments
- Investing in alternative energy technologies
- Maintaining dialogue with oil producers

Long term measures listed above can reduce the likelihood or severity of oil supply interruption, but they are of limited help once oil supply is curtailed, prices skyrocket, people panic and begin hoarding oil and cancel investments in the face of market uncertainty. Thus emergency response measures that could be introduced to alleviate the gap in the oil supply and demand include demand restraint, fuel switching, surge production and emergency oil stocks.

Emergency oil stocks are a powerful and direct defence against oil disruption. The most compelling reason for government oil stock piling is that it can be used reliably at their discretion and during an emergency to make up for the shortfall caused by interrupted oil supply. These stocks also serve as a deterrent to politically or economically motivated supply disruptions and form a key tool of foreign policy.

1.2 International Scenario

International practices outline the stock piling reserve of USA and EU at a large inventory base. Following the Arab – Israeli war and the Arab oil embargo of 1973, most industrialized countries have a system of maintaining petroleum reserves to deal with sudden disruption in supply. The European Union (EU) requires member countries to maintain stocks of petroleum products at a level corresponding to at least 90 days average daily internal consumption in the preceding calendar year. Stocks are maintained in the form of crude oil and intermediate finished products.

An association of 29 countries, the International Energy Agency (IEA) mandates the participating countries to maintain emergency reserves equivalent to at least 90 days of net imports, with the flexibility of meeting this requirement through crude oil and refined products. Participating countries guarantee a minimum obligation by holding stocks as Government emergency reserves, through specialized stockholding agencies or by mandating minimum stock holding obligation on Industry.

Globally, three approaches are followed for meeting stockholding requirements viz. Industry stocks, Government stocks, and Agency stocks. While some countries use only one category of stockholding to meet their minimum obligation, other countries use a combination of these approaches. Types of inventory stockpiling of major nations are presented at Table 1.1.

Table 1.1: Types of inventory stockpiling stocks and global scenario

Type of Stock	Owner	Remarks / Countries
a. Industry stocks	Stocks held by Industry, for commercial intent or otherwise.	
b. Government stocks	Typically financed through budget allocation of respective Governments and held exclusively for emergency purpose.	Czech Republic, Ireland, Japan, Korea, New Zealand, Poland and the USA.
c. Agency stocks	The stock holding arrangement involves establishing a separate agency endowed with the responsibility of holding all or part of the stock obligation. These are government sponsored schemes and held under co-operative cost sharing arrangement of the industry.	Belgium, Finland, Hungary, the Netherlands, Portugal, Spain, Austria, Denmark, France, Germany and Switzerland.

1.3 Indian Context

Until 1998 an empowered committee namely Oil Coordination Committee (OCC), under the aegis of Ministry of Petroleum & Natural Gas (MoP&NG) was responsible to manage crude oil and petroleum products both in terms of supply and demand including stock piling, while Indian Oil and Gas sector was under a protected regime otherwise referred to as the Administered Price Mechanism (APM). Oil marketing companies during this period were assured of their returns on investments through APM, while also developed storage facilities for inventory of crude oil and products following OCC mandate. Thus under the assurance of APM, oil marketing companies along with OCC and MoP&NG undertook several proactive measures such as advance planning, procurement of additional cargos to top up inventories, tapping additional supply sources etc. and thus the oil and gas sector did not witness any major supply side disruption.

Earlier years 1998 to 2002 witnessed dismantling of APM in a phased manner and thus effective 1st April 2002, oil marketing companies in a free market scenario were expected to have crude oil and petroleum products stock piling only with a commercial intent. However owing to bare minimum storage inventory post APM and based on the projected demand of petroleum products and increased import dependency for crude oil, MoP&NG decided that strategic storage of crude oil is an imperative for energy security.

Government of India recognised the intent to deal with contingencies arising out of supply disruption of crude oil and Union Cabinet on 7th January 2004, while noting the need for a strategic crude oil reserve of 15MMT approved construction of strategic crude oil reserves of 5 MMT under Phase I storage program which was equivalent to about 14 days cover in consumption basis, then. Thereafter, the Expenditure Finance Committee (EFC) while deliberating on financing of crude oil reserves, inter alia observed that the storage capacity should be split in two parts, viz. a) a core critical sovereign reserve; and b) a secondary or subaltern reserve in which oil marketing companies could participate.

On 6th January 2006, Cabinet Committee on Economic Affairs (CCEA) accorded its approval for Phase I storage program entailing 5 MMT storage capacity which was funded from the existing funds available with Oil Industry

Development Board (OIDB). In order to implement and operate these strategic petroleum reserves (SPRs) a Special Purpose Vehicle (SPV) namely Indian Strategic Petroleum Reserves Ltd. (ISPRL) was created under the aegis of IOCL and subsequently transferred to OIDB. Crude oil from these reserves would be released under the advice of an Inter-ministerial Empowered Committee in the event of any short term disruption.

SPRs under Phase I of the strategic storage program of Government of India are being taken up at following sites for a total storage capacity of about 5.33 MMT storage facility is being implemented and scheduled to be commissioned by 2015/16:

Table 1.2: Locations of phase I storage program

Location	Storage Technology	Capacity
Vishakhapatnam	Underground Unlined Rock Caverns	1.33 MMT
Mangalore	Underground Unlined Rock Caverns	1.50 MMT
Padur (Udupi)	Underground Unlined Rock Caverns	2.50 MMT

The envisaged storage capacity at Vishakhapatnam was initially 1.0 MMT. However, during the basic design stage, based on site assessment and cost analysis there upon, it was found to be technically suitable to have an enhanced storage capacity with relatively reduced cost, thus the storage capacity was augmented to 1.33 MMT. This decision was also beneficial in the sense that it offered a cushion effect to other two storage sites, in terms of total storage capacity.

Engineers India Ltd. (EIL), the consultancy organisation under the aegis of MoP&NG have been involved in these SPR projects since 1999, starting with preparation of Pre Feasibility Report indicating possible storage locations and cluster of refineries these conceived storages would cater. Subsequently, EIL undertook preparation of Detailed Project Reports with cost estimates for Project Appraisal and Approval by Cabinet. Effective 2008 EIL has been mandated to execute these SPR projects at the three aforesaid sites as the Project Management Consultant.

In India, several options are available to have stock piling to hold the equivalent of 90 days of net import. The Phase I storage program by the Government of India is to be maintained by ISPRL as Government stocks. The Phase II storage program would entail a mix alternative of Industry stock, with two splits viz. a) obligatory stock holding mandated by GoI and b) voluntary commercial stocks held by each entity. With several alternatives of such arrangements being available worldwide, these entities could adopt the Agency stock arrangement, and ISPRL would be mandated to hold stock on behalf of these entities against payment of a storage fee. International practices suggest that the mechanism of agency stock ensures energy security interventions and concurrent revenue generation as well. Depending on the locational advantage of storage installations and it's connectivity with the hinterland refineries etc. enhances the utility quotient of such inventory stockpiling and thus attracts better return on investment.

In line with the Integrated Energy Policy of Govt. of India and as per the Approach Paper prepared by MoP&NG in December 2009, the storage facilities are required to be enhanced by another 13.32 MMT under Phase II. EIL was assigned to carry out a Pre Feasibility Study for selection of sites for creation of storage facilities for both crude oil and petroleum products, with type of storage and maximum possible capacity of storage for each selected site.

Further to the Pre Feasibility Studies, EIL was engaged to carry out Detailed Feasibility Studies and prepare DPRs for storage of crude oil at four selected sites namely Padur (underground unlined rock caverns), Rajkot (underground concrete tanks), Bikaner (solution mined

salt caverns) and Chandikhol (underground unlined rock caverns) during July 2011.

The Detailed Feasibility Studies have been completed and the DPRs for the four selected sites namely Padur (2.5 MMT), Rajkot (2.5 MMT), Bikaner (3.75 MMT) and Chandikhol (3.75 MMT) have been submitted in April 2013. Currently, the proposition of Phase II storage program is under active consideration by the Government of India (Refer Figure1.1).

Table 1.3: Locations of phase ii storage program (Planned)

Location	Storage Technology	Storage Capacity
Chandikhol, Odisha	U/G Rock Caverns	3.75 MMT
Bikaner , Rajasthan	U/G Salt Caverns	3.75 MMT
Padur (Udupi) , Karnataka	U/G Rock Caverns	2.5 MMT
Rajkot , Gujarat	U/G Concrete Tanks	2.5 MMT

In fact, reckoning India's emergence as a large refining hub supported by a vast coast line, its geographic vantage position along the trade route between oil producing countries of the Middle East and oil importing countries of Asia Pacific provides a significant opportunity to be developed as a regional storage hub in the APAC region.

The existing infrastructure of offshore oil terminals located across the coast line of India and corresponding pipeline connectivity offers a unique advantage for receipt and dispatch of crude oils through very large crude carriers (VLCCs), which further reduces the freight cos

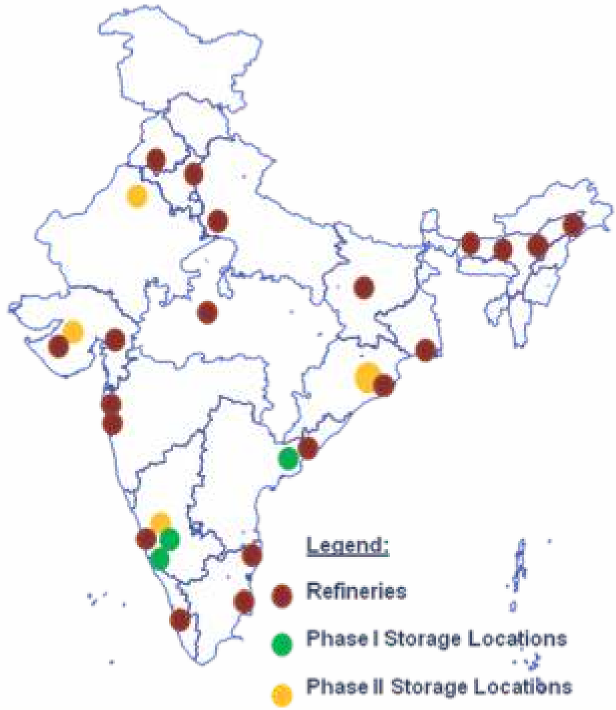


Figure 1.1: Indicative Locations of SPRs of both Phase I and Phase II storage program

Possible advantages that would accrue to the oil producing countries of Middle East by locating crude oil storages in India could be outlined as under:

a) Security against production disruptions;
b) Low cost storage in comparison to the floating storages;
c) Compartmentalized, on demand storage facilities;
d) Integrated offshore infrastructure for receipt and dispatch;
e) Availability of a significant market in terms of clients seeking crude oil supply.

Recognizing the win - win proposition, several Middle East counties have expressed their keenness to invest in the storage projects of India, thus bringing in necessary capital investment offering substantial commercial gain. While this would reduce Government spend; creation of additional storage facilities would mean accelerated economic development within the country. However, a regulatory framework would be necessary for mandating and overseeing creation, maintenance and emergency responsiveness of these stock piling alternatives.

1.4 Significance of the SPRs

In order to create such storage facilities for hydrocarbons, several technologies are available worldwide. Among all the storage options, underground unlined rock caverns, underground salt leached caverns or underground concrete tanks are highly cost effective when the inventory of storage is very large, and have been successfully adopted by several countries for stockpile and buffer storage purpose. Aboveground steel tanks are however the preferred options for industry stocks, when the inventory to be stored is not very large and meant as a feedstock for refineries.

While in USA, federal stocks are stored mostly in underground salt caverns, in Scandinavian countries, the underground unlined rock caverns are the preferred option. In case of Germany, France and UK salt caverns are the preferred alternative for hydrocarbon storage. In far-east countries, such as South Korea, Singapore, Japan, China etc. the storage facilities are developed in underground unlined rock caverns. In South Africa the concrete tanks are the adopted technology for the purpose of the crude oil stock piling. Adoption of the technology for storage has been mostly driven by the product to be stored and the geological setting of each country coupled with the advantage of available infrastructure.

The adopted underground storage technologies considered during the present pursuit are intrinsically safe. The underground storage facilities have distinct advantages such as the stored product is located at a depth and is fully isolated and secure; external fire does not affect the underground stored product; safety hazards on account of sabotage, storms, earthquake and explosions are practically nullified; surface land requirement is low and underground cavern storages require very low maintenance. The storage facilities have very minimum impact on the environment and for large storage capacities the capital cost per ton of storage is low. By products of underground storages such as the leached salt or rock debris have high economical value as well.

Be it federal stocks or storage of crude oil, natural gas and products for commercial purpose, the concept is widely practiced throughout the world. The IEA member countries, its affiliate countries and some large economies have realized the necessities of buffer stocks and have invested in creation of underground storage facilities / stock holding mechanisms for ensuring steady supply of stored products during exigencies. Thus far, the stored inventory has also been used for deriving commercial value through various mechanisms. The approaches of funding mechanisms and deriving commercial return from the stored inventory have ensured that the proposition of stockpiling of crude oil, natural gas or products are self sustainable. Therefore, while existing IEA member countries have SPRs in place, the growing economies like India and China are pursuing these initiatives for meeting supply exigencies.

Energy security interventions across the globe ensure and reinforce the interdependence between both supply and

demand sides. Crude oil being the most sought after commodity, petroleum economics govern the world trade relations and there has always been a tendency to balance out between the oil producers and consumer nations. Therefore, one significant approach towards energy security is to have an increased interdependence in terms of cross investments. While the oil consuming companies/nations take up investment in upstream sectors of oil economies, the oil producing companies/ nations tend to make investment in downstream assets in oil consuming countries such as crude oil storage inventory. These cross investments and mutual dependence throughout the hydrocarbon value chain go a long way for international harmony both in terms of supply side revenue generation and demand side consumer assurance.

Considering the realignment of petroleum economics hinged through the shale asset revolution in USA to start with, India is strongly poised to secure its energy sources, as the market will be more supply oriented. Given the economic surge that India is currently treading and to cater to the refining hub that India has become over a period of time, stock piling of crude oil is a strong imperative. Thus underground storage alternative for having such large inventory of crude oil will become need of the hour and will result in significant economic development across the value chain.

1.5 Underground Storage Technologies

In view of the present geo-political scenario of the world, energy security of a nation has gained paramount importance. In order to ensure energy security, federally owned oil stocks are stored in huge underground storage caverns. Strategic locations are selected with an option of providing the most flexible means of the nation's oil & gas transport network. These networks could be through interstate pipelines, through ships/ barges for coastal shipments or through dedicated fleets of crude carriers for cross country freight shipments.

Advantages of underground storages are essentially based on aspects such as Space Saving, Economical, Environmental Friendliness and Strategic Safety.

Selection of a storage concept for underground storage of crude oil & natural gas is made according to:

- Storage requirements;
- Geological setting of the site;
- Subsurface rock mass quality;
- Hydro-geological regime of the site;
- Storage product loading and unloading facilities
- Safety and environment conditions

Other possible value added propositions of underground storage technologies are suitable utilization of above ground land parcel, re-working of the muck as construction aggregate and evaporated brine as feedstock to chemical industry. Harnessing renewable energy such as solar energy and wind energy are other possible alternatives on aboveground land.

In general, five different types of storage concepts are in use:

- Type 1:Unlined underground rock caverns
- Type 2:Unlined underground salt caverns in salt beds and salt domes
- Type 3:Pore space storage in depleted oil & gas reservoirs or deep aquifers
- Type 4: Underground Concrete Tanks
- Type 5:Lined rock caverns

Type 1: Storage in Underground Unlined Rock Caverns

An established technology successfully adopted in many countries, the principle of storage essentially employs ground water pressure for containing the product within an unlined rock cavern. Based on a site campaign involving geological, geo-physical, geo-technical and geo hydrological investigations, it is imperative to establish that rock formations in conjunction with ground water conditions are competent for construction of caverns and suitable to store the hydrocarbons.

Figure 1.2: Unlined rock cavern

Type 2: Storage in Underground Salt Caverns

Salt caverns are created out of salt formations by a process called "Solution Mining". Essentially, the process involves drilling a well into a salt formation, then injecting massive amounts of fresh water. The dissolved salts are pumped out as brine and disposed. Besides being the most economical way to store petroleum products for long periods of time, the use of deep salt caverns is also one of the most environmentally safe options. Rock salt provides an excellent impervious environment for oil, fuel and gas storage- it is insoluble in hydrocarbons and does not show any chemical reactivity with oil and liquid fuels.

Type 3: Storage in Depleted Oil & Gas Reservoirs and Deep Aquifers

This type of storage involves usage of depleted reservoirs for storage of hydrocarbons by pumping the products in the reservoirs. Formerly a product bearing reservoir formation overlain by an impermeable rock formation satisfies the confinement requirements; it would be either a stratigraphic trap or a structural trap or both. The permeability and porosity conditions of the product bearing strata also conform to the storage requirements. Owing to its wide availability, this type of storage forms one of the most predominant storage options for natural gas.

In case of deep aquifers; the water bearing pore spaces are confined underneath an impermeable cap rock formation. The products are pumped into the pore spaces and contained within the formation.

Type 4: Storage in Underground Concrete Tanks

These are mounded concrete tanks developed for locations wherein the storage alternative is not supported by presence of competent rock, shallow ground water regime and adequate ground recharge possibility. Worldwide, such storage facility has been created only in South Africa and is in operation.

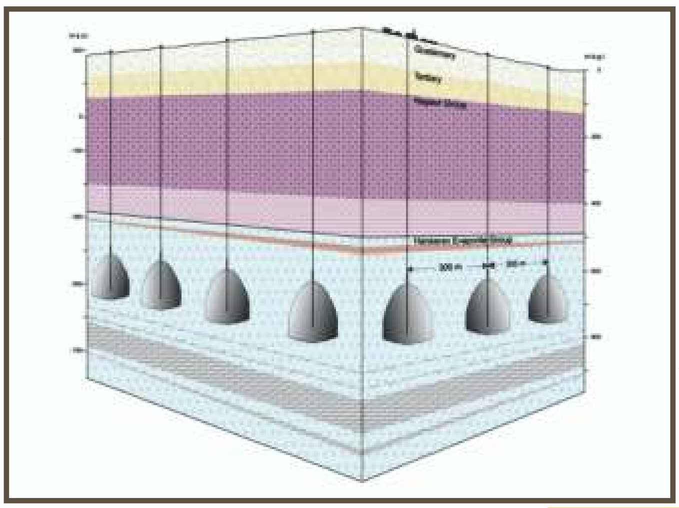


Figure 1.3: Underground salt caverns

Figure 1.4: Underground concrete tanks

Type 5: Storage in Lined Rock Caverns

This storage concept is suitable for any storage product without restriction and the product is stored in a neutral atmosphere as it is not in contact with rock formation and ground water. Thus quality is not disturbed. The lining for these storages is of steel alloy and reinforced concrete.

This technology provides advantages in terms of environmental aspect, improved safety and a wide variety of product storage option but is very expensive and not considered hereunder.

A comparative matrix of the four major types of storage facilities is presented at Table 1.4

Table 1.4: Comparison of types of crude oil storage

Storage Technology	Siting Requirements	Security	Safety	Economy
Solution Mined salt caverns	Thick salt formations and water for leaching	E	E	E
Unlined Rock Caverns	Competent rock and stable ground water conditions	E	E	G
Underground Concrete tanks	Hard soil /weak rock and low ground water table	G	G	E
Conventional steel tanks	Large land requirements	A	A	A

Note: E - Excellent; G - Good; A - Average

Under Phase I storage program, unnlined rock cavern storage technology has been adopted for creation of the SPRs and the details are enumerated herewith.

1.6 Structure of the Book

This book is structured into eleven chapters out of which chapters 2-9 deal with the details of design and construction of underground unlined rock caverns for storage of crude oil. "Chapter 10 deals with major project accomplishments and key project learning's". Chapter 11 and 12 give a brief introduction to the concept of underground concrete tanks and solution mined salt caverns for storage of crude oil.

Chapter 1 gives the overall scenario of oil and gas development and challenges in our country and outlines the requirement of underground storages. Basic requirements of underground rock caverns are discussed in detail in Chapter 2. Details of various site investigations carried out during different stages of execution of project are discussed in Chapter 3 while geological and hydrological aspects of rock caverns are described in Chapter 4 and 5 respectively. Chapter 6 discusses in detail geotechnical design of unlined rock caverns starting from availability of input parameters to final monitoring and verification of design during the construction stage. Case studies of some of the existing projects along with lessons learnt during construction are also described in detail which will prove to be a good learning point for all the engineers dealing with underground structures.

Design and construction of concrete works required in an unlined rock caverns are discussed in Chapter 7 which includes mainly design of tunnel and shaft plugs. Process related requirements along with other operation requirements of unlined rock caverns are discussed in Chapter 8. Chapter 9 is included in the book to specifically discuss construction requirements for such large projects along with discussion on associated construction problems and its coordination with active design cycle for tunnels and caverns.

Chapter 10 discusses various project achievements, general project description and important learning's gained from execution pint of view. Storage of crude oil in the underground concrete tanks is discussed in Chapter 11 while storage of these products in solution mined salt caverns is discussed in Chapter 12.

BASIC REQUIREMENTS

R. Rath, A. Usmani & A.Nanda

CHAPTER - 2

2.1 Principle of Storage

The basic principle of storage in unlined underground rock caverns is the hydraulic confinement. Therefore the rock caverns are planned at a design depth such that sufficient hydrostatic pressure is available within the surrounding rockmass so as to counter the vapour pressure of the stored product, which would otherwise escape through the fissures and cracks. In order to secure hydraulic gradient vector from the rockmass towards the cavern, a water curtain system is provided consisting of galleries located above the crown of the cavern. Boreholes with pre defined patterns and orientations are drilled from the water curtain tunnel so as to intersect all the pervasive joints of the surrounding rock mass. A well saturated rock mass and the ground water gradient vector flowing into caverns, ensures sealing of the stored product from leakage as shown in Figure 2.1. Specific attention is also warranted to ensure that the rock mass remains saturated with water even while excavation works are in progress, so that conditions of de-saturation are eliminated. During operation the seepage water is pumped out and sent to treatment plant.

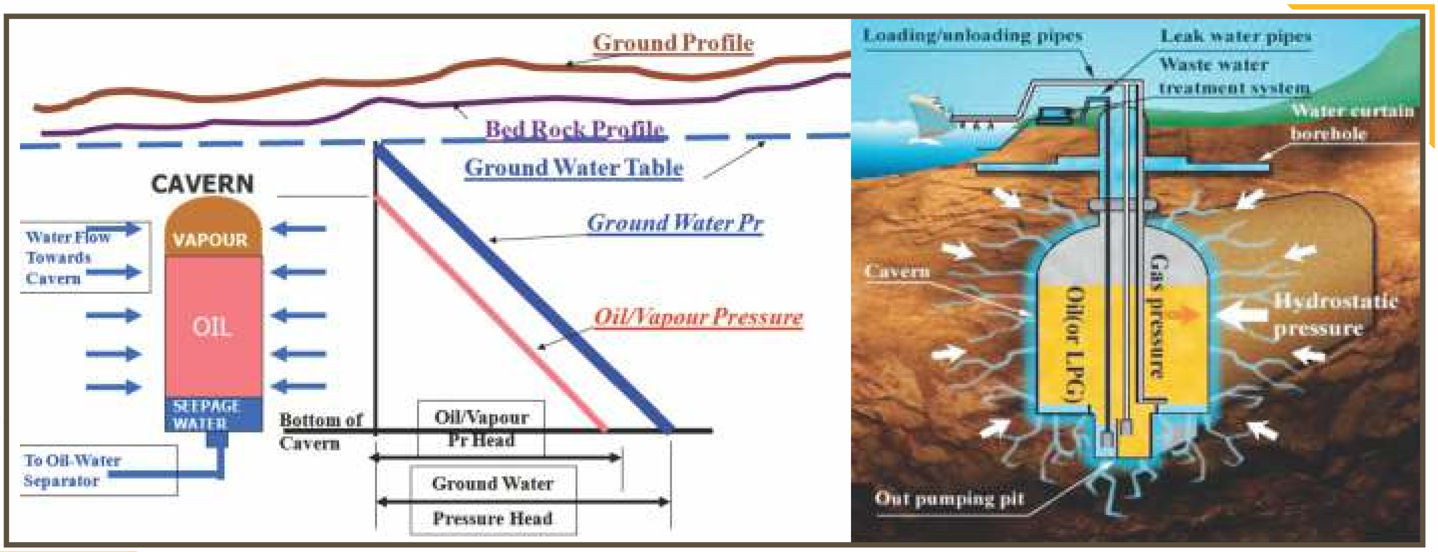


Figure 2.1 : Principle of crude oil storage in underground rock caverns

2.2 Layout

Unlined underground rock caverns are constructed in such a way that the stored product remains confined within the caverns with an umbrella of sufficient hydraulic pressure surrounding it. Layout of storage caverns depends primarily on two aspects viz. the attributes of major discontinuities that characterises the geological setting of the selected site and orientation of maximum and minimum horizontal stresses encountered at the site. In

principle, the longitudinal axis of caverns is aligned parallel to the direction of major principal stresses, however it is also verified that the attributes of major features are not adversely oriented with respect to the caverns.

Each underground rock cavern storage unit primarily consists of caverns, shafts, pump pits water curtain tunnels, access tunnels and water curtain boreholes. The U shaped storage units are connected to the surface by vertical shafts. Arrangement of storage unit is planned either with single shaft system or double shaft system

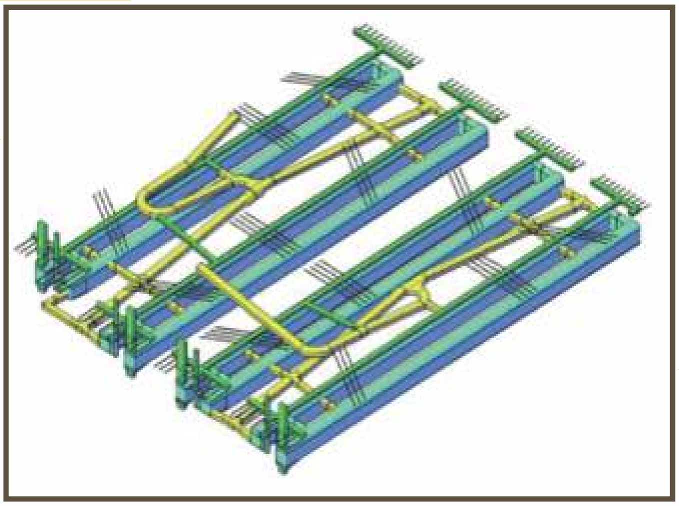


Figure 2.2: Layout of storage cavern with two shaft concept

attached with one unit or two legs of cavern. This concept of single shaft system and double shaft system mainly depends on the concept of crude intake and output casings passing through single access or separate accesses. This also depends on the dimension of the shafts which can be accommodated at a particulate site as well as operational requirements. A typical layout of cavern is shown in Figure 2.2 which shows each storage unit with two shafts, one for intake of crude oil and the other with submersible pumps for crude outlet. In the same context, Figure 2.3 shows another layout of caverns with a single shaft concpt.

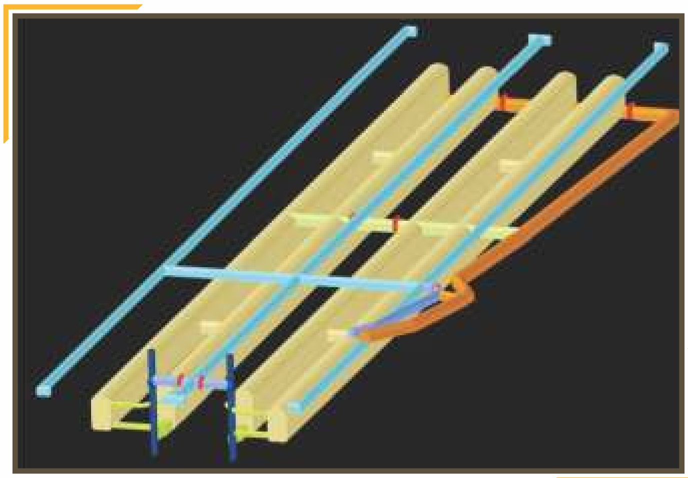


Figure 2.3: Layout of storage cavern with single shaft concept.

Unlined rock caverns are large underground excavations running into several kilometres depending on the storage requirements and selected site conditions. A typical storage installation involves several units of caverns excavated along with access tunnels, water curtain tunnels and shafts, wherein dimensions entail access tunnels running up to 3000-4000 m, several water curtain tunnels spanning 4000 m, a significant number of water curtain boreholes drilled from water curtain tunnels, shafts excavated vertically up to a depth of 75-100 m and lastly U shaped caverns running into another 4000 m. While a large amount of excavation work is involved in construction of these underground caverns, limited above ground excavation works are undertaken mainly for land grading purpose such that the process facilities and other associated utilities and offsite systems are built for operation of storage installations.

In order to secure the stored products, the cavern and shafts are closed at the end of construction phase with thick concrete plugs with specific design for ensuring gas tightness. The underground process facilities include submersible crude oil pumps and seepage water pumps located within the shafts, instrumentation cables within the shafts, and hot oil circulation pipe with nozzles laid on the cavern floor. In addition to underground facilities, different process related facilities like crude oil receiving system, crude oil pumps & casings in shaft, crude oil heating systems, nitrogen bank, seepage water pumping and treatment system, compressed air system and flare stack are amongst the main components of an underground storage system.

2.3 Investigations

Extensive and planned investigations are necessary to minimize geological surprise during construction of such large underground projects. These are planned and executed at various stages of the project; feasibility stage, detail design stage and construction stage. Details of the investigation carried out before and during the execution of these projects are covered in Chapter 3.

2.4 Underground Works

Underground storage caverns are generally U-shaped with an approximate "D" shaped cross section connected to a shaft carrying submersible pump installations and pump pit, located at end of one leg of the cavern. The storage units are connected to the surface by vertical shafts. While the roof is horizontal along full length of the caverns, inverts of the cavern units are designed to have a gradient of 1:250 from the intake shaft to the pump pit located at the outlet shaft so as to ensure that the sloping floor facilitates free flow of crude oil. During construction, in order to create multiple excavation faces, cross tunnels are designed and constructed between adjacent legs of caverns, however an optimum no. of concrete plugs is also factored in the overall layout. Rock cavern section ranges approximately from 30 m x 20 m (H x W) at the outlet end where it is connected to the pump shaft, down to 22 m x 20 m (H x W) at the shaft inlet connection. Depending on the envisaged storage capacity and geological setting of the selected site, length of the caverns generally vary between 300 m to 900 m.

The underground caverns and other associated facilities are excavated using drill & blast method wherein the typical heading and benching approach is adopted. In consideration of the constraints of equipments and their reach, top-heading is usually designed as 8 m in height while the subsequent bench heights vary from 6m to 8m. While top heading is excavated through the horizontal blasting pattern, subsequent benches are excavated either through vertical blasting or horizontal blasting pattern. Owing to the in built sloping cavern invert, height of the cavern varies and given the fact that the cavern roof is horizontal, height of the benches vary. This adopted preference is governed by the availability of plant and machineries. For the purpose of ease in construction and to ensure access for multiple faces for excavation, storage caverns are connected at the upper and lower levels by connecting galleries. All connecting galleries between the two legs of a cavern unit are designed to be equipped with concrete separation walls in order to align the product movement. However, all connecting galleries from the cavern unit to access tunnels are designed to have concrete barriers so as to ensure gas tightness. Crude oil storage is designed to operate at vapour pressure of 1.3 to 1.5 bar. A pressure above atmospheric pressure is always maintained in the cavern, to eliminate leakage of air into the cavern. The cavern is designed to resist vacuum pressure and also designed for accidental load case for an internal transient explosion of a defined case of 1 MPa.

Vertical shafts which house casing and pipes are normally rectangular or circular in shape and connect directly to the cavern or indirectly through a set of inlet / outlet tunnels. One operation shaft and one oil inlet shaft is designed as per U –shaped caverns. Shafts are located at one end of the cavern leg through which oil is taken in and as per the operational requirements shafts are normally 4m x 4m to 6m x 12m in size while circular shafts are 6m to 8m in diameter.

In order to ensure continual saturation of surrounding rockmass, water curtain system is designed and built overlying the storage caverns so as to recharge the permeable joints and allow water to flow from the rock mass towards the cavern. This objective is met through

the water curtain system along with the water curtain bore holes, which are designed to intersect the predominant pervious joints sets. Water curtain tunnels are designed to be D-shaped in cross section with dimensions of about 6 m width and 6 m height. The water curtain boreholes drilled from water curtain tunnels are designed to have a length of about 50m to 75m with an inclination of 50 downwards. In case of scenarios involving the necessity of overlapping effect on a larger rockmass and concrete plugs located in the connecting gallery, bore holes with 100m length are also drilled from water curtain tunnels. In general with a design spacing of 10m to 20m, boreholes are drilled perpendicular to the water curtain tunnel axis. However based on the geological mapping of the water curtain gallery, appropriate orientation, spacing and length of the boreholes are decided during the detailed engineering and construction stages of the project. With a flexible approach, clusters of boreholes are designed for drilling with excavation progress of the water curtain tunnel. During construction stage, to avoid de-saturation of the rockmass water curtain boreholes are charged in advance say for a distance of about 50m from the active excavation face. This also reduces the risk of fluctuations in ground water level during the construction activities. Temporary water supply arrangements are made during the construction stage to feed the water curtain boreholes with necessary manifolds. To eliminate the risk of interruption in water supply, an arrangement is made wherein the water curtain system is fed from a surface water storage facility. Prior to tightness testing of the caverns, the temporary water supply system to the water curtain bore holes shall be disconnected and water shall be filled up to the drive way level.

Access tunnel are constructed from the ground surface with portals at suitable locations so as to provide access to plants and machineries for excavation of the rock caverns and execution of underground civil and installation works. These accesses are designed with the objectives of overall construction schedule as well as safety aspects that needs to be addressed during the excavation phase. The access tunnels are designed with a slope gradient of 1:8 in linear segment whereas in curvilinear segments invert of the tunnel is designed to be horizontal. The main access tunnels are designed to allow for two way traffic with heavy dump trucks. Access tunnels are D-shaped cross section tunnels having height of 8m and width of 8-12 m.

Concrete barriers are provided to isolate the product from the external environment in tunnels and shafts and to separate units. Concrete separation walls are provided for circulation of crude oil, vapour and inert gas.

In order to facilitate installation of pumps and instrumentation in the shafts, casings are installed in the shafts. The shafts are backfilled with mass concrete or with water through which casing and piping pass through. A support framework is provided on top of the pump pit that acts as a guide to the casings that are lowered from the top. Additional structural support to casings/pipes shall also be provided between the pump pit and the concrete barrier.

2.5 Aboveground Process Facilities

Above ground process facilities associated with the underground storage installations are as enumerated below:

- Cavern Shaft Tops
- Heat Exchangers
- Metering Skids
- Booster Pumps
- Boiler
- Pipeway above ground
- Buried Pipelines and Electrical Cables
- API Oil Separators
- Effluent Treatment Plant

- Fire Water Tank
- Fire Water Pump House
- Fire Station
- Control Room
- Standby Power Generator
- Outdoor Switch Yard
- Sub Station Building
- Compressed Air System
- Nitrogen Plant
- Cooling Tower
- Maintenance Workshop
- Diesel Oil Tanks and Pumping System
- LPG Mounded Storage and Pumping
- Storm water reservoir
- Raw water tanks and pumps
- Drinking water sump and pumps
- Flare & Flare stack

The main systems include crude oil receiving system, submersible crude oil pumps & casings in shaft, crude oil heating systems, nitrogen systems, seepage water pumping and treatment system, compressed air system, flare and control system.

Crude oil is pumped from the Crude Oil Terminal (COT) or Very Large Crude oil Carriers (VLCC) through offshore oil terminals (SPM) to the inlet shafts of the cavern. Crude oil is pumped out of the underground storage caverns using submersible pumps located in outlet shaft and pumped to the COT or Refineries through connecting pipelines. Seepage water pumps installed within the shafts are used to pump the accumulated seepage water out which is conveyed to the Effluent Treatment Plant (ETP) for necessary treatment and reuse.

After testing of caverns, the caverns are inerted to bring oxygen content to below 5%. In order to prevent sludging in caverns, the crude oil is circulated and heated through a hot oil pipe located on the floor of the cavern. Sludge in the pump pit is removed by direct injection of steam into the water in the pit.

2.6 Construction Aspects

Construction of underground works is carried out by the drill and blast method with fully grouted un-tensioned rock bolts and fibre reinforced shotcrete as the principal means of support. Steel sets and ground anchors may be adopted in exceptionally poor conditions. The typical construction cycle entails survey, drilling of patterned blast holes, charging of holes with detonators, blasting, defuming, scaling, mucking, geological mapping, rock support and grouting as required.

The basic design of caverns is checked, validated, updated and modified during the construction based on the actual geological, geotechnical and hydro-geological conditions encountered during excavation.

At the end of excavation the underground caverns and shafts are sealed with large concrete plugs to ensure gas tightness. The plugs are constructed in the tunnels connecting the caverns and in the shafts. The tunnel plugs have a size of 8 mx8 m with a thickness of 3-5m and the shaft plugs have a size varying between 4 m to 12 m and thickness between 2.5 to 5 m. Special cooling arrangement has to be provided during casting of the plugs to ensure no minor cracks are formed after setting of concrete. The plugs are reinforced concrete structures with M40 concrete. The tightness of these plugs are ensured by a system of rock mass and contact grouting arrangements between plug and rock. The shaft plugs have pump and instrument casing passing through the plug. The space above the plug in the shaft is filled with water or concrete.

The water curtain system is the most critical part of the storage. The water curtain boreholes should be charged at least 40-50 m ahead of the cavern excavation to ensure saturation of the joints. The water curtain tunnel along with the boreholes also serves to update the geology and hydrogeology and decide on requirements of pre-grouting and any other specific requirements for excavation of the caverns.

Typical construction schedule for 1.0-1.5 MMT storage is around 36-42 months for excavation and support and another 6 months for the underground concrete and another six months for pre-commissioning and commissioning works. Thus construction schedule in such projects requires careful planning, deployment of necessary equipment and qualified manpower and strictly following all quality and safety procedures.

2.7 Key Project Learnings

The design and construction of large storage caverns is a complex work requiring the coordinated efforts of geologists, hydro-geologists, geotechnical and rock mechanics engineers for the underground works and structural, piping, instrumentation, rotating and process engineers for the cavern mechanical and above ground works. The coordination between the geo-engineers and the other engineering disciplines can be difficult as these two groups are not used to working together and requires a coordinated but flexible management approach.

The design philosophy for underground works should follow an observational approach, i.e. the original design assumptions are to be updated by means of the results from mapping and monitoring during every stage of construction, to allow for a safe and optimized design. Further, the interface between design and construction processes should be given special attention with the objective to achieve consensus between the design and production teams in the organization. An active input of constructability aspects into the design process is considered as a crucial factor for a speedy construction. This is facilitated by development of an experienced design interface manager.

Risk management is one of the critical activities for successful implementation of large construction projects. Risk management is a systematic process of identifying hazards and associated risks. Risk can be managed, shared, transferred or accepted using systematic risk management systems. The result of risk assessment is a set of risk registers. These registers identify the risks including ownership and management of risks. A dedicated risk manager should be engaged for the design and construction period.

At the tendering stage the contract documents should include the geotechnical reference conditions. The main objective of the Geotechnical Reference Conditions is to give bidders a single, concise and workable description of the ground conditions upon which they shall base their tenders and give them a clear idea about the associated risks. These reference conditions also establish a geotechnical baseline to be used in assessing the existence and scope of differing site conditions. The Geotechnical Reference Conditions should provide a clear contractual arrangement for the allocation of the risks arising from differing site conditions and give the contractors the necessary understanding of the conditions and the rationales for selected solutions as the basis for their design and construction. They also serve to constitute the base for the contractors estimate of cost and time and to serve as the basis for verification of site conditions during construction and the associated compensation of cost and time as well as possible claims.

Geotechnical and hydro-geological monitoring are vital during construction. These are required not only to validate the design assumptions and to optimize the design, but to also check the quality of works and ensure safety and hydraulic containment. The quality of geotechnical monitoring is a serious concern.

Safety and quality is a key concern is all underground works and in particular for large storage works, where very large underground excavation works are carried out under tight schedules. Deployment of dedicated quality and HSE managers is vital for the successful implementation of these projects. There is ample scope to improve the quality of underground excavation and rock support works.

As discussed earlier, it is impossible to assess the complete geological, hydrogeological and geotechnical conditions at the start of the project. Hence the tender documents should provide flexibility to handle variations in the ground conditions. The adoption of risk management procedure goes a long way in this direction. A pragmatic approach towards contract handling is required, in particular for handling large variations in some items including requirement of new items which were not envisaged. Poor contracts with a rigid approach along with varying geological conditions are a major reason for delay in most underground projects and these aspects should be carefully visualised and taken care during contract preparation.

List of applicable codes and standards is given in **Annexure I**.

SITE INVESTIGATION S

C.Kumar, S.Pal, G.Kannan & R.Rath

CHAPTER - 3

3.1 Introduction

Site investigation is a key prerequisite in planning, design and construction of underground unlined rock cavern storage facilities. Since the construction of large underground caverns pose diverse & complex geological challenges, extensive and planned investigations campaigns are necessary to minimize geological surprises and associated risk quotients of such projects during construction. As it is well acknowledged that prediction of likely scenario of ground condition for any underground project is not possible in totality, these investigation campaigns are planned and taken up during the span of the project at select identified stages. Therefore objective of the site investigations is to determine the geological, hydrogeological and geotechnical conditions of the select project site that affect the safety, cost effectiveness, design, and execution of underground project.

3.2 Investigation Campaigns

From the conceptualization stage through feasibility studies to the basic design stage and throughout the construction phase, site investigation schemes are designed to provide adequate level of information appropriate to the particular project development stage.The investigation program is aimed to prepare, collate, generate and analyse engineering geological and rock mechanics data for site characterization in different stages of investigation and also to identify potential geohazards that may exist at project sites. In general, the site investigation program is carried out in five stages spanning the project life cycle:

Stage I: Pre-Feasibility Studies

Stage II: Detailed Feasibility Report

Stage III: Supplementary Site Investigation during Basic Engineering Design

Stage IV: Detailed Engineering stage

Stage V: Construction stage

Results of the investigation campaigns undertaken during each stage are carefully analysed and suitably adopted for subsequent stages of the project. Concurrently, the collated information is also often used to revisit the assumptions for an on the go design approach for these projects. The site investigation programs conceived for each stage of the project are outlined here under:

3.2.1 Stage I: Pre-Feasibility Studies or Desk Top Studies

Pre-Feasibility studies are undertaken to determine whether a conceived project finds merit both in terms of suitability of selected site and the economic considerations. If so, pre-feasibility studies provide information to determine whether planning should proceed to the feasibility phase. Adequately precise and detailed geological information and maps are the basic pre-requisite during the pre-feasibility or desk top studies stage. The Preliminary study usually starts with following information/ database:

- Collection and study of available reports, published literature on regional geology, and site specific geological setting etc., pertaining to the select location;
- Topographic sheets corresponding to the select site with a scale of 1:50,000 / 1:25,000 and published geological maps;

- Reconnaissance survey of shortlisted sites with focus on morphology, inferred geological setting, topographical features, access and approach to the site, the nature of land and brief about the local populace etc.;
- Satellite imageries and / or aerial photographs of the selected site for terrain appreciation and lineament mapping;
- Assessment of the selected site so as to have maximum possible storage capacity considering various risk factors including secured inventory holding capacity and operational aspects based on the inferred geological setting, hydro-geological regime and geotechnical condition of the site;
- Study of available infrastructural connectivity viz. pipeline network (both present and future), ports and offshore oil terminals and refineries in the near proximity so as to integrate the inventory stockpiling and use existing facilities for filling in, storage and evacuation of the stored product;
- Assessment of the selected site in terms of adequate space for disposal of excavated muck with possibility of having an optimum design stack height;
- Availability of common utilities such as water, power and other associated requirements for construction of the facilities and beyond for the operational purpose.

Thus evaluation of several factors such as technical parameters, infrastructural set up and the proximity of storage facilities either to the coastline for transhipment or to a refinery / pipeline network etc. are performed as part of pre-feasibility studies and suitable sites are finalized.

3.2.2 Stage II: Detailed Feasibility Report

Investigations during feasibility studies are designed to provide information at a level such that critical geological features and corresponding hydrogeological implications are established along with the geotechnical conditions of the selected site for the conceived project. The type of investigations including the scheme of undertaking the selected investigations are generally planned considering the terrain of the selected site. The macro level approach with adequate coverage of the site helps to delineate suitability of the exact location for siting the planned storage facilities. The available sufficiency in data permits selection of the most favourable site at a micro level within the constraints of morphology and geological setting, determination of the general layout of the facilities best suited to the prevailing site conditions, evaluation of the influence of hydrogeological regime on the design and construction of the facilities. The collated information further helps assessment of the likely environmental impact owing to the storage caverns. In consideration of the above factors, the geotechnical conditions of the site along with the various design assumptions help to ascertain the capital costs for developing the planned project in sufficient detail so as to allow a comparative cost estimate and further investment decision.

Investigations during detailed feasibility study generally entail three broad aspects namely developing understanding of the geological setting, collating information for assessment of geotechnical condition and carrying out specific exploratory investigations to ascertain hydro-geological regime of the site. These integrated investigation schemes cover the following activities:

- Coordinated effort involving multiple agencies so as to obtain geological condition & inferred geological attributes, ground water regime, hydrological and soil data including seasonal variations, insight into the possible geological hazards, seismicity and prior regional experiences etc.;

- Remote sensing studies to have information about landforms, watersheds, lineaments, soil and rock type boundaries, outcrops, erosion features, etc.;

- Detailed geological mapping to obtain distribution of rock types, discontinuities and geological features including interpretative profiles etc.;

- Surface geophysical investigations primarily comprising of seismic and electrical resistivity surveys;

- Subsurface exploration by undertaking core drilling (vertical or inclined) of depth varying from 100 to 200 m depending on the terrain and the target depth for locating the storage facilities below the Mean Sea Level (MSL);

- Field & Laboratory testing of rock selected samples for geotechnical characterizations;

- Drilling of destructive holes up to a design depth of about 50 m for performance of hydrogeological tests such as drawdown and interference tests;

- Performance of water pressure and pumping tests to collect hydrogeological parameters of the select rock formations along with it's permeability etc.;

- Performance of In-situ stress measurements to identify maximum and minimum in-situ stress along with directional attributes;

- Estimation of other hydrogeological parameters i.e. physical, chemical & micro-biological water quality parameters of ground water, rainfall data, sources of water in the vicinity of the project site along with test results for water quality and establishing it's suitability for usage during construction stage.

Relevant information on the regional and site specific conditions is collected as part of the detailed feasibility study efforts to support the rationale for plan selection, project safety and environmental assessments including description of potential borrow areas and quarries, accessibility to sources of construction materials and suitable dumping site for the excavated muck etc.

3.2.3 Stage III: Supplemnary Site Investigation during Basic Engineering Stage

The basic engineering design (BED) stage happens after a feasibility study has been completed and investment decisions are made to take up execution of the project. BED studies are developed to reaffirm the basic planning decisions made in the feasibility study, establish or reformulate the scope of the project based on current criteria and costs, and formulate the design memoranda which provides the basis for the preparation of plans and specifications for project implementation along with developed contractual frameworks.

Investigations performed during the BED stage are designed in a manner so as to collect sufficient detail beyond the feasibility stage to assure that the project can be implemented with the design assumptions. The emphasis of this stage is towards site-specific studies which provide more detail and in-depth of information.

Investigations during basic engineering stage involve following activities:

- Updation of site specific geological maps, interpretative geological sections, soil and rock classifications, distribution of rock types, structural attributes and interpretations of geological features with it's impact on the conceived project;

- Preparation of composite map with desired rock mass characterization i.e. estimate of Rock Mass Rating (RMR), Rock Mass Quality Index (Q) and

Geological Strength Index (GSI);

- Subsurface exploration through core drilling at selected locations with due consideration of the overall plot plan and the underground layout of the storage caverns;

- Surface geophysical survey as well as selected well logging to correlate and calibrate geophysical data with inferred geological information;

- Performance of complete classification & index testing to establish engineering properties and geotechnical characterization through field & laboratory testing;

- Performance of additional water pressure and pumping tests at select locations with specific focus on the overall plot plan and underground layout of the storage caverns so as to further optimise and finalize hydrogeological model.

Results of all geotechnical investigations performed as part of basic engineering stage is aimed to study geological, geotechnical & hydrogeological condition developed in sufficient detail to establish final design and operating requirements for implementation of the project along with the drawn specifications and contractual frameworks.

3.2.4 Stage IV: Detailed Engineering Stage

Prior to commencement of the construction activities, Investigations are mandated to be undertaken by the contractor in order to focus on specific considerations such as the Portal and Shafts of the underground storage facilities and 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 contract, the pre-project and basic design stage engineering & design reports for the selected plan are carefully reviewed. Investigations during this stage of the project include the following activities:

- Updation of interpretative geological maps for each component of the underground storage caverns including portals, shafts and access tunnels etc.;

- Confirmation of identified main geological features such as dykes, faults, folds, shear zones etc. delineated and interpreted during previous investigation campaigns;

- Subsurface exploration through core drilling for the purpose of identification of geological features in the areas selected for locating the shafts and portal(s);

- Preparation of composite map with rock mass characterization i.e. estimates of RMR, Q and GSI;

- Development of interpretative 3-D geological model from updated geological map, inferred structures, interpretative geological sections, block models with superimposed facilities; The derived model is continuously updated with every add-on information during the entire construction stage;

- Confirmation of geomechanical characterization of the site along with the finalized layout by performing additional field and laboratory tests;

- Confirmation of hydrogeological characteristics by performing additional water pressure and pumping tests;

Results of all the investigations performed as part of mandatory site investigation campaign prior to commencement of construction are used to prepare a comprehensive geological, geotechnical & hydrogeological model corresponding to the overall layout and with adequate details to derive sub-models for each specific component of the underground facilities.

3.2.5 Stage V: Construction stage

Water curtain tunnels which are excavated before the cavern excavation, acts as a pilot tunnel revealing the actual encountered geology that is interpreted to be encountered during cavern excavation. These details are utilized for hydrogeological tests, grouting trials etc and for assessment of grouting requirements in water curtain tunnel and the underlying caverns.

Lineaments and major discontinuities predicted as part of the interpreted 3 D geological model developed before construction are probed / cored in advance during excavation of water curtain tunnel and also during the cavern excavation. Coring / BHTV imaging is also performed in select water curtain boreholes so as to reduce the uncertainty quotient of geological features within the rockmass surrounding the storage caverns. The interpretative 3 D geological model is updated through this active design process with specific objective to focus on the critical segments of the caverns otherwise referred as geological hot spots. Additional field investigations and laboratory tests that are deemed necessary for fulfilling the design and construction requirements are also undertaken during the construction stage.

3.3 Scheme of Investigations

Scheme of Investigations for underground storage caverns is dependent on a no. of factors, thus necessitating an integrated approach to perform in a select hybrid model over the span of the project. Therefore, a detailed outline of the investigations is presented at **Annexure II**. These investigations are perfomed in various combinations as per the stage of the project. A typical scheme of bore hole over a layout is shown in Figure 3.1.

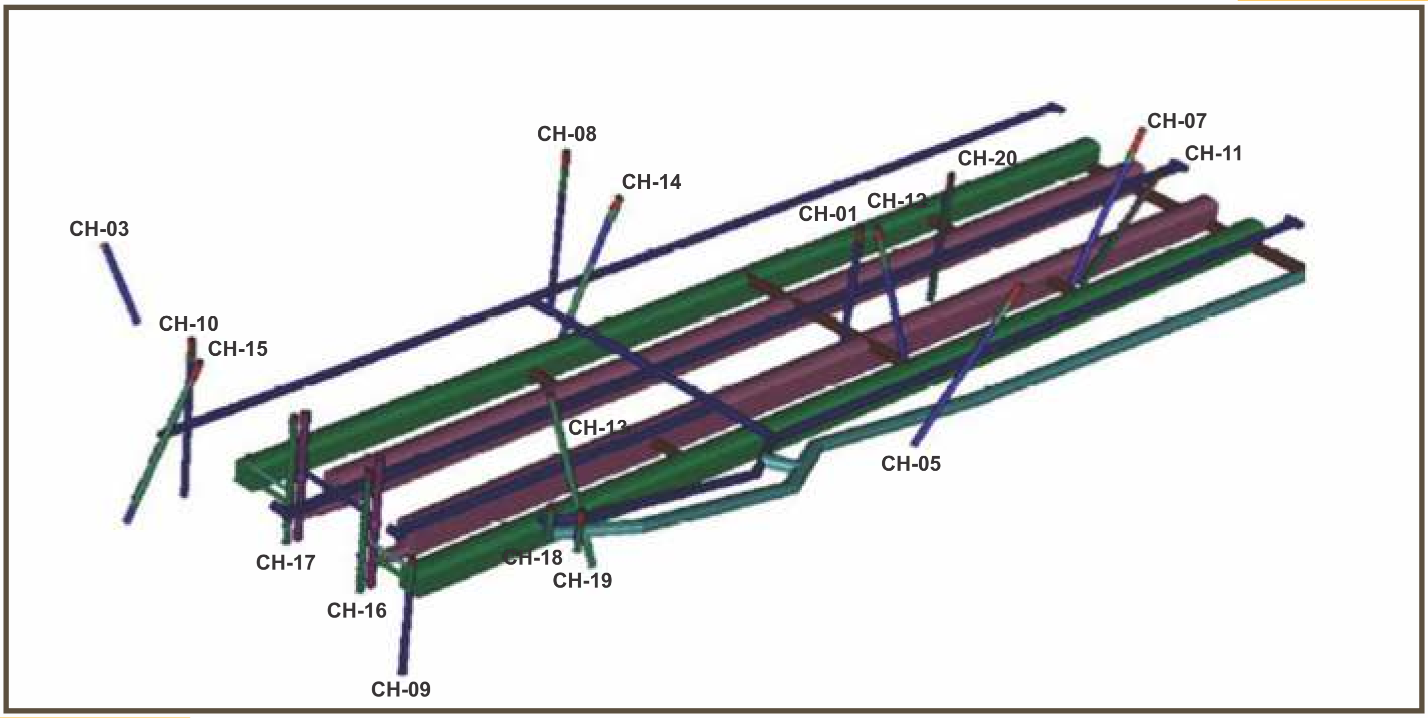


Figure 3.1: Layout with investigation boreholes

Case Study No. 3.1 Scheme of Investigations

As one of the case scenarios, presented herewith is the investigation campaigns performed for a underground storage project at Padur (Udupi) located on the western coast of India in the state of Karnataka. The presented case study outlines the results of investigation program including the role of investigations during construction stage. Also deliberated herewith are some key considerations that resulted from these investigation campaigns leading to the on the go design approach and updation of interpretative 3 D geological model for the project.

(i) Basic design stage

The proposed site at Padur, in the Udupi district is located on the west coast in south western part of Karnataka state. Geographically the position of site is at latitude 13°13' and longitude 74° 47' on a hilly terrain. The Initial Site Investigation (SI) was carried out in 2005 during the Detailed Feasibility Study, the second, Supplementary Site Investigation (SSI) was carried out during the basic engineering design stage in 2009 and the third, Mandatory Site Investigations were carried out before the actual construction of the project in 2010.Key results of the investigation program are as follows:

(ii) Geological condition

Geomorphology: Topography of the area exhibits an undulatory hilly terrain traversed by a narrow central valley oriented roughly east-west. The fringing plateaus on either side have an elevation difference of about 20 m. The plateau and the flanks towards the valley are characterized by a number of gneissic hilly outcrops orientated North–South with an elevation of about 30 m. The central valley and the valley flanking the northern part of the site is characterized by vegetation and paddy fields.

Litho-stratigraphy: As part of the geophysical investigation, interpretation of the seismic refraction survey data has been carried out with special attention to find out any low velocity zone within the basement rock along the seismic lines. The seismic refraction survey data reveals four distinct layers with different seismic velocity contrast with depth. The abstract of seismic velocities are summarized as under

- Top most layer comprises of overburden lateritic soil / silty soil, of varying compactness. This top layer is very thin having variable thickness along profiles. This layer has seismic velocity of the order of 500 m/sec to 1100 m/sec.
- Second layer comprises of weathered Granite / Granite gneiss having seismic velocity of the order of 2000 m/sec to 2400 m/sec, which increases with depth. Lower velocity indicates higher degree of weathering, whereas high velocity indicates lower degree of weathering along with fracture and joint in the rock mass.
- Third layer comprises of fresh Granite/ Granite gneiss (lower velocity < 3500 m/sec indicates lesser degree of fracture/ Joint, whereas the higher velocity in the range of 3700 m/sec to 4500 m/sec represents massive nature of rockmass.

- Fourth layer comprises of massive Granite / Granites Gneiss having seismic velocity of the order of 5100 m/sec to 6900 m/sec forming basement.

At some places very low resistivity zones possibly highly jointed or fractured rockmass of Granite / Gneissose Granite under saturated condition have been identified below high resistivity zones. These low resistivity zones are found to be having a reasonable correlation with the geological logging of cores with joint sets. However, having adopted an integrated approach of analyzing the data along with surface geological mapping, no major structural weakness zone was expected at the site.

Seismicity: The site falls within the Zone III classification as per Seismic Zone map of India where the seismic coefficient is about 0.04 g.

(iii) Rock Types & Discontinuities

The project area comprises of banded and granitic gneisses, migmatites along with true intrusive granites and few mafic intrusions. The banded gneisses consist of white bands of quartz-feldspar (felsic bands) alternating with dark bands containing hornblende, biotite and minor accessory minerals (mafic bands). The granites are porphyritic to granular with typical quartz vein system and at times with intrusive properties. The mafic intrusives are in the form of doleritic dykes of varying thickness in the above parent rock.

Details of the structural discontinuities within the rock mass as determined from site investigation mapping reveal three major discontinuities (sub-vertical) and one sub-horizontal discontinuity as shown in Table 3.1. The sub-vertical discontinuities are persistent and oriented in almost north-south and east-west direction with a dip of about 80-85 degrees both sides. Major tectonic and geomorphic features are also aligned parallel to these discontinuities. Mafic dykes are also found roughly oriented parallel to these discontinuities indicating these intrusions are both syn-tectonic as well as post tectonic. Sub-horizontal joints are oriented east to N60°E and dipping about 5-15 degrees both ways.

Table 3.1: Joint sets

No..	Joint set number	Dip Direction	Dip (°)
1	Joint set 1	250/70	85
2	Joint set 2	100	85
3	Joint set 3	200/20	85
4	Horizontal Joint	150/330	15

(iv) **Geotechnical condition**

Core Drilling: Six core holes (three vertical & three inclined) were drilled during the detailed feasibility study, and six core holes (four vertical & two inclined) were drilled during the supplementary site Investigation (SSI). The inclined bore holes were drilled at an approximate inclination of ~30° with vertical. The core logging included lithology, information on weakness zones, core recovery, RQD and joint. All other core holes except one which was falling in the lineament zone in the valley, show RQD and core recovery above 90 % in cavern area.

In-situ Stress Measurement: Hydro fracturing tests were conducted during the detailed feasibility study. The detailed analysis of the data indicates the following results for the virgin stress field (unit in MPa):

$$Sh = 0.075 + 0.0575 . z$$

$$SH = 0.475 + 0.1025 . z$$

$$SV = 0.026 . z$$

With Sh the minor horizontal stress, SH the major horizontal stress and SV the vertical stress and z the depth in meter below ground level.

Mean orientation of the induced fractures suggests a direction of N156E (±14 deg) for SH. The horizontal to vertical stress ratio is about 2 for the minor horizontal stress (Sh/SV) and about 4 for the major horizontal stress (SH/SV) at the depth of the storage.

Q-classification: Majority of the excavation work is expected to be made in the rock mass with an average Q value greater than 40, corresponding to "good/excellent" rock.

Rock Mass strength: Based on the laboratory testing performed during site investigations and subsequent analysis, the following intact rock and rock mass parameter design values as well as stress levels have been determined as shown in Table 3.2.

Table 3.2: Rock mass properties

Property	Value	Remark
Intact Rock		
Bulk density (t/m^3)	2.68 ± 0.75	Mean ± Standard deviation
Uniaxial compressive strength, UCS (MPa)	134 ± 43	Mean ± Standard deviation
Tensile strength (MPa)	6.7 ± 2.4	Mean ± Standard deviation (indirect tensile strength test)
Young's modulus(GPa)	62 ± 4.0	Mean ± Standard deviation
Poisson's ratio	0.23	Form laboratory tests
Mi	28	For Gneiss
Rock stresses at cavern roof		
Vertical stress (MPa)	1.4 to 2.2	Rock cover varies from 50 to 80m

Max horizontal stress (MPa) Min horizontal stress (MPa)	5.2 to 8.3 3 to 4.7	Rock cover varies from 50 to 80m Rock cover varies from 50 to 80m
Rock Mass (Typical Value at Cavern Level)		
GSI	90	Mean GSI is 90
Rock-mass strength (MPa)	90	Rock Lab, H&B
Friction angle (0)	50	Rock Lab and correlations
Cohesion (MPa)	7	Rock Lab and correlations
Deformation modulus (GPa)	50	Rock lab and correlations
Hoek and Brown rock mass Strength Parameters (Typical values at cavern level)		
Mb	17.91	Rock Lab
S	0.2775	Rock Lab
A	0.500	Rock Lab

(v) Hydrogeological condition

Investigation results show low to very low hydraulic conductivity as the caverns are located in the competent rock mass of granitic gneiss. While surface mapping shows few joint sets, the northwestern part of the site area is reported to have a zone with jointed rock mass. From investigations, these zones exhibit conductivity of the order of 10-7 to 10-6 m/sec. Joint patterns of similar nature were also expected to occur in the northern part of site area. In addition to the above, some local jointed zones which may encounter higher inflows and required pre-grouting were expected.

(vi) Investigation during construction

Some of the water curtain boreholes drilled perpendicular to water curtain tunnel were imaged by BHTV (Borehole televiewer) as a part of construction strategy to understand the complete geology beyond the tunnel and above the storage caverns. These were extrapolated to predict the geology that would be encountered in cavern heading. Based on the geological setting revealed from the excavation of water curtain tunnel and a systematic geological mapping of the face and walls conducted during cavern excavation, actual geological, geotechnical and hydro-geological conditions were regularly updated and modified. Figure 3.2 shows the geological model updated after completion of third bench excavation.

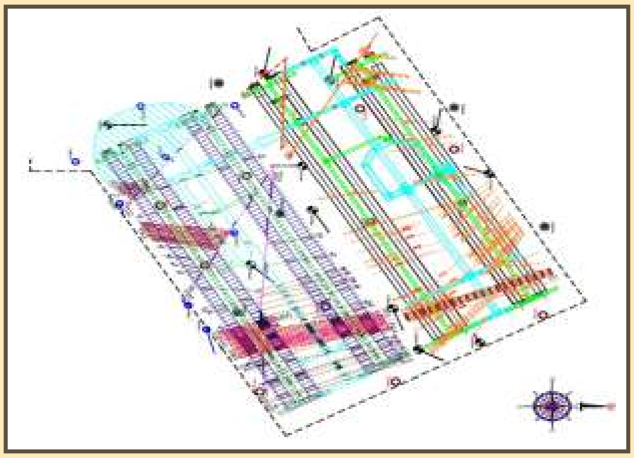


Figure 3.2: Updated geological model after bench 3 excavation

GEOLOGICAL CONSIDERATIONS

S.Pal, G.Kannan & R.Rath

CHAPTER - 4

4.1 Geological Setting

Geological setting is one of the prime considerations for siting underground rock caverns so also has an important contribution towards successful construction of the facilities. Given the fact that geology plays a significant role starting from the conceptual stage of underground storage caverns through the design stage till the completion of the project, geological prediction forms an integral part of any sub surface project.

As it is well appreciated that no quantum of investigation can ascertain the geological setting of any underground project, interpretative geological profile and inferred 3 D models representing the geological setting becomes an imperative. Therefore, with the constraint of time and cost, during the initial stage of any underground project; limited investigations are undertaken from a site assessment perspective. Thereafter, depending on the adopted execution philosophy mandatory investigations are carried out to supplement the earlier understanding. Further, with every stage of project as the construction progresses, the excavation mapping results help to re-visit the assumptions and thus the predicted model is updated. Unlike the hydro power projects mostly located in the complex geological settings of Himalayan terrain, the underground storage caverns are located in peninsular India. However, the dimension of these caverns and owing to its length and parallel disposition offer significant challenge both for design and construction. The constrained interpretation of unfavourable discontinuities with changing trend and attributes within the rock mass between two parallel caverns remains the most challenging aspect to predict. Therefore, with an on-the-go design approach and continuous assessment of revealed geological setting along with tell tale signatures of geotechnical monitoring plays a crucial role in successful completion of any underground storage project.

Selection of sites for creating underground rock caverns commences during the pre feasibility study stage which entails a desk top study comprising of available literature, remote sensing imageries, maps and reports. Subsequently, during the detailed feasibility stage, as part of the site investigation campaign a detailed engineering geological mapping is conducted along with other investigations such geophysical surveys, core hole drilling, hydrogeological tests and in-situ stress measurements etc.

Geological mapping is one of the major tools for characterization and assessment of the selected sites for understanding the geological setting of the project site. In the detailed feasibility stage a detailed engineering geological mapping of the site is undertaken with inputs from remote sensing techniques and on-site traverse. To commence the mapping process, identification of geological windows like road cuttings, quarries is required. And the identified windows need to be well marked at ground as well as on available contour map of the area.

Geological mapping for the selected site preferably covers the following aspects:

i. Regional Geological Setting: This being the primary consideration during the conceptualisation stages of the project, needs a thorough evaluation as part of the desk top study. With a broad base consideration, in addition to the other logistic factors; in general storage locations are characterised by Peninsular gneissic complexes or Charnockites and Khondalites of the Eastern ghats.

ii. Geomorphology: Morphology of the selected terrain has a significant bearing in precise siting of the conceived storage facility. Given the fact that

morphology is a manifestation of the geological features and lithotypes, consideration of the ground elevations, presence of valleys and rolling topography along with the inferred features are crucial in locating the underground facilities such as the portal, access tunnel, water curtain tunnel, the shafts and the storage caverns.

iii. Major rock type with variants : Competent rock types are the basic premise of rock caverns, therefore during the feasibility stage through geological characterizations, litho types are mapped and the properties of the litho types form basic input for design considerations.

iv. Nature and average thickness of soil cover: Thickness of over burden For eg. Laterite with a thickness higher than 8 metres on the plateau and locally less on the slopes

v. Mineralogical and textural description based on outcrops eg. banding (gneissic structure) with alternation of mafic bands (predominant dark minerals) and felsic bands (predominant light minerals) due to differential erosion.

vi. Degree of weathering: whether rock outcrops are fresh/slightly weathered or moderately or highly weathered.

vii. Discontinuities: in terms of orientations, persistence, spacing and nature (roughness, aperture, filling etc.)

viii. Tectonic structures: All major geological structures like shear seams, igneous intrusions, faults etc. are interpreted in relation to geomorphology of the area i.e whether these features are cutting across the hill or manifested as valley. Analysis of Lineament using satellite pictures are done. Lineaments are identified and strike measured, at a large scale around the site

ix. Rock mass assessment: rock conditions are assessed on the basis of surface outcrops using Q index (Barton et al, 1974). Q index is the product of the three factors:

a) the potential size of rock blocks

b) the geomechanical quality of the contacts between the blocks

c) the initial state of the rock mass as regards with water and stress.

All the above details from outcrops are systematically recorded with respect to the numbered outcrops so that the geological information is reported along with spatial attributes. Based on the geological mapping, schemes for geophysical investigations are decided which broadly include seismic and electrical resistivity surveys. Based on assessment of the geophysical interpretations and the geological mapping results, drilling of core-holes (vertical and inclined) is planned. While during the coring continuous geological logging of cores is undertaken, drill holes are logged subsequently through geophysical logging transducers. Thus good understanding of geological setting is absolute necessity to firm up the course of investigations and in turn frame geological model of the project.

4.2 Geological Model

A geological model or better represented as an engineering geological model for underground storage caverns is about integrating the surface and subsurface geophysical and geological observations including the physical assessments through the core drilling and creating an interpretative disposition of the various factors such as litho types, characteristic features, elevations super imposed with the design facilities. Like all other geo-models, it is multidisciplinary in nature with inter-operability and dynamically updatable knowledge base about the subsurface attributes.

The model is used for scheduling the construction activities by qualifying and quantifying the geological regimes likely to be encountered as the excavation progresses. The geological interpretation forms an intrinsic part of the model which predicts the likely expected behaviour of the rock mass under various case scenarios and is developed by integrating structural geology, rock classes and engineering properties of the rockmass. The geological model along with engineering geological parameters forms one of the major inputs in basic design of the caverns for various aspects such as orientation, size, shape and support requirement of the access tunnel, water curtain tunnel and the storage caverns. The storage caverns are oriented in a direction so as to encounter minimum adverse geological conditions and in-situ stress regimes in the rock mass while undertaking the underground excavation. The length of the cavern is designed to accommodate the requisite volume of storage and the extent of suitable geological setting available at the selected site.

The development of model generally involves the following steps:

1. Preliminary analysis of geological context of the domain of study described under geological characterization in previous section.
2. Interpretation of available data and observations as point sets.
3. Construction of a structural model describing the main rock boundaries (shear seams, intrusions, faults)
4. Definition of a three-dimensional mesh honouring the structural model to support volumetric representation of heterogeneity.

Based on geological model, adversely oriented discontinuities or lineaments, if any, intersecting the access tunnels, water curtain tunnel and storage caverns are inferred in advance and identified as probable Geological hotspots. The geological model is virtually transformed into a 3-D model of the project facilities using suitable software. In the descriptive model, a relatively important part is devoted to distinguish and clarify:

- the observations and findings
- the necessary assumptions due to missing or incomplete data
- the limitations and the validity of these assumptions

In addition, 2D plates are used concurrently:

-Geological Map

-Longitudinal Cross Section

-Perpendicular Cross Section

These plates, made in accordance with the 3D model, represent 2D sections, easier to manipulate and reproduce than the 3D model.

Case Study 4.1 On the Go design Approach for Interpretative Geological Model

The continuous evolution of geological model through step wise specific investigations can be well demonstrated by an example of one of the projects in south west coast of India. During the routine site investigations of a storage cavern involving geological, geo-physical, geo-technical and hydro-geological investigations it was established that rock formations in conjunction with ground water conditions are competent for construction of rock caverns and suitable to store the hydrocarbons. However, in one of the drill holes, a dolerite body was found to be intruding the parent rock formation in the form of a dyke with zones of hydrothermal alterations along the contact. This was included in the geological model (Figure 4.1) as an intrusion within cavern alignment.

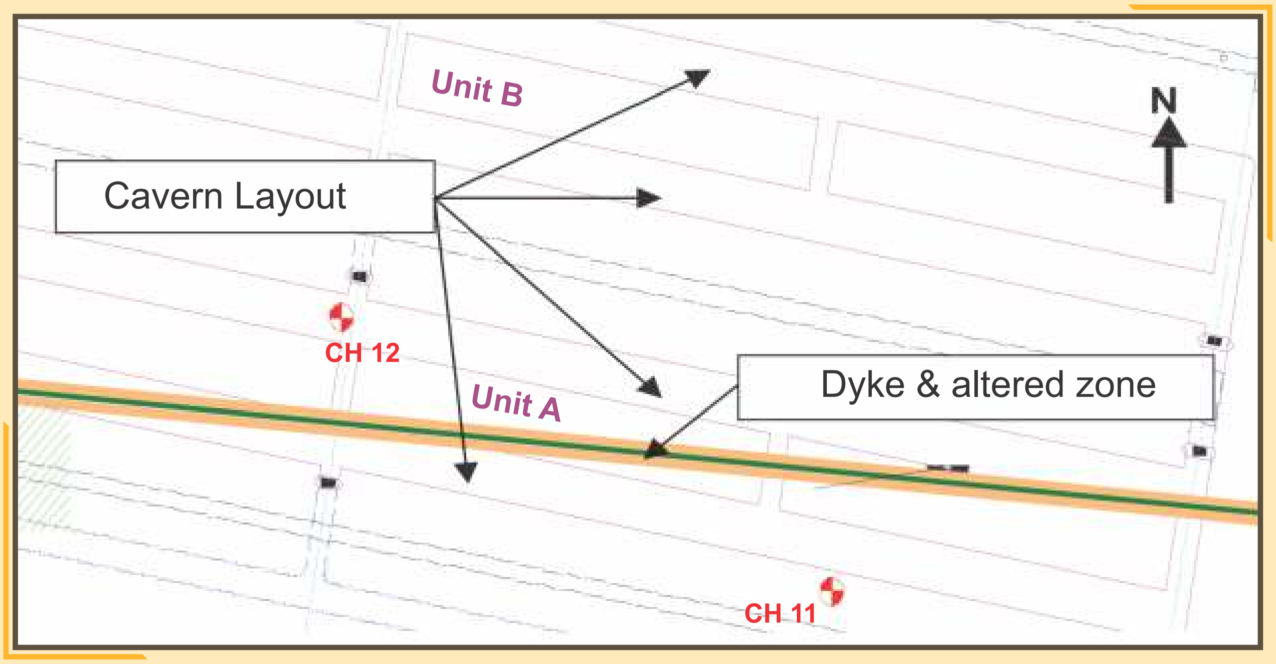


Figure 4.1: Geological model at investigation stage

The dyke was sub-vertical. So, the alignment of the dyke with respect to cavern remained to be confirmed during additional site investigations in the pre-construction stage through inclined bore hole (CH 12) along the predicted alignment. The feature was not negotiated by the drill hole. So the other possible alignment of dyke body- across the cavern alignment was looked into. Accordingly, one more inclined borehole (CH 20) was drilled to ascertain the orientation of the inferred dyke and the geological model. The finding of core hole revealed that the hydrothermally altered dyke is oriented N-S with a westerly dip transecting the caverns across the alignment (Figure 4.2). The composite 3-D model was then made including other lineaments as predicted from remote sensing imageries (Figure 4.3).

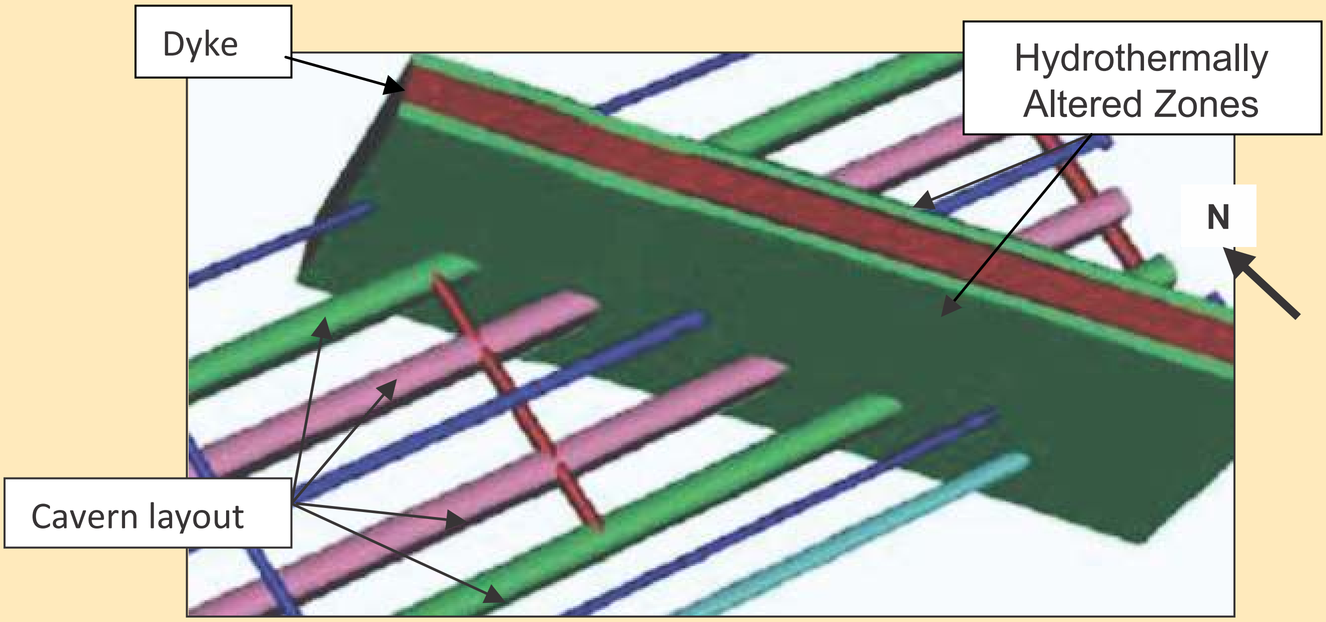


Figure 4.2 : Geological model after additional investigation

During excavation of water curtain gallery which was likely to negotiate the dyke body; thin dyke bands were reported which was sub parallel to the alignment of galleries as well as the caverns. In order to confirm the disposition of the dyke as well as to assess the geological conditions associated with dyke in tunnel grade, a horizontal investigation hole of 69 m was cored along tunnel alignment. It confirmed the oblique model with the thin dykes being offshoots of the major dyke. Investigation hole revealed condition better than design stage with sharp contact on west and altered wider contact on east. The thickness of dyke along cavern was inferred to be 32 m. No water seepage condition was observed along contact.

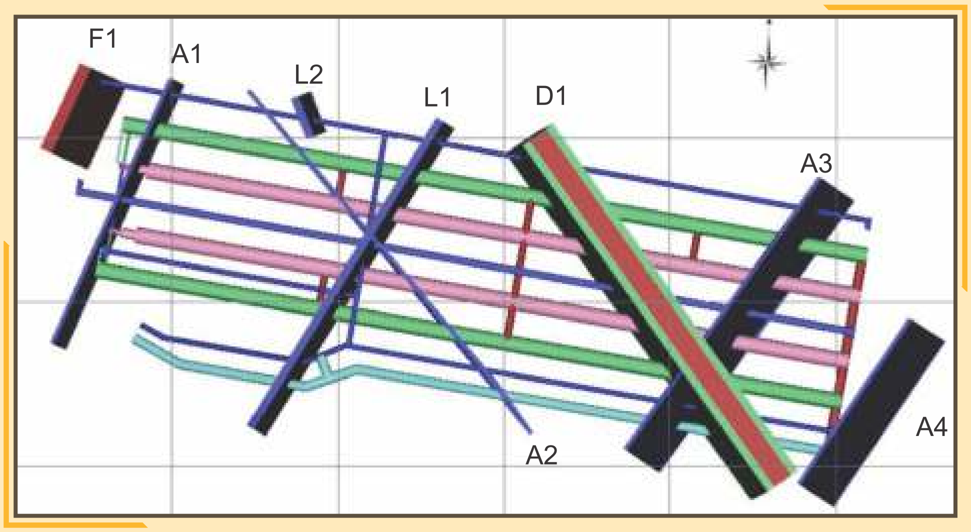


Figure 4.3: 3D Geological model at detail design stage

4.3 Excavation Face Mapping

Geological mapping of excavated tunnel faces otherwise known as excavation face mapping is one of the key activities of the excavation cycle. This is because the tunnel support philosophy is based on empirical rock support design, which is derived from Q-system of rock mass classifications along with wedge stability analysis. The typical rock support for the underground excavation is derived from the corresponding rock mass classes (Q Values) and comprises of rock bolts and shotcrete. In addition, based on the excavation mapping for anticipated critical wedge failures, recommendations for spot bolting is also included as part of basic design.

Excavation face mapping is performed to register the tunnel geology in a 3-D format. A cavern being about 28m to 32 m height, is generally excavated through drill and blast method in three or four stages. Since the constraint of plants and equipments' reach to a certain elevation is the guiding principle, the construction of caverns is undertaken through top heading and subsequent benching approach. Further, depending on the deployed equipments excavation is taken up either in full face approach or in a staged manner comprising of mid face and side slashing. Depending on the construction schedule, in order to derive applicable rockmass classes excavation face mapping is undertaken with adequate consideration and supplemented through inferences and interpretations. Engineering geologists map the excavated face by every round of blasting with the following main objectives:

i. To pickup all geological features like shear seams, dykes etc.

ii. To record the discontinuities with orientations and other parameters.

iii. To derive rock mass classification on the basis of prominent and relevant unfavourable joint parameters.

iv. To check necessity of feature specific rock supports such as spot bolts

v. To check and infer persistence of features in continuity to adjacent faces.

Based on the inputs of excavation face mapping engineering geologists prepare the Rock Support Recommendation Sheet (RSRS) which essentially takes into account

i. Category of Design rock support to be adopted for the tunnel chainage under consideration.

ii. Spot support required like modifications, if any, in length, direction & spacing of rock bolts, thickness of fibrecrete and sequence of support implementation.

Depending upon local geology, the spacing of rock bolts is either decreased by installing spot bolts or suitable adjustments of rock bolt spacing is adopted. The orientation of bolts, perpendicular to tunnel periphery, often needs to be altered depending on disposition of feature such that feature can be properly stitched using the bolts. The length of rock bolts in adverse case scenarios is increased for anchoring unstable block with the competent rock-mass. These site specific adoptions are undertaken through an approach better referred as geometrical analysis. In certain rock conditions like hydrothermally altered zone, where longer exposure of rock after excavation deteriorates the conditions, sealing by fibrecrete forms an immediate support requirement even before installation of rock bolts.

INSIGHTS

In one of the projects 30-40m wide hydrothermal zone was found associated with 20-30m thick mafic dolerite dyke. In the cited altered zone the rock strength was found to be depreciating with increased time of exposure. So, sealing fibrecrete immediately after blast was provided for the entire zone.

Probe holes are major tools of investigations during construction and are an absolute necessity for predicting tunnel geology ahead of the excavation face or behind the walls.

The probe holes are normally destructive holes drilled ahead of tunnel face wherein prediction is done mainly by interpreting drilling time / hindrance and nature of return water / sludge. The systematic probing technique is often intervened according to encountered / anticipated geological condition. The frequency is generally increased in vicinity of geological hotspots predicted in the interpretative 3 D geological model. Normal directions of probeholes along tunnel alignment are altered depending on orientation of anticipated geological feature relative to tunnel geometry. Whereas the investigation hole by coring are required to assess the tunnel grade geological conditions associated with critical geological feature.

In another project, lot of investigation holes had to be recommended to ascertain extent and nature of a shear zone behind the side wall of cavern for treatment plan of feature subsequent to large sidewall wedge failure along the feature. The designed advance rate for the construction progress was modified as per encountered rock class and localised blast control was adopted as per presence of local geological features.

4.4 Updation during excavation and other aspects

The predictive 3D geological (structural) model prepared before start of construction is dynamic in true sense as it is continuously refined with inputs available in course of ongoing excavations. The updating is done through updating overall Q-map and structural map.

The water curtain tunnels act as pilot tunnels before commencement of the cavern excavation. Selected water curtain boreholes are cored/imaged by Bore Hole Tele Viewer (BHTV) revealing additional information about orientation of the discontinuities around the cavern.

Based on the excavation face mapping of the access tunnels, water curtain tunnels, and the derived information from the water curtain boreholes, the 3D geological (structural) model is updated on a routine manner as the excavation of tunnel progresses. The predictive 3D dynamic geological model is revisited based on the excavation progress; the process helps to identify the locations of geological hot spots in advance. This ensures assessment of necessary special design and pre-grouting requirements. Difficult rock conditions if any are probed, rock mass assessed and supported accordingly with special design supports. Pre-grouting of the rockmass is also taken up in these conditions if warranted.

Figure 4.4 shows the 2-D geological map of one of the projects updated after completion of major portion of water curtain tunnel place above storage caverns. The main dyke is shown in green colour and small shears encountered along the predicted lineaments are also shown.

INSIGHTS

In one of the projects, a horizontal investigation hole of 69 m was cored along tunnel alignment to ascertain the attributes of a 30 m thick mafic dyke. This anticipatory approach helped to suitably adopt the rock support design for the affected segment. More so, owing to the revealed geology of the mafic dyke the excavation plan was modified to adopt a cautious and relatively lesser pull.

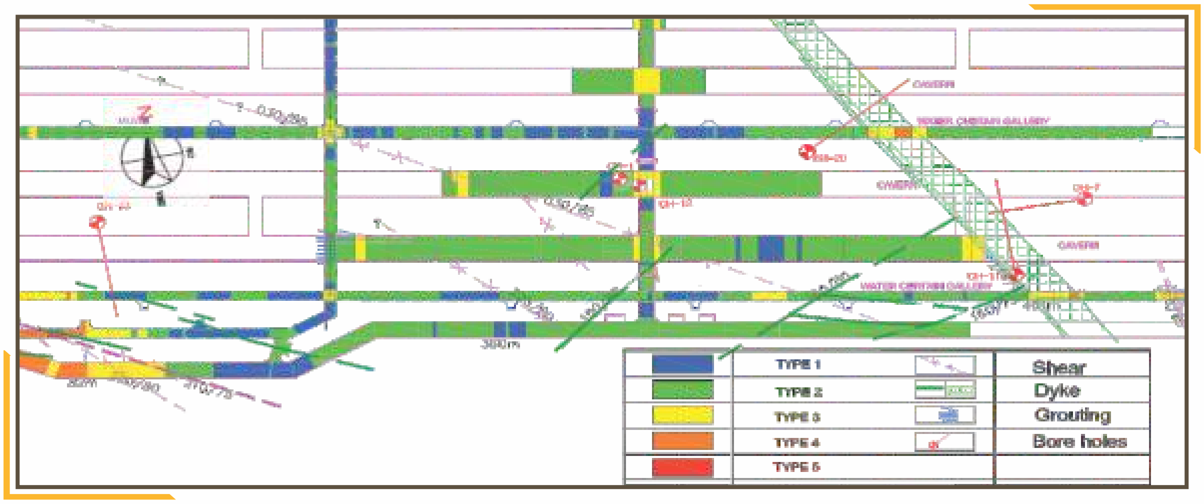


Figure 4.4: Geological model updated during construction

Storage caverns typically 30 m in height (and 20 m width) are being excavated in 4 stages; one top heading and 3 benches of about 8 m height each. Based on the 3D geological model, geological plans are prepared at every level viz., heading and all the bench levels. Longitudinal section is prepared along the caverns on both the walls. Sub-horizontal weak geological feature and any other feature which run sub-parellel to cavern and dip steeply towards the cavern wall is given special importance. The interpretation of core logs and BHTV data from the large number of water curtain boreholes drilled above the storage caverns are of utmost importance, as they are drilled 20 m above and beyond either side walls of the cavern.

Before excavation of cavern top heading, detail design of the cavern was revisited based on the updated 3D geological model and the prepared geological plans and sections. Special design support and pre-grouting requirements are recommended based on the geological information and exposed water curtain tunnel for the predicted geological hotspots in the cavern. The necessary provision is kept in contract for such difficult rock conditions. The support evaluation of heading is made keeping in mind the rock mass which may be encountered in the subsequent benching with the help of L-sections. 3D Geological Model of the cavern is updated at every subsequent benching stage and the geological plans and longitudinal sections are updated to predict the chainages of weak zones at every bench. These are reviewed and discussed amongst the designers. Special Technical Assessment Meetings (STAM) are arranged for this purpose at site before start of bench excavation. This aids in stability check of caverns at every stage during construction]

The Geological assessment during such evaluation comprises of:

i. Summary of geological mapping such as 3D Geological model, geological tunnel logs prepared from face maps, structural geological maps, Geological plans at relevant bench level, L – sections showing geological projections upto cavern invert on both the walls, Geological cross sections especially for Geological hot spots are produced. Inputs from BHTV/core log data from water curtain boreholes are also used in these assessment.

ii. Recommendations for additional investigations, if any, are evaluated and recommended.

iii. Rock mass classification maps and rock support maps are prepared for the relevant bench segment.

iv. Observations on any changes in the geological data are recorded and discussed.

v. Major joint sets are statistically analysed based on actual excavation data (face maps and tunnel logs). Rosette plot and contour plots of joint sets are prepared. Joint sets are characterised based on frequency, joint condition, filling and persistence.

4.5 Geological Uncertainties

The necessity for continuous updating of geological model, stage wise revisiting the support evaluation based on geological assessment point towards the intensity of uncertainties /surprises involved with factors in the underground storage cavern projects. Like all other underground works, no amount of investigations is sufficient to rule out geological uncertainties. Nevertheless the systematic approach of investigation campaigns, development of interpretative 3 D geological model, constant updation of the model with all additional information generated in course of construction and rechecking of the stability before each stages of excavation helps minimize the surprises to manageable extent.

The major surprise element involved is otherwise quite obvious for common geological features:

i. Variation in orientations: the small scale variation in different benches may be due to factors like warping of foliation or dip reversals in steeply dipping features whereas more severe variation are due to continuity of regional feature in en-echelon pattern and displacement by other feature or faulting.

ii. Variations in thickness of feature by pinching and swelling: while pinching out of a particular feature is always favourable for engineering purpose, unforeseen swelling of features like shear zones become very critical for stability of structure whose rock support has been designed by modeling the feature with lesser thickness.

iii. Time factor: Certain features have relatively rapid evolution causing negative impact within estimated life of the project or for the matter within the construction period of the project. Gradual washout of infillings of an otherwise stable discontinuity also leads to instability associated with increased seepage along the feature

Case Study 4.2: Updation of Geological Model through Investigations during Construction

During investigation stage of one of the projects on west coast of India, 30m thick fractured mafic dyke (TD5) with dip of 85° towards 70° was encountered in a corehole and was studied in detail. It was classified as class IV rock as per Q system and was structurally projected along its strike disposition. A Fracture zone (FR4) was found on either side of this dyke with clayey silt infilling of about 5-10mm. The layout of storage cavern (060° - 240°) was chosen (along with all other relevant geotechnical parameters) such that this dyke cut across almost perpendicularly all the storage caverns. The dyke rock was hard & fractured and at contact with parent rock, it was fresh and tight in general but at some places, sandy silt particles were found in the contact. Very little amount of water was intersected in this region. Dyke and fracture zone together constituting of about 50-60 m thick (Figure 4.5) was supposed to be critical zone during construction.

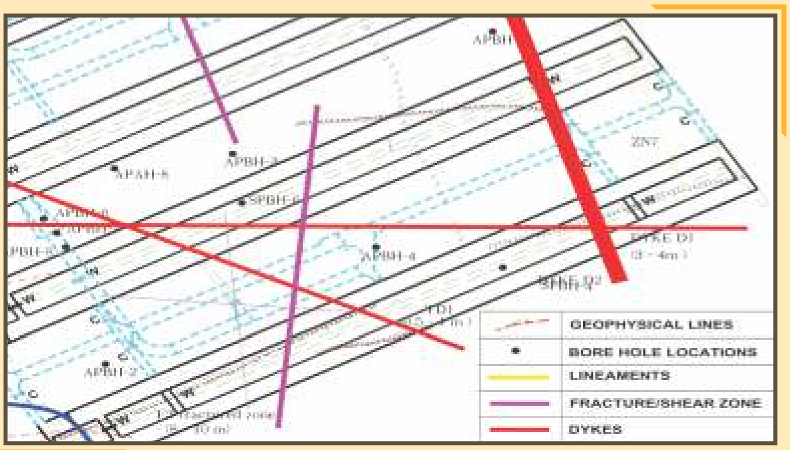


Figure 4.5: Geological model of the project during investigation

A geological model was developed at cavern heading level and the rock support design for the caverns was updated. Intersection of these dykes in the caverns along with other critical geological locations were classified as Geological hot spots and marked in the geological model and issued for construction and planning purposes. However, during cavern heading excavation, sudden change in the position of the dyke TD5 was observed in one of the storage caverns (Figure 4.6).

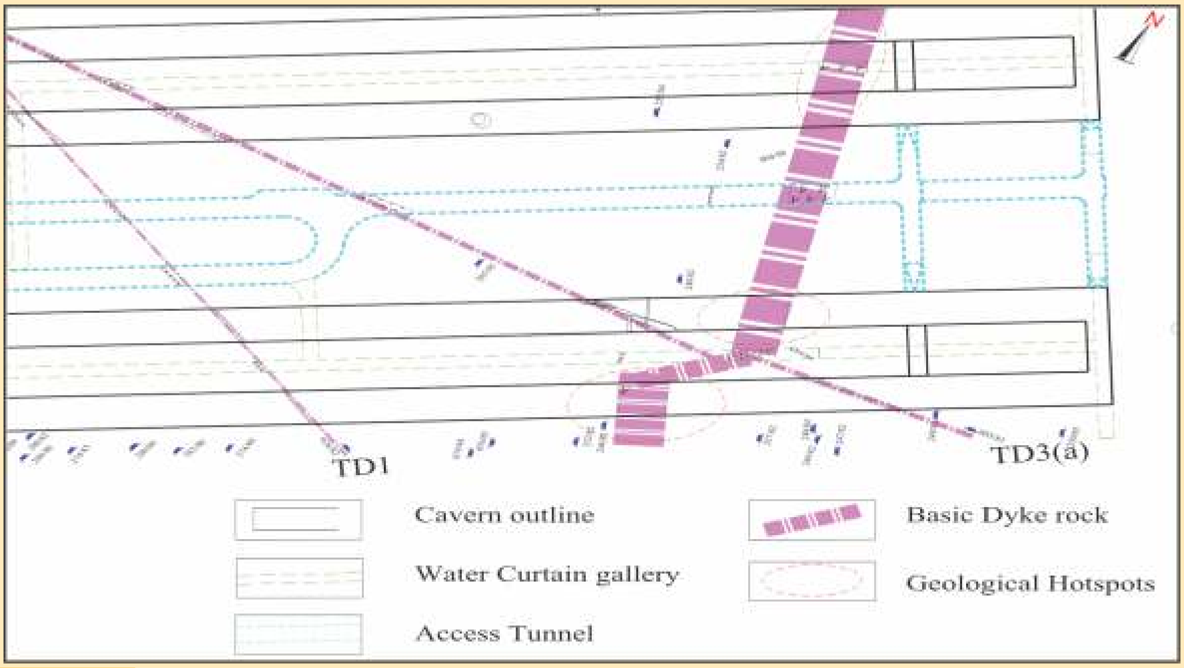


Figure 4.6: Geological model showing 30m thick dyke as geological hotspots of the project

The dyke was reported to have an offset about 50 m from the original orientation and encountered in the shifted position in one of the caverns, with same orientation. Apprehending that shifting could be due to dragging of feature within rock pillar between the two adjacent storage caverns, 3 water curtain boreholes drilled in this rock pillar just above the observed shift in dyke was examined with BHTV. It was revealed that the shifting of dyke occurred in the rock pillar and the orientation of dykes, which were split in to numerous dyke swarms, were found to be both perpendicular as well as parallel to the cavern alignment. The parallel orientation of the dyke swarms along with its steep dip towards north wall in one of the caverns was found to be critical. This zone in north wall of cavern was physically found to be fractured and crushed with silty clay infilling and was classified as rock class V where as in south wall of the cavern, no shifting in dyke was found and the dyke remains perpendicular to the cavern wall and is classified as class IV.

The overall geology in this segment was reported to be quite complex and it was not very clear whether the shifting in dyke was caused by displacement along the narrow dyke (TD3) or dyke was emplaced on pre-existing fracture system and during emplacement dyke found its own way along the weaker zone explaining the ductile mechanism observed. Therefore the observed geological uncertainty warranted more deliberation and assessment with following key aspects:

i. Nature of the affected segment namely either brittle or ductile;

ii. Sequence of dyke emplacements;

iii. Associated fracture zone (FR4) is either syn-tectonic or post tectonic.

In order to ascertain the extent of branched out dyke passing parallel to the cavern, 4 nos. of probe holes of about 15 m length were drilled. The probe hole result showed that these dykes were extended up to a distance of 10 m behind the storage cavern wall. In addition, these branched dyke rocks were highly irregular and discontinuous.

Further in order to ensure the stability of the cavern, Design verification was necessitated during excavation taking into consideration the following factors evolving out of the change in orientation of the feature TD 5:

i. The fractured condition of the north wall segment wherein the dyke was reportedly oriented parallel for certain extent

ii. The changed orientation of dyke (085° towards 160°) as one major discontinuity set for wedge analysis.

Case Study 4.3: Adverse Geological Conditions and Design Interventions

In another project, on the eastern coast of India, a large wedge failure occurred in northern wall of one of the storage caverns, during excavation of the last bench (Bench 3). The wedge failure extended over approximately 80m length and up to 20m high (Figure 4.7). Some cracks in shotcrete were also observed on the same wall, as extension of failure, as well as on the opposite wall of the cavern. The failure took place as toppling wedge resulting along a slickenside shear plane.

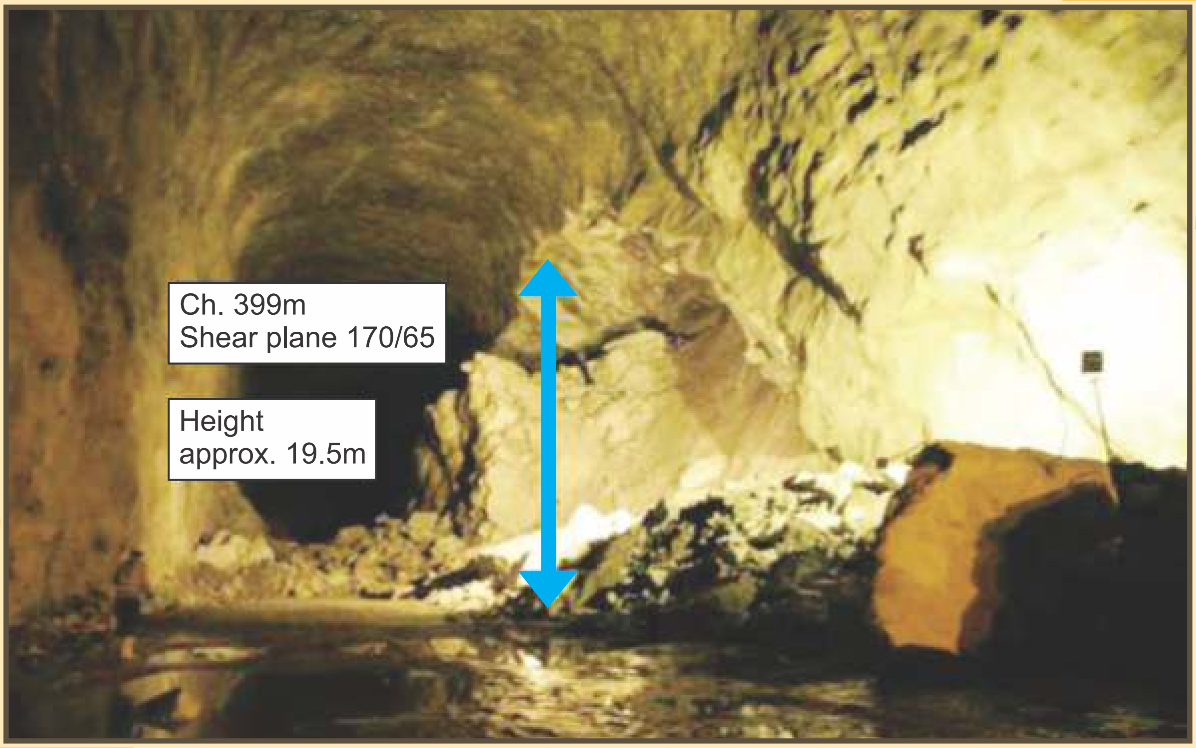


Figure 4.7: Photograph of the cavern wall failure zone

Immediately after the wedge failure, additional investigations were carried out in order to study the actual geological conditions within this area of special concern (Figure 4.8) to check the extent of zone of failure and to rule out similar failure in any other zone. The results of geological mapping, additional investigation drillings (Figure 4.9) in the pillar between adjacent cavern legs as well as inspections during the excavation of the failure rock mass have indicated that there is sheared material of substantial thickness present behind the apparent failure plane. Therefore, although the failure occurred along a distinct shear plane, rock support needed to be checked and verified for a case of a shear zone rather than a shear plane only.

The rock mass in this area consisted of banded to foliated, medium to coarse grained, strong and only slightly weathered, grey coloured Khondalite. The area was characterised by intersecting multiple shears and two shear zones which appear in part to have some pyroxene veins or dykes contained within them. The material along and within the major shear was brecciated and contained clay-rich fault gouge resulting in weak rockmass. The rock mass was damp to dry at the time of failure with no apparent water inflows.

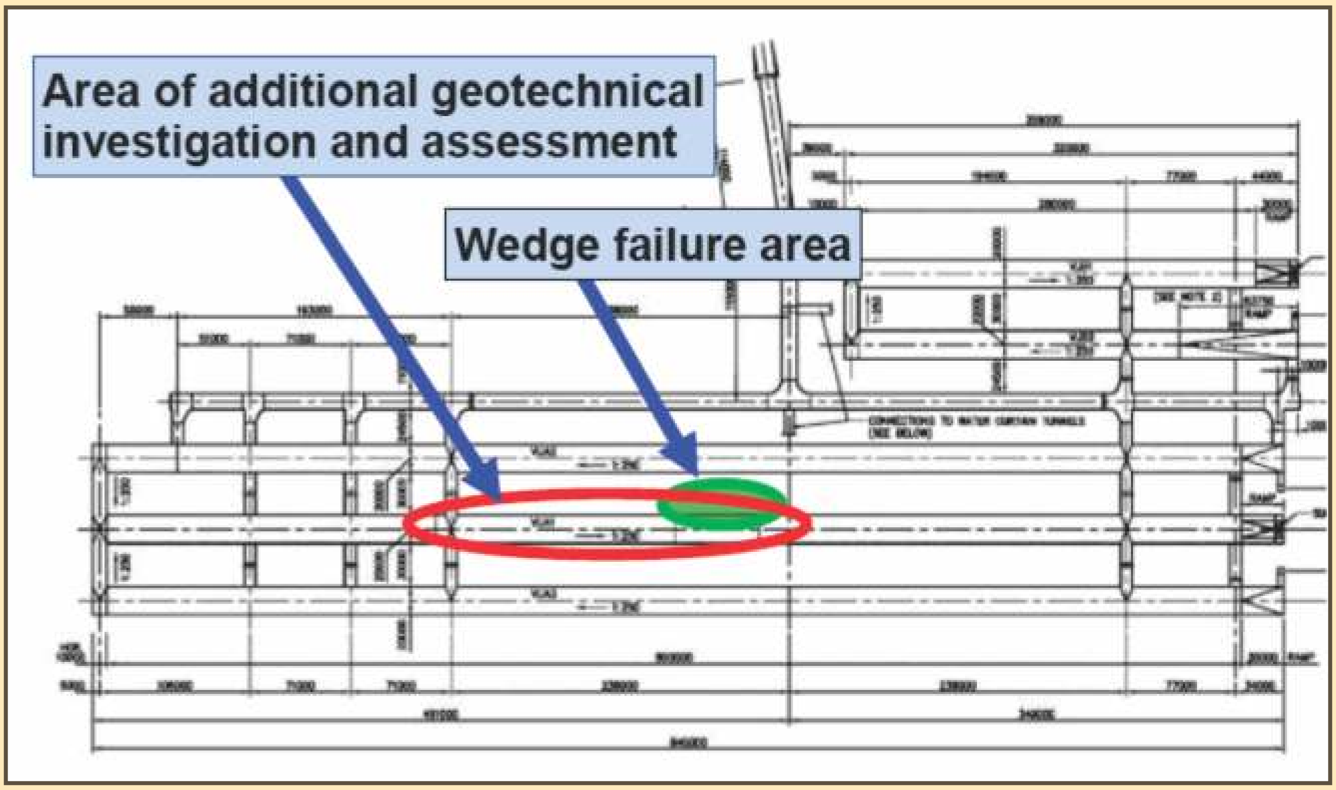


Figure 4.8: Location of wedge failure area and additional assessment zone along the cavern north wall

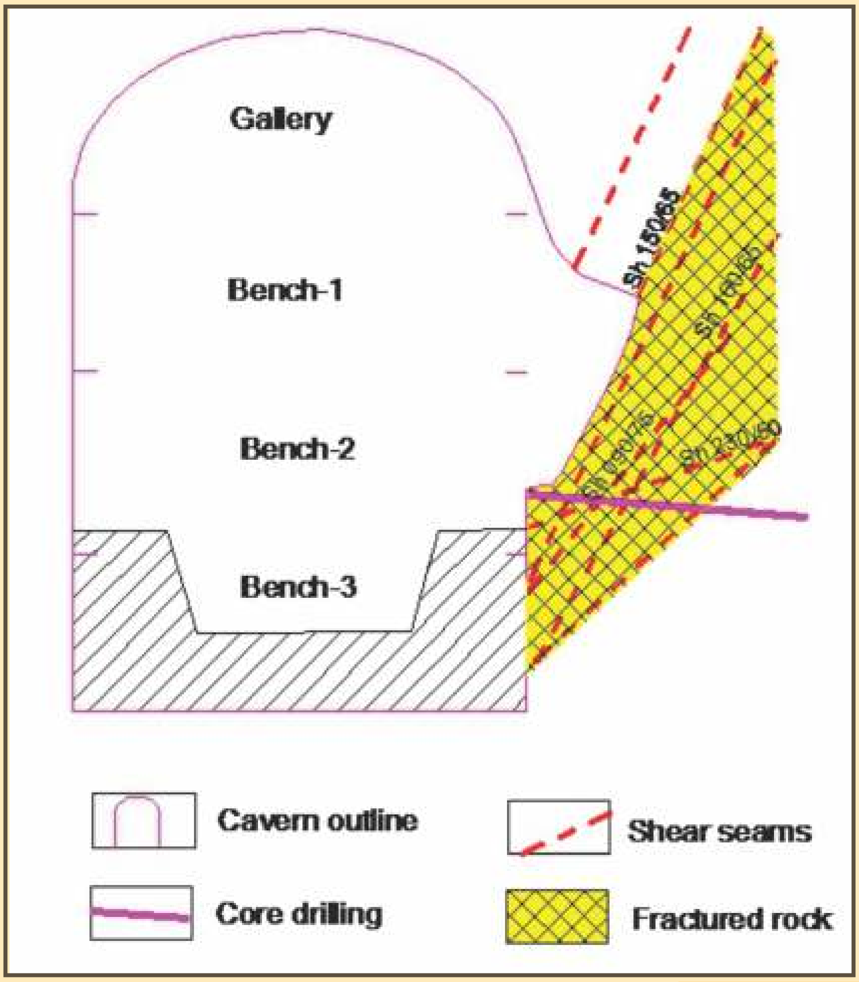


Figure 4.9: Typical finding of investigation holes

The rock mass was characterized into three different qualities, depending on the fracture intensity and characteristic discontinuity surface conditions (Figure 4.10):

i. Red/pink shaded structures indicated the major faults/shears and composed of very highly fractured to sheared/disintegrated rock with frequent clay filling and coatings, local zones of more blocky conditions may exist within the scale of the structure. This rock mass is typically called very poor rock or sheared rock.

ii. Orange shaded regions represent highly fractured to crushed rock mass conditions with local clay filling and common clay coatings. Local shears may be present within this rock mass with increased clay filling thicknesses. This rock mass is typically called poor rock.

iii. Green shaded regions represent fractured rock mass conditions with improved rock mass quality (fair to good). Local shears or faults may exist in these regions with increased clay coatings and minor filling. This rock mass is represented by fair/good rock.

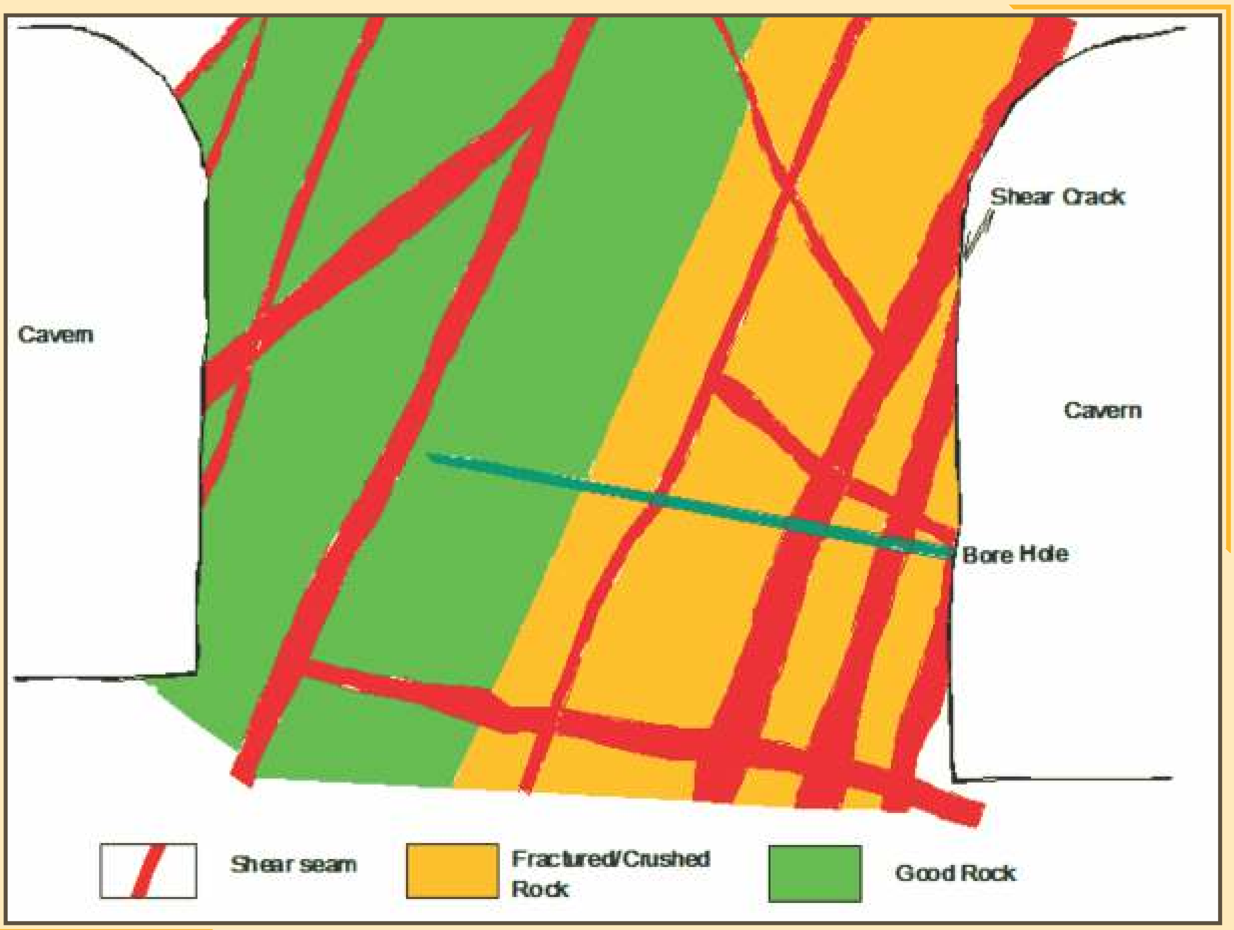


Figure 4.10: Geological assessment of rock pillar between caverns

The orientations of main geological features in the cavern south wall indicated an intersection of two main sets of shear features. One running acutely along the cavern axis (070-180°) at an orientation of about 170/75 (direction/dip) and another one crossing the cavern with an orientation of about 070/60. The general geological setting in this area was created by a toppling wedge situation intersected by the crossing shear zone. Poor and very poor rock was to be expected at the intersection of the two faults. The poor rock mass condition was likely created by corner effects and two-directional shear movements within the intersecting shears.

The rock mass encountered during Bench 3 excavation in this area was the poorest rock mass so far encountered in the entire project site. Excavation was carried out by excavator without the need for blasting. Stress relaxation during excavation of Bench 3 caused movement within the poor rock in the lower section of the pillar, resulting in shear deformation along the toppling wedge geometries much higher up at cavern spring line level

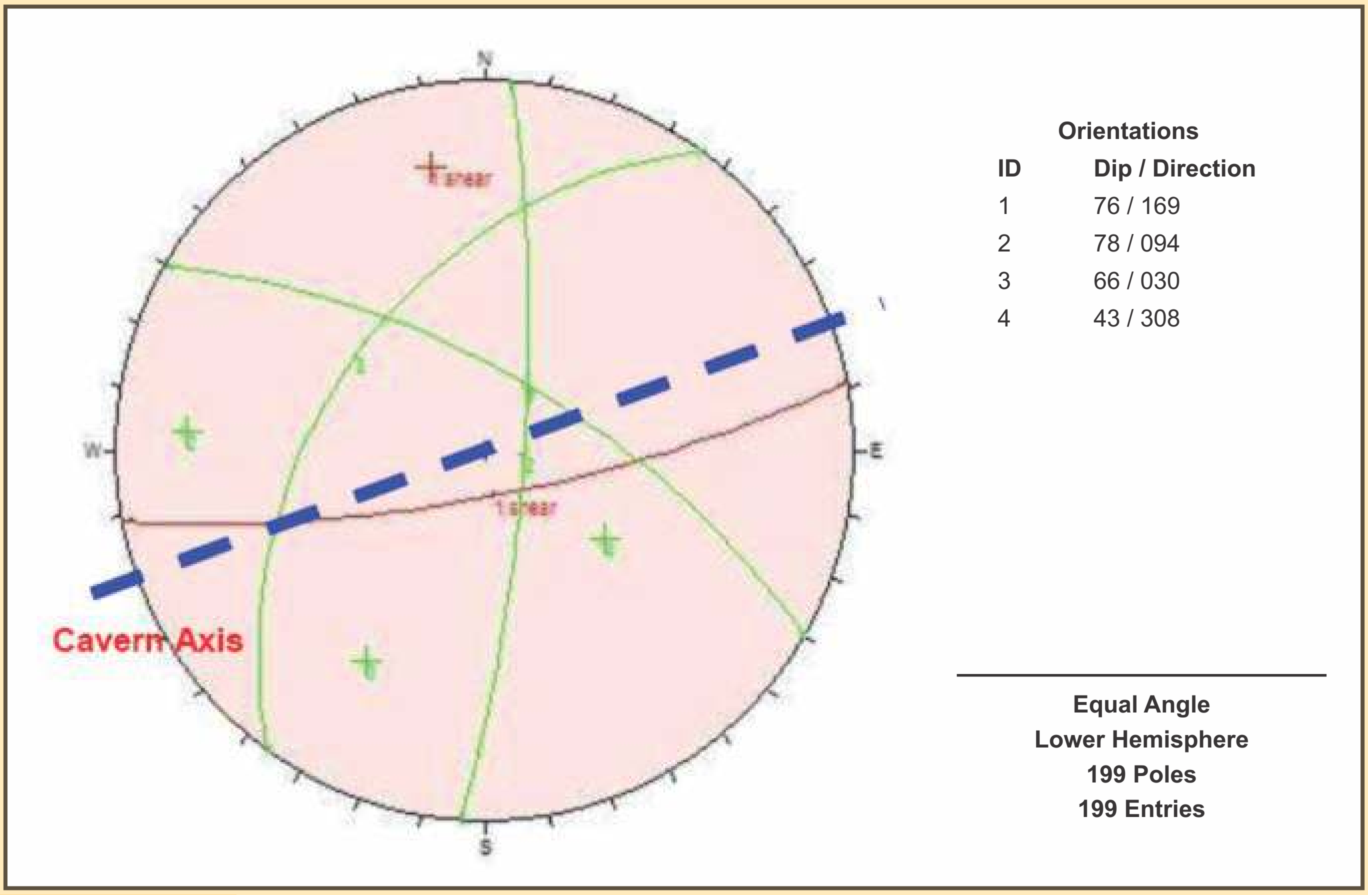


Figure 4.11: Major plane plot showing averages of major joint and shear systems

or even higher. The geology of area was reconstituted through several geological sections and joint plots (Figure 4.11) . The reconstituted geology was used for modelling the rock support analysis through FEM. The red line in Figure 4.11 represents the plane of the major shear in the lower hemisphere and the green lines the planes of the major joints. Poles to planes of joints and shears are represented by the green and red crosses respectively. The conditions of the joints are mostly rough, planar to undulating with clay coatings or clay fillings. Only very few joints have slicken-sided surfaces.

The two geological uncertainties associated with this sudden failure were:

- The waviness of shear plane made it almost parallel to the cavern wall at the stretch
- The intensity of shearing increased with depth.

The management of the failure zone essentially comprised of:

- Reducing section at lowest bench thereby avoiding the adverse feature to daylight at cavern wall
- Additional reinforcement for anchoring the failure plane with relatively fair rockmass.

HYDROGEOLOGY AND GROUTING

G.Kannan, A.Usmani & A,Nanda

CHAPTER - 5

5.1 Introduction

The basic principle of storage of crude oil in unlined rock caverns is the hydrogeologic containment. Thus the rock caverns are planned at a depth such that there is sufficient hydrostatic pressure to counter the vapour pressure of the stored product. In order to secure the water flow from the rock mass towards the cavern a water curtain system is provided consisting of galleries located above the crown of the cavern. Boreholes are drilled from the water curtain tunnel to intersect all the joints of rock mass. A saturated rock mass and ground water flowing into caverns, ensures proper sealing of the stored product from leakage. Particular attention is given to ensure that the rock mass remains saturated with water even while excavation works are in progress.

Provision of water curtain above the cavern with bore holes charged to a head equivalent to the design pressure, provides the required hydrogeologic containment. Maximum operating gas pressure at the cavern crown and the vertical distance between water curtain gallery and cavern should satisfy the requirement of hydraulic gradient greater than 1.0 at cavern roof level (Aberg, 1977).

This chapter presents the hydrogeological aspects that need to be considered in the design & construction of the large storage cavern as shown in Figure 5.1. The hydrogeological investigations & studies, analyses, grouting & construction aspects are discussed along with case studies of two large unlined crude oil storage caverns.

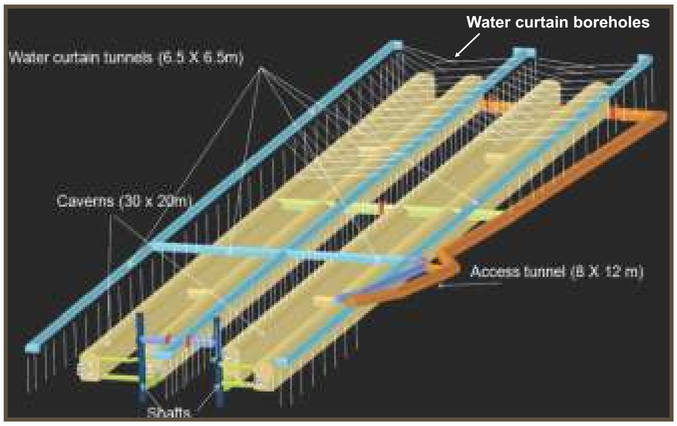


Figure 5.1: A typical cavern layout along with superimposed water curtain system.

5.2 Hydrogeological Investigations

A set of predefined hydrogeological investigations are performed during the exploratory investigation stage. Water pressure tests such as packer tests are conducted in coreholes drilled from surface using hydraulic inflatable packers for every 12 metres segments by Lugeon tests and Injection fall – off test methods. Other water tests like pumping in and pumping out tests are also performed in few selected set of coreholes to estimate transmissivities. From the hydro-geological test results, a hydraulic conductivity profile is prepared for each core hole. Based on inputs from geological model and geophysical investigations, a conceptual hydro-geological model of the site is prepared.

Ground water level is monitored right from investigation stage from existing wells and the coreholes drilled during exploratory investigation campaign. Groundwater level data of at least one continous year is recommended to be collected from the project site. These data set is used as a background data reflecting the seasonal variation of ground water level and further build up over the duration of the project for monitoring the level over the operational satge of the storage installation.

Water quality tests are performed for the groundwater samples from these wells as a background data during investigations so as to benchmark with the quality of ground water during cavern excavation and further during cavern operation after crude filling. Water quality tests such as physical, chemical and microbiological tests are carried out, while chemical tests involve typical cation and anion tests and trace elements which are hazardous to drinking; microbilological tests involve total aerobic bacteria, total anaerobic bacteria, sulphate reducing bacteria and slime forming bacteria etc.

5.3 Hydrogeological Model Studies

Lineament ananlysis through topographical studies or satellite imagery studies form part of the investigation campaign and offers key inputs to hydrogeological tests / studies. The meteorological data, such as average annual rainfall of the project site for the past few years and the terrain analysis offers insight about the possible quotient of recharge / infiltration to the subsurface or the bedrock. Given the basic assumption of competent rockmass for unlined rock caverns the lineaments in such hard rock terrain are probable water bearing structures through which infiltration takes place to the bedrock. Based on the exploratory investigation results and the interpretations from terrain analysis integrated with sub surface inferences through geophysical studies, core logs and the developed geological model for the project site, a conceptual hydrogeological model is prepared.

Normally the hard rock terrain is characterized by a thin layer of soil/lateritic material, followed by a thin layer of weathered bed rock and subsequently massive or jointed rock with a low to very low hydraulic conductivity. Very few geological features in the massive bedrock are actually water bearing and they are highly hydraulically conductive. Hydraulic conductivity studies are carried out through packer tests (water pressure tests) during investigation for each identifiable litho type / sub surface layers. Hydraulic conductivity profiles from all the short and long duration water pressure tests are analysed and the hydro-geological properties of each litho unit are described. Normally the soil layer has high hydraulic conductivity of about less than 10-4 m/sec. Weathered bed rock and jointed bedrock with major water bearing geological features (shear contact and dyke contact) show hydraulic conductivity in the range of 10-5 to 10-8 m/sec. However, massive bedrock (without joints) shows hydraulic conductivity less than 10-8 to 10-10m/sec. With these key inputs, a conceptual hydrogeological model is prepared and the same is continuously updated throughout the construction stage along with the 3D interpretative geological model and other water pressure tests conducted during construction of water curtain tunnel. During construction, at times investigations are recommended to confirm certain specific water bearing geological structures.

The lowest ground water level of the project site is ascertained based on the monitoring of groundwater level throughout the year especially in the dry season. The highest ground water level of the project site is also ascertained especially during peak monsoon / rainy season. These two key information / input form the basis for the design of water curtain system and finalising the elevations of water curtain system and crown of the storage caverns.

During excavation, a minimum hydraulic head equivalent to 20 metres of water above the horizontal water curtain level is required to be maintained in order to ensure hydraulic gradient >1. This is to prevent de-saturation of rock mass surrounding the storage cavern. Thus impediment of uncontrolled inflow of groundwater in tunnels as well as conservation of groundwater is necessitated.

The hydro-geological model studies are performed to check the flow pattern around the storage caverns so as to confirm hydrogeologic containment and estimate the seepage rates based on the data collected during the exploratory investigation stage. Finite element analysis is carried out to estimate hydraulic gradient, seepage in the caverns during construction and make up water requirement during construction. These analyses are carried out for a no. of design considerations and conditions of operation viz. completely empty caverns at atmospheric pressure, maximum normal vapour pressure for storage caverns, storage cavern units under different pressures, etc. Based on these hydro-geological model studies, criteria for grouting are established including requirements of probing, tentative grouting scheme for likely grouting segments.

5.4 Water Curtain System

Based on the model studies, the water curtain system is conceptualised and is normally oriented parallel to the alignment of storage caverns and boreholes are drilled in the direction perpendicular to major joint sets so as to achieve maximum seepage flow towards the cavern. A water curtain system comprising of series of boreholes with a regular pattern drilled from the water curtain gallery above the storage galleries are shown in Figure 5.2.

Excavation of the water curtain galleries are carried out using drill and blast method and typical size of the water curtain gallery is designed to optimize the construction, completion and testing of the water curtain system. The section and the size of the water curtain tunnel is designed so as to accommodate the water curtain boreholes, additional test boreholes and extension of boreholes in ceratin specific cases.

Probe drilling and geological face mapping are carried out during excavation of the water curtain gallery. The geological face mapping and observations during excavation of the water curtain galley enables to adapt the design considerations for detail engineering of the water curtain system. Based on the results of the excavation mapping, through an active design process, details of the water curtain system such as length and orientation of the boreholes, spacing of boreholes and inclination of boreholes are designed. Stringent monitoring of the ground water table is carried out during construction. Grouting in water curtain gallery is minimized and only employed where leaking water endangers work safety or creates unacceptable drawdown of the ground water table.

Water curtain gallery is excavated in advance of storage cavern excavation, thus the water curtain tunnel acts as a pilot / investigation tunnel revaling the geological setting likely to be encountered during cavern excavation. Water curtain system comprises of a significant no. of water curtain boreholes drilled from the water curtain tunnel before start of cavern excavation. The water curtain system is designed to extend so as to provide a cover of about 20m on all sides of the U shaped storage caverns. In case scenarios involving separation distance between two storage galleries and in consideration of operational flexibility and ensuring resilience of construction and other activities surrounding the project sites; vertical water curaton bore holes are also designed and drilled from the water curtain galleries.

The water curtain boreholes are pressurized with water to prevent lowering of ground water table when the storage cavern excavation is under progress and subsequently during operation. As a design mandate, the water curtain system is charged at least 50 m ahead of cavern excavation so as to prevent de-saturation of the rock mass surrounding thestorage caverns. It is important to note that a de-saturated rockmass with air traps and bubbles is extremely difficult to re-saturate and often become a significant risk for the storage inventory of crude oil. About 10% of these water curtain boreholes are cored or investigated through BHTV studies revealing geology 20 m beyond all the caverns. The 3D geological model of the project is updated and detailed geological informations are collected before cavern construction. Packer tests such as Lugeon/Injection - fall off tests are also performed in the entire water curtain boreholes. These hydraulic conductivity data thus generated before cavern construction helps in updation of the hydrogeological model.

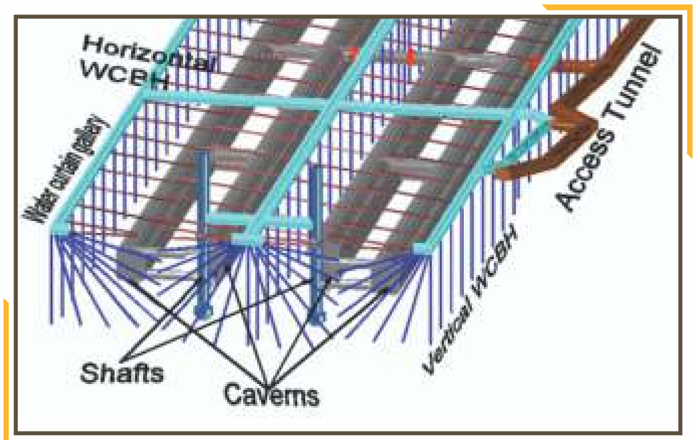


Figure 5.2: Typical representations of horizontal and vertical water curtain boreholes.

5.4.1 Water curtain boreholes

Based on the results of excavation mapping, total number of boreheols, location, length, spacing, orientation and the diameter of water curtain boreholes are designed in the water curtain galleries. However, as per design considerations the minimum diameter of the bore hole is maintained as at least 95 mm. The boreholes of horizontal water curtain are drilled about 1 m above the invert of the water curtain gallery and the boreholes extend beyond the cavern contour lines. The boreholes of the vertical water curtain system are drilled from water curtain gallery inverts. The upper part of all sub-vertical boreholes are protected by a casing, which extends above water curtain gallery invert in order to prevent sludge from being carried into the hole during construction phase. Vertical water curtain boreholes extend below cavern invert elevation (lowest invert) as specified in Design. Maximum lengths of the water curtain boreholes are typically about 150 m. with typical spacing of 20m between the holes.

During construction, depending on the segment specific requirements additional water curtain boreholes are specified to be drilled from the water curtain tunnel. These are required for improving the efficiency of water curtain system, to conduct water pressure tests so as to gather additional information on local hydrogeology, to undertake underground instrument installations, to cross select joints or to counter balance the effect of locally unfavourable flow pattern. Provisions are also kept for additional boreholes to be drilled from surface. These alternative is exercised when it is not possible to drill additional boreholes from the water curtain galleries. These holes are vertical, inclined or horizontal with a minimum diameter of 95 mm. Most of these holes are incorporated within the water curtain borehole monitoring network.

Based on design considerations, the water curtain tunnel are connected to the ground surface through monitoring wells and suitable instrumentation network and form part of the ground water monitoring system. The monitoring well is used for monitoring the hydraulic head in the water curtain and performing water sampling during operation and injecting chemicals in to the water curtain for water treatment (bacteria, etc.). The monitoring is drilled vertical from the surface into the water curtain gallery crown and are equipped with PVC or stainless steel casing. The minimum inner diameter of the casing is 180 mm facilitating easy sampling procedure. This casing is sealed on all length and stick out at surface level where it is protected. The Instrumentation wells are required in case of bringing out cables of instruments installed in the Water Curtain Gallery through the shafts.

5.4.2 Drilling of Water curtain Boreholes

The boreholes are drilled by either rotary or percussion method, while drilling with compressed air is not acceptable; drilling with only water hammer is recommended. The drilling equipment should be capable of drilling a length up to 150m. Provision for coring / BHTV is kept for 10% of the total number of boreholes. Any borehole that penetrates a buffer zone of 10m around the cavern walls and roofs are rejected and backfilled with cement grout. Drilling procedure is performed with clear water and during drilling of each borehole flow rate measurements are recorded in litre/minute.

Upon completion of drilling, the holes are thoroughly flushed with fresh water and all mud and cuttings removed. In order to avoid air trap and possible leakage pathway, direct air flushing is strictly prohibited. The boreholes are equipped with sealed casing in PVC of minimum 100mm inner dia. and 3 m in length. Each borehole is equipped with mechanical packer, individual well head pressurizing equipment including closing valve, water meter, manometer and non-return valve. The individual wellhead equipment is made available and ready to install prior to starting the drilling of water curtain boreholes. The detailed design of the pressurizing head and water injection set up is designed before hand for implementation.

The accuracy of positioning of boreholes is a critical consideration and is maintained at 5 cm. A deviation survey is carried out after completion of first 20 boreholes by deploying a borehole deviation survey instrument. Subsequently deviation surveys are carried out for at least 20% of the borings and form the basis for acceptance of the boreholes. The error in determining the deviation from the theoretical location of borehole is maintained at less than 5% of the drilled length at any point of time. Any borehole found to fall outside the allowable tolerance are replaced by an additional borehole complying to design consideration.

5.4.3 Dismantling of water curtain boreholes and filling of water curtain galleries

After construction of all underground facilities, the water curtain galleries are designed to be flooded. Prior to commencement of flooding, the water curtain galleries are cleaned, permanent monitoring equipments are installed & commissioned and borehole water injection equipments including all temporary lines and supports are dismantled. However, identified sensitive boreholes are maintained with uninterrupted water supply and arrangements are made to dismantle the water injection lines of these select boreholes under water. This is achieved through scheduling these activities intermittently during the flooding of the galleries. Based on specific hydrogeological conditions identified during construction a dismantling and flooding plan is prepared. A detailed record is maintained to note the quantities of water supplied for filling the access tunnels and water curtain galleries, daily water levels in tunnels and also water level (resulting out of seepage) in storage caverns during filling of water in the water curtain tunnels and access tunnel.

After water curtain system is flooded, the permanent water supply to the water curtain system as specified in the design is maintained throughout the period till completion of cavern acceptance test (CAT), commissioning and subsequent cavern operation.

5.5 Probing and Grouting

The storage caverns and water curtain galleries are constructed in competent bedrocks characterised by very low permeability. However, during excavation caverns intersect highly permeable dykes and joints leading to excessive seepage, thus necessitating grouting. Grouting is required not only to arrest seepage during construction, but to prevent water level drawdown and to maintain the decreasing hydraulic gradient around them. Water curtain boreholes in water curtain tunnels are also pressurized with water to about 4 bars pressure during cavern excavation to maintain water table close to the natural ground water level that prevailed before construction which also contributes seepage. Ground water levels are monitored from surface piezometers and underground manometers and the adequacy of hydraulic confinement is checked continuously throughout the construction. The ground water levels are analyzed with respect to excavation sequence, the encountered seepage and schedule of grouting activities performed underground. Hydro-geological hotspot locations are identified and maps of mandatory probing zones and mandatory pre-grouting zones are prepared for gallery and every bench of the storage caverns from the updated hydro-geological model.

During the excavation progress, hydro-geological model is constantly updated by:

1. Structural projections of permeable features;
2. Results of water pressure tests performed in water curtain boreholes and derived hydraulic conductivities of the water bearing features;
3. Results of probing and the grouting scheme adopted for defined segments;

4. Observing the seepage locations & permeability values of all water curtain boreholes and manometer holes drilled from underground; and

5. Correlating all the above data.

Seepage maps are prepared at every stage of construction which then forms part of hydrogeological model.

Since pre-grouting is always preferable and is more efficient than post grouting, stage wise systematic pre-grouting is performed mandatorily at the hotspot locations before every bench excavation so as to avoid high localized seepage threatening construction and hydraulic containment.

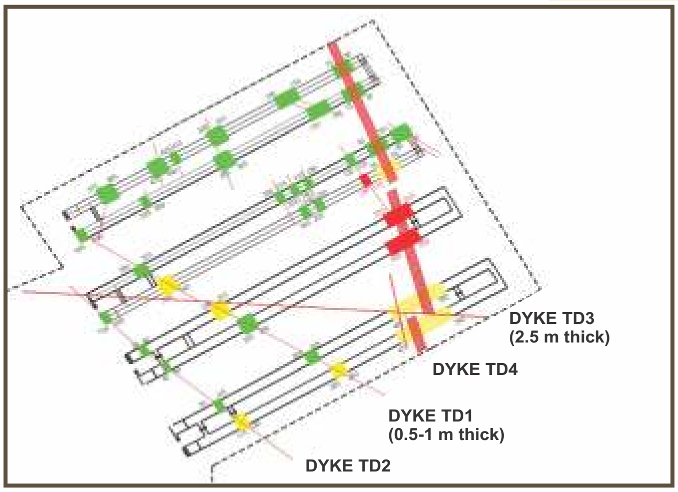


Figure 5.3: Mandatory pre-grouting zones along the permeable zones based on geological model

Mandatory pre-grouting zones identified during the assessment for a storage project are shown in Figure 5.3 and Figure 5.4. shows typical section and profile showing stage wise pre-grouting of cavern gallery and benches.

Additional probing, pre-grouting and post-grouting recommendations are also made based on the observations during excavation and are carried out before further excavation progress is made. Seepage to the caverns and water supply quantity requirements to water curtain system are related to the residual hydraulic conductivity of the rock mass obtained after grouting.

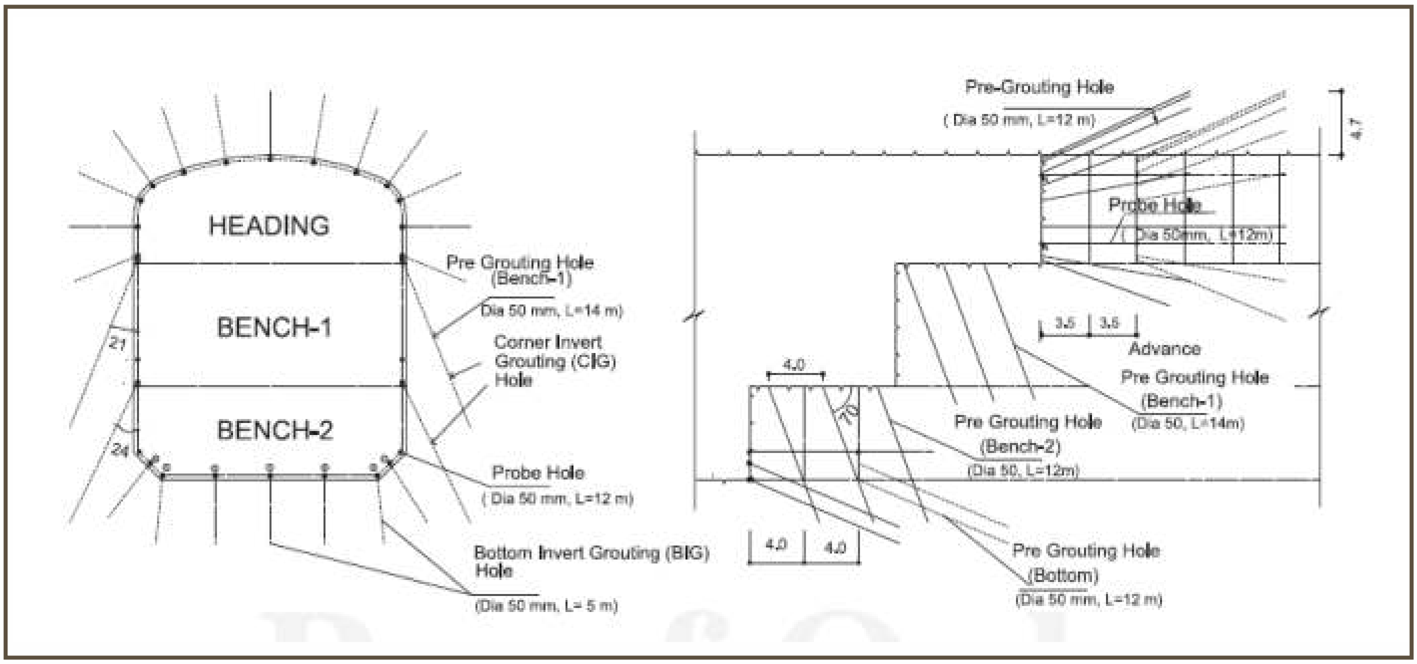


Figure 5.4: Typical systematic pre-grouting adopted during various stages of cavern excavation.

Pre-grouting is a preferred approach and is carried out from top heading by modifying the grout fan as suited to disposition of the identified feature. In case of persistent features overlapping grout fans are adopted for grouting from alternate faces.

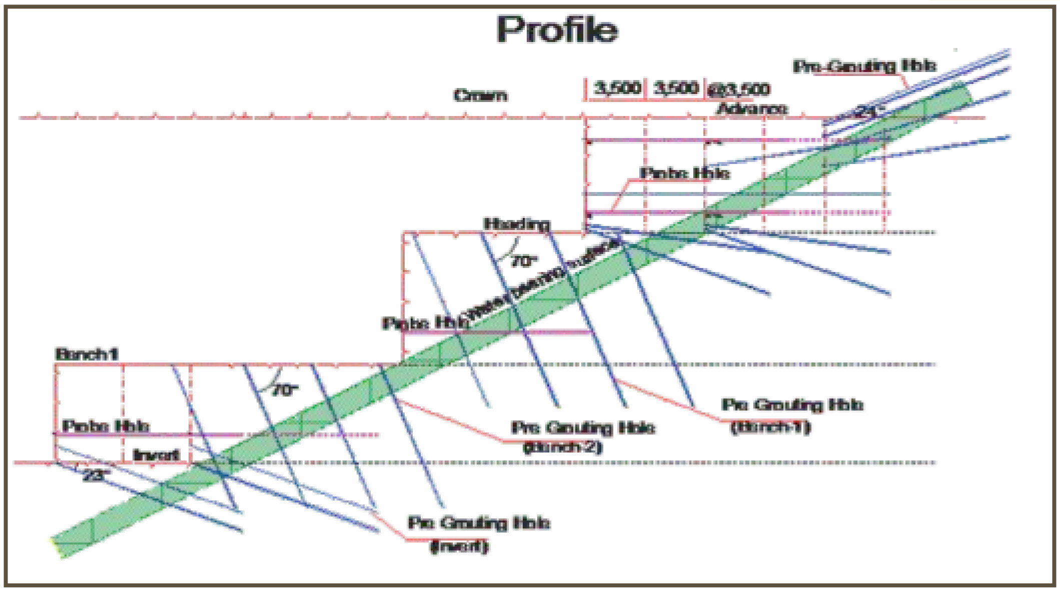


Figure 5.5: Typical pre-grouting scheme crossing a geological feature

Once the disposition of major hydrogeological features are established on the basis of excavation mapping, the probing schemes are optimized for subsequent bench levels and grouting activities are concentrated in the zones where the features are anticipated to be negotiated in the respective elevations (Figure 5.5). Accordingly, pre-grouting plan of all benches are prepared. Side wall pre-groutings from higher bench levels were carried out in the identified zone with sub vertical grout holes directed to intersect the feature and constitute grout curtain to cutoff wall seepage. Invert pre-grouting from last bench is carried out with target to cutoff seepage up to depth of 5m from invert.

Case Study 5.1 : Seepage Estimation

This case study enumerates a complex geological condition encountered at one of the storage project sites, located under a hill with a maximum elevation of 80 m above mean sea level. The investigated area consists of a landscape which is gently sloping down into two valleys. The elevation goes from +30 msl in the valley to +80 msl in the hill slope. The bedrock is granite gneiss overlain by laterite. Systematic hydrogeological investigations were carried out for this site followed by representative numerical studies in order to ascertain flow variations of the area. The hydrogeological investigations mainly consisted of water pressure tests, which included mainly short duration tests, long duration tests and interference tests. Furthermore, additional information was collected from daily water level measurements in the boreholes as well as observations on drilling fluid losses and gains.

The investigation data revealed main bed rock formations having an average permeability of 3.5×10^{-8} m/s. The investigation also exposed presence of sub-vertical dykes and lineaments in and around the area having high permeability values of the order of 10^{-7} m/s. The main findings from the investigation were the identification of major highly permeable sub-horizontal joint sets having permeability of the order of 10^{-4}m/s. These observations pointed to the heterogeneous permeability distribution as shown in Figure 5.6, and possibly towards the existence of compartments, each with a different hydrogeological behaviour.

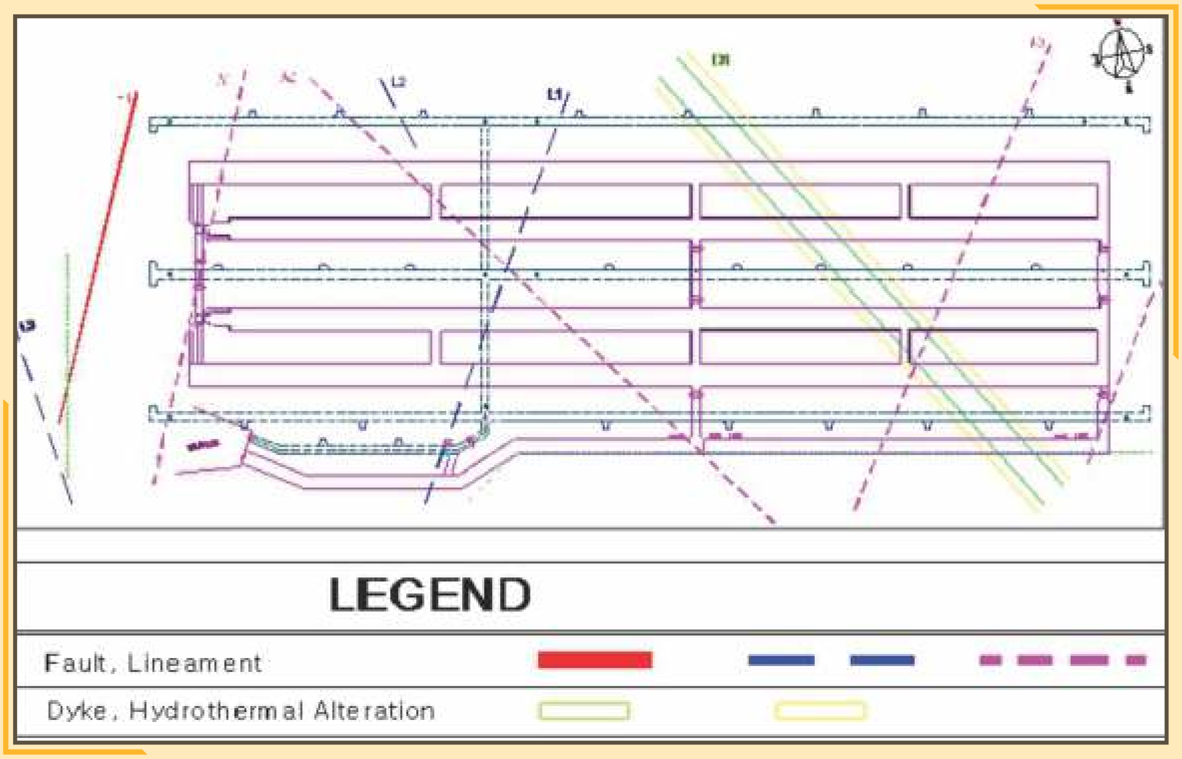


Figure 5.6: Layout and geological map of the underground storage system

The occurrence of these high permeable sub horizontal joints, which is difficult to detect ahead of excavation works, constitutes a risk of desatuartion during construction. Furthermore, such joints are rather difficult to grout and their residual permeability may be in excess of the average bedrock permeability. In case of insufficient natural lateral supply for these joints, such situations may lead to insufficient containment during operation. It was therefore considered to implement a vertical water curtain borehole system not only in between both storage units, but also in between the storage unit and the surrounding access tunnel, as well as a peripheral vertical water curtain system surrounding the underground storage units in order to maintain a continuous flow of water directed toward the caverns.

The numerical studies were carried out for the storage site, taking into account the existence of available geological/hydrogeological conditions. In this approach, individual effect of each of the major features, which were categorized into three main parts; the bed rock, dykes/lineaments and permeable horizontal joint set were considered. The numerical modeling was carried out considering main bed rock formation having an average permeability of 3.5×10^{-8} m/s and representing 80% of the total cavern area based on the investigation findings. The dykes and lineaments covering approximately 10% of the area were modelled having average permeability of 10^{-7}

m/s and major highly permeable horizontal joint sets having permeability of $5x\ 10^{-7}$ m/s (which was considered achievable post grouting of these joints based on previous experiences) covering 10% of the cavern area. It is assumed during the study that while the sub-vertical features influence the full cavern section, the highly permeable sub-horizontal features influence a limited section of the cavern and therefore this needs to be incorporated in the analysis.

In this study, 2D models, consisting of two units of caverns separated 60 m apart were modeled as shown in Figure 5.7 . Each unit of cavern consisted of two legs which are 900 m in length. The caverns are U-shaped in plan with an approximate 'D' shaped cross section and with a pillar width of 30 m. The caverns are 20 m wide and 30 m high and are aligned at an angle of 50o with respect to the maximum principal horizontal in-situ stress. The caverns are located 20 m below the water curtain tunnel which is 6.5x6.5 m in size.

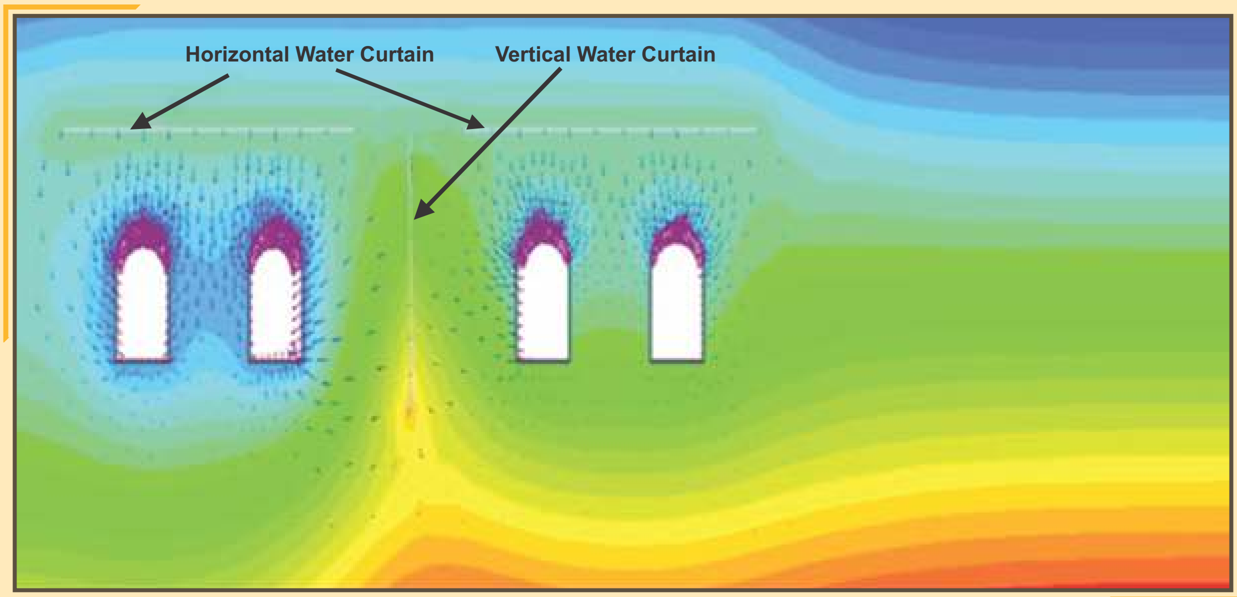


Figure 5.7: Flow patterns with a horizontal and vertical water curtain system

The natural ground water table was taken 75 m above the invert of the cavern and numerical analyses were carried out under both atmospheric and operational conditions. The maximum pressure considered during operations is 0.16 MPa. The distance between cavern and left/right side model boundary is taken as ten times the width of excavation to eliminate the effect of flow conditions on the cavern boundary. The top boundary of the model was assumed as natural water table level. All other boundaries were considered to be no flow boundaries. Water curtain was located 20 m above the invert of the cavern and was charged with a maximum hydraulic head of 47 m, worked out after taking all hydrogeological conditions into account. One horizontal water curtain gallery running perpendicular to cavern axis and extending 20 m beyond the outer cavern wall and three vertical water curtain galleries; one central, one along the northern boundary of the site and a third along the southern boundary of the site were considered in the model as shown in Figure 5.7. Spacing between horizontal and vertical water curtain boreholes was taken as 20 m.

The seepage estimation for this case is shown in Table 5.1. It is observed from the results that while the overall bedrock covers 80% of the area, the final seepage values are mainly governed by the presence of dykes and

horizontal joint sets. It is also inferred from the analysis that the permeability values considered for the horizontal joint set ($5x10^{-7}$ m/s) are based on efficient grouting mechanism of the same during excavation, which otherwise can significantly change the final estimated values of seepage in caverns

Table 5.1: Seepage values for different rock formation

Rock formation	Seepage Estimation (m^3/hr)	
	During Construction (Atmospheric Pressure)	During Operation (Max pressure) Pmax = 1.6 bar)
Bed Rock	24	16
Dykes & Lineaments	67	45
Horizontal Joint Set	59	39

Case Study 5.2 : Testing and Grouting

The project under case study is an unlined rock cavern, geologically seated in a stable Peninsular Gneissic rock complex comprising of granitic gneisses along with few mafic intrusions such as dolerite dykes of varying thickness. The project area has undergone extensive weathering and has lateritic soil at the top followed by weathered and fresh bed rock. The massive bedrock found in the project site is occasionally associated with dykes; fracture zones, open joints and few clay coated joints (Kannan et al, 2013). The cavern in particular is located in a hard granitic gneiss rock with relatively very few joints. Three joint sets are found prominent in the rock mass: two sub-verticals and one sub-horizontal. The sub-vertical joint sets are persistent and oriented in almost north-south and east-west direction dipping 80-85 degrees both ways. The dykes are found roughly oriented parallel to these discontinuities. Sub-horizontal joints are oriented east to N60E and dipping about 5-15 degrees both ways. The caverns are oriented N60E. Four different types of mafic dykes are encountered in the bed rock during construction in the project area (See Figure 5.3). Two of them TD1 & TD2 are thin (0.5 to 1m) and highly permeable. Their contact is slightly weathered with occurrence of 5-10 mm soft gougy material. These dykes are fresh & hard and moderately fractured. Third dyke TD3 found was about 3m thick and the contact is fresh & tight and is totally dry. This dyke intersects the other dykes present and its thickness varies and found branched at these intersections. The fourth kind of dyke namely TD5 found was about 30m thick and intersect all the caverns perpendicularly. This dyke rock is hard and fractured and at contact with parent rock, it is fresh and tight in general but at some places, sandy silt particles can be found in the contact. Very little amount of water is intersected in this region. Fracture zone (FR4) is found on either side of this dyke with clayey silt infilling of about 5-10mm. Dyke + fracture zone consists of about 50 - 60 m thick and would make it a very critical zone during construction.

As per hydrogeological model from investigations, three distinct hydro-geological zones were recognised in the site; surface permeable lateritic soil, permeable weathered gneissic bed rock and tight massive bed rock. The results from hydrogeological investigation shows that the hydraulic conductivity of the massive bedrock is of the order of 10-10 m/sec and weathered bedrock was found to be the order of 10^{-6} m/sec. However, the hydraulic conductivities of the intruded dykes were found to be of the order of 10^{-5} m/sec to 10^{-7} m/sec. The hydraulic conductivity of the sub-horizontal joint sets at south west corner of the project was of the order of 10^{-5}m/sec and higher. Pumping interference tests were carried out in the investigation core holes to establish seepage water quantity assessment in the caverns. The site receives more than 4000 mm of rainfall annually during monsoon (Kannan et al, 2013); most of them drain as surface run-off to the valleys located north and south. All the caverns are located about 60 m below the ground surface and about 40 m below the lowest natural ground water level (Kannan, et al, 2013). The ground water level fluctuation is monitored for monsoon and non-monsoon periods for which piezometers are installed on the 20 investigated coreholes spread all over the project area before, during and after construction.

About 550 water curtain boreholes are drilled at 20 m intervals in total ranging from 50-75 m length adequately covering all the caverns shafts and storage areas in the access tunnels. Water pressure tests (Lugeon/Injection fall off tests) were conducted in all water curtain boreholes and the water bearing zones are delineated in the hydro-geological model. These huge amounts of hydraulic conductivity data were grouped and statistically analysed along with the surface investigation holes and their geometrical means were taken for further hydrogeological analysis. Thus the hydro-geological model was updated along with updated geological model based on the above analysis and the seepage assessments are reviewed once again and the grouting requirements are ascertained with respect to certain hydrogeological hotspots. All the water curtain boreholes are pressurized with water to about 4 bars pressure during cavern excavation to maintain natural ground water level. However, in order to check proper functioning and efficiency of the water curtain system, certain hydraulic tests are conducted. Simple pressure observation test: wherein a set of continous 10 water curtain boreholes were closed from water supply after cavern heading excavation and further pressure loss was observed for the next 48 hours. Sensitive and nonsensitive boreholes were delineated from this test.

Pressure build up test: wherein a set of 5 continuous boreholes, 2 on either side of sensitive boreholes were closed from supply and then pressure restored in only the centre sensitive borehole and the pressure build up in the 2 adjacent boreholes on either side was observed for the next 48 hours. The interconnectivity of water curtain boreholes were tested for efficiency, and based on requirement additional water curtain boreholes were drilled at 10m intervals and some places at 5 m close intervals.

Water curtain efficiency test: wherein all supply to alternate water curtain boreholes is closed and the pressure loss/pressure buildup was observed till the pressure is stabilized and the remaining alternate boreholes are tested similarly and additional water curtain boreholes are recommended wherever required based on the results.

Pre-grouting is invariably preferred over post grouting in high seepage segments. Thus a mandatory probing scheme was devised before every excavation round at the beginning of excavation of water curtain and access tunnel. Later on when more detailed geological attributes were revealed with progress of excavation, mandatory probing was optimised to certain predicted sections based on the updated geological model. Probeholes were drilled typically 10-12 m length, covering three rounds of excavation. Further probeholes are drilled ahead with an overlap of one excavation round. Pre-grouting was performed when the seepage in probeholes exceeds certain seepage criteria decided based on the acceptable design seepage, 0.3 litre/per minute/metre/bar of static water

pressure. However, before cavern excavation starts, almost complete geological attributes are known especially about sub-vertical joints and features. Therefore based on the updated 3D geological model and hydrogeological model, certain mandatory pre-grouting zones were prepared (see Figure 5.3) based on prior probing and grouting results, where such high permeable zones intersect the caverns. Systematic pre-grouting was performed in these zones wherever the caverns intersect them during various stages of construction of caverns, namely heading, bench 1, bench 2 and bench 3. The grout design is planned in such a way that grout holes reach 5 m beyond all sides of caverns including roof, wall and invert (refer Figure 5.4).

Generally in cavern heading, umbrella pre-grouting (Kannan, et al, 2013) was carried out ahead of excavation face. For benches below; wall pre-grouting (corner invert pre-grouting) was performed on the sidewalls from one bench higher before excavation, so that grout spreads on all sides and controls water seepage entering from all sides of the cavern. Finally on the cavern invert, complete bottom invert grouting was performed, the grout holes reaching 5m below the cavern invert. At some high seepage and low lying ground water level area, this bottom invert grouting was performed before every bench stage so as to reduce the seepage and maintain the natural ground water level during bench excavations. Weirs were constructed across the caverns in these high seepage segments to measure seepage before and after grouting. At some places, post grouting was also required based on weir seepage measurement and the allowable seepage criteria. Grouting is required not only to arrest seepage during construction, but to prevent water level drawdown and to maintain the decreasing hydraulic gradient around them. The resultant seepage and hydrostatic pressure (groundwater level) is based on the extent and efficiency of grouting crucial to such storage projects. Grouting at two particular cases are discussed in detail in the following sections.

Grouting in vertical and horizontal jointed zones

During construction of the cavern in this project, a high permeable dyke TD2 of thickness 1m was encountered in some cavern sections with an hydraulic conductivity of the order of $5X10^{-5}$ m/sec. Their geological properties are already described in the sections above. Heavy seepage along with problem of lowering of the ground water table at this dyke contact was encountered. This dyke was found to be hydrogeologically connected to highly persistent sub-horizontal joints and also to a fault zone 200 m south of the project. This dyke was not envisaged during investigation stage and during excavation of water curtain tunnel and access tunnel prior to cavern, this was encountered. During pre-grouting, prior to excavating the face containing TD2, probeholes were drilled and huge seepage of more than 400 litres per minute were encountered and the hydraulic conductivity was estimated to the order of 5×10^{-5} m/sec. Accordingly extensive grouting was performed in these segments. Grout mix with Ordinary Portland Cement (OPC) was used to grout these high seepage segments. High grout consumption was noted during pre-grouting of this section of the order of 20 tonnes of cement. In this TD2 dyke area, ground water level was measured thorough monitoring wells and it was about 40 m above the water curtain level as the water curtain bore holes here were charged to upto 4 bars. Grouting of the rock mass was carried out using 50 mm holes extending 5 m inside the rock mass at invert level and 10-14 m inside the cavern side walls as shown in Figure 5.4. As described earlier, seepage measurements in these sections were measured using weirs and the seepage obtained in one of the cavern section was 1.8 litre per minute (lpm) in Bench1 and 2.02 lpm in Bench2 measured over 30m width weir. It was observed that increase in seepage from Bench 1 to Bench 2 is around 12%. The total seepage after complete cavern excavation and grouting was measured to be around 7 lpm/100m cavern width vis-a-vis the allowable design seepage of 9 lpm/100m cavern width.

During excavation of cavern in the southwest corner of the project, some persistent sub-horizontal joint sets were encountered and were spaced 5-10 m apart. About 5-6 such joints were observed in each of the cavern wall of unit 4 in south west side. These horizontal joints were found persistent more than 200 metres and found terminated by the dyke TD2 on one side and extend outside the cavern and beyond the project area. They are probably hydro-geologically terminated by an interpreted fault in the other side which lies about 300 m beyond the boundary of the project to the south and southwest. Before excavation of each of this cavern segment, these horizontal joints were actually probed and predicted through vertical investigation hole from water curtain tunnel at the top.

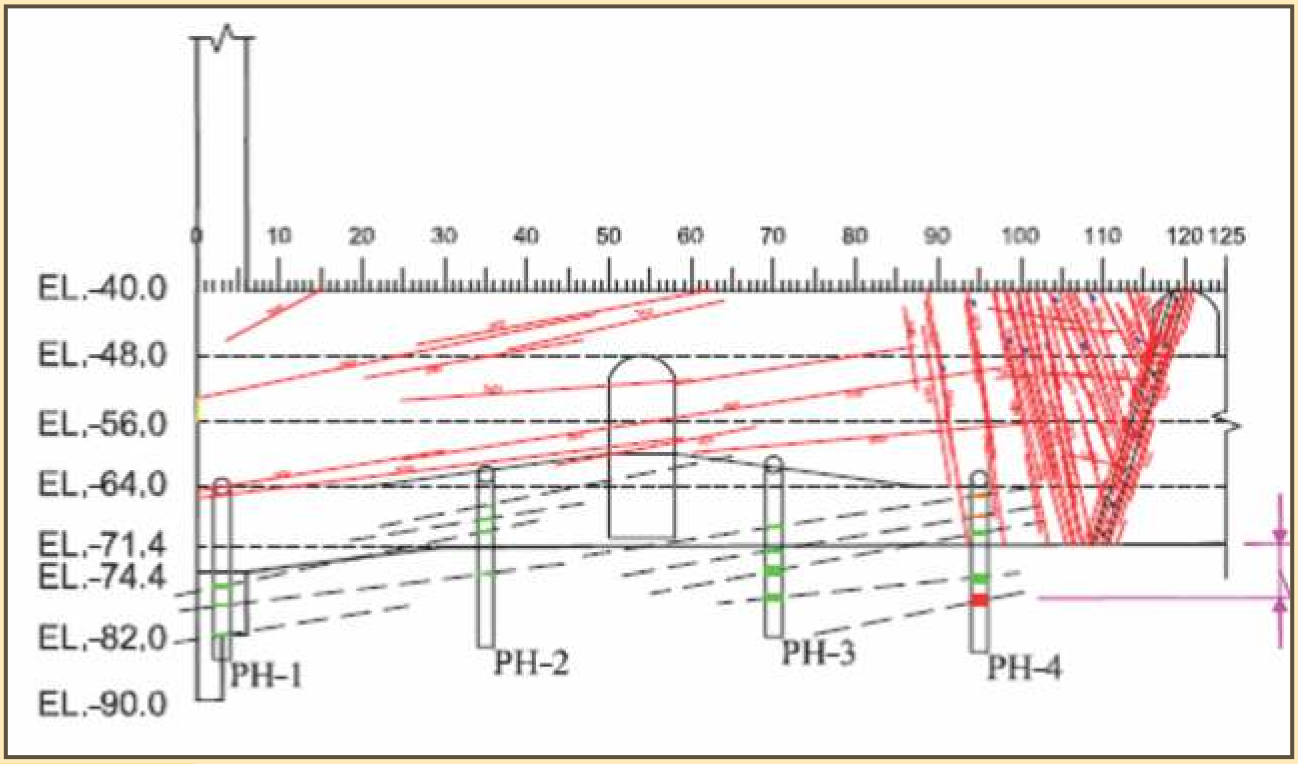


Figure 5.8: Plan and section of geological model of the area

During top heading cavern excavation, systematic probing was performed and the pre-grouting scheme was locally adopted for these joints and pre-grouted. During benches excavation, corner invert pre-grouting was performed which were found very effective at every stage of bench. Cement grout with OPC was performed as the joints were very open. All the horizontal joints from top to bottom were grouted in stages during cavern excavation and the seepage was slowly pushed to lower benches resulting in high water pressure of 6-7 bars at the cavern invert level. As the water was continuously being pushed due to grouting, water pressure in the horizontal joints below the cavern went on building up and posed a threat to the stability of invert of the cavern. Accordingly four vertical probe holes of about 21 m length at about 25 m spacing was performed from bench-2 invert of cavern covering about 15 m below the final invert of the cavern (see Figure 5.8). Series of sub-horizontal joints with pressure varying between 2-7 bars were encountered in these probe holes up to a depth of 14.0 m below the final cavern invert level.

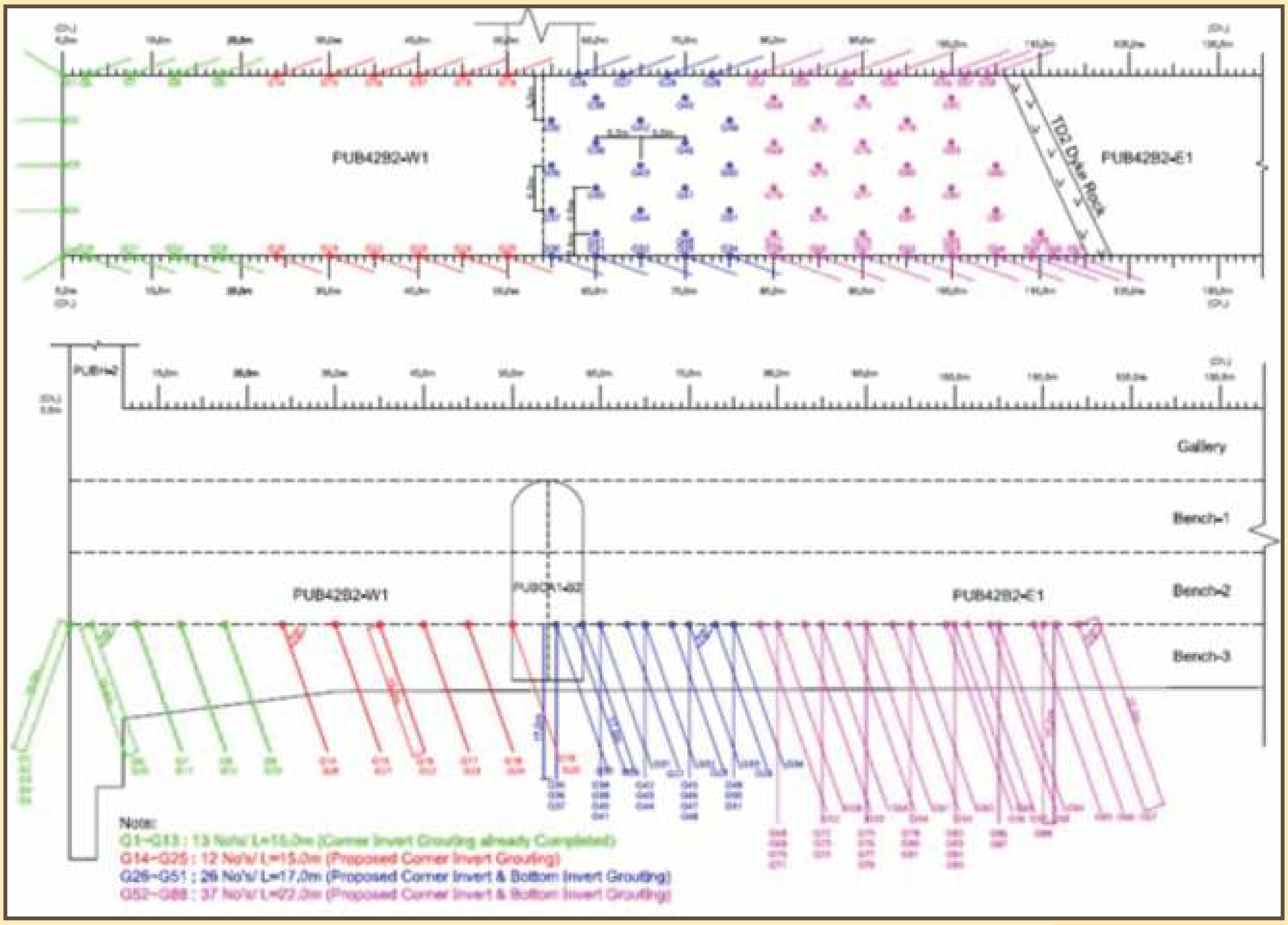


Figure 5.9: Plan and section of pre-grouting scheme adopted for the final cavern invert

Specific high pressure grouting scheme was applied to arrest large seepage coming from the horizontal joint sets below the cavern invert (see Figure 5.9). High seepage of more than 400 litre/minute was encountered in some of the probeholes.

Grouting of the rock mass was carried out using 50 mm holes at 5 m x 5 m grid on the invert of the cavern. The holes were drilled vertically extending 7 m-14 m below the cavern invert deepest being closer to TD2 dyke and 7 m in the pump pit side. At the corner invert, the holes are drilled at 70 degree with same lengths spreading beyond the cavern. Grouting was performed with OPC cement at pressures more than 10 bars negating the hydraulic pressure of about 7 bars. Grouting started from pump pit side and closed at D2 dyke side increasing the pressure and seepage towards TD2 side. After pre-grouting the final cavern invert was excavated. The seepage was measured across these sections using weirs after complete cavern excavation. The total seepage was reduced to 3 lpm in this section after grouting. Thus the grouting was successful and there is no cavern invert stability problem and the cavern is standing stable for more than a year now.

Pump pit grouting

Pump pits are the pits excavated inside the cavern at the lowest elevations in order to collect crude oil and seepage water before pumping them back to the surface through pipe lines in the shaft. These pump pits are narrow in section difficult to excavate and grout in case of huge seepage occurrence. The pump pit in southern most cavern unit exactly lies in the high pressure open sub-horizontal joints zone and further aggravates the construction problem. Pump pit in this storage cavern is extending 20m below the final cavern invert level and the sub-horizontal joints here are dipping westerly towards this pump pit and to the south wall. After excavation of final cavern invert is completed, three probe holes of 22 m were drilled from that level inside the pump pit location extending 5 metres below the final pump pit elevation. Huge water seepage of about 400 litre per minute with high pressure was observed through these probe holes.

The pressure measured in these holes was more than 8 bars which threatens the stability of pump pit considering the size of the pump pit. High pressure fan pre-grouting was performed from the top of the pump pit level surrounding the pump pit all along the walls to prevent high seepage and to counter the high water pressure. The pre-grout holes are extended 5 m below the pump pit final invert level to give further bottom cover. During pump pit excavation no seepage was encountered neither after completion of the cavens excavation, indicating a successful pre-grouting campaign.

DESIGN OF UNDERGROUND ROCK CAVERNS

A.Usmani, C.Kumar & A.Nanda

CHAPTER - 6

6.1 Introduction

Design of underground structures is not a new phenomenon in the field of rock engineering; however design of large unlined rock caverns extending to a length of few kilometers, passing through varying geological conditions and planned as robust structure for fifty years of its intended life is a complex process, yet to be fully established. Construction of large underground caverns requires in depth understanding of geo-materials and use of advanced instrumentation technologies. Most of the underground hydrocarbon storage projects are constructed as unlined rock caverns using the principle of hydraulic confinement. Long term structural stability of these caverns is main concern, calling for long term operational requirements to be combined with a robust design. Thus key of a stable underground structure design lies in accurate determination of geological and geotechnical parameters and active control of construction. Further due to hydrogeological containment requirements, these kind of underground projects have many restrictions on the extent of drilling/coring that is allowed during investigations and construction period, thus making them different from the other underground projects.

An underground storage project mainly comprises of caverns, access tunnels, water curtain tunnels with water curtain boreholes, shafts and pump pits etc. The caverns are excavated using drill & blast method using heading down benching approach. The top-heading are usually designed as 8 m in height while the subsequent bench height varies from 8 m to 10 m. Top headings are excavated using horizontal blasting approach while benches are excavated with either vertical blasting or horizontal blasting pattern. Storage galleries are connected at the upper and lower levels by connecting galleries. All connecting galleries between the two legs of a cavern unit are equipped with concrete separation walls in order to avoid the product by passing the defined routing. All connecting galleries from the cavern unit to access tunnels are equipped with concrete barriers to ensure tightness. Crude oil storage is designed to operate at vapour pressure of 1.3-1.5 bar. A pressure above atmospheric pressure is always maintained in the cavern, to eliminate leakage of air into the cavern. The cavern resists vacuum pressure and is also designed for accidental load case for an internal transient explosion of 1 MPa (10barg).

6.2 Geotechnical Parameters: Tests and Interpretation

Design methodology of underground structures is based on parameters evaluated by geological, geophysical, geotechnical, and hydrogeological studies that involve mapping of outcrops, geological mapping through vertical and inclined core drilling, laboratory testing, in-situ stress measurements, geophysical surveys, and hydrogeological tests. Different levels of investigations planned during such type of an underground storage project are detailed in Chapter 3 "Site Investigations". The basic geological and geotechnical model of the project are prepared based on the investigation in feasibility stage which is continuously updated.

Geotechnical design parameters values are established based on the various in-situ and laboratory rock mechanical studies along with correlation with recorded Rock Quality Designation, Q values. Different parameters estimated from investigations include unconfined compressive strength, density, cohesion, angle of internal friction, deformation modulus and in-situ stresses. It is also noted that since water curtain system comprises of numerous water curtain boreholes drilled in water curtain tunnel before start of cavern excavation, it provides to be a major source of rock mass information. Water pressure

tests are conducted in all water curtain boreholes and the water bearing zones are delineated in the hydrogeological model.

6.3 Basic Design of Underground Works

The design philosophy for underground works follows an observational approach, i.e. the original design assumptions are continuously updated by means of the results obtained from geological mapping and geotechnical monitoring during construction, to allow for an optimized and safe design. When an excavation is carried out underground, rock loads are activated from the roof, sides and bottom with different magnitudes. The magnitude of these loads depends on rock type, joint system, in-situ stresses and method of excavation. Various empirical methods exist in the literature to determine these load magnitudes viz. Terzaghi (1946), Barton et al. (1974) and Bieniawski (1984). Once the loads coming on the rock are established, analysis and design of the support system is carried out for given field conditions.

Analysis and design methodologies in rock engineering can be divided into four broad classes; viz observational approach, empirical methods, analytical methods and lastly numerical methods. The term observational method is coined by Terzaghi (1948) and provides a "learn as you go" alternative. Empirical methods such as RMR, GSI and Q system find major place in rock engineering as they are mainly based on previous experience as derived from construction of structures on rock of similar characteristics, Analytical methods like rock-support interaction curves and limit equilibrium methods are based on the assumptions of continuous, homogenous, isotropic and linear elastic behaviour of rock mass. Numerical methods now days find a lot of utility among designers due to availability of many commercial software and availability of high speed computers. Numerical methods help engineers to gain understanding of governing deformations and failure mechanisms of a structure while exploring different alternatives rather than making absolute predictions about its behaviour. These are very helpful when analytical solutions are not available and parametric response of the rock structure needs to be analysed with changes in input data. In addition to all these methods described above, analysis of kinematic admissibility of potential wedges or planes intersecting the excavation faces, carried out using kinematic analysis is also very significant in case of underground structures. The final design of the caverns is worked out taking into account combination of all these approaches.

The layout of underground rock cavern is finalized considering the results of geological investigation, the stability of neighborhood facilities and the storage capacity that the volume of crude oil may be accommodated along with operational requirements. Rock mass classification systems constitute an integral part of empirical tunnel design. They are traditionally used to group areas of similar geomechanical characteristics, to provide guidelines of stability performance and to select appropriate support. The primary objective of all classification systems is to quantify the intrinsic properties of the rock mass based on past experience.

Despite a plethora of empirical classification systems, two systems are most commonly used for mine and tunnel design. The first system is the Norwegian Geotechnical Institute's Q system by Barton et al. (1974) and the Rock Mass Rating System (RMR), proposed by Bieniawski (1973). The RMR and Q systems have evolved over time to better reflect the perceived influence of various rock mass factors on excavation stability.

The support philosophy of underground rock caverns is based on empirical rock support design, which is derived from Q-system of rock mass classifications along with wedge stability analysis and finite element modelling for stress-deformation studies. Barton et al (1974) of the Norwegian Geotechnical Institute proposed a Tunnelling Quality Index (Q) (based on evaluation of a large number of case histories of underground excavations) for determination of rock mass characteristics and tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000.

INSIGHTS

Norwegian Tunnel Quality Index Q

Q is defined as

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

RQD is defined as Rock Quality Designation, Jn is the joint set number, Jr is the joint roughness number, Ja is the joint alteration number, Jw is the joint water reduction factor and SRF is the stress reduction factor The first quotient (RQD/Jn), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400.

The second quotient (Jr /Ja) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to tunnel stability. When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly, although rock wall contact after small shear displacements have occurred become very important factor for preserving the excavation from ultimate failure.Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability. The friction angles are little lower than the residual strength values for most clays, and are possibly down-graded by the fact that these clay bands or fillings may tend to consolidate during shear, at least if normal consolidation or if softening and swelling has occurred.

The third quotient (Jw/SRF) consists of two stress parameters. SRF is a measure of: i) loosening load in the case of an excavation through shear zones and clay bearing rock, ii) rock stress in competent rock, and iii) squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter. The parameter Jw is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible outwash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient (Jw/SRF) is a complicated empirical factor that describes the 'active stress' condition. Numerical values of these parameters are presented in **Annexure III**.

Sometimes it is difficult to relate RQD to other measurements of jointing, as RQD is a one-dimensional, averaged measurement based solely on core pieces larger than 10 cm. Therefore many simulations using blocks of the same size and shape penetrated by a line (i.e. borehole) at different angles have been used for such estimations. The first such attempt was made by Palmstrom (1974) when the volumetric joint count (Jv) was introduced to estimate rock quality estimations.

INSIGHTS

Relation of RQD Vs Jv

The following, simple expression between RQD and Jv was then used to estimate rock quality designation of the rock face:

RQD = 115-3.3 Jv

(RQD = 0 for Jv > 35, and RQD = 100 for Jv < 4.5)

where Jv is the sum of the number of joints per unit length for all joint (discontinuity) sets known as volumetric joint set.

This expression was included in the introduction of the Q system by Barton et al. (1974). It is observed that correlation between RQD and Jv is rather poor, especially, where many of the core pieces have lengths around 0.1m. However, when Jv is the only joint data available (no borehole or scan line logging), above relation has been found to be an alternative transition for finding RQD from Jv, where, for instance, RQD is required in the Q and the RMR classification systems.

INSIGHTS

Excavation Support Ratio (ESR):

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton et al (1974) defined an additional parameter which they called the Equivalent Dimension, De, 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: De = 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. Barton et al (1974) suggest the following values for ESR:

Table 6.1: Values of ESR for different excavation utilities

Division	Excavation category	ESR
A	Temporary mine openings	3-5
B	Vertical shafts with circular sections rectangular and square section	 2.5 2.0
C	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations	1.6
D	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
E	Power stations, major road and railway tunnels, civil defense chambers, portal intersections.	1.0
F	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Based on the equivalent dimension of the excavation, De and Q value of the rock mass, initial support requirements of the cavern or tunnel is evaluated. Typical tabular representation of support requirements for an underground rock cavern is shown below in Table 6.2.

Table 6.2: Typical Q-system reinforcement pattern for cavern

INSIGHTS

Rock classes	I	II	III	IV	V
Rock mass quality Q	40≤100	10≤40	4≤10	1≤4	0.1≤1
Reinforcement category	2)	3), 4)	5)	6)	7)
Shotcrete Type	Unreinforced shotcrete(S)	Unreinforced shotcrete(S)	Fibre reinforced shotcrete(Sfr)	Fibre reinforced shotcrete(Sfr)	Fibre reinforced shotcrete(Sfr)
Thickness, t	4 cm	4-5 cm	5-9 cm	9-12 cm	12-25 cm
Rock Bolt Type Systematic bolting (B)	Spot Bolting (sb)	Systematic bolting (B)	Systematic bolting (B)	Systematic bolting (B)	Systematic bolting (B)
Spacing, S	3.0~4.0m	2.3~2.5m	2.1~2.3m	1.7~2.1m	1.3~1.7m
Length, L	5.0m ~ 7.0m	5.0m ~ 7.0m	5.0m ~ 7.0m	5.0m ~ 7.0m	5.0m ~ 7.0m
Reinforced ribs of	X				
Cast concrete lining (CCA)	X				

INSIGHTS

Rock Bolt Calculation

The length (L) of rock bolts is also verified from the excavation width (B) and the excavation support ratio ESR for a particular tunnel or cavern tunnel or cavern.

$$\text{Bolt length (L)} = (2 + 0.15B)/ESR \qquad \text{(Barton et al, 1980)}$$

For a typical 31 m high cavern, rock bolt length can be estimated as

$$= (2 + 0.15 \times 31.0)/1.3 = 5.12\ m \approx 6.0\ m$$

Based on the review of above two set of support methodologies, final length of rock bolt length is decided. For all classes of rock type (I-V), fiber reinforced shotcrete is applied to the entire length of walls, as proposed in Q-system. The reinforcement standards proposed by this approach are typical to any tunnel or cavern, which are applied mapping a particular section, calculating Q value, and deciding the reinforcement pattern accordingly. In case of support requirement at intersections of tunnels and cavern, a minimum extent of intersection area is considered as 5 m from intersected point. Further Q-value at the intersection is calculated using 3.0 x Jn (Joint Set Number) on the intersection zone. The length (L) of rock bolts in the intersection part is estimated from the diagonal span (B) and the excavation support ratio ESR as discussed earlier.

Design of underground structures necessitates evaluation of structural safety of cavern or a tunnel according to various analyses based on initial investigation and subsequent construction data along with adequate methods of classifying rock mass to rationally perform design and construction of tunnels or underground cavern. Thus stress distribution around an excavation is checked by conducting stress-deformation analysis of rock mass for different tunnel sections taking into account effects of reinforcement on the rock mass. This aids in deciding type of reinforcement and adequacy of excavation methods before commencing actual excavation through interpretation of various physical properties of rock mass using different numerical analysis tools. Selected locations in tunnels or caverns, where size and geology of rock mass varies are analyzed for stress-deformation behavior under full loading conditions.

Analyses are generally carried out considering different aspects of rock mass behaviour, both during construction and in the long term over the entire service life, together with the construction sequence and processes. The support philosophy of all underground openings within the project is based on staged excavation and incremental installation of rock support measures. The benefits of progressive support enhancement are explicitly considered in developing a cost effective and practical construction sequence.

On the basis of the results of geotechnical investigations, various engineering properties of the rock mass like Rock Tunnelling Quality Index (Q index), RMR, UCS, modulus of deformation, cohesion, angle of internal friction and Hoek-Brown parameters (m,s,a) are first interpreted based on the results of the field and laboratory test data and literature review of experience of similar projects in the past. After a reasonable assessment of these properties along with engineering judgment of the designer, various numerical analyses using different finite element software are carried out. The magnitude and direction of the maximum and minimum in-situ stress in the field are obtained from the hydraulic fracturing test. In order to cater for the variation of horizontal in-situ stresses, different K-ratios are used to define both initial in-situ stress conditions as well as boundary conditions during the calculation. RocLab (Rocscience Inc.Toronto) software from Rocscience is used to estimate rock Hoek-Brown paramaters for relevant rock mass parameters. For stability analysis either Mohr-Coulomb's or Hoek-Brown's failure criterion, as appropriate shall be used.

The design of the excavation-support system for the tunnels/caverns are based on the fundamentals that rock mass is the main bearing element of the structures and rock mass and the excavation-support system form a composite structure. Failure mechanisms in rock are normally of two types viz; structurally induced instabilities and instabilities caused due to overstressing of the excavation boundary. In case of underground caverns two types of analysis approaches viz. continuum mass and discontinum mass are considered. In case of continuous approach rockmass around an opening is assumed to be homogenous and isotropic as a simplified assumption and analyses ranging from elastic to elasto-

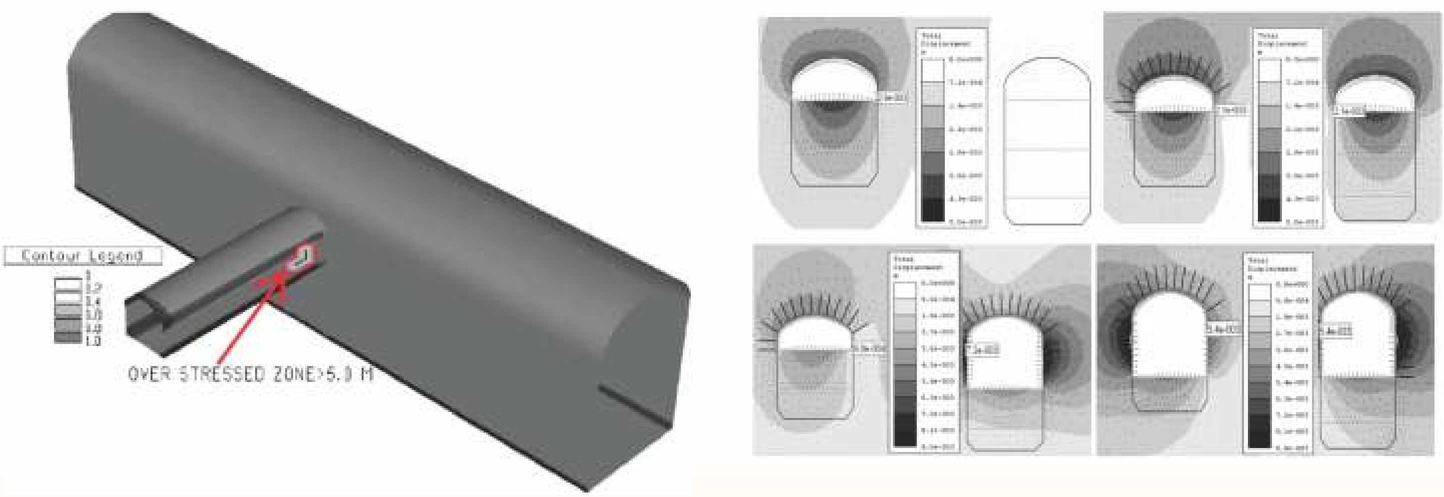


Figure 6.1: Different types of 2D and 3D analysis for tunnels and caverns

A Navratna Company

carried out. These kinds of analyses are carried out on different commercial software packages available like Phase 2.0, MIDAS etc for two dimensional and Examine 3D etc (Figure 6.1). for three dimensional. These analyses provide the first hand idea of the stress-deformation behaviour of the rock mass around an opening.

INSIGHTS

Resolution of Stresses along the Excavation

Directions of the principal stress and the tunnel axis can be applied directly when stability analyses for storage cavern are performed in three dimensional methods; however, if those analyses are carried out by two dimensional methods, the maximum and minimum horizontal principal stresses are recalculated considering directions of the face and tunnel axis.

In that case, following axis transformation methods are used. If the degree between the tunnel axis and the maximum horizontal principal stress is Θ, then stress component of tunnel axis direction, Saxi, and the stress component of tunnel face direction, Sec may be obtained from the following equations derived from basic elastic theory.

$$Saxi = S_H \times \cos 2\Theta + S_H \times \sin 2\Theta$$

$$Ssec = S_H \times \sin 2\Theta + S_H \times \cos 2\Theta$$

By using above equations, the results of axis transformation on the maximum and minimum horizontal principal stresses (S_H, S_h) measured from the investigation holes are arranged. Since boundary effects are very important in case of finite element analysis, so generally boundary conditions are taken as 6 times larger than tunnel (3 times that of tunnel diameters) according to Kulhawy's theory (1974). Actual construction sequences are vital part of any numerical modeling exercise and should be simulated closed to real conditions. Especially, the processes of excavations and installations of reinforcements are major variables affecting tunnel constructions thus it should be rationally simplified and used for input data in numerical analysis.

INSIGHTS

Numerical Modeling of Caverns

Generally excavation stages for modeling are defined in terms of initial stress on original rock mass followed by different excavation stages for heading and benches of cavern gallery. This is followed by installation of shotcrete & rock bolt as per the original excavation sequence. The Hoek-Brown empirical failure criteria (Hoek and Brown 1980) have been widely used for determination of rock-mass parameters from intact rock properties. Some other well-known empirical approaches for rock mass behavior have also been provided by Bieniawski (1978), Barton et al. (1980). It is observed that since 3D discontinuum modeling of cavern stability is more complex (Yazdani et al. 2012) and time-consuming, 2D continuum modelling using finite element software provides much simple and quick results that are easy to analyze.

During cavern construction, some relaxation of the rock mass occurs prior to the installation of ground support. In order to simulate this relaxation, two modelling stages are generally implemented for excavation increment. The relaxation is based on the concept of allowable tunnel deformation prior to support installation as reported by many researchers (e.g., Ghee et al. 2006) and can vary from one problem to other.

Rock support during numerical analysis is applied in terms of fully grouted rock bolts (applied normal to the excavated boundary face) of 25 mm diameter and minimum yield strength of 500-550 MPa along with fiber-reinforced shotcrete of minimum compressive strength of 40 MPa and shear strength of 1 MPa positioned as per actual design requirements. Stability of a particular design is verified by analysing stress concentration and deformation vectors along excavation boundaries. The absolute displacement values are also verified with respect to permissible displacements limits prescribed for various cavern sizes in the literature.

Further in order to consider the effects of joints on the excavated faces, discontinum modelling is carried out using commercial softwares. In addition to stress-deformation analysis, stability of underground openings is also verified in terms of formation of triangular wedges from intersection of different joint sets. This is usually carried out using commercial software Unwedge, which based on the steronet projections, determines the wedge geometry, forces acting on a wedge, sliding direction of the wedge, normal forces on each wedge plane, resisting forces due to joint shear strength and finally factor of safety. Based on these analyses, if necessary, rock bolt length, bolting patterns and shotcrete thickness as suggested by the Q-system are re-adjusted.

In addition to stress deformation studies using finite element modelling, stability of possible wedges (Figure 6.2) is also verified based on stereographic representation of various joint sets around an excavated opening. Wedge stability is first analysed with no rock support conditions and subsequently under full designed support consisting of grouted rockbolts and shotcrete. Major and minor intersections of cavern-tunnels are analysed separately using three dimensional approaches and intersection reinforcements are finalised as per the extent of yield zones around excavations.

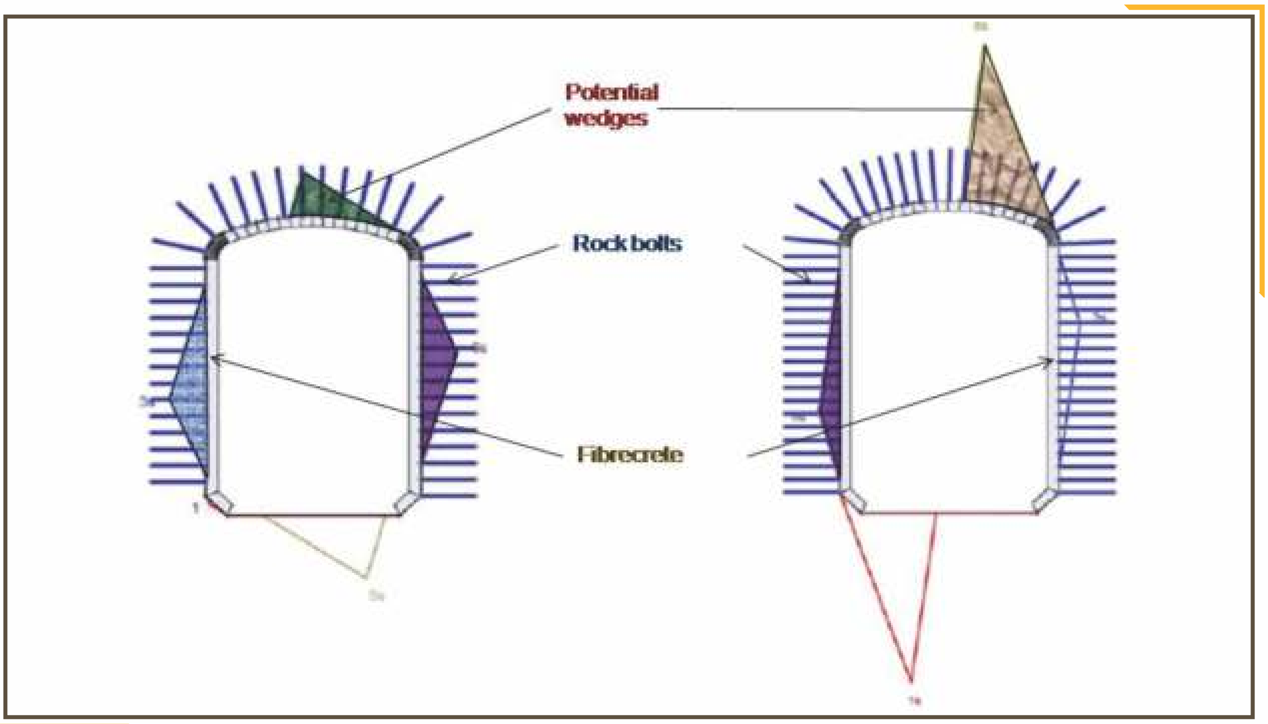


Figure 6.2: Unwedge analysis for cavern

Lastly, the interface between design and construction processes should be given special attention with the objective to achieve consensus between the two parties of the project organization. An active input of constructability aspects into the design process is considered as a crucial factor for a speedy construction.

6.4 Geotechnical Monitoring and Instrumentation

When an underground opening is excavated into a natural undisturbed rock mass, the stresses near the opening are disturbed and redistributed. Thus, considering the dynamic nature of underground construction and rock-mass behavior, timing (of the installation of the monitoring targets) and interpretation (of recorded data) are vital for understanding changing stability requirements of the excavated structures.

During the preliminary design stage, it is necessary to make simplifying assumptions for the complex, in situ rock-mass characteristics. These simplifications always introduce sources of uncertainties and eventually potential-failure probabilities into the design. The inherent uncertainty associated with excavated rock caverns makes it necessary to regularly monitor their performance to verify the overall stability, or otherwise, relevant areas classified as geologically and/or geotechnically sensitive. Sources of uncertainty in these structures include inadequate detail of the rock mass (variable time-dependent behavior of rock masses, presence or intrusion of groundwater, variation in geology within walls), human error (observation, computation testing errors), and operational mining variation from the design (overcutting of faces), etc. Monitoring of the cavern structures therefore becomes an important tool for locating any potential failures in the ground before the unstable rockmass becomes hazardous and thus provides an opportunity to look again at the initial design parameters to redesign/optimize, if required, to balance the excavation stages. This review also helps to provide necessary measures for managing the potentially hazardous zones and thus lead to a stable design configuration. Well-designed monitoring programs can also help to differentiate among normal elastic movements, inconsequential dilation, and incipient cavern wall failures. Excavation methodologies on the construction site for underground rock caverns are based on stage excavation. Progressive support optimization, based on actual behavior of the excavated structure, is an essential part of underground engineering, leading to important time- and cost-effective support solutions. This can be done by analyzing stress-deformation response of the caverns during excavation. In case of underground rock caverns, displacement monitoring is carried out using two types of instruments: (1) optical sensors and (2) a set of grouted multipoint borehole

INSIGHTS

Geotechnical Monitoring Scheme

Extensometers can be installed either from water curtain tunnel towards cavern side or directly from the cavern walls and roof. In order to study effects of heading excavation on water curtain tunnels, extensometers are drilled down from water curtain tunnel towards cavern. However in some projects effects of only cavern heading and benches are studied and thus extensometers are placed directly through the cavern surface.

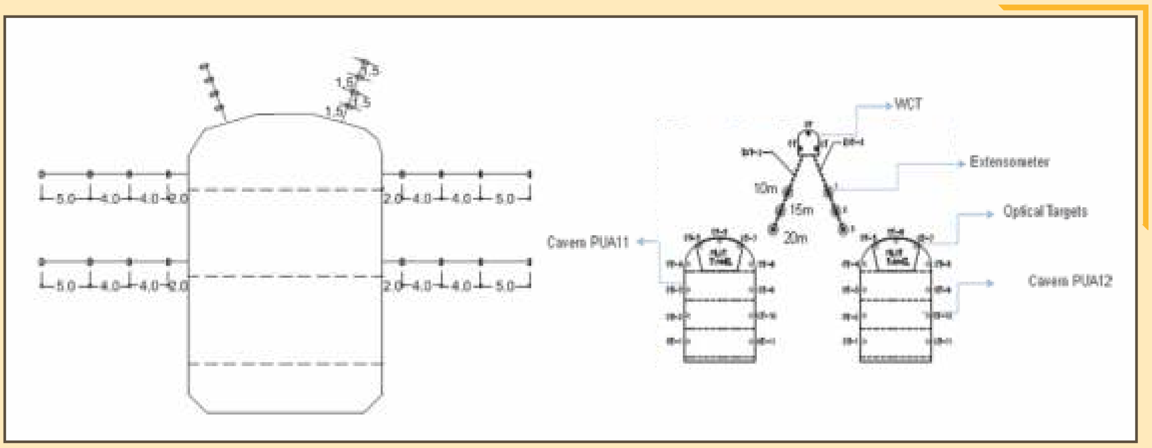


Figure 6.3: Scheme of extensometer and optical targets for a cavern

In underground projects, at a particular section in a cavern, five to six sets of grouted extensometers are installed with two in the top heading and three to four each on the left and right walls (both sides). Maximum extents of rock-mass covered using extensometers vary from 5-6 m from crown and 12-15 m in walls. The extent of coverage of extensometers varies from site to site and is planned so it should not lead to source of additional leakage. The displacements are measured using a vibrating-wire transducer, which is connected to a steel rod installed in the rock. Extensometers are drilled through 100-mm holes. Each extensometer rod is grouted where the rod end is located in rock to develop strong bonding between the rod and surrounding rock mass and thus minimize the chances of slippage between extensometer rod and surrounding rocks The deformation monitoring of excavation surfaces of cavern using 3D measurement of optical targets in x-y and z direction is other main geotechnical monitoring method in order to assess the rock mass behavior during excavation. Optical targets are fixed to reference points through bolting to the rock surface. The bolts are installed through a drill hole of diameter 25 mm upto a depth of 220 mm from rock surface. Monitoring section within the caverns are typically arranged at every 25.0 m of interval with five targets within the top heading (one in crown and two at each side) and 4 targets along each side wall as shown in Figure 6.4. However, additional section of optical targets is installed based on observed geological features in updated geological map.

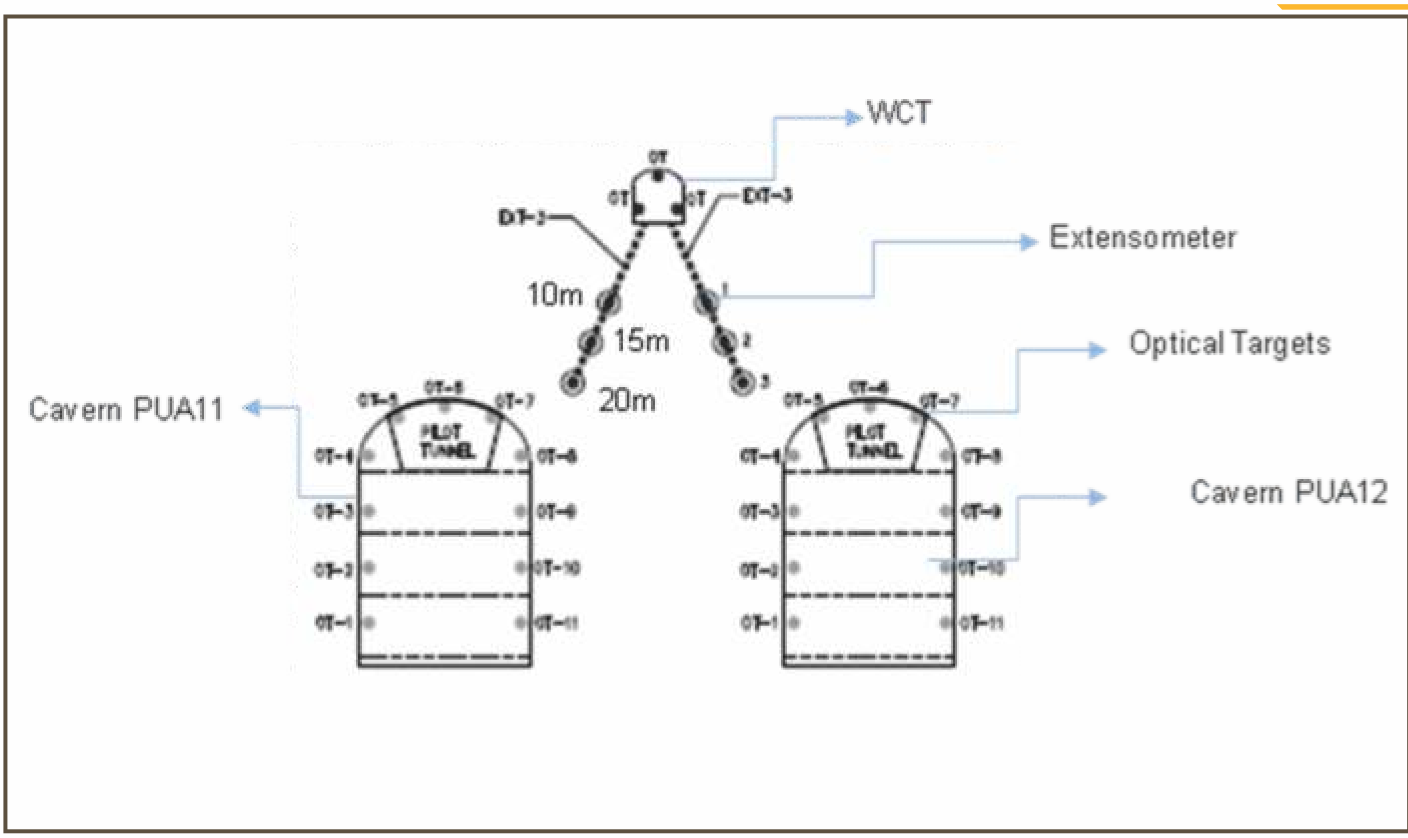


Figure 6.4: Extensometer placement (through water curtain) for cavern monitoring.

Case Study 6.1: Realignment of an inferred Dyke

One of the major challenge observed during the design of cavern during construction was change in the position of dyke TD5 (Refer Chapter 4 (Case Study 4.2) for details). The fractured dyke was about 30 m thick and in general perpendicular to the cavern alignment but it offsets suddenly about 50 m from the original orientation and encountered in one of the caverns in parallel orientation for about 50 m and dips steeply towards the wall of the cavern and possesses stability problems in the cavern wall. The probe hole drilled inside the cavern wall shows that these dykes are extended up to a distance of 10 m behind the cavern wall. Borehole Televiewer (BHTV) was also conducted in the water curtain boreholes drilled above the cavern to study the geometry of the shifted dyke in the intermediate cavern pillar. Special designs were performed and additional supports were recommended to ascertain stability of cavern from wedge sliding as well as wall stability.

i Initial Design

The fractured mafic dyke TD5 was encountered in a corehole and was studied in detail during investigation stage itself. Its orientation, thickness, geological, geotechnical and hydro-geological properties were well understood. It was classified as class IV rock as per Q system (Barton et al, 1974). The layout of storage cavern was chosen (along with all other relevant geotechnical parameters) such that this dyke cut across almost perpendicular to all storage caverns. Accordingly, rock supports were designed for this 30 m thick mafic dyke zone after detailed numerical analysis & unwedge analysis.

The construction sequence for the upper water curtain tunnel was planned with probe holes ahead of excavating this dyke. The dyke zone was classified as class IV rock after excavation. As the dyke intersected water curtain tunnels (as expected) obliquely (almost perpendicularly) and with very little ground water seepage, it was not considered a big challenge geotechnically. Rock support in form of 100 mm shortcrete with 5 m rockbolts and 1.5 – 1.75 m spacing centre to centre was applied to stabilize this dyke zone.

ii Design challenges:

The dyke offsets about 50 m from the original orientation and encountered in the shifted position in one of the caverns. But their orientation was same. The shifting could have taken place in the rock pillar between the two adjacent storage caverns. To confirm this, 3 water curtain boreholes drilled in this rock pillar just above the observed shifted dyke was examined with BHTV. It revealed that the shifting in dyke does occur in the rock pillar and the orientation of dykes, which were split in to numerous dyke swarms, were found to be both perpendicular as well as parallel to the cavern alignment. The parallel orientation of the dyke swarms along with its steep dip towards north wall in one of the caverns was found to be critical. The fracture zone FR4 found along either direction of the TD5 dyke seems unchanged in orientation and got overlapped with parallel alignment of dyke in this zone. This zone in north wall of cavern was physically found to be fractured and crushed with silty clay infilling and was classified as rock class V where as in south wall of the cavern, no shifting in dyke was found and the dyke remains perpendicular to the cavern wall and is classified as class IV.

A close study on the northwall geology and BHTV studies suggest that the offset of the dyke is ductile and at the offset, it locally splits into number of small dykes (dyke swarms) and some of them run parallel to the cavern orientation for about 50m and some of them runs perpendicular to the cavern same as original dyke but overlapped with similarly oriented fracture zone. To ascertain the extent of branched out dyke passing parallel to the cavern, 4 nos. of probe holes of about 15 m length were driven. The probe hole result shows that these dykes are extended upto a distance of 10 m behind the dyke. However, the persistence, thickness and orientation of each branched dyke rock were still unknown. In addition, these branched dyke rocks were highly irregular and discontinuous.

iii Design verification during excavation

With the shifted orientation of dyke in north wall of cavern, stability of wedge analysis was performed. This wedge analysis result shows that due to shift in orientation, the wall wedge weight and apex height has been increased. Also as the rock mass around this zone is highly fractured, closely jointed and of complex geology, numerical analysis using Phase2 software was carried out to access the north wall stability with assumed rock support. The analysis models were developed from respective surrounding rock mass and geometry of mafic dyke of about 10m thick placed on the north wall of cavern with the properties of dyke varying from poor to very poor. Results of the FEM analysis show that Stress level observed around the cavern are moderate and stable. Yielded zone varied between 2-3m and is restricted close to central part of north wall of cavern. Limited yielding of rock bolts was noted. Maximum displacement observed during excavation of bench 2 was estimated between 4-8 mm. Overall displacement is relatively small and acceptable. Strength factor observed between the pillars is around 2 and the shear stress is very close to zero. This state ensures the stability of pillar between two caverns.

The results of the analyses both from unwedge & FEM indicate that there is no cause for concern for wedge sliding as well as wall stability. However, due to uncertainties based on the fact that the branched dykes were highly irregular and their persistence, orientation, etc were still unknown, one row of 12 m rock bolt (25 Φ) was recommended at both bench 2 and 3 level, which will cross over the dyke zone of about 10m. In addition shotcrete thickness of 100 mm was proposed on north wall. To check the integrity of applied recommended systematic rock bolt, pull out test with a load 1.5 times the design load i.e. 20 MT was performed on 5 rock bolts of 5 m in length on north side of wall. The result of the pull out test show a maximum deformation of 4.3 mm at ultimate load and none of the rock bolts failed at this load.

iv Geotechnical Monitoring

The readings taken from two borehole extensometers installed from water curtain tunnel located above the cavern at this zone were observed during the heading excavation. These extensometers are anchored at 4 points. The result from extensometer shows a deformation of about 1mm at crown, which stipulates the stable condition as there is no significant deformation/movement of the rockmass at the crown region during and after the excavation. In addition, before start of bench 2 excavation, one extensometer (3-point anchor) each on north & south wall were installed in this zone to observe the deformation in wall. A deformation of about 3 mm was observed which is well within the acceptable limit.

A Navratna Company

Case Study 6.2 : Wedge failure & Remedial Measures

This section describes a large wedge failure which occurred (Figure 6.5) in the Northern side wall of the central storage cavern VUA1 located in the east coast of India, approximately between chainage 360 m and Ch. 440 m during excavation of the last bench (see chapter 4 for details). The wedge failure extends over approximately 80 m in length and is upto 20m high. Additional investigations were carried out in order to study the actual geological conditions within this area of special concern. Subsequently, special case rock support assessment was performed in order to determine rock support for this area. However, results of geological mapping, additional investigation drillings in the pillar between VUA1 and VUA2 as well as inspections during the excavation of the failure rock mass have indicated that there is sheared material of substantial thickness present behind apparent failure plane.

Figure 6.5: Pictorial view of the cavern failure

Therefore although the failure occurred along a distinct shear plane, rock support needs to be checked and verified for a case of a shear zone rather than a shear plane only. Owing to the actual nature and geometry of the sheared material, detailed design assessments in terms of geometry, rock support, excavation method and sequence were performed within the wedge failure zone and further beyond towards the higher chainages.

i **Special Design Considerations Post failure**

The FEM analysis was carried out with the software Phase2.0. The detailed analysis of the required additional rock support in the sidewall of the VUA1 Bench 2 and the redesign of the rock support in the VUA1 Bench 3 was carried out considering the geotechnical assessments. The analysis models were developed from the respective rock mass and geometry of discontinuities, which were simulated as distinctive shear zones derived from geotechnical information. Groundwater recharge through the water curtain tunnels had also been considered in the analyses.

Stress levels around the cavern, and in particular above the failure zone, were moderate and acceptable. As expected, relaxation occurs around the failure zone in the rock mass close to the side wall. Stress concentration occurs in the north wall at the bottom of the wedge geometry indicating that rock bolts should safely extend beyond this zone.

Significant yielding of existing rock bolts occurred which required recommendation of additional rock support. The overall total displacements were relatively small and average in the range of about 25 mm. However near the failure zone, the overall total displacements are in the range of 45 mm and about 90 mm along the exposed

failure plane, which was mostly attributed to relaxation. A typical geotechnical model near the failure zone and further beyond with some key results with respect to stress around the cavern are shown in Figure 6.6. The results showed that along the side wall, deformations are concentrated along the northern side wall over the height of potential wedges. In addition, the toppling wedge situation along the southern side wall indicates increased rock mass deformations. Therefore, additional rock support was required in this area of concern for both the northern as well as the southern side wall.

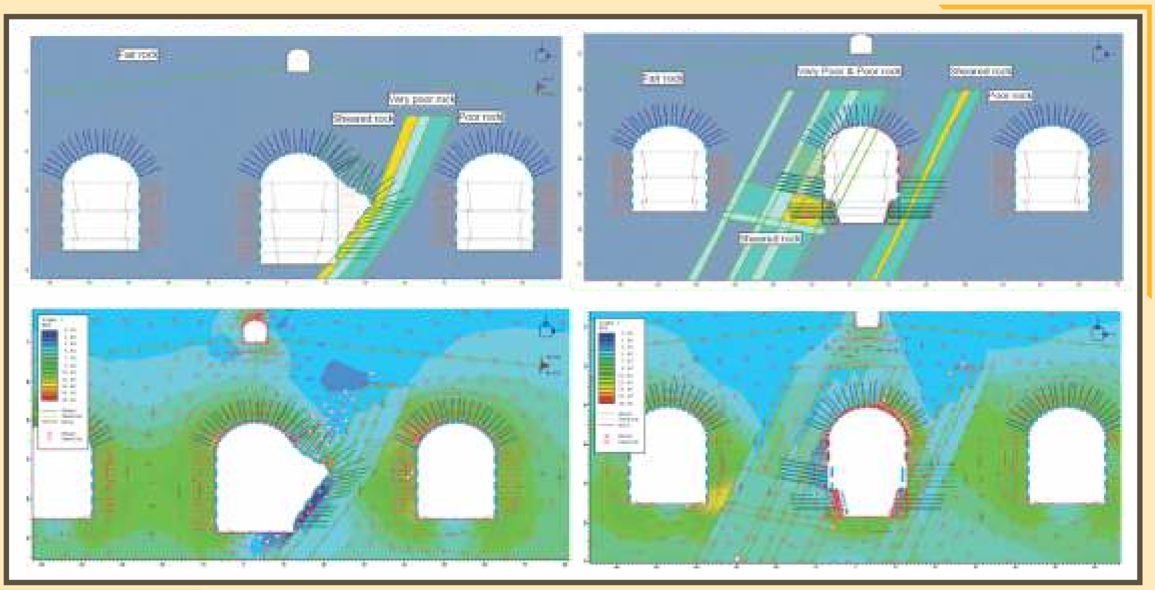


Figure 6.6: Finite element model phase2 & stress concentration around cavern

Proposed excavation sequence i.e. leaving part of bench 3 side slashing behind on both side walls to be excavated in two stages in the remaining portion of area post failure and recommended additional rock support are shown in a typical sketch in Figure 6.7(a & b). After reviewing geological data within the area immediately after wedge failure, some initial installation of additional rock support was recommended in following steps as:

i. Perform scaling of any loose material and clean the surface of dirt and laitance

ii. Apply the first layer of plain shotcrete of 50 mm.

iii. Install the rock bolts of diameter 32 mm and length varying from 8 to 12 m.

iv. Apply the first layer of wire mesh to the surface.

v. Apply the second layer of plain shotcrete of 100mm thickness.

vi. Apply the second layer of wire mesh to the surface.

vii. Apply the final layer of plain shotcrete of 50mm thickness.

viii. If there is a shotcrete berm at the location, then apply an additional layer of 75 mm thickness of shotcrete.

ix. Apply the third layer of wire mesh strip for the berm location.

x. Apply the final layer of plain shotcrete of 75 mm thickness to create the shotcrete berm.

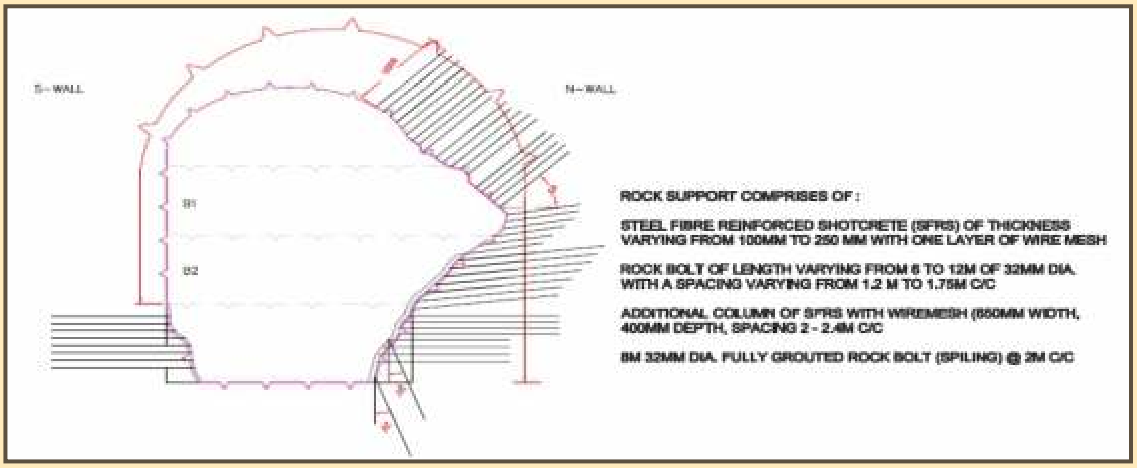


Figure 6.7(a): Typical rock support for part excavation of Bench 3

In additional to the support installed immediately after wedge failure, additional rock support was recommended after detailed rock support assessment in the form of :

i. Rock bolt L=12 m, diameter 32 mm, spacing 1.5 m c/c, inclined upwards,

ii. Base layer shotcrete top-up to 250 mm with plain shotcrete and one layer wire mesh (T4-150 mm),

iii. Additional columns of SFRS (650 mm width, 400 mm depth, spacing 2.4 m c/c).

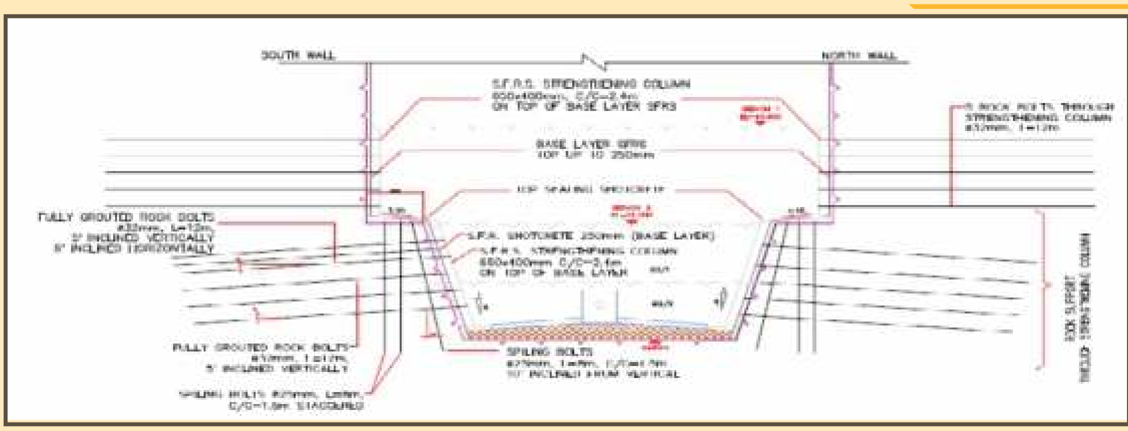


Figure 6.7(b): Typical rock support for part excavation of bench 3

ii Geotechnical Monitoring

All optical 3D monitoring sections were re-activated within this stretch of VUA1. All targets within 50 m on either side of a bench excavation were to be read on a daily basis. Before bench 3 excavation, eight additional three point rod extensometers (5/12/20 m) on the north wall and four on the south wall were recommended. For Bench 3 excavation works within cavern VUA1, 3D monitoring including the extensometer readings were to be carried out on a daily basis. A deformation of about 10 mm was observed which is well within the acceptable limit.

Case Study 6.3: Portal Slope

This case study deals with the analysis and design of slopes for the portal of an underground crude oil storage cavern site. The site selected for the slope study is characterized by residual soils and granitic rock formations, located in the south-western part of India. It is observed that in tropical residual soils, most hill slope failures are caused by rainfall and thus it is important to consider hydrological conditions when attempting to analyze the stability of slopes in such material. Combinations of shallow slopes with lower overburden and high steep hill slope with large overburden were seen at the site along with varying combinations of lateritic soils and weathered rock formations.

In the present study the portal for a crude oil storage facility consists of two parallel tunnels; main access tunnel and water curtain access tunnel starting from main portal entrance as shown in Figure 6.8. These tunnels serve as the main access for construction of main storage galleries situated deep inside for the storage of crude oil.

Two phased stage investigation programme was carried out at portal location to minimize uncertainties while planning for stabilization of the slopes all along the portal. The first phase of the investigation was part of the overall investigations carried out for the storage site and was mainly confined to investigations using core recovery and destructive hole drilling at the portal location to ascertain soil, weathered rock and bedrock cover thicknesses along with their interfaces. The second phase of the investigation was specific to slope area and determined shear strength properties of the laterites and weathered rock.

The coreholes examination revealed that bedrock is mainly composed of granite gneiss which is covered with top soil that consists of anthropogenic soil cover, laterite, residual soil and weathered rock with thickness varying from 5 to 20 m. The average RQD value for the bedrock was found to vary from 71 to 87%. Standard penetration tests were also carried out in soil part of the cored holes to evaluate their penetration resistance. Standard penetration resistance of upper soil layer (laterite) was found to be of N value of 34; corresponding to very stiff to very hard categories while for weathered rock test could not be completed since number of blows was recorded to be greater than 50 for 10 cm penetration. However boundary between rock and soil interface (1–2 m section) was inferred to be consisting of weaker zones of silty clayey material having highly permeable layers. It was observed at site that tunnel portal is located in steep cut and has shallow cover where rocks are mostly completely weathered and highly jointed. In addition, it was also suspected that portal stability will be further affected by the surface drainage water and working loads further reducing the stability during construction. There are also chances that collapse or subsidence of earth surface due to lack of soil bearing capacity can occur since ground loosening may start due to blasting and excavation of portal area. Under such conditions, portal area needs to be selected and designed in consideration of actual ground conditions, climate, surrounding environment, constructability and maintenance convenience.

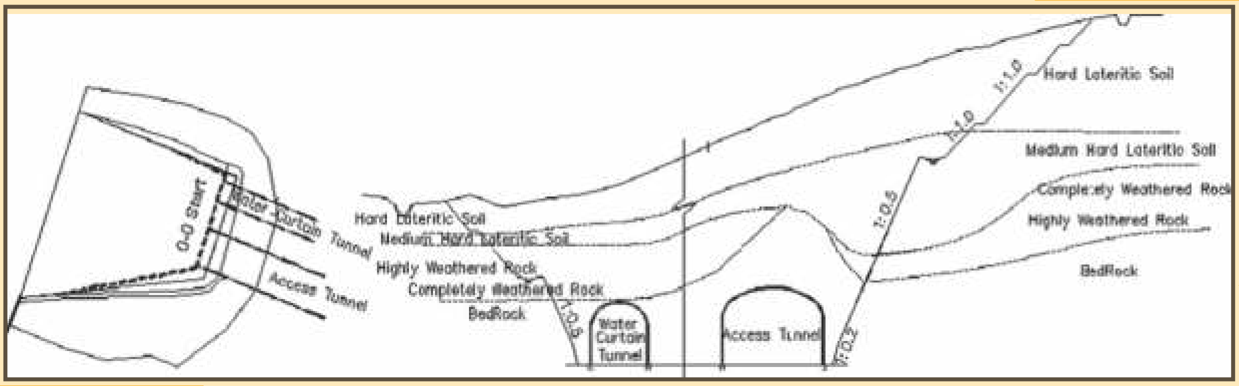


Figure 6.8: Plan and section of portal along with access tunnel and water curtain tunnel

In order to design an optimal and stable portal structure, different models of slope embankments were analyzed using varying combinations of thicknesses of laterites and weathered rock along different sections of the slopes. Slope stability analysis was mainly carried out for the slope at entry of access and water curtain tunnels along with sections representing right and left hand side of tunnels. This study was carried out using SLIDE software (RocScience 2008), an interactive limit equilibrium slope stability program that uses the method of slices to calculate a factor of safety. In the present study, Bishop simplified method with a circular slip surface was used for slope stability analysis along with Mohr– Coulomb failure parameters. Bishop simplified method is easy to use and gives relatively accurate results and agrees favourably (within about 5%) with the factor of safety calculated using finite element procedures (Albataineh 2006). SLIDE analysis in general involves a critical surface search, in order to attempt to find a slip surface with an overall minimum factor of safety to analyze the stability of soil slope. The models were mainly created from the results obtained from final investigation results along with sections referring to centre of water curtain and access tunnels and sections representing maximum slope height along with different soil, weathered rock combinations. In these models thicknesses of hard laterite and medium laterite were varied from 2m to 10 m, thickness of weathered rock from 1m to 6 m and depth of bedrock from 10 to 15m.

Considering the international practice followed and information provided by Sreekanatia (1993) on local behaviour of laterites, a factor of safety of greater than 1.5, 1.2 and 1.1 were adopted for dry season, rainy season and seismic loading respectively. Cut slope stability analyses were carried out by selecting areas with greatest height or poor geological conditions as a main cross-section. As per the adopted design criterion in this study, a soil slope of 1:1, weathered rock slope of 1:0.5 and bedrock slope of 1:0.2 was selected in the present modeling for numerical evaluation of slope stability. It was further planned from a stability point of view to install 1 m wide berm every 5 m interval in the soil region and 3 m wide berm at the boundary between rock and soil or below the interface. However, during construction, berm width can be adjusted as necessary to obtain workspace for equipment entry and underground tunnel requirements.

The results of the analysis in terms of factor of safety for different slope combinations are shown in Table 6.3. It is observed from Table 6.3, that slope stability is ensured in terms of factor of safety for all conditions except for saturated conditions along the larger sections, i.e. on right hand side of water curtain tunnel and access tunnel. It was also observed during the analysis that the critical failure surface cuts across the laterite surface to finally intersect completely weathered rock layer for Case 6. Based on results of numerical studies, it was inferred that

Table 6.3 : Different slope cases analysed

S. No	Location	Dry (FS≥1.5)	Saturated (FS≥1.2)	Seismic (FS≥1.1)	Remarks
1	Rear part of AT at starting point	3.26	2.41	2.89	Stable
2	Rear part of WCT at starting point	2.84	1.89	2.47	Stable
3	Left side slope of tunnel at starting point	4.1	2.38	3.15	Stable
4	Right side slope of WCT at starting point	1.72	0.91	1.54	Unsatble in s taurated condition
5	Right side slope of AT, 10 m outside	1.89	1.09	1.67	Unsatble in staurated condition
6	Right side slope of AT,at starting point	1.76	0.99	1.57	Unsatble in staurated condition
AT= Access Tunnel, WCT = Water Curtain Tunnel					

slope stability of a saturated condition cannot be guaranteed for the available ground conditions and further strengthening in terms of specific reinforcements needs to be determined. Since the right hand side of access tunnel (Case 6) was observed to be the most endangered section due to its height and thick zone of completely weathered rock, it was planned to carry out reinforcement trials on this side of slope section. The reinforcement in the form of passive rock anchors fully grouted along its full length providing 100% bond length and extending beyond the slip circle surface was adopted as the primary reinforcement for increasing the slope stability. Passive rock anchors, 25 mm diameter untensioned bars were planned to be installed at an inclination for a length crossing slip surface zone and covering whole of the completely weathered rock region. Rock anchors which only develop resisting force after some movement within the slope has taken place are considered as passive support elements in numerical models and they normally do not incorporate initial loading or tensioning. In a rock anchor support, the orientation of the applied force is always parallel to the orientation of the rock surface; thereby rock anchors were placed at an inclination of approximately perpendicular to slope surface. In order to decide on the optimum pattern of rock anchors position along slopes, three different arrangements of anchor reinforcement were reviewed, initially for Case 6, as it represents the most critical section. These combinations were analyzed only for saturated conditions. In the first pattern, passive rock anchors were installed along the slope right from the top of the hard lateritic soil to the bottom highly weathered rock for a length extending beyond the critical slip surface.

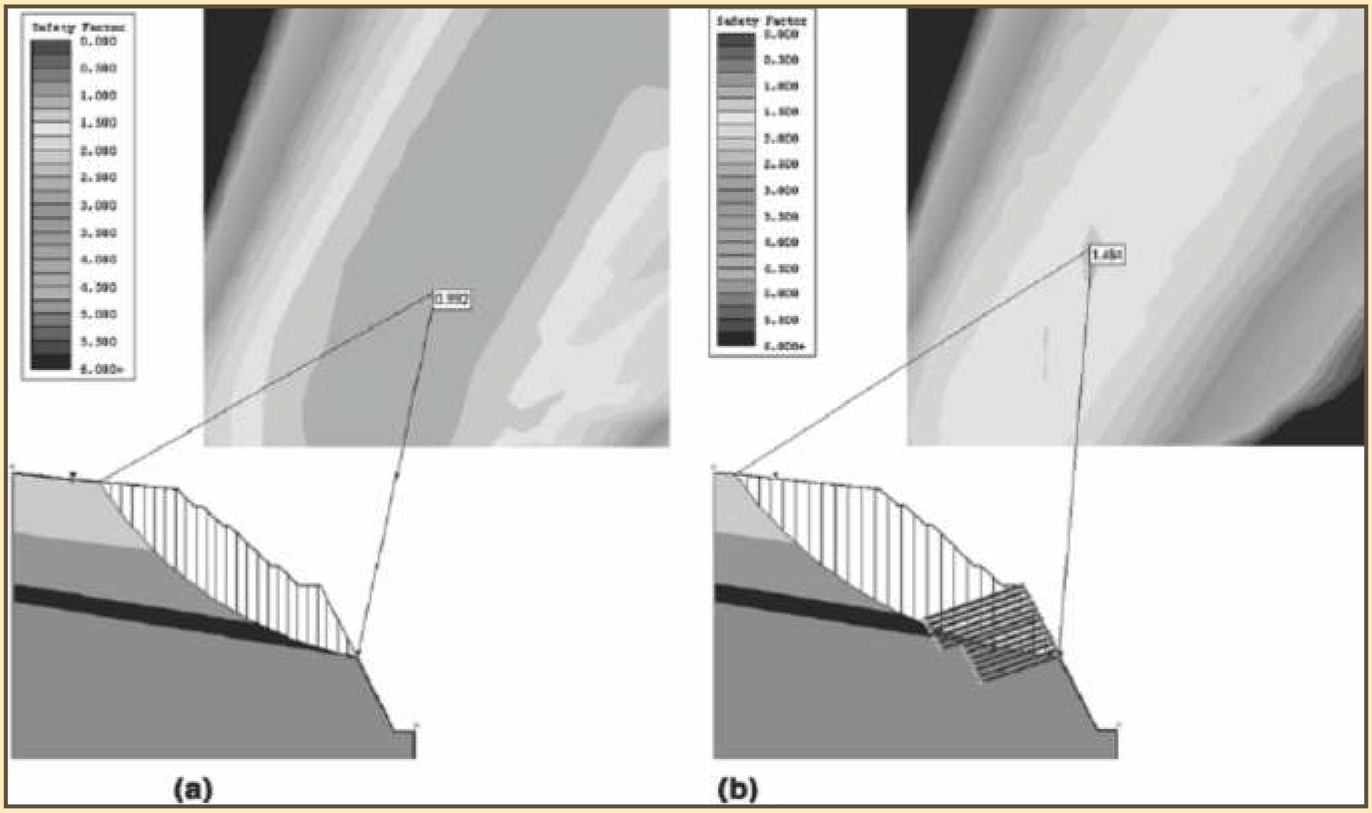


Figure 6.9: Critical slip surface obtained (a) with and (b) without reinforcement for case 6

Rock anchors were placed nearly perpendicular to rock surface in lower portion and vertically down in the above lateritic soil layers. In the second case, rock anchors were placed nearly perpendicular to rock surface only below

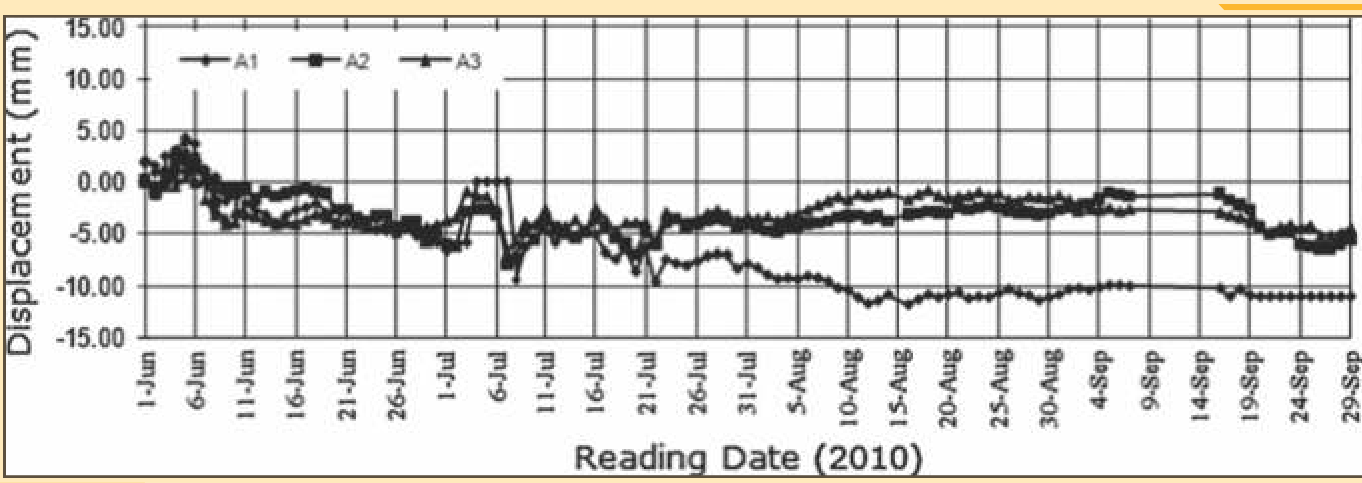


Figure 6.10: Convergence monitoring above the centre of access tunnel

3 m berm (Figure 6.9) at the interface between the medium hard laterite and completely weathered rock, while in the third case inclined anchors were placed in top and bottom layers only. It was inferred from numerical analysis results that while stability is ensured in last two cases under saturated conditions, the most favourable conditions in terms of construction difficulty were envisaged in installing rock anchors only in the lower 3 m berm part considering large area and height of the slope.

The optical monitoring data in form of resultant displacements for a slope point were plotted against the actual date of recording for overall assessment of slope movements as shown in Figure 6.10. The figure shows plotted distribution of convergence measurements for "centre of access tunnel" and on "right hand side of access tunnel" for main monsoon period (June to September) at site which was critical to stability of slopes. Timing of convergence readings coincided with start of excavation of portal of access tunnel and water curtain galleries. Some higher values of displacement were recorded during this period (Figure 6.10) which was mainly induced as part of blasting activities in tunnels just beneath the slope location. It was also observed form the results that any displacement in slope movement was substantiated with similar pattern of measurement recorded inside tunnels which showed overall accuracy of measurements.

CONCRETE WORKS

A.Usmani, A.Nanda & C. Singh

CHAPTER - 7

7.1 Introduction

Underground rock caverns are large D-shaped caverns used for storage of crude oil employing the principle of hydrogeologic containment. Underground storage facilities involve concrete works mainly for tunnel and shaft plugs for containment, cavern floor for circulation, encased hot oil pipe concrete blocks for ensuring de-sludging and partition wall within caverns for unidirectional circulation. Within the storage caverns, crude oil remains in contact with the concrete surface, therefore, all the concrete works are carried out using minimum M40 grade sulphate resisting portland cement (SRPC).

7.2 Concrete Plug Design

Concrete plugs in crude storage facilities (Figure 7.1) are a critical component used to contain the crude oil inside the rock cavern. These plugs are designed as gas tight to prevent escape of oil, vapour, gas and to withstand differential pressure occurring on account of different fluid pressures stored on either side of the plugs. There are generally two types of plugs required in underground storage; tunnel plugs and shaft plugs (Figure 7.1). In case of tunnel plug, it is vertically located at a select chainage of access tunnel to isolate crude oil, forming a separation between stored crude oil within the cavern and water in the access tunnel. On the contrary, the shaft plug is horizontal in section and is covered by a long column of water or concrete while oil and vapour is retained beneath the plug. Plug construction involves substantial volume of concrete, which results in development of large heat of hydration. Therefore probability of cracks occurring due to heat of hydration of the cement needs to be avoided. Provision is made in the design stage as well as during the construction process toprevent the development of large temperature gradients caused by hydration of cement using concrete cooling arrangements. In order to ensure tightness of the plug for prevention of any gas leakage, contact grouting of the plug is carried out at concrete and rock interface until a proper seal is established.

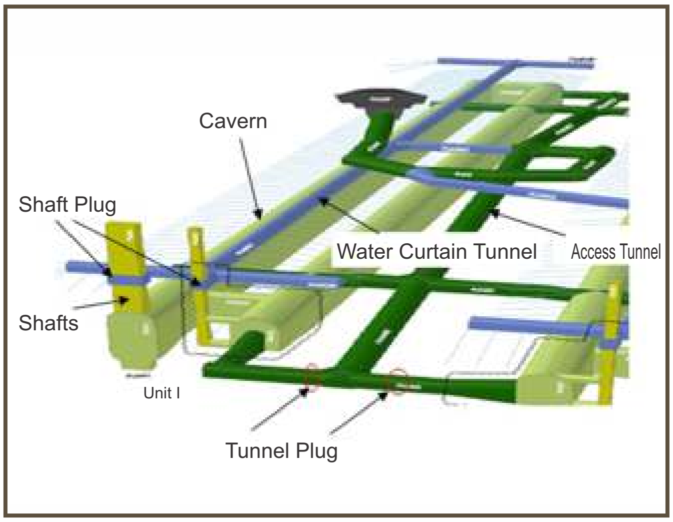


Figure 7.1: Sechematic layout of underground rock caverns

Plugs are designed to resist failure from mainly five modes viz. mechanical jacking of rock surrounding the plug key, shear failure through concrete along contact rock, deep beam flexure failure, failure due to excessive seepage around plug and long term chemical & physical breakdown of concrete, grout or surrounding rock. Thus, there are three stages of design for concrete plug starting from structural analysis and concrete design for worst loading conditions to transfer the load safely to the surrounding rock followed by thermal analysis to study heat generation and cooling system design and lastly design of grouting system which provides desired long term sealing to plug. The concrete plug thickness and reinforcement are designed to safely transfer the load coming on the plug into the surrounding rock. This is supported by a cooling system, designed to reduce risk of thermal cracks that can develop during the strength gain in the concrete followed by grouting arrangement to ensure gas tightness of plug (after shrinking of the concrete). Cooling system is provided first to limit the risk of thermal cracking during hardening of the concrete and later to produce an injectable gap after shrinking of concrete for grouting material to percolate in the space between the plug and rock interface and ensure tightness of the plug.

The total pressure acting on plug is transferred to the surrounding rock by means of a key which is cut into rock surface all along the periphery of the tunnel walls (Figure 7.2). Plug is retained both due to the cut in the rock (key-in) and due to the forces of friction and adhesion occurring along the surface in contact of concrete with the rock.

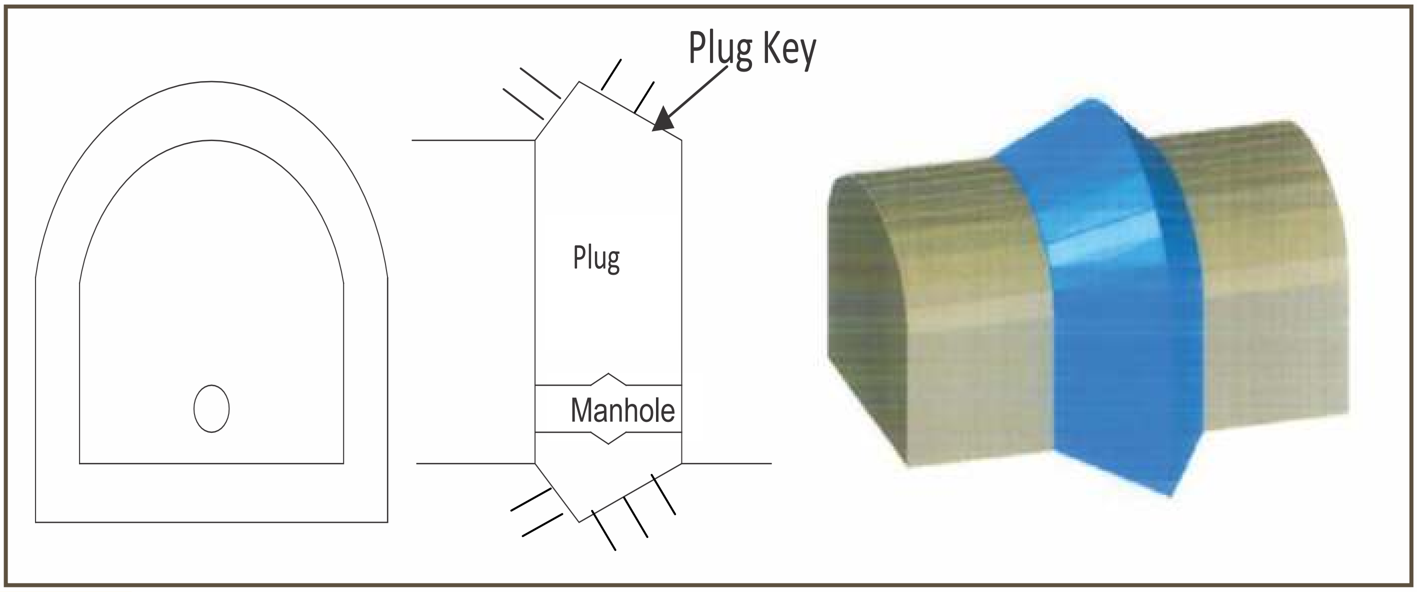


Figure 7.2: Plan, sectional and 3D view of a tunnel plug along with a circular manhole

Design of plug is based on long-term operational requirements as well as construction methodology adopted specific to each plug either in vertical or horizontal plane. Thus design philosophy is different, in case of a shaft plug and a tunnel plug. Access tunnel plug is designed for construction of plug in a single stage to avoid creation of any joint of weakness; hence the construction sequence has no impact on the design philosophy of the same. However, in case of shaft plug, plug is constructed in two stages or more in order to avoid heavy shuttering loads required to cater for several hundred tons of concrete, reinforcement, piping, cooling and grouting pipes loads. The plugs are constructed in specially excavated plug key-ins designed for the purpose. Primarily in order to construct stable plugs, plug key-in locations are chosen in rock mass which is massive and less jointed with high rock quality. Selected location is identified such that nil or minimum seepage is encountered and is not very close to any tunnel/cavern junction areas in order to reduce effect of three dimensional stresses.

7.3 Access Tunnel Plug Design

Tunnel plugs are dimensioned as per the size of access tunnel which is typically 8 m X 8 m (D Shaped cross section). Tunnel plugs are 3 m to 5 m thick and extend into rock surface with a key depth of 1m to 2 m as shown in Figure 7.3. In underground storage projects, tunnel plugs are mainly designed to resist crude oil pressure, which also includes pressure of the vapour phase above the oil surface which is typically around 1.5 bars. These plugs are also designed to resist hydraulic pressure due to water head on outer side of plug used to maintain hydraulic containment around the caverns. Since in case of any explosion inside the caverns, plugs can also be affected by generated pressure wave; an accidental load of 1.0 MPa is also considered during the design stage. Therefore, the tunnel plug is designed to withstand maximum loads considering the aforesaid requirements. Design of tunnel plug is checked for minimum crack width requirements prescribed for liquid retaining structures which is limited to 0.2 mm as per IS 3370 (Part 2: 2009). A manhole in the centre of the plug is also provided to access other side of the plug in order to facilitate removal of man and machinery after the completion of plug as shown in Figure 7.3.

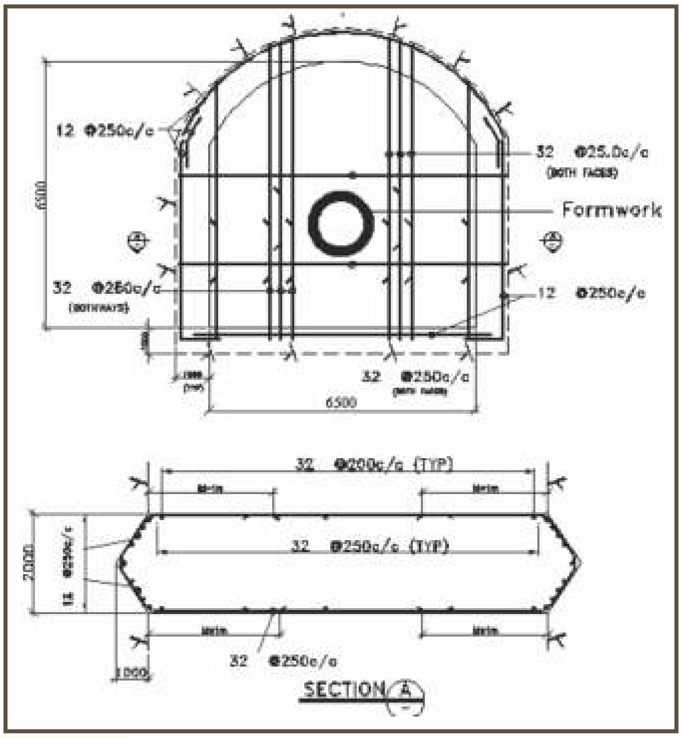


Figure 7.3: Reinforcement arrangement inside a tunnel plug

7.4 Shaft Plug Design

In case of shaft, plugs are located at an interface where crude oil is located in the lower zone below the plug and the upper zone is filled either with mass concrete or water column. Shafts normally vary from 4m to 12m in size and can be circular or rectangular. These plugs are designed to withstand differential pressures on either side of the plug and also to support load of casing pipes through which submersible crude oil pumps, seepage water evacuation pumps and other instruments are lowered. These casings loads are quite large and vary from 200-300 tons, which is required for operational requirement of storage structures. In an underground storage project, shaft plugs are designed to resist worst combination of pressure coming from the water / concrete column, self weight and weight of the casings and carrier pipes. The plug is characterised by a trapezoidal shape (bottom layer and top layer) as shown in Figure 7.4, and is sealed in an enlarged part of the shaft section known as plug key. This widening provides a good seat for the plug and distributes the unbalanced load into the surrounding rock.

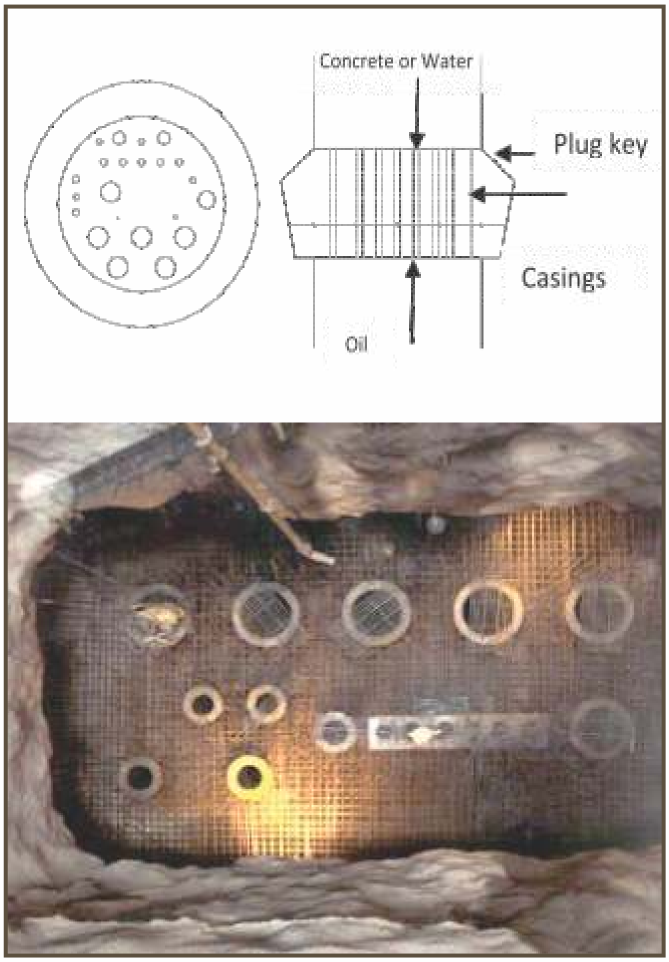


Figure 7.4: Plan and sectional view of a shaft plug along with casings

INSIGHTS

Designs of Tunnel and Shaft plug

Plugs are made using M40 concrete mix with sulphate resisting portland cement due to its contact with oil surface. Plug key-ins are excavated inside the rock mass which supports plug load by bearing and transfers the loads to sound rock below. In case of tunnel plug, governing load due to water pressure for actual head of water as per the tunnel plug elevation is considered. Since shaft plug construction is carried out in two stages of 1.5 m and 2.5 m each for a usual 4 m thick shaft plug, weight of second 2.5 m layer of concrete is assumed to be taken care of by first layer of concrete during design.The concrete plug is modelled as four nodded plate elements using finite element analysis. Plug reinforcement is calculated considering all final loads coming on the plug with entire plug thickness. Shear capacity of concrete is checked against shear stresses developed after final loading and after casting the total thickness of the plug. Further adequacy of the reinforcement is also checked against moments and shear forces developed after casting of the first layer of concrete with all empty casing loads transferred to it. Due to staged construction methodology, a check for the induced stresses developed in steel due to first layer and second layer of concrete is made. If the total stress developed in steel after casting the final concrete layer is higher than the strength of steel, reinforcement obtained earlier is revised accordingly. The plug is analyzed as simply supported around its periphery along the rock surface and is modelled as a plate using plain stress analysis. This model provides the main support reactions from which distribution of the uniform pressure both in horizontal and vertical direction is determined. Plug reinforcement is determined from the distribution of tensile and compressive forces on the entire thickness of the plug. Most of the reinforcement is confined around the periphery of the plug surface due to concentration of tensile forces as the load is transferred to the rock through the key. However, some minimum compressive reinforcement is also provided along the middle layers to counteract large compressive forces.

7.5 Cooling Design for Concrete Plug

It is well known that heat is produced during hydration of concrete. The temperature continues to rise within the structure as long as the rate of heat development is higher than the rate of dissipation to its surroundings. The cast in situ concrete structure expands in volume during this initial phase, and later it contracts as it cools. It is observed that volumetric changes within such a structure do not take place freely due to the presence of different restraints that would also result in thermal stresses. Therefore restraint, in addition to the above mechanisms described determines the risk of thermal cracking. Restraint is often differentiated into internal and external restraints, with restraints such as reinforcement in case of tunnel plugs and reinforcement and piping in case of shaft plug play a major role in restricting this free expansion of concrete along with the rock surface.

In order to ensure tightness of plug, risk of thermal cracking is reduced by using cooling arrangement in the plug concrete during and after casting of concrete till temperature in concrete reduces to ambient levels. Tightness of the plug is also achieved through a combination of cooling arrangement used to shrink concrete plug (as part of secondary cooling) followed by grouting to fill any gap created while cooling of concrete.

It has been reported by many researchers that several deleterious reactions occur in concrete during hardening stage, which can jeopardize durability of the concrete structure. One of those damage mechanisms is the delayed ettringite formation (DEF) which is a result of high temperature while the concrete hardens. DEF may result in internal cracking of the concrete, even years after construction. Construction codes in many countries for this reason restrict the maximum concrete temperature while hardening or setting of concrete. Different standards around the globe define the maximum allowable limits for temperature to prevent cracking, like British standard BS 8007:1987 recommends maximum concrete temperature must not exceed 65°C, French standard NF EN 1992 as 70°C, Irish standard IS 326:2004 as 60°C while ACI 207.2R allow a maximum temperature of 70°C. Thus for different types of concrete plugs, DEF is to be avoided, especially due to the risk of additional sulphate attack, in order to ensure the functionality and tightness of the plug. Thus, maintaining the temperature of the concrete below 60°C during and after the concrete casting is primarily important for durable plug design.

In order to study the behaviour of concrete in confined conditions, a set of adiabatic to semi-adiabatic temperature tests are carried out on 1m x 1m x 1 m cube samples on site in confined conditions, with the concrete filled in a box closed on all sides. The temperature of concrete in the adiabatic box, monitored on hourly basis continuously for seven days at one of the sites is shown in Figure 7.5, for four sensors. Four temperature sensors are placed inside the cube, with one at the centre, three on the side faces of the cube. One sensor is also placed outside the cube to measure ambient temperature. As observed, the sensor for ambient temperature reported a constant temperature of 30-31°C, the sensor 1 located in the centre of the cube reported max temperature rise of 64°C, while the side face reached to a maximum temperature of 57°C.

Heat hydration analysis of concrete plug and subsequent cooling design are carried out using a combination of laboratory studies on cubic samples discussed above along with numerical studies using available software packages. Numerical analysis is usually conducted to evaluate analytically the effect of casting temperature on heat of hydration in mass concrete plug constructed using sulphate resistance portland cement. Concrete plug is modelled using 3D element and analysis carried out as per heat transfer module of the software which entails the process of calculating the change of nodal temperatures with time due to heat source/convection. Design of the cooling system is carried out based on the results of the numerical analysis along with specific project requirements; however the final system of pipe arrangement is governed by availability of space and construction sequence. ACI 207.4R-93 and ACI 207.2R-07, also suggest a basic methodology for calculation of flow and cooling pipe arrangements for mass concrete structures, which is based on old but good detailed investigations conducted during construction of dams and other mass concreting works. However, this approach does not take into account the effect of restraint, which is considerable for tunnel and shaft plugs.

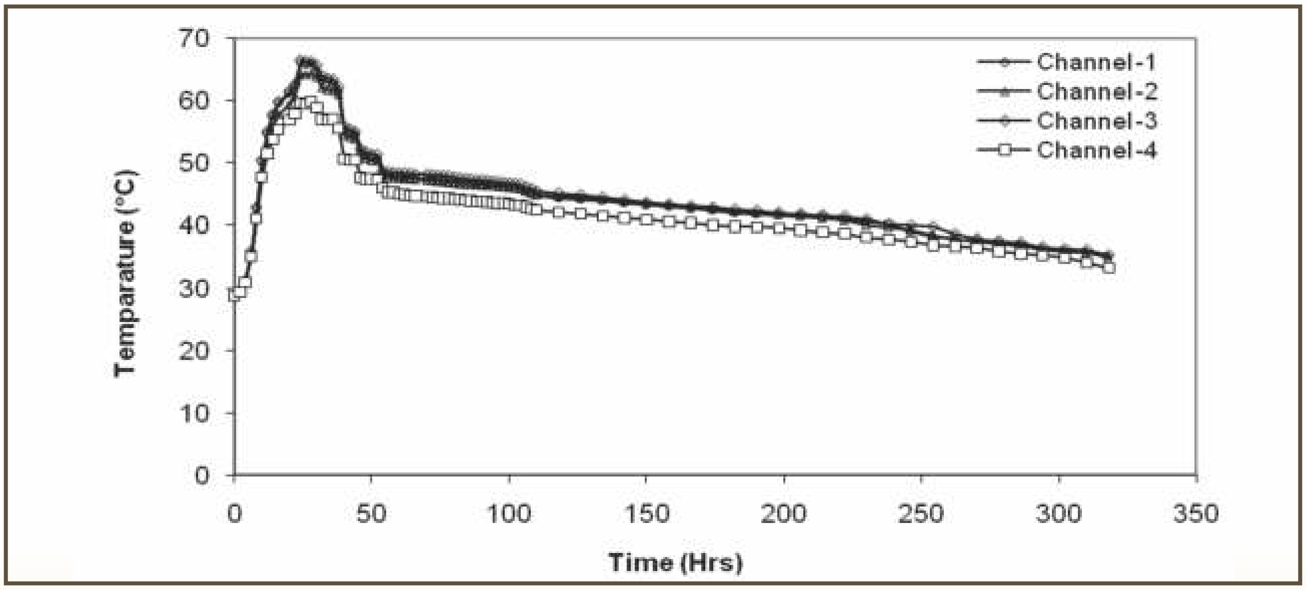


Figure 7.5: Temperature variation of sensors during adiabatic test

The arrangement of the cooling pipes is chosen from experience and in an iterative process. A denser array of cooling pipes are placed in the bottom of the tunnel plugs close to the restraining rock foundation. In case of tunnel plug, it was analyzed that in the bottom part of the tunnel plug, close to the restraining rock foundation, cooling pipes are placed at a dense array (distance of about 500 mm) for effective cooling as shown in Figure 7.6.

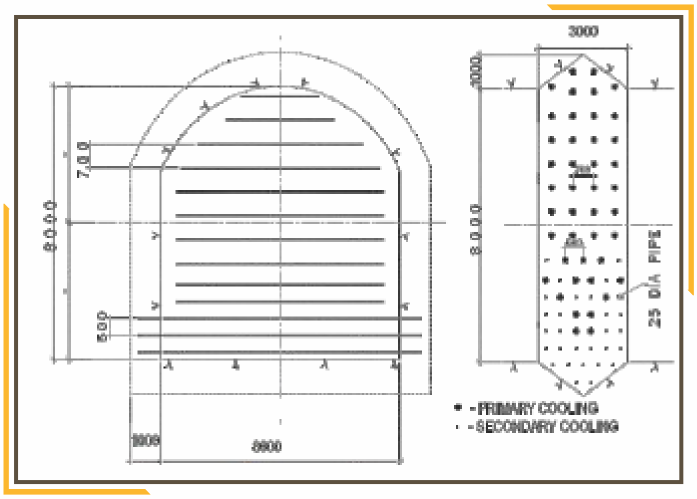


Figure 7.6: Plan and sectional view of cooling arrangement with cooling pipe

It is observed that due to low heat transfer capability of concrete, larger distance between the individual cooling pipes would result in an increased temperature in between the cooling pipes and thus higher risk for thermal cracking. However, as we move higher up in the tunnel plugs, the crack risk lowers due to larger distance from the restraining border. In these parts of the tunnel plugs, the distance between the individual cooling pipes is increased up to 700 mm as shown in Figure 7.6. Thus the maximum temperature between the individual cooling pipes and the geometry of the plugs are a significant factor for cooling design. In case of a tunnel plug, the geometry of the plugs and the concrete properties allow for a maximum spacing of 800 mm. The required distribution of the cooling system into individual cooling coils calls for a maximum length of 60m to 80 m of cooling pipes. The temperature of cooling water is set to 10°-20°C while required flow in a cooling coil is set between 20-27 (X 10^{-3}) m^3/min in order to limit the increase of water temperature in the coil. However, in case of the higher length of coils, the flow and consequently the operating pressure had to be increased in order to achieve the desired cooling effect. This would largely increase the cost for achieving a water-tight cooling system, especially because of more complicated joints. Soon after the concrete in the tunnel plugs is mature (28 days of hardening), the surfaces of the plugs are isolated and temperature of the cooling is slowly reduced in the tunnel plugs to 5°-8°C. The reduction of the temperature results in thermal contraction of the tunnel plug and widens the gap between rock and concrete at the top and the sides of the plug. This reduction of the temperature is achieved in slow pace in order to avoid cracking in this phase. When the complete plug is cooled down to the desired temperature, contact grouting is performed. While the grout gets hardened, the plug gets restored to ambient temperature and results in compressed plugs.

7.6 Grouting of Plugs

Grouting is performed in tunnel and shaft plugs after completion of casting of the plug to achieve complete gas tightness of the plug. Once the plug key is excavated as per the design requirements, rock mass grouting is performed to grout all the blast induced cracks near the key-in surface in the excavated damaged zone. The rockmass surrounding minimum 5 m of the plug area on all sides is grouted with higher pressure of up to 10 to 15 bars as shown in Figure 7.7. OPC cement is used for this purpose, however depending on the rock type in case scenarios micro cement is also used.

Rock mass grout holes are spaced 0.5 m on all sides. The packer is put as close to the surface of the holes (about 50 cm in depth).While casting the plug, several measures are taken to avoid shrinkage cracks within the concrete, however, contact between the rock mass and the

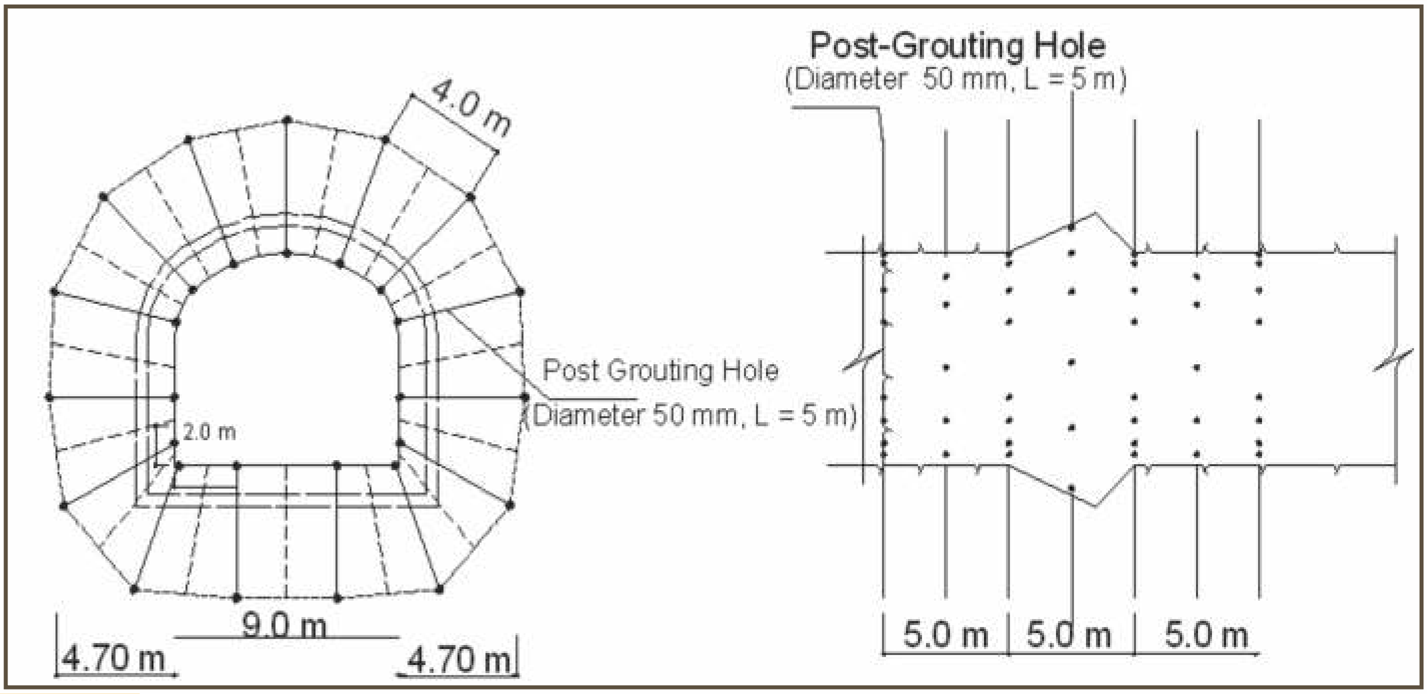


Figure 7.7: Plan and sectional view of post grouting arrangement around a tunnel plug

concrete still retains some voids due to several reasons viz. differential expansion of rockmass and concrete during casting of plug, shrinkage of concrete plug, uneven contact surface between plug and the rockmass, gaps created in the contact especially at crown due to gravity settling of cement during casting and gaps created in the contact possibly due to improper compaction which is practically difficult to achieve at times.

These gaps in the contact are grouted after casting of the plug to achieve water and gas tightness of the plug. Therefore, for the purpose of contact grouting, grout injection tubes are embedded inside the plug before casting.

Once the concrete plug attains its design strength (28 days after casting), it is cooled again to 5-8° C. The embedded cooling pipe networks used for the primary cooling are used for the secondary cooling. This reduced temperature allows the plug to shrink and create more gaps artificially in the contact between the plug and the rockmass. Two stages of contact grouting are performed at this stage, one followed with another. Grouts are injected in the gaps between concrete plug and rockmass during cooling and once set, the cooling stops and the plug attains normal ambient temperature. At this stage, the plug expands to its normal size and the grouted contact between the plug and rockmass are further sealed resulting in tight plug.

Two stages of contact grouting are performed to ensure complete gas tightness of the concrete plug. The first stage is carried out with ordinary portland cement and later one with microcement. First stage contact grouting acts more as back fill concreting and fill any large voids such as created in the roof and upper wall area of the plugs. These are implemented by embedded GI pipes inserted about 50 cm inside the rockmass. These pipes are placed at 0.5 m spacing all along the sides similar to rock mass grout holes. The leftover hole portion used for rockmass grouting are used for this grouting instead of fresh drilling through the GI pipe into the rock mass. During secondary cooling of the plug, the gaps created

due to shrinkage in the contact are filled primarily with this grout at 5 bar pressure. During grouting of these pipes, the grout first fills the GI pipes and the hole and the annular space between pipe and the hole in the rock mass, it overflows and the grout spreads along the contact between plugs and the rockmass.

Second stage contact grouting starts once the first stage grout is completed and gains strength (normally two days) under secondary cooling. This is performed with microcement so that it penetrates the leftover small spaces of the first round contact grout with OPC. For this, specially designed perforated grout injection tubes are embedded inside the plug. They are placed in three peripheral rows in contact with rock at 1 m interspacing distance. These pipes are made of PVC which can withstand the high pressure created by the hardened concrete. They have got an outer protective layer which prevents cement slurry entering the grout pipes during casting of plug. While grouting, the grout pressure of 3 bar from inside the tubes, opens the protective layer covering the perforations and allows the grout to flow out of the tubes in the contact.

In case of shaft plugs, secondary cooling is not performed during contact grouting. Here since the plug is horizontal due to gravity, the lower contact gets better tightness than its tunnel plug counterpart. However, except secondary cooling, the same procedure for contact grouting is followed as outlined above for tunnel plugs.

Before second stage contact grouting with micro-cement, water is injected through perforated grout injection tubes to check the injection tube function, quantify the volume of leak and void space in the contact. The tightness test is conducted in two stages, one before and one after second stage contact grouting. The grouts are injected through each individual injection tube which has got both inlet and outlet hose coming out of the plug. Grout left in the pipes is removed by flushing at a pressure less than the pore opening pressure of these injection tubes, before initial set of micro-cement. Water tightness test is performed in the central row injection tubes at 3 bar water pressure. If the central pipes are found to be leaking through the contact or water seeping across the plug through blast cracks, grouting is performed through the outer row tubes again, at normal temperature. The grout injection tubes are specially designed for re-grouting for this purpose. Every time immediately after grouting, the grout material inside the tubes is flushed at low pressure so that the pipes can be used for re-grouting. Thus the plug tightness is achieved through contact grouting. Finally, after successive tightness tests, the central row injection tubes are also grouted with micro cement at 7 bar pressure.

7.7 Cavern Floor & Hot Oil Pipe Encasement

The cavern floor is designed as reinforced concrete. The invert paving does not have any structural functions other than creating a proper clean floor with tight tolerances for the storage area. The paving supports the circulation pipe in the main storage galleries which is used to transport heated crude oil all along the cavern. In order to prevent any fine material to be washed away and go down to the pump pit, all the floors of the storage areas are lined with a blinding concrete layer. Cavern floor is designed for a minimum thickness of 300 mm in order to withstand load of construction vehicles and carrier pipes as shown in Figure 7.8. Concrete floor is underlain by a sub-base and base layer consisting of granular material which allows the drainage water to pass through to rock below and prevent damage to floors. Concrete floors are sloped both in cross-section (10%) and longitudinally (≈1 in 250) towards the outlet gallery in order to achieve gravitational flow of oil inside the caverns. Hot oil pipe (≈700 mm dia) is encased in reinforced concrete block having minimum concrete cover of 300 mm on either side and is anchored to rock underneath to resist thermal expansion/ contraction of concrete pipe support and steel pipe.

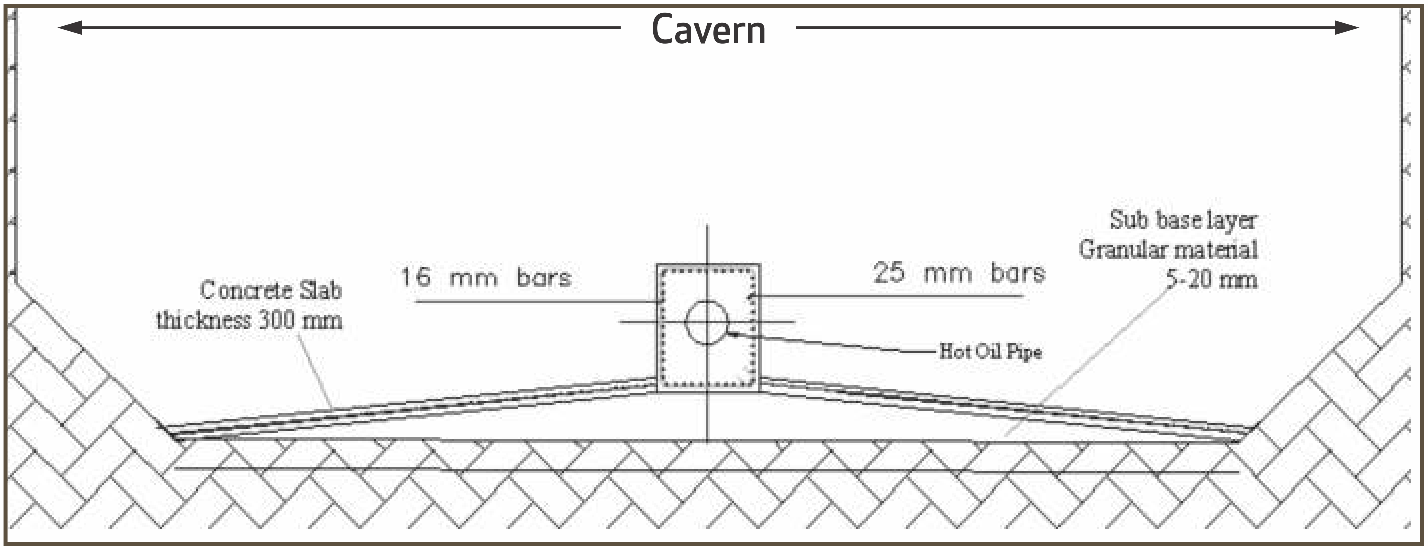


Figure 7.8: Sectional elevation of cavern concrete floor and hot oil circulation pipe

7.8 Partition Walls

Partition walls are constructed in the interconnecting galleries between the two cavern legs so as to avoid shot cut between the two legs and allow the fluid to flow its normal gravitational flow along the longitudinal slope of the cavern leg. Partition walls are designed as 400-1000 mm thick RCC walls, anchored into the rock surface. These walls are designed to take pressure resulting from crude oil head on either side. They are anchored into the rock surface using set of 25 mm rock bolts inserted 1 m deep inside the rock surface and where required excavated 300 mm deep all along the rock periphery.

PROCESS DESIGN

N.S.Vasudev

CHAPTER - 8

8.1 Introduction

The storage of crude oil in unlined rock caverns is based on the following basic principles:

- Crude oil is lighter than water and not soluble in water, also.
- The storage caverns are located below the surrounding ground water level at sufficient depth so as to ensure hydrogeological containment.

As the storage caverns are located below the surrounding ground water level, the oil is confined in the cavity. Due to natural fissures in the rock, water continuously percolates towards the cavern, thus preventing oil from leaking out. Water leaking into the cavern ("seepage water") is drained to a pump pit located in the deep end of the storage units, and by the seepage water pumps automatically gets pumped out from the storage cavern. Crude oil storage facilities are designed typically to operate at following vapour pressure conditions:

Operating Pressure Relief Valve	1.5 Kg/cm^2g
Maximum Operating Vapour Pressure	1.3 Kg/cm^2g
Minimum Operating Vapour Pressure	0.1 Kg/cm^2g

A pressure above atmospheric pressure is always maintained in the cavern, to eliminate possibility of ingress of air into the cavern and also to ensure that leakage of vapour out of the cavern can be detected above ground. The cavern resists under pressure and is designed for an accidental load case of internal transient explosion of 1MPa. The normal storage temperature is in the range 25° to 30° C. The temperature in the surrounding rock is approximately 28° C, which is the crude temperature during long term storage without any additional heating.

By allowing a span of normal storage temperature, the need for heating the storage under normal conditions is reduced to a minimum and performed intermittently, if necessary.

As intake of crude into the cavern is at a temperature close to the storage temperature / ambient temperature, the risk of thermal stratification is minimized. Seepage water into the cavern is also at a temperature close to the storage temperature, hence thermal loss from this storage during normal storage conditions is brought to a minimum.

8.2 Above Ground Process Facilities

In an underground unlined rock cavern storage, following process facilities are installed above ground:

i. Shaft Top Equipments
ii. Heat Exchangers
iii. Metering Systems
iv. Booster Pumps
v. Boiler
vi. Pipe way above ground
vii. OWS and close blow down system
viii. Buried Pipelines and Electrical Cables
ix. Effluent Treatment Plant
x. Fire Water System with Storage and pumping facilities
xi. Fire Station
xii. Control Room
xiii. Emergency Power Generator
xiv. Outdoor Switch Yard
xv. Sub Station Building

xvi. Instrument & Plant Air System

xvii. Nitrogen Plant and Storage System

xviii. Cooling Tower

xix. Non Plant Buildings

xx. Fuel Oil Storage and Pumping System

xxi. LPG Mounded Storage

xxii. Contaminated Rain Water System

xxiii. Raw water Storage and Pumps

xxiv. Drinking Water System

xxv. Flare

xxvi. UPS for Control & ESD System

xxvii. EPBX, SCADA, Telecom System

xxviii. CCTV-Surveillance and Access Control System

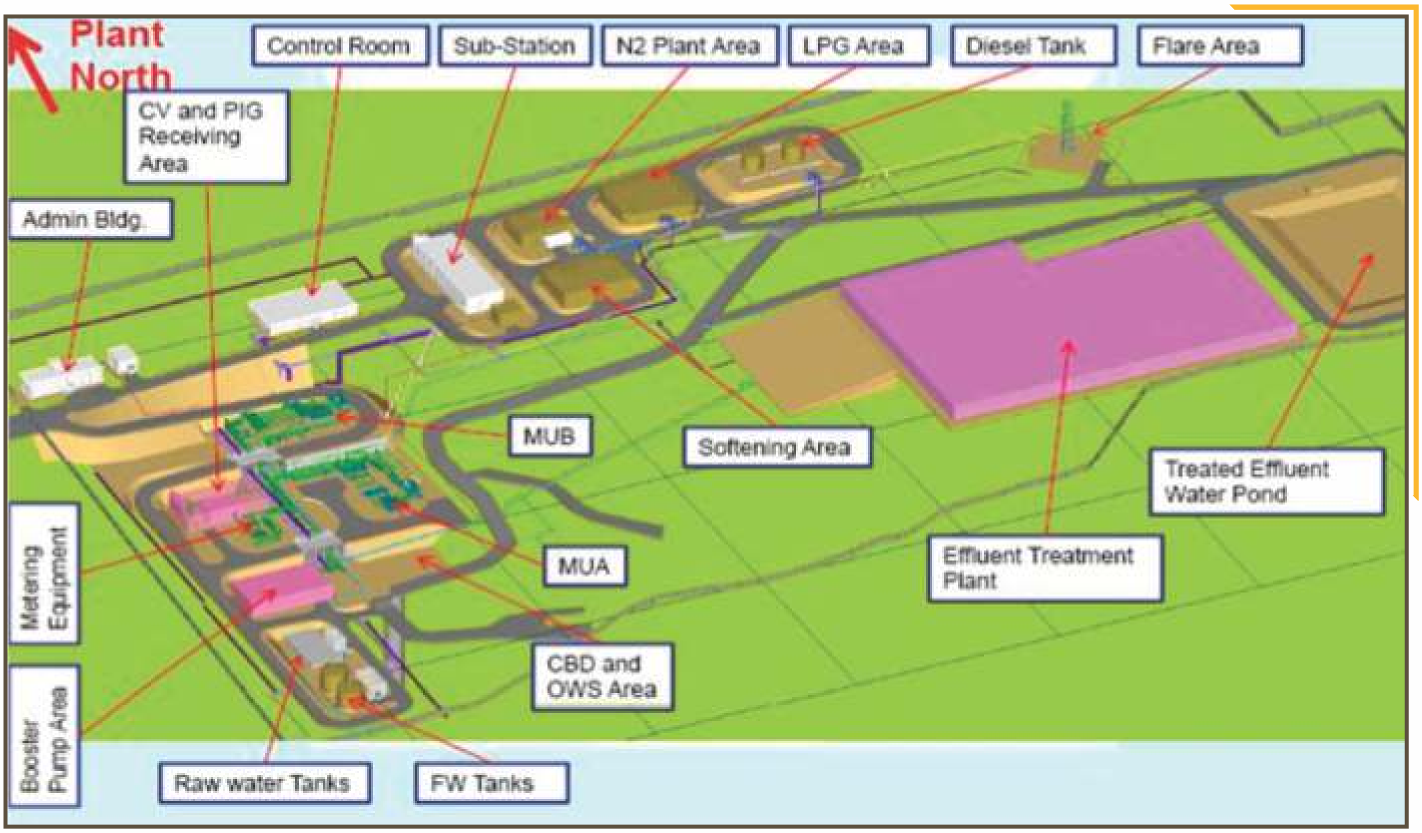


Figure 8.1: Typical layout of above ground storage facilities for crude oil storage

8.3 Crude Oil Receiving System

The crude oil is imported to caverns by a crude oil pipeline, either from the existing Crude Oil Terminal (COT) or directly from an SPM. As per a typical design, maximum flow rate for cavern filling is considered at 10,000 m^3/h. The required battery limit pressure at the cavern facility before metering varies 4 to 5 kg/cm^2 (g). Normally three bidirectional flow meters in parallel are used to measure the flow rate into and out of the caverns. They are equipped with strainers with air eliminators, as well as an integrated on-line automatic sampling skid allowing product samples to be taken for off-line laboratory analysis. A master meter in the form of Turbine flow meter is also installed for verifying the ultrasonic flow meters.

Each cavern unit consists of two legs in "U" formation. Crude Inlet to the cavern is through the inlet shaft i.e. at the starting of the "U" formation. The outlet shaft which is located in the other leg of the cavern i.e. at the farthest point of the "U" configuration consists of Submersible crude pumps, Seepage water pumps, Instrumentation and Pump pit.

8.4 Crude Oil Pumps and Casings

The crude oil is pumped out from the cavern at a considered maximum flow rate of 6400 m^3/h and is exported by the same pipeline that is used for supply. With the help of submersible pumps, crude oil is pumped to grade level. Typical configuration of pumps is as follows:

Type of Pumps	:	Vertical Submersible Crude oil
No. of Pumps	:	4 Working + 1 Standby
Capacity	:	1600 m^3/hr (each pump)
Differential Head (m)	:	100-200

These pumps are used for pumping out crude from the bottom of the cavern pump pit to the top. The capacity of the submersible pumps is identical in all the caverns to ensure inter-changeability. The pumps are designed for a discharge pressure which is adequate for providing the required suction pressure to the booster pumps at the booster station. The pumps are capable of handling most critical crude quality and for pumping out of melted waxes and crude sludge during de-waxing operation.

In order to maximise crude transfer to nearest COT or tanker, two units of cavern storages are, in general considered in operation simultaneously. The crude oil is sent to the booster pump station and is discharged at a maximum flow rate of 6400 m^3/h into the pipeline which is used for both filling & evacuation operation. The evacuated crude oil from the booster pump pass through the ultrasonic metering system for measurement purpose.

All crude oil & seepage water pumps, level and temperature transmitters and steam lines for the caverns are installed in individual casing pipes in the shaft. The casings are designed to withstand an explosion, inside the cavern up to a pressure level of 1 MPa (explosion in the vapour phase). During normal operation the pressure inside the casing pipes is equalized to the cavern vapour pressure by vapour balance pipe connections at the cavern shaft top.

8.5 Crude Oil Heating Philosophy

Crude oil heating is performed when the crude oil volume in cavern is < 15% of total volume to enable bottom of the crude to be heated up in order to melt the sludge/wax formed, and to decrease the product viscosity for making pumping easier and de-clogging of crude oil pumps. A hot oil pipe encased in concrete with perforations at every 10 m is used for heating and de-sludging. One unit for de-sludging operation is used at any given time. Heat exchangers are used for heating of crude oil from caverns. Inlet temperature varies from 28° C to 60° C and outlet temperature and maximum allowable pressure drop are maintained not to exceed 70° C & 1 bar respectively.

Boilers are used for steam generation and steam is required for:

i. Heating crude oil during de-waxing / de-sludging

ii. Steam injection to pump pits for improving fluidity

The considered boilers typically consist of two units working at an operating pressure of around 6 kg/cm^2 (g), including all required valves, fuel burner and control unit. Typical considered capacity of boiler is 9 tons hour each; however the capacity depends on the number of caverns that is required to be heated at one time. The boiler is also designed to have two furnace fuel oil tanks (Diesel) of 10 days capacity. The design considerations also have recovered condensate to be reused as boiler feed with a provision for by-passing it in case of contamination.

Sludge or crude oil waxes settled in pump pit are removed by direct injection of steam into the water. It is considered that 6 kg/cm^2 g(H) pressure steam at cavern top is adequate for heating. Steam is injected at a rate of 4-5 tons /hr. Injection at the bottom of the pump pit creates

turbulence and mixing of sludge into the water. Steam injection nozzles are also guided on the circumference of the crude oil and seepage water pumps so that in case of clogging of a section of pumps, same are heated by stem and de-clogged. Steam injection nozzles are also used for injection of additives to minimize bacterial growth in the oil & water interface.

8.6 Nitrogen System

With the prime objective of inertization, nitrogen is required for the underground crude oil storage to perform the following activities:

- Removal of oxygen from the cavern prior to crude oil filling, such that oxygen content is brought down below 5%.
- Maintaining cavern pressure during crude oil evacuation, such that pressure is maintained at 0.3 kg/cm2(g) (min) during pump out.
- Maintenance activities in the pump shaft.
- Quicker dispersion of hydrocarbons.
- Flare header purging.

The nitrogen requirement to reduce the oxygen content below 5% during commissioning is met by using Pressure Swing Adsorption (PSA) inert gas generators. During normal operation, maximum nitrogen requirement is at the time of crude pump out @ 6,400 m3/hr.

As the vapour pressure of the stored crude oil is below atmospheric pressure, it is necessary to inert the vapour phase with nitrogen. Moreover, nitrogen is also required when the cavern pressure drops with crude oil withdrawal. In order to meet a huge requirement during withdrawal of crude oil, PSA inert gas generator skids or liquid nitrogen stored in vessels are used.

8.7 Seepage Water System & Treatment

The seepage water is removed from caverns by submersible seepage pumps installed inside casing pipes of duplex stainless steel anchored in the concrete plug above cavern roof depth and extending down into the cavern pump pit.

Seepage water is pumped to the water treatment plant from seepage water tank using Effluent Treatment Plant (ETP) feed water pumps.

Effluent treatment plant (ETP) of sufficient capacity is designed in such a way, so as to meet the quality of water for reuse in

- Make up fire water tank.
- Supplementary make up water to the water curtain.
- Irrigation purpose in green belt area.

This considered plan is to achieve zero discharge for the storage facility. Capacity of the Effluent Treatment Plant is dependent on the seepage rate of the water inside the cavern and varies from 30 to 150 m^3/hr. Sequential batch reactor are used for the water treatment.

8.8 Flare System

Flaring of gas and vapour is required during filling, during circulation and during heating. Further PSVs are provided to release the vapor to flare in case of uncontrolled pressurization during last stage of filling or during sudden raise in pressure inside the cavern. The flare system and the PSVs are designed for the flare load corresponding to maximum crude inlet capacity, i.e. 10,000 m3/h. Height of the flare stack is decided based on ground level radiation as per API 521 and as per state pollution control board norms. One LPG mounded bullet of adequate capacity is required for pilot ignition of flare system. Steam from the boiler is provided to have smoke less flame.

8.9 Compressed Air System

Compressed air is required in the storage installation complex for the following main requirements:

- Instrument Air to operate the various instruments in the facility and also for purging of some control panels
- Plant Air for operating hose stations for various miscellaneous uses in the complex for pneumatic operated fire dampeners, etc.

Compressed air required for all of the above uses, is generated at a centralized location in the topside of cavern and distributed to the various users through headers. Two qualities of compressed air are produced and distributed:

- Instrument Air comprises compressed air cooled to ambient temperature and dried to remove water to meet stringent atmospheric dew point requirements.
- Plant Air comprises compressed air cooled to ambient temperature and though not containing any entrained water droplets, is saturated with water vapor at the supply conditions.

The typical requirements of compressed air unit are as follows:

- Maximum pressure of 8.0 kg/cm^2,
- Normal pressure of 6.5kg/cm^2 and
- Minimum pressure of 5 kg/cm^2

8.10 Instrumentation and Control Philosophy

The main controlling components involved with the underground storage and above ground process facilities consist of the following:

i. Distributed Control System (DCS), SCADA and APPS System for pipeline and tankfarm management system for crude oil inventory control

ii. Emergency Shut Down System (ESD)

iii. MMI System (Man Machine Interface)

iv. Fire and Gas

v. Fire Alarm System

vi. Video Surveillance System (CCTV)

vii. HVAC Control System

viii. Boiler Control System

ix. HCs/VOCs monitoring

x. Monitoring of exhaust gas from boiler and DG set (Ambient Air Monitoring System).

Normally all monitoring and control of main systems are performed from the control room. Operation of the main functions in the plant is done in organized operating sequences such as:

i. Crude Oil Intake

ii. Crude Oil Discharge

iii. Crude Oil Recirculation

iv. Crude Oil Heating

v. Seepage Water Discharge

vi. Flaring Cavern Vapour

vii. Inerting Cavern Vapour

A typical photograph of above ground process facilities at one of the storage installations is shown in Figure 8.2.

Figure 8.2: View of above ground storage facilities(during construction)

CONSTRUCTION OF ROCK CAVERNS

C.Kumar, D.K.Saraswat, R.Rath, Vijay Shahri & K.Y.Mallesh

CHAPTER - 9

9.1 Introduction

Construction of an underground storage cavern involves excavation of rock mass involving construction of tunnels and caverns running into several kilometres with a stipulated time period for completion. Therefore, it requires proper planning, adequate deployment of skilled manpower and specialized equipments, strict adherence to the quality and safety procedures, etc. Underground facilities, mainly include tunnel portal at the start of access tunnel, water curtain tunnel & caverns as shown in Figure 9.1. Further vertical shafts for various pump installations are also constructed from surface which connect the water curtain tunnel and cavern at various levels.

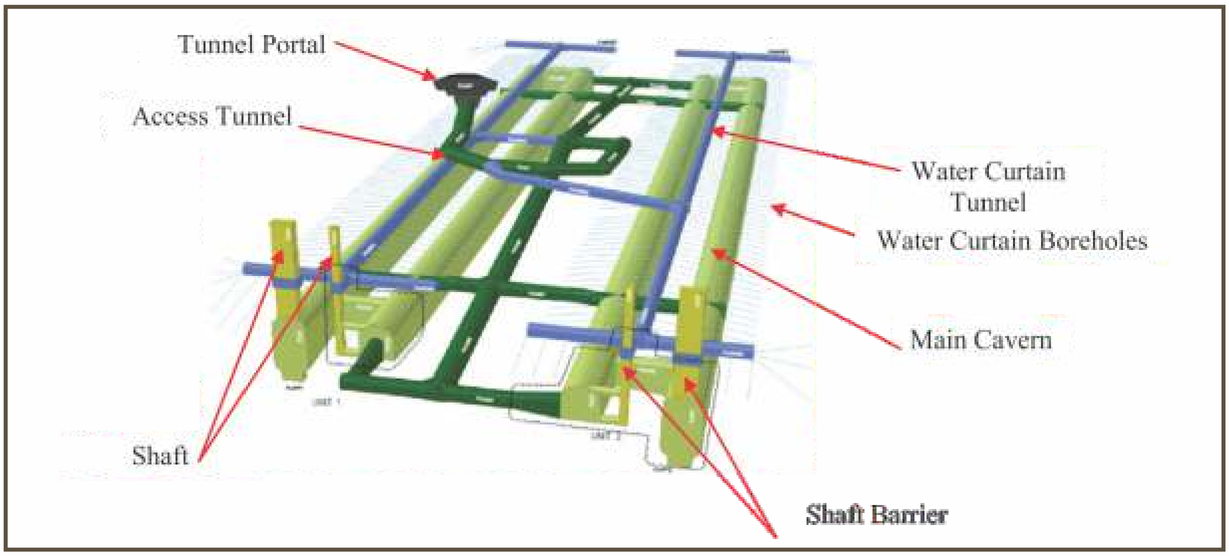


Figure 9.1: Overall underground facilities

The excavation plan involves construction of entry portal for access followed by excavation of access tunnel, shaft; excavation of water curtain tunnels and associated boreholes; excavation of the caverns in stages followed by concrete plugs to seal the cavern. Concurrently, water curtain boreholes are drilled from the water curtain tunnel and supplied with water through a temporary pressurization system for rock mass saturation around caverns. The cavern construction also involves in-cavern mechanical works such as hot oil circulation pipe and associated anchor blocks. In order to carry out evacuation of crude oil and seepage water from the cavern pump pit, submersible pumps, seepage water pumps and instruments are installed through pipe-casings lowered from the surface through the shafts.

Depths of cavern at a particular site along with availability of land around a cavern site are two most important considerations while deciding construction methodology at the site. Excavation of underground storage caverns is carried out using the drill & blast method, followed by installation of rock support, including fully grouted un-tensioned rock bolts & fibre-reinforced shotcrete. Controlled blasting techniques are adopted to minimize

overbreak/underbreak along the rock surface as well as to obtain a smoother profile at cavern face. All tunnels except the main cavern i.e. access tunnel, water curtain tunnel, connection tunnels, etc. are advanced using full face excavation and are paved with reinforced concrete. The main storage gallery is constructed by the heading-and- benching method in stages with the top heading being taken up first followed by excavation of the subsequent benches. The number and sizes of the headings and benches are decided as per the approved construction methodology and the proposed equipment. Special precautions are taken at the intersection of tunnels and shafts with the caverns. On completion of the excavation, the caverns are isolated and sealed by installation of concrete plugs. This ensures containment of the stored product while the shafts provide the necessary inlet and outlet pumping facilities.

In view of the cavern size and excavation sequence, muck removal from the cavern face forms an integral part of the construction planning. Therefore, while a suitable site of adequate holding capacity is selected for muck disposal; the lead distance between the construction site to the disposal site is also very important.

9.2 Underground Construction Scheme

9.2.1 Access Tunnel & Portal

Location of the portal is selected and designed considering topographical and geological conditions, climate conditions, surrounding environment, the drivability of the vehicle and maintenance convenience, etc. ensuring easy and exclusive access, so as to have independent access to the dumping areas for muck disposal.

Construction at tunnel portals is carried out carefully in view of the poor rock conditions/ multiple joint sets likely to be encountered near the ground surface. Where necessary, the excavation is initiated by driving pilot headings until sound rock is reached. Controlled blasting and short blasting rounds are also adopted. Steps are also undertaken to ensure slope stability near tunnel portal including rock/ soil reinforcement, provision of canopy, etc. Additional support measures such as shotcrete with wire mesh, long rock bolts, steel ribs etc may be required in poor ground condition near portal.

The access tunnel is designed to handle / accommodate the access of men and material into the cavern during construction;entry of all equipments required for the underground works as well as for the removal of muck, simultaneously with the movement of men and material. The accesses are designed with the objectives of time schedule as well as safety during the excavation phase. For a typical storage of cavern of 1.0 - 1.5 MMT storage capacity, one access tunnel and for 2.0 – 3.0 MMT two independent access tunnels originating from separate portals are considered so that the underground works are carried out independently through mutually exclusive units.

Determining factors in the dimensioning of the access tunnel are the ventilation requirements of the caverns as well as the chosen method and equipment for mucking. The access tunnel allows space for ventilation ducts and for free passage of the intense two-way heavy construction traffic. In order to facilitate mucking operations, the invert of the access tunnels is planned to be paved and properly de-watered. The typical portal at one of the storage sites along with access tunnel intersection is shown in Figures 9.2 & 9.3.

After the completion of underground works, access tunnel is closed by a wall with one steel door and one steel powered rolling shutter. The wall comprises of RCC beams and columns, with fill-in by brick work.

Figure 9.2: Portal for storage caverns

Figure 9.3: Access tunnel intersection

9.2.2 Water Curtain System

The access to the water curtain gallery is routed through access tunnel and is constructed around 20 m above the cavern roof. Determining factors in dimensioning of the water curtain gallery are the equipments to be used for excavation, mucking as well as the method and equipment for water curtain borehole drilling. A water curtain gallery also consists of a series of water curtain boreholes of about 95-100 mm diameter and 50-100 m length with a spacing of 5m to 20m. For horizontal water curtain, boreholes are sub-horizontal and the water curtain boreholes extension above the cavern is determined in the design. Water curtain boreholes are drilled approximately 1 m above the water gallery invert. The water curtain boreholes are charged through temporary arrangements for pressurization of boreholes, so as to ensure saturation of rock-mass ahead of the cavern construction. Cavern excavation is allowed with at least 50 m of advance saturation of the rock mass through a charged water curtain system. Once the cavern excavation is completed, the temporary arrangement of pressurization is dismantled and the water curtain gallery is filled with water up-to a defined level.

Water curtain gallery which is excavated before the cavern excavation, also acts as a pilot tunnel exposing the actual geology to be encountered during cavern excavation and is also utilized for hydro-geological tests, grouting trials etc for assessment of grouting requirements in water curtain tunnel and the underlying caverns. Coring and borehole imaging in water curtain boreholes is also performed in select boreholes to collect relevant geological details of the rock mass that facilitates updation of the interpretative geological model.

A typical water curtain tunnel along with temporary pressurization manifold of pressure gauge is shown in Figures 9.4 & 9.5.

Figure 9.4: Water curtain tunnel

Figure 9.5: Temporary pressurization system for WC boreholes

9.2.3 Storage Caverns

Storage caverns are designed as U-shaped in plan, with an approximate "D" shaped cross section. It is connected directly or indirectly with a shaft that carry pump installations and the pump pit is located at the end of its one leg. Shafts are planned either as a single shaft both for intake and outlet of product transfer or as two separate shafts, one for intake and one for outlet of product. In consideration of the planned storage capacity, site condition and other aspects, each leg of the U-shaped cavern is designed to have a typical length of about 300m to 800 m; with maximum length of about 900m. The caverns are designed to have a span of 20 m, height varying from 20 m to 30 m and with a pillar width of 30 m between each leg of the caverns,

The caverns are planned to be excavated in multi stages involving top heading and two or three benches as shown in Figures 9.6 & 9.7. The top-heading is designed as 8 meters in height. Since the height of the cavern varies due to the sloping gradient of the floor, height of the benches also varies. Excavation of the heading and benches are undertaken either through full face excavation or in a combination of pilot and side slashing advance mechanism. Conventional drill and blast technique is used with horizontal drilling of blast holes for ensuring smooth control blast, reduced blast induced damage to the cavern wall and safe working condition.

Figure 9.6: Cavern heading & bench excavation

Figure 9.7: Photograph of complete cavern

The cavern roof is horizontal along the full length of cavern. The invert of the cavern unit is kept inclined from the intake to the pump pit to ensure a free flow of crude and also to facilitate dewaxing /desludging operations. In order to ease in construction sequence, cross tunnels are constructed between the caverns. These cross tunnels are further separated using concrete partition walls so as to achieve desired flow routing for crude oil circulation.

9.2.4 Shaft

The shafts are excavated from top down to bottom and offer possibilities of additional ventilation and emergency exit from the cavern.

The excavation of shaft starts from poor weathered rock conditions near surface until reaching down to fresh rock. Top section of the shaft is excavated by excavator & jack-hammer and drill & blast, if required. Rock shaft near the surface is stabilized using vertical & inclined rock bolts (spiles) and a reinforced concretering beam, in addition to the normal support. A concrete retaining structure is constructed above the rock shaft up to the finished ground level. These reinforced concrete structures are used to support the gantry crane rails as shown in Figure 9.8 and, at a later stage, for installation and support frame for the pipe sleeves in the shaft. The shafts below the shaft top structure are excavated by drill and blast method. Shaft is supported using typical rock support of rock bolt & shotcrete.

Figure 9.8 : View of shaft (top & inside)

9.2.5 Underground Mechanical & Concrete Works

In order to install pumps and instrumentation in shaft, pipe casings are installed in the shaft. These casings are used for installation of different types of submersible pumps, instrumentation and other process requirements. Installation of large diameter casings of length approximately 100 m requires construction of large supporting structures on the shaft top so that casings can be lowered in pieces. A reinforced concrete support framework is provided on top of shaft. In addition, structural supports to casings pipes are provided between the pump pit and the concrete barrier. Underground concrete works consist of concrete plugs in tunnel & shaft, separation wall in cross tunnel, concrete floor in storage area, backfilling of shaft, spillway in cavern, encasing and anchoring of hot water pipe etc. The main access tunnel plug (to be closed at the end of the cavern construction) is provided with a temporary manhole to facilitate access through the plug during casting of the concrete plug.

9.3 Underground Construction Activities

Construction of underground storage caverns involves excavation using drill & blast method, supports using rock bolts and shotcrete, casting of concrete barriers and walls, pavements and water curtain systems. Typical set of activities during drill and blast cycle are shown in Figure 9.9, which involves surveying, probing, drill & blast, mucking, scaling, geological mapping, rock support (rock

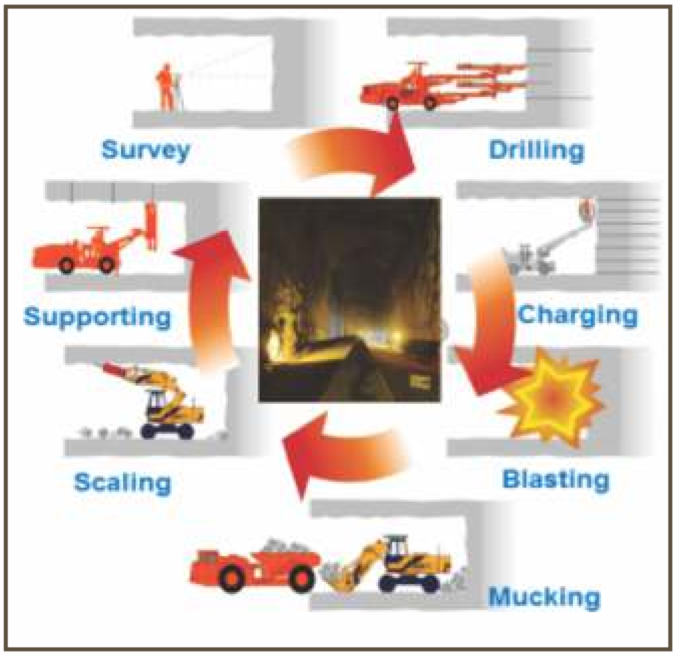


Figure 9.9 : Typical drill & blast cycle

bolt & shotcrete) and grouting as and when required.

9.3.1 Drill & Blast

Drill pattern is decided based on the requirement regarding smooth and control blasting, local geological features and optimization of the pull or advance of excavation. Drill pattern and charge concentration are continuously modified throughout the construction period, to suit the local geological conditions. Prior to start of major construction activities, suitability trials are carried out in similar geological conditions, to develop and finalize the drill and blast methodology. During the suitability trials, vibrations are also monitored in order to optimize the blast design and drilling pattern in accordance with blast quality requirements and blast vibration acceptance criteria.

Blast design consist of finalizing the location of blast, drilling pattern including diameters, spacing, depth & orientation of drill holes, types, strengths & quantities of explosives proposed for use in each hole along with delay for each blast, distribution of the charge in the holes, priming of each hole and stemming of holes, type, sequence & number of delays, delay pattern, blast quantities, methodology for disposal of misfired explosives etc. Controlled blasting is carried out to achieve minimum rock damage and a uniform surface at some of the critical locations such as shaft plug, barrier location etc.

9.3.2 Scaling & Mucking

Scaling is the first operation to be carried out after blasting, face ventilation and muck pile water spraying. All loose and likely to be get loosened rock pieces are scooped through scaling. The walls and roof are washed down to permit careful inspection of rock surface. Scaling is carried out by experienced rock workers in presence of geologist & safety in charge. Additional scaling (if required) is also carried out after excavation until final rock support is installed. An elevating platform is made available at all times for inspection and scaling for the tunnel and cavern.

All excavation muck materials are wetted to reduce dust. Mucking is carried out as per the construction methodology and the muck is disposed at the approved disposal area. Mucking equipment is sized in accordance with the tunnels to be mucked as well as the time constraints. In the case of oil storage caverns, the space available allows for the use of large wheel loaders with face shovels. The access tunnel on the other hand is more restricted owing to smaller dimension. The loaders use a side tipping action and haul the muck to a loading niche, which is blasted specifically for loading. In longer tunnels turning bays are also designed for the mucking trucks.

The haul roads in the project, including those in the tunnels and caverns are paved and maintained to reduce wear and tear on the trucks.

The surface of the spoil dumps is leveled and compacted for the same reason. Safety and quick turnaround are also considered on spoil dumps. Adequate arrangements are made to ensure that trucks are standing correctly when dumping and that they do not fall off the edge of the tip while reversing or dumping. The mucking operation is one of the most critical activity that have a significant bearing on the completion of the project as scheduled.

9.3.3 Geological Mapping

The excavation mapping is performed by experienced engineering geologists using the markings in portal, access tunnels, water curtain galleries, caverns and shafts as reference. The engineering geologists classify the rock mass by adopting pre approved rock mass classification system such as Q-system and consequent adoption of the design requirements for rock support which include shotcrete and pattern rock bolts. Depending on the specific site conditions involving combination of critical joint sets leading to a possible wedge failure, should there be additional requirements, the engineering geologist decides to install additional rock supports, as the need be including provision of spot bolts. The excavation mapping is performed and recorded as per the preapproved templates adapted to the actual excavation geometry.

As part of the excavation cycle, geological mapping of the rock surface is performed for the face, crown and both the side walls of tunnels and walls of shafts after each blast. Results of excavation mapping are presented in the form of geological face map, geological model, rock mass quality model, as-built maps, recommendation of rock support etc.

9.3.4 Rock Support

The support philosophy of all underground tunnels is based on staged excavation,

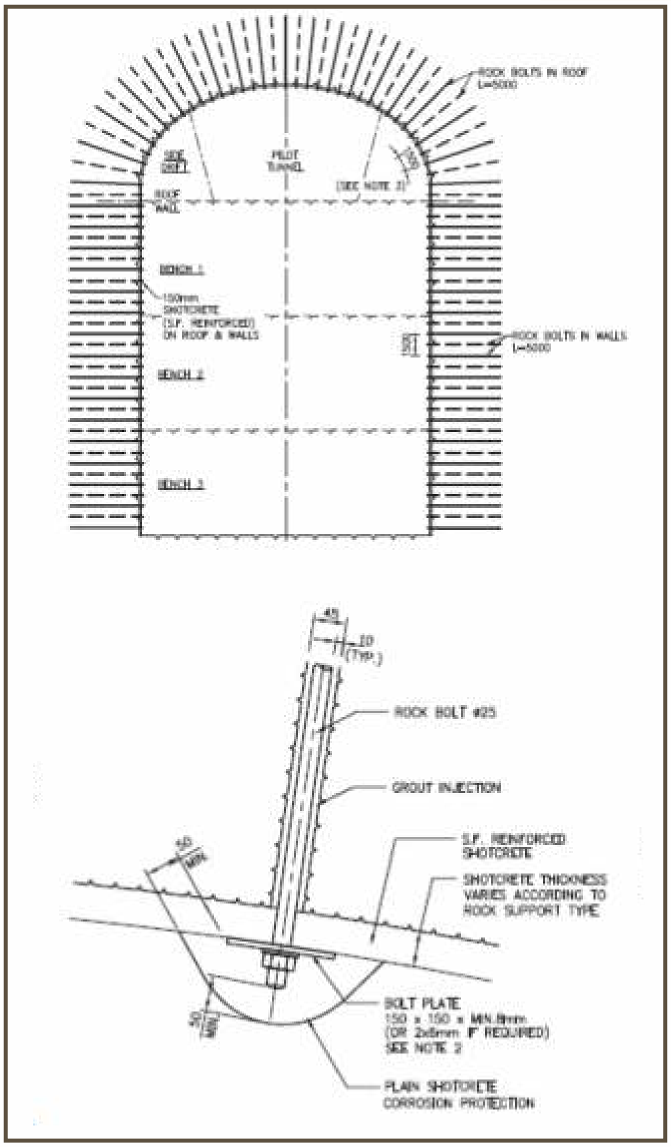


Figure 9.10: Typical cavern rock support

incremental installation of rock support measures and verification by monitoring. The rock support essentially consists of rock bolt of length around 4-6 m and fiber shotcrete of thickness 50-150 mm depending on the prevailing ground condition. Based on the actual geological, geotechnical and hydro-geological conditions encountered during excavation, modification of rock supports is done and implemented at site, thus leading to cost effective and practical rock support instalation. A typical rock support arrangement along with rock bolt detail for cavern is shown in Figure 9.10.

9.3.5 Rock Bolt

Rock bolts are essentially used to limit and control rock mass deformation around an opening so as to achieve safe and stable excavation. Rock bolt used in underground excavation works is an untensioned reinforcing element consisting of a rod embedded in a cement grout filled hole. The rock bolts are fitted with a plate and a nut at one end and are fully grouted with cement grout. Rock bolts are generally of 25 mm diameter and around 5-6 m long reinforcing bar with yield strength not less than 500 N/mm2. Modification in the length, orientation, spacing etc. of rock bolt are carried out to suit the encountered geological settings and variations in the cavern profile.

Rock bolts are installed along a specific pattern which depends on rock class assessment and comply with recommended and approved design, drawings and method statement. Spot bolting is adapted to local conditions, joint orientations, block size etc. and is installed as soon as possible just after the excavation. In case of difficult rock conditions, longer lengths of the rock bolts i.e 8-12 m are used for rock mass stabilization. Bolt length and angle of installation are finalised as per actual geological conditions and work under tension against most probable movements of any anticipated wedge failure.

As a requisite for the intended use of these excavation as hydrocarbon storage no borehole is left unfilled on completion of the construction beyond or outside the storage cavern perimeter. This requirement applies in particular to bolt holes drilled in the storage volume, which need to be fully encapsulated with grout or resin. In order

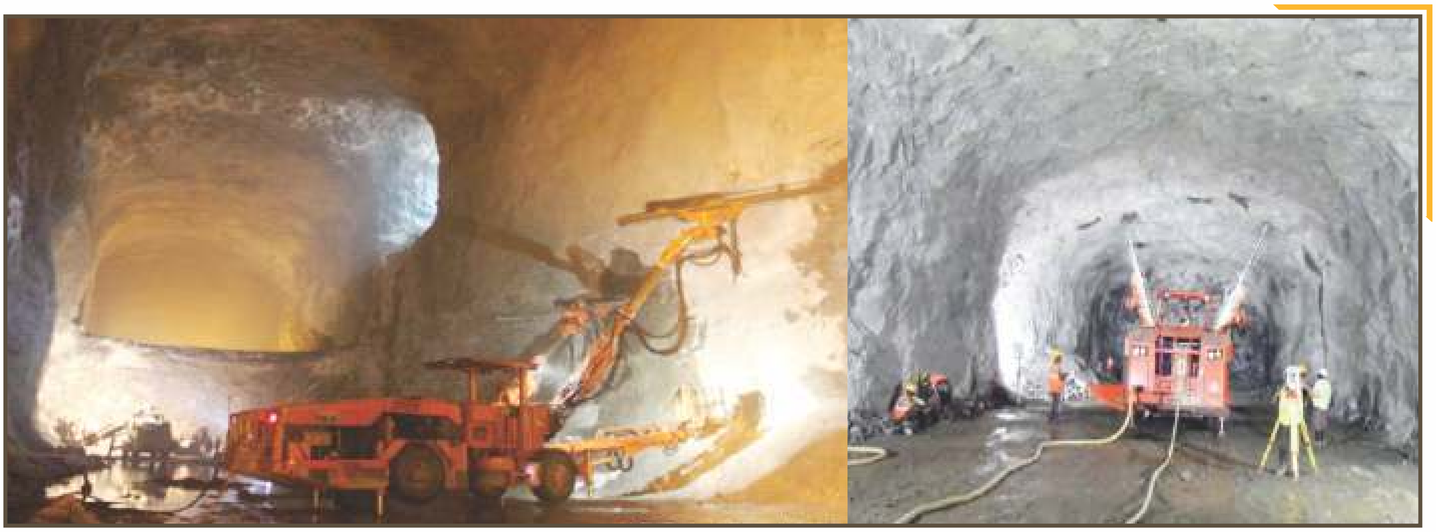


Figure 9.11 :Typical rock bolt installation on wall & roof of caverns

9.3.6 Fibre Reinforced Shotcrete

Shotcrete is defined as a mixture of cement, aggregate, steel fibre, water and accelerators mixed in design proportion. It is applied by spraying at high velocity from a spray nozzle on to a clean surface as shown in Figure 9.12, to form a layer of pneumatically applied concrete on that surface.The shotcrete is used for protection and supporting rock surfaces after excavation, to fill the cavities caused by over break or weathering. It further acts in conjunction with the rock bolts to provide structural stability.

Minimum fiber content for fiber reinforced shotcrete is kept not less than 50 kg/m3 of fresh concrete. The length of steel fibers is fixed as per the design requirements but is at least 6mm longer than the maximum aggregate size. Specialized fiber feeder equipment, consisting of a drum and screen mechanism that uniformly screens the individual fibers into the shotcrete mixture is used for getting uniform mixture. Different physical tests are conducted on shotcrete to determine compressive strength, setting time, slump test, fiber content, toughness criteria, density, bond strength test, in place thickness of shotcrete etc. Bond strength requirement between rock and shotcrete is kept greater than 0.5 MPa and between two successive shotcrete layers is kept greater than 1.0 MPa. Suitability trials are conducted in advance on prepared samples of shotcrete by taking out cores from sprayed mix and testing them for verification of physical properties.

All loose materials, debris, dirt or other foreign matter is thoroughly cleaned befor applying shotcrete to allow good bond between the rock and the shotcrete. Cleaning is generally performed by jetting, but slickenside surfaces of exposed rock are sometimes sand blasted. All holes and cracks are filled up to avoid any movement of the crack. Alignment control devices such as ground wires, guide strips, depth gauges, depth probes or forms are used to establish the limits and thickness of shotcrete placement. A thin initial coat of shotcrete is applied to the selected work area before start of layering of shotcrete. The work area is of such size so that the surface can be maintained wet with fresh shotcrete so that the initial set does not occur until final layering of the area is completed. Once the initial bonding or wet coat is applied to the entire work area, a second pass over the area is performed at a slower rate. After completion of the limited work area, rebound and overspray is removed from adjoining areas.

Shotcreting is laid to a maximum thickness of 50 mm and a minimum thickness of 25 mm in one operation. In case of a layer of shotcrete is to be covered by succeeding layers, the successive layer is laid only after initial setting of the previous layer. All loose materials are removed by broom or hacking and surface is cleaned with an air water jet. The surface is thoroughly sounded by hammer for hollow areas resulting from rebound pockets with poor bond. Such areas are cut and replaced during second layer deposition. Where shotcrete is used with wire mesh, the mesh is tightly anchored to the rock support. Minimum diameter of wiremesh is taken as 5 mm which varies as per reinforcement requirement.

Figure 9.12: Shotcreting of cavern wall & roof in progress

9.3.7 Probe Holes and Grouting

Probe holes are required in advance of blasting of work face to detect weak zones / water bearing zones in the rockmass that would require grouting. Requirements for pre-grouting are furnished during construction based on the geological mapping of the water curtain gallery and the predictive geological model prepared from face mapping of underground excavation.

Pre-grouting by cement based grout is the preferred method of achieving the required tightness of the rock mass. The optimal length of drilling and the most suitable grout mix design are decided on site through an iterative process. Whenever seepage is detected after a blast, systematic grouting of the excavated zone is carried out. In the event of excessive seepage exceeding the design critical value, post-grouting is planned to be performed.

9.3.8 Geotechnical Monitoring

Displacement measurements are carried out in underground excavation to monitor rock mass behavior during construction of the caverns and for in-situ verification of design assumptions. Two different types of instrumentations are used to measure the deformation i.e. convergence monitoring of rock mass surface by optical targets and internal rock mass deformation by set of extensometers. Convergence monitoring is carried out all along the cavern length spaced at regular intervals of 15-20 m. However, deformation measurement by extensometer is only limited to geological hot spots as observed during investigation and subsequently substantiated during the excavation of other components (access tunnel and water curtain tunnel), specifically from the excavation of water curtain tunnel which is located 20 m above the cavern crown.

Monitoring starts immediately after placement of the rock support and observation continues daily upto 15 days after excavation or till stabilzation. After that, weekly readings are performed until the end of the construction period, provided that movements have converged. In the case of continuous movements, daily readings of the monitoring results are continued. Lifting equipment with a reach up to the crown of cavern is required to be available on site for installation, maintenance and re-installation of optical targets.

9.3.9 Dewatering

The dewatering of excessive water either from regular seepage or from ingress of water during construction through water bearing structure is carried out using a set of pumps and drain lines laid along the tunnel and cavern floor. Excessive seepage often result out of a sudden outburst of water through holes of any excavated faces as shown in Figure 9.13. Such volumes of water collected inside the tunnels or caverns as well as those stranded on the ramp become detrimental to the concrete floors and increase wear and tear of haulage trucks used in construction

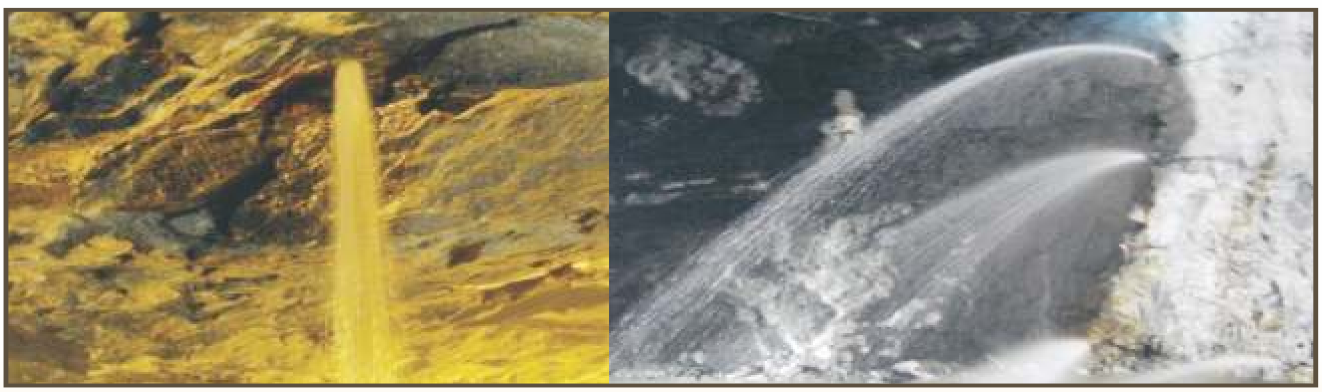


Figure 9.13 : Encountered seepage through water bearing structures

9.3.10 Ventilation

Ventilation inside tunnels and caverns is required to remove fumes from diesel powered equipments, rock particles suspended in air and gases resulting from the detonation of the explosives during construction so as to provide a safe working environment. This is achieved solely by a continuous supply of fresh air to the working face using air ducts as shown in Figure 9.14.

Figure 9.14: Ventilation ducts inside access tunnel

9.3.11 Muck Disposal

Figure 9.15 : Overall layout showing dumping sites & muck disposal sites

A speedy and continuous dumping of muck generated in large quantity is a crucial activity during excavation of underground storage caverns. Therefore a systematic plan is made for logistics of the muck disposal and to maintain the haulage road and disposal area as shown in Figure 9.15 in such a way that muck disposal can be performed at all times. A muck disposal plan consider the quantum of muck to be generated, swelling factors, dumping areas and possible re-utilisation of muck as construction material etc. The muck disposal is carried out with required safety precaution and maintaining suitable slopes for haulage. Re-working of the muck is beneficial for the proposed excavation works as it offers both space availability and possible revenue

9.4 Storage Inventory

Given the intent of inventory stockpiling of crude oil, it is imperative that accurate volume of the large underground storage caverns need to be established. With the considered dimensions of 30m high, 20m wide and 300m to 900m long caverns along with the uneven cavern profile owing to acceptable overbreak, it is exteremely difficult to establish the storage capacity of each leg of the caverns by any manual method. Further, it is also essential to have an acceptable accuracy level for such measurements as the inventory holding of crude oil involves a significant capital investment much beyond the cost of creating these facilities.

Therefore, the most advanced 3 D laser profiling survey technique is used to scan the undulating surface of the cavern from the concrete paved invert through the cavern side wall and the roof. Through this digital scanning method, a complete 3 D volumetric model of the storage caverns is created to derive the overall volume of the caverns and the total storage capacity.

In a similar manner, as in case of Tank Fram Management System (TFMS) for above ground conventional steel tankages, strapping tables are derived for the storage caverns as well. With reference to the cavern invert, volumetric calculations are used to derive the inventory holding for each corresponding incremental increase of depth of crude oil stored in the caverns. Through automatic level sensors housed in the shaft pump pit, the storage inventory is measured during crude oil filling, evacuation and operational storage holding period. These level instrumentation mechanism is integrated with the Emergency Shutdown Valves & the Tank Farm Management System for crude oil metering, operation and custody transfer purpose.

9.5 Health, Safety & Environmental Aspects

HSE policy & objectives are developed to demonstrate commitment to ensure health, safety and environmental aspects of the underground rock cavern storage projects and the line of operations. A comprehensive, planned and documented system for implementation and monitoring of the HSE requirements are evolved at site. The monitoring for implementation is done by regular inspections and compliance to the observations thereof.

The normal safety precautions that apply during ordinary construction work are also applicable to underground work. However there are several areas which need particular emphasis:

a) Working with explosives.

b) While working under loose rock or unstable ground, scaling is a vital operation. Wherever there is uncertainty, strong measures are used. Therefore, experience is absolutely critical for decision making in these circumstances.

c) Working in close proximity to mechanical equipments in confined spaces is always fraught with danger. Proper guardrails, screens are installed and caution signage prominently displayed.

d) Air Quality – To ensure optimal functioning of men and machinery, the quality and quantity of fresh air is monitored and maintained.

e) Noise – High noise levels have an extremely detrimental effect on the working environment, particularly in confined spaces. Care is taken to adopt adequate measures to minimize noise from drilling and where compressed air is used.

f) Fire and Smoke – All necessary steps is taken to protect against fire hazards and smoke extraction.

g) Communication and Access control system – A key consideration, a robust communication system and access control system is also maintained at the tunnel portal and records of all personnel working underground at any given time is reported on a continuous and sustained basis.

h) Electricity – Sizing of switchgear and cables are based on actual and prospective loadings. All circuit breakers / fuses are rated for 110% of normal load and cables are properly terminated.

i) Emergency exit – Considerations and provisions are provided, in the pump shafts for emergency exists, should the need arises on as early as possible basis.

9.5 Equipment for Underground Construction

Equipments used for underground construction including that for drilling, blasting, scaling, mucking, conveying, rock support, concreting, dewatering, hoisting, ventilation, illumination, testing and monitoring etc. are planned as per the approved construction methodology and is made compatible with the geological conditions, tunnel dimensions, construction sequence, time schedule and safety requirements.

The underground works require a large excavation within a short construction period. Typical construction schedule for 1.5 MMT storage is around 48 months. This tight construction schedule requires careful planning, deployment of necessary equipment and qualified manpower and strict compliance of all quality and safety procedures. The major construction equipment for a 1.0 to 1.5 MMT storage facility are listed below and a pictorial view is shown in Figure 9.16.

i. 2/3 boom jumbos (5 numbers)

ii. 20m^3/hr Shotcrete robots (4 numbers)

iii. 6.5m^3 Loaders (4 numbers)

iv. 25 MT Dumpers (25 numbers)

v. Drilling rigs for water curtain bore holes (3 numbers coring/destructive)

vi. Grouting equipment (2 numbers)

Figure 9.16: Pictorial depiction of drilling jumbos and dumpers

As part of a well structured excavation schedule and construction progress, the following excavation production rates are achieved per month for a project site:

i. Access tunnels : 30,000 m^3

ii. Water curtain tunnel : 6,000 m^3

iii. Cavern heading : 60,000 m^3

iv. Cavern benching : 1,00,000 m^3

9.6 Above Ground Construction Works

The above ground construction works include the above ground process and utility facilities and a typical set of above ground facilities are shown in Figures 9.17 and 9.18. Construction of above ground process facilities are well established as part of the refinery projects and have been dealt at length by many such project compendiums thus not presented herewith. However, there are some specific items which pertain to underground storage caverns. These include installation of submersible pumps and instrumentations in the shafts. After completion of plugs and dismantling of the water curtain system, the caverns are tested for tightness by carrying out a Cavern Acceptance Test (CAT) using air at the desired operating pressure. After successful completion of the CAT, the caverns are inerted with nitrogen before crude intake and commissioning. The testing of large caverns with capacities exceeding 1.0 million cubic meters is a complex task requiring large compressors and accurate instrumentation to establish tightness of caverns. After the testing, intertization of such large caverns with nitrogen requires large capacity nitrogen generators.

Figure 9.17 : Compresser unit arrangement and instrumentation facilities

Figure 9.18: View of above ground process facilities

A Navratna Company

9.7 Project Execution and Schedule

Like any other project, planning and execution schedule forms a key consideration for underground storage projects. However, owing to the subsurface uncertainities associated with these large underground storage projects, due consideration is made for the unlikely events of the project encountering adverse geological conditions. Similarly, intergartion of the storage installations to the existing refineries or pipelines or port infrastructures etc. entail pipeline connectivity which also comes with additional consideration of Right of Use enroute the pipeline route. Often, this forms a major bottleneck along with the land acquisition for the storage facilities. Therefore, due consideration is also kept for such activities which comprises of liasioning with state govt. and local bodies for smooth transition of land availability.

Further like any other project, underground storage projects require a host of statutory permissions such as Environmental Clearances, Consent for Establishment, Mining, Blasting & Storage Licenses, Forest Clearances, etc. These aspects are considered under the Pre-Project activities and needs to be taken up prior to commencement of execution of project.

With appointment of a Project Management Consultant (PMC); a duration of about 15 months are considered for performance of the aforesaid pre-project activities and pre-award activities which include pre-bid engineering, pre-qualification, evaluation and engagement of contractors. Depending on the quantum of works and nature of project different split scope matrix with clearly demarcated interface is developed. Broadly, for underground rock cavern project the underground civil works including cavern shaft and in cavern mechanical works are designated as Part A contract. The above ground process facilities including pumps and in shaft equipments are designated as Part B contract. The pipeline intergartion along with offshore oil terminals if any are clustered under Part C contract.

In case scenarios, for a large underground storage installation of say 2.5 MMT storage capacity the underground excavation works contract is split into two mutually exclusive parts and designated as Part A1 & A2. Being underground excavation works, these part of the projects are executed under a design and built item rate contract, which enables to take care modifications and changes in the ground conditions leading to escalated time & cost of the project. The Part B & Part C contracts are taken up under the normal route of project execution say either through EPCM or EPC / LSTK mode.

For a typical underground storage cavern project of 1.0 to 1.5 MMT capacity, around 42 to 48 months of duration is envisaged for the works and 3 to 6 months for commissioning. Summary of duration against major activities is listed below:

i. Mobilization of equipment, personnel, etc.: 3 months

ii. Underground Excavation : 30 months

iii. UG Concrete & Mechanical Works : 9 months

iv. Testing & Pre-commissioning: 3 months

v. Aboveground Process Facilties: 24 Months

vi. Pipeline Integration : 24 Months

vii. Commissioning : 6 months

In addition, a time duration of around 15 months is considered for pre project activities that include supplementary site investigation, performance of basic engineering, preparation of bid document, evaluation and award of bid packages for underground civil works. Concurrently, the above ground process facilities and pipeline integration system are taken up such that the final commissioning is completed within the envisaged project schedule. In addition, a contingency of additional six months should be considered for subsurface uncertainties.

PROJECT ACOMPLISHMENTS

H.P.S.Ahuja

CHAPTER - 10

10.1 Introduction

Taking the oil security concerns of the country, the Government of India approved the setting up of Strategic Petroleum Reserves (SPRs) of 5.0 MMT (36.65 Million Barrels) in January 2004. To build and operate these SPRs, a Special Purpose Vehicle (SPV), was created. The SPV which was initially a subsidiary under Indian Oil Corporation Limited(IOCL) was named Indian Strategic Petroleum Reserves Ltd. (ISPRL). In January 2006, the financing mechanism for funding the building of the reserves was approved by the Government and ISPRL was made a wholly owned subsidiary of Oil Industry Development Board (OIDB).

Subsequently, the total capacity of the SPRs was increased to 5.33 MMT and the reasons are covered later in this Chapter. The SPRs are at Visakhapatnam (1.33 MMT), Mangalore (1.5MMT) and Padur (2.5 MMT). Their supply zones are shown in the Figure below:

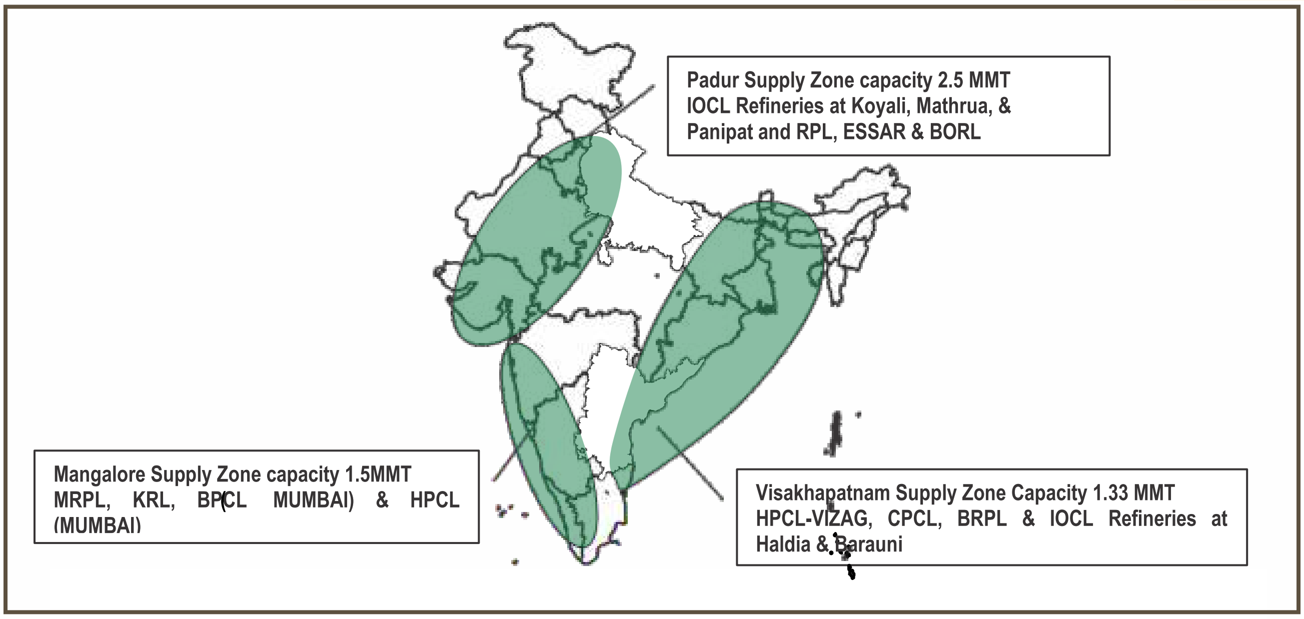


Figure 10.1: Locations of SPRs of Phase-I storage program along with supply zones

For the construction of the SPRs at all three locations, ISPRL engaged Engineers India Ltd. (EIL), as the Project Management Consultant (PMC).

When the decision to build the reserves was taken in 2004, the total crude oil storage capacity in the country was 7.261 MMT. Thus, ISPRL was required to add almost 69 % of storage and as per the government approval, the projects were to be completed in a period of 6 years. The projects not only required creation of India's largest underground cavities but also required excavation rates, which were well beyond those ever achieved in this country. Apart for the technological and logistical challenges there were the difficulties of acquiring land and obtaining environmental clearances for technologies which were new to the country. The implementation of these exceptional projects gave invaluable experience to

the engineers and geologists of the country. The underground works are considered by all those who have had the opportunity to see them, as marvels of engineering.

The major achievements of the Projects of ISPRL are as under:-

1. The caverns are the largest underground caverns excavated in the country. To create these storages, approximately 22 million tons of hard rock was excavated.
2. The total tunneling carried out is in excess of 29.8 Kms. At Padur which is the largest storage facility constructed, over 13.5 Kms of tunneling was competed in a span of 40 months.
3. The quantum of explosives used for excavating the storage facility is in excess of 8400 tons.
4. 86 Kms of bore holes have been drilled for creating the water curtains and the monitoring bore holes.
5. 3,75,000 cubic meters of concrete and shot crete has been used in these projects which interestingly is more than the concrete used to build Bruj Khalifa, currently the world's tallest building.
6. The quantum of steel used is over 36,500 tons which is 40 % more than the steel used to build the famous Howrah Bridge in Kolkatta.
7. These Projects has had an envious safety record and specifically the Padur project site have achieved 14 Million safe man hours.
8. The employment generated by these projects for skilled and semi skilled manpower is over 52 million manhours.

During the construction of the reserves, some National records were set in underground excavation:-

1. At Visakhapatnam on 29th April 2010, 9184 cu mts of hard rock excavation was excavated in a single day
2. At Mangalore in October 2012, 1,21,263 cubic meters of hard rock was excavated in a single month.
3. At Padur between February 1st 2012 and April 30th 2012, 6.16 lakh cubic meters of excavation was carried out.
4. ISPRL was awarded the Asia Oil & Gas Leadership Awards under the category Best Employer Brand Organization (Oil Storage) in February 2014.
5. Received National Safety award from National Safety Council for Visakhapatnam.
6. Received Best Managed Industry Award from Hon. Chief Minister of Andhra Pradesh for Visakhapatnam.

Despite the various hurdles faced, these projects have matched the global benchmarks of gestation period which ranges from 7-8 years from award of job.

10.2 Adopted Execution Philosophy ensured Excellent Bidder Response

Initially a single Lump sum Turnkey (LSTK) approach for execution of the projects at each location was proposed. With this approach, it was necessary for contractors to form consortiums, as no Indian contractor had all the experience required for executing the job. The LSTK contractor apart from project management skills, was expected to have experience in major underground works as well as experience in building and commissioning of oil terminals. The contractor was also expected to have knowledge of hydrogeological containment which was very essential for containment of the product within the caverns. No Indian contractor even of the stature of Larsen & Tubro (L&T) or Hindustan Construction Company (HCC) had such experience. These companies were forced to form consortiums for bidding for the ISPRL projects.

South Asia LPG company Private Limited (SALPG) a joint venture between HPCL and TOTAL had faced a similar experience at the start of their project. The Single LSTK approach was unsuccessful and resulted in an offer which was almost 80% higher than their estimates. SALPG had thereafter modified the execution philosophy and split the job into two parts, so that Indian bidders could participate in the bidding process.

Based on the experience in SALPG, it was decided to change the execution philosophy for the ISPRL projects. The execution philosophy was thus revised, and the underground civil works would be executed through an item-rate contract and the above ground works would be executed through another contract on LSTK basis.

On 2nd March 2007, the execution philosophy was further revised and the hydrogeological containment requirements was removed from the scope of the contractors and added to the scope of EIL/Back-up consultant.

In line with the revision in execution philosophy, through a global Notice Inviting Tender (NIT) bids were invited for prequalification of the bidders for the underground civil works of Visakhapatnam. The bidder response was excellent and 13 parties responded out of which 10 were selected for the next round of evaluation. The excellent bidder response also helped ISPRL to adopt the Reverse Auction method for determining the L1 bidder for the Visakhapatnam project and later for the Mangalore and Padur projects also. This offered a significant savings in the Capital Investment for these projects.

10.3 Enhancement in Cavern Capacity at Visakhapatnam.

In the case of underground storage caverns, geological surprises cannot be ruled out. Presence of dykes and other faults can increase the cost of cavern construction, as added rock support/ grouting/ shotcrete might be necessitated under such conditions. Thus there is no certainty that an identified site will ultimately be able to accommodate the desired level of storage capacity at an economical cost. Keeping this is view, when the first project at Visakhapatnam was taken up, it was decided that the capacity would be increased to the level that the site permitted. By doing this ISPRL was ensuring that even if the conditions at other sites were not favorable, the target of creating 5 MMT of crude oil storage cumulatively at the three sites at an economical cost could be achieved.

Further, the marginal cost for additional capacity in underground caverns is generally low, as the fixed cost gets spread over larger volumes. The site permitted addition of another gallery of 840 meters length. The addition of the gallery would increase the capacity by 33%. It was estimated that an increase in capacity by 33% would lead to only 15% increase in project capital cost, if the process parameters, such as pumping-in and pumping-out rates, were not changed. ISPRL then decided to change the capacity.

Subsequently it was established that the geology at Mangalore and Padur sites were favorable for construction of the capacity envisaged in the DFR and ISPRL would therefore end up with storage in excess of 5.0 MMT. It was decided later; to share the additional capacity with HPCL

10.4 Introduction of Geotechnical Reference Conditions.

An important concept that was introduced for the first time in India was the concept of Geotechnical Reference Conditions (GRC). Underground works usually involve a number of uncertainties with respect to the type of geology that can be encountered during excavation. Although prior to commencement of the works, data is obtained about the geology of an area, it is not feasible to map the complete area. It has been observed that in large projects, the geology could vary drastically from one section to another. In view of the uncertainties, contractors load their bids for the risks. The risks perceived by different contractors can vary drastically.

Basic philosophy of the GRC was to ensure that all bidders

can base their estimates on a well-defined set of site conditions with assurance that equitable compensation will be made, if changed conditions were encountered. By providing the bidders with the GRC, ISPRL could expect the lowest reasonable bids with a minimum of contingency for unknowns

The GRC was an interval of values determined by ISPRL in association with EIL and the Foreign Back up Consultant (FBC). All variations of a relevant parameter within this interval were at the Contractor's risk. If the actual geotechnical conditions encountered were found to be outside the reference frames set out, the contractor was entitled to compensation for extra cost, extension of time, or both in accordance with the provisions of the contract. The amount of compensation would be based on the difference between the actual value of the parameter and the nearest limiting value of the relevant reference frame, or the difference between the actual value and the baseline value, as relevant.

10.5 Reverse Auction Process for Procurement

One of the major achievements of ISPRL has been the successful implementation of Reverse Auction for determination of the L1 bidder for its contracts. While Reverse Auction was a tool which has been used successfully in the past, by a number of organizations, to the best of our knowledge it was never used for item rate contracts, having a large number of items with individual estimates with respect to the quantities to be executed.

Before the Reverse Auction method could be finalized, it was necessary for ISPRL to address a number of issues which included; items like compliance to CVC guidelines, security of the process, avoidance of speculative bidding, ensuring bidders quote only one figure despite it being an item rate contract.

All these issues were addressed, through some innovative methods. Presentations were made to CVC officials and after their concurrence, the Reverse Auctions were conducted. Speculative bidding was avoided by providing the GRC to the bidders. Diesel was made a free supply item with a certain limit. Escalation was provided for steel and no deviation to the bidding documents was permitted.

ISPRL/EIL officials were issued commendation letters by Ministry of Petroleum and Natural Gas for this initiative. By adopting Reverse Auction for determining the L1 bidders, ISPRL has been able to save a significant amount in the capital investment.

10.6 Rock Debris, a Major Issue in Cavern Projects.

Rock debris is a major issue in large cavern projects. The quantum of rock debris being generated by the various projects is as follows:

- Visakhapatnam : 5.4 million tons
- Mangalore : 6.3 million tons
- Padur : 9.8 million tons

The rock debris generated during these projects can fill 230 acres of land to a height of 10 meters (32 feet). Rock debris disposal is therefore a major issue in rock cavern storages. The generation is maximum during the bench excavation phase. During this phase the quantum of excavation can be as high as 10,000 tons per day by each contractor.

10.7 Key Important Leanings

i The time period of 6 years provided for the execution of rock cavern storage projects is very tight if, land acquisition and statutory clearances are included in it.

ii Geology of underground works can change drastically even in geologically stable areas. Adequate provisions need to be kept in the contracts and the budgets to address change in geology.

iii It is advantageous to pave the access tunnels with concrete. Although this is an additional expenditure, it more than compensates by enhancing the safety of the site and speeding up the underground works.

iv Muck (rock debris) disposal can become a major impediment during the execution of a rock cavern project and needs to be addressed at the conceptual phase. Permission to be obtained for Crusher plant installation and operation, along with the initial statutory approvals for the project.

v A rock debris height could reach up to 50 meters on the project site to be specified in the UG contract. A specific disposal site should be identified before commencement

vi Large compartments can save costs and are excellent for strategic reserves, but can be a problem if commercial use of the caverns is thought of.

10.8 Knowhow Dissemination

Given the large and complex nature of the projects, along with associated sub surface uncertainties, the teams from ISPRL and EIL could gain excellent insight about the various nuances that are encountered while executing underground rock caverns projects. The learning and hands on experiences have been captured in the form of several technical papers and presentations during the course of the project execution. The teams of ISPRL & EIL have been privileged to be associated with these coveted underground projects and having undertaken the Phase - I storage program are best poised to take up the underground projects envisaged under Phase - II storage program: likely to be initiated by Government of India.

PART -II

Alternate Underground Storages


KATO

UNDERGROUND CONCRETE TANKS

A.Usmani, A.Nanda & C.Singh

CHAPTER - 11

11.1 Introduction

As per international practice followed by import dependent countries, strategic petroleum reserves are created in large underground storage facilities which are economical, safe and secure for a large inventory of crude oil. Location and type of such underground storages are decided depending upon the geological setting and proximity to an existing refinery and pipeline network, Storage of crude oil in large underground concrete tanks is one such alternative. The principle of storage of crude oil in underground concrete tanks system essentially employs primary containment by underground monolithic reinforced concrete and secondary containment by an external HDPE membrane. In the unlikely event of any leakage of crude oil from the concrete tank, the secondary containment ensures collection of the leaked crude oil without polluting the surrounding ground terrain. In order to ensure that the tank is impermeable, an acrylonitrile internal coating is also applied to the floor and walls of the tank.

11.2 Underground Concrete Tanks

Underground concrete tanks are normally square in plan measuring 150 m to 200 m in cross section and around 26 m deep in the centre and 8 m deep at the edges as shown in Figure 11.1. Concrete tanks mainly consist of floor slab and roof slab directly resting on the column heads spaced 8000 mm apart. Edges of concrete tanks are made up of retaining walls of 8 m height as shown in Figure 11.2. Tanks are covered at the top with a fill material or aggregate layer of 1 m.

Construction of tanks commences from excavation of natural soil or rock followed by construction of floors and

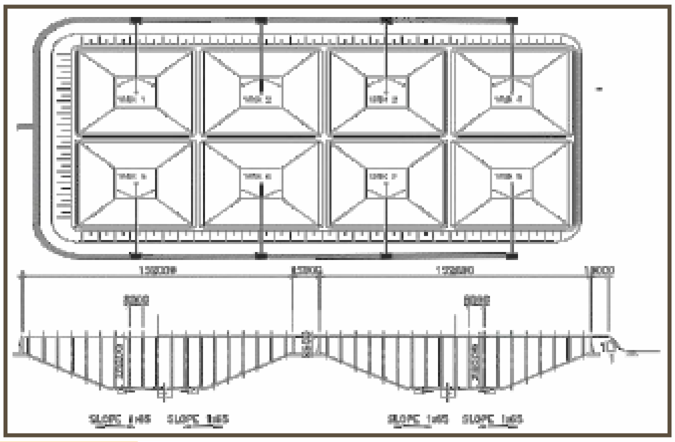


Figure 11.1 : Schematic plan and section of storage tanks

erection of long slender columns without any joints as shown in Figure 11.3. The sides of the tanks are made of 1000 mm thick retaining walls. The liquid and vapour containment is ensured using 2 mm thick HDPE membrane which completely envelopes the tank wrapped all around the tank periphery. A porous no-fines layer is constructed below the tank (laid on the HDPE membrane) which, in the event of the first containment suffering a leak, comply with the EPA requirements for monitoring, and collect the crude oil for back filling it to the tank.

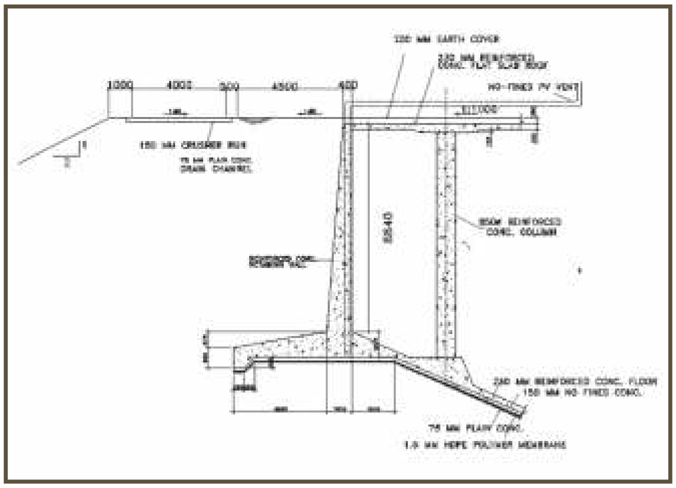


Figure 11.2 : Detailed section of retaining wall

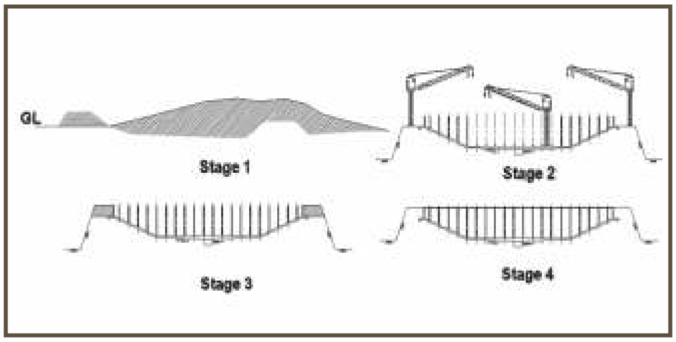


Figure 11.3 : Different stages of concrete tank construction

11.3 Site Investigations

A major requirement for underground concrete tanks lies on availability of weathered rock or hard rock within the upper few meters of ground profile. This ensures that no problems with either of bearing capacity or settlement are encountered for the portions of the tank which are founded on these ground profile. Different types of site investigations are planned for designing underground concrete tanks.

Geotechnical investigations carried out at site mainly include boreholes to a maximum depth of 30 m, standard penetration test in soil and decomposed rock in the upper portion of each borehole, electrical resistivity tests as well measurement of ground water levels using simple test and-pipe piezometers in selected boreholes. In addition some laboratory tests for finding basic index properties of soils and petrographic analysis and strength testing on rocks is also carried out.

The main geotechnical concern for an underground concrete tank site is obtaining sufficient quantity of material suitable for the construction of the integral embankments and specifying methods of placing and compacting this material which will minimize settlement of these embankments after completion of tanks. The weathered rock material available at a site is broken down to a granular material that can be placed and compacted conventionally in layers using a performance specification based on dry density and moisture content. In case of weathered material not available, blasting for excavation is undertaken. The blasted material is processed (by screening and or breaking) to produce a fill material that is placed as a rockfill.

In case when the tank site is underlain by good rock at shallow depth, bearing capacity and settlement of the foundations becomes secondary considerations and the most critical foundation design parameters pertain to stability and compressibility of the integral bunds. The stability of the bunds is controlled by shear strength of the rock fill to be used in the construction of the embankments and the magnitude of the horizontal displacement that is likely to be encountered during a considered earthquake eventuality. The compressibility of the bunds is controlled by compressibility of the rock mass (drained elastic modulus) which is controlled to a large extent by construction procedures including grading and selection of rock and compaction methods. Some of the main geotechnical parameters derived from these investigations include allowable bearing pressure on rock below tank floor, allowable bearing pressure on rock fill of integral bunds, elastic modulus for rock, elastic modulus of compacted rockfill, modulus of sub grade reaction and angle of internal friction of rock fill material.

11.4 Analysis and Design of Concrete Tanks

The principle of storage of oil in underground concrete tanks essentially employs primary containment by underground monolithic reinforced concrete tanks and secondary containment by external membrane manufactured from HDPE. The tank structures are designed in accordance with IS456 (Code for Practice for plain and reinforced concrete), and the 28 day cube strength of concrete is taken as 40 MPa for reinforced concrete work. Where ever required IS 3370 (Part I and II – Code of practice for concrete structures for storage of liquids) and BS 8007 are used as applicable codes. Design is carried out in accordance with IS 3370 where crack width is 0.2 mm for liquid retaining portion of the tank. The tank floor, walls, columns and roof slab act as monolithic reinforced concrete throughout. The connection between roof slab and retaining walls is also monolithic construction. The concrete mix, based on use of sulphate resisting cement, ensures complete resistance against either chemical or corrosion attack occurring in main tank structure.

For underground concrete tanks a water based acrylonitrile or acrylic co-polymer coating is also applied based on research carried out and practical experience gained from tanks constructed earlier in South Africa. The coating requires careful application with regard to surface preparation and removal of dust and laitance prior to application. The coating is applied in the layers of a primer, undercoat and a topcoat. The external polymer membrane used is an extruded high density polyethylene sheet (HDPE) having an average thickness of 2.0 mm.

During analysis a maximum wave height of 1 m from trough to crest at the sides based on the available model studies is considered for analyzing wave action that could develop as a result of sloshing of the oil in tank during earthquake. The construction of floor of the tank is monolithic reinforced concrete slab designed in accordance with IS 3370 & IS 456. The design ensures that the slab is structurally adequate to cater for the bending moments and stresses arising from column loads, settlements and differential settlements as well as limiting cracks (0.2mm) due to shrinkage and thermal effects.

Design of the retaining walls is mainly carried out with horizontal earth pressures from the embankment fill and horizontal liquid pressures from water and crude oil and comply with IS3370 & IS456 including limiting crack widths to 0.2mm. Underground concrete tanks are leak-proof and designed as uncracked section as per IS: 3370 assuming liquid up to the full height of the wall irrespective of provision of any over flow arrangement. Parts of such structures not coming in contact with liquid are designed in conformance to IS: 456 except ribs of beams of suspended floor slabs & counter forts of walls (located on the side remote from the liquid) and the roof which is designed as uncracked section. No increase in permissible stresses in concrete and reinforcement is allowed under wind or seismic conditions for such structures.

The support columns are designed for roof dead and live loads, construction vehicle loads and seismic loads. The columns are cast in a single lift to avoid construction joints. The roof structure is designed as monolithic reinforced concrete slab 300 mm thick with drop panels at column locations. The joints between roof slab and retaining walls are also monolithic construction. The roof is designed for dead load of slab, 250 mm of earth cover, live loads, operating vapour pressures and limiting the crack widths to 0.2 mm.

Different loadings considered for all structures (irrespective of the material employed for construction) include Dead Loads (weight of floors, roofs, partitions, stairways and fixed service), Equipment Loads, Live Loads (as per IS: 875), Horizontal liquid pressure from water and crude oil, Horizontal earth pressure from embankment fill assuming granular material, Operating Loads (equipment during plant operation), Wind Loads (as per IS:875) Seismic Loads (as per IS: 1893), Hydrodynamic Forces (takes into account fluid movement inside the tanks), Impact & Vibratory Loads and Soil and Hydrostatic Pressure. The factors of safety against uplift are taken as 1.2.

11.5 Above Ground Process Facilities

For the performance of storage facilities, including loading and unloading of stored product, inventory management etc. the following above ground installations are envisaged:

a. Evaporation Ponds
b. Metering Skids
c. Diesel driven Booster Pumps
d. Diesel driven Can Pumps
e. Pipe way above ground
f. Buried Pipelines and Electrical Cables
g. API Oil Separators
h. Fire Water Tank
i. Fire Water Pump House
j. Fire Station
k. Control Room
l. Standby Power Generator
m. Outdoor Switch Yard

n. Sub Station Building
o. Compressed Air System
p. Nitrogen Plant
q. Maintenance Workshop
r. Diesel Oil Tanks and Pumping System
s. LPG Mounded Storage and Pumping
t. Storm water reservoir
u. Raw water tanks and pumps
v. Drinking water sump and pumps
w. Flare Stack
x. Foam Generation Units
y. Boiler
z. Jet Mixer

Case Study 11.1: Underground Concrete Tanks at Rajkot

The presented case study outlines the detailed feasibility studies undertaken for the proposed underground concrete tank storage facilities near Rajkot, Gujarat.

The selected site for the storage is located near village Gavridad, in the Rajkot District of Gujarat. The site is approx. 15km North of Rajkot and is approachable by State Highway No. 24 connecting Rajkot and Morbi, The project site is located on the right side of the village road connecting Gavridad and Anandpur. The nearest railway station is Rajkot, located nearly 20 km from the project site.

The site is located about 100kms from the existing COTs of M/s IOCL, BPCL, Reliance and Essar Oil at Vadinar, thus with a proposed new pipeline, the storage facility will have access to all the offshore oil terminals (SPM) located off Vadinar for filling of tanks. Through these pipeline the feedstock can be provided to all the refineries in the region. Through existing pumping station of Salaya Mathura crude oil Pipeline (SMPL) of IOCL, which is around 5 Km from storage site, will be used for further connectivity for usage of other refineries viz. Mathura and Koyali.

A study to this effect was carried out in year 2000 by EIL along with M/s TVE of South Africa engaged as the Foreign Back-up Consultant (FBC) and a Detailed Feasibility Study Report was prepared. The present study is an updation of the DFR 2000, where in the storage tank configuration and the associated pipeline integration scheme have been modified allowing more flexible operation and a wider integration to the existing facilities such as pipelines and refineries. While, EIL has undertaken the study related to the present DPR, M/s TVE of South Africa has reviewed EIL's basic design and provided inputs with regard to the storage technology, as the FBC.

Based on the suitability of underground concrete tank storage facilities at the selected land parcel, a total storage capacity of 2.5 MMT has been planned at Rajkot which would cater to the refineries located in the western regions of the country.

The principle of storage of crude oil in the UCT storage system essentially employs primary containment by underground monolithic reinforced concrete tanks and secondary containment by an external HDPE membrane. In the unlikely event of any leakage of crude oil from the concrete tank, the secondary containment ensures collection of the same without polluting the surrounding ground. To confirm the efficacy of the installation, monitoring bore holes that penetrate the aquifers are installed on the periphery of the tanks and are used to monitor traces of hydrocarbon vapours or liquids, if any in the ground or water table. The system meets the American Environmental Protection requirements detailed out in the report.

Based on the available results of the geotechnical investigations conducted at site (DFR, 2000) and new topographic survey (2012), it has been established that the bearing strata and rock formations at site in conjunction with ground water conditions are competent for construction of the underground concrete tanks measuring 152m x

152 m in plan, with the depth varying from 8 m at the edges to about 26 m in the centre. The tanks have been located in a manner so as to use the sides of the hills as a part of integral embankments as well as to optimise the cut and fill quantities during construction. In order to have a total storage capacity of 2.5 MMT and adequate operational flexibility of catering to shipment parcels, a configuration of Eight Tanks have been envisaged.

For the contemplated storage, M40 grade of concrete is used for all structural members of the tanks with minimum concrete cover of 40 mm for adequate protection against corrosion. Each tank is designed with a double entry pipe system so that the contents can be circulated to provide a degree of mixing. Further to avoid any stratification and consequent sludge formation in the tank, a specially designed high capacity jet mixer is planned to be used in each tank.

Settling water within the crude, at the bottom of the tank, is drained through a 800 mm dia pipe located centrally (enclosed in concrete) to the oil water separator, from where water is pumped to open evaporation ponds These ponds are designed with a combined surface area to evaporate the anticipated water drained from the tanks within a reasonable length of time. The system has zero effluent production. A flare stack has been provided as the facility is envisaged to be operated frequently.

Double block and bleed valves are provided to prevent any intermixing of the two qualities of crude high sulphur and low sulphur which are being stored in the ratio of 75:25. The temperature fluctuations are limited in the vapour spaces due to natural thermal insulation provided by the tanks being underground.

A no fines concrete drainage layer below the tank collects any unlikely leakage of oil between the primary and the secondary containment of the tank and routes it to a central drain. The vapour space of the tank is vented through a PV valve located 7.5 m above the tank roof terrace level and 15 m from the edge of the periphery road around the roof.

The system is equipped with diesel driven can pumps capable of delivering the crude oil from any one of the tanks to the Gavridad pump station where the pressure will be boosted for delivering crude oil to Mathura Refinery through Salaya Mathura Pipeline. A booster station has been planned at Vadinar along with a 100 km new pipeline so as to fill the storage facility through any of the existing SPM located off Vadinar.

A cluster of booster pumps have also been planned at the storage facilities so as to evacuate the crude oil to the COTs of respective refineries located at Vadinar. Keeping in view the demand of the region, no export of crude oil is envisaged through SPMs.

With an execution philosophy involving three contract packages of Item rate contract for underground concrete tank facilities, one LSTK contract for above ground process facilities and one LSTK contract for pipeline integration facilities, the project is envisaged to be completed in a period of 66 months, which includes 15 month duration for pre bid engineering, tendering and award; followed by 41 months of construction of the facilities including 10 months of commissioning.

Based on the basic design performed for the underground storage and associated above ground process facilities, the capital cost estimate has been worked out to be Rs. 1897.40 crores, with an additional approximate cost estimate of Rs. 1303.56 crores for the pipeline integration purpose.

Based on the aforesaid, the configuration of underground facilities including design basis for underground storage caverns and proposed layout, associated process design including above ground plot plan, planning schedule and cost estimates have been worked out and presented. In addition, for connectivity from the storage facility to the nearest pipeline and refineries, a pipeline Integration scheme has also been presented. The design meets the requirements laid down by US Environmental Protection Agency for underground tanks holding hazardous liquids.

A Navratna Company

SOLUTION MINED SALT CAVERNS

R.Rath & A.Nanda

CHAPTER - 12

12.1 Introduction

Salt cavern storage alternative for storage of hydrocarbons viz. natural gas, crude oil, heating oil, diesel, gasoline, kerosene, ethylene, propylene, LPG (butane / propane), and compressed air or hydrogen takes advantage of the natural sealing properties of rock salt formations against gaseous media and non-aqueous liquids. Though, rock salt deposits occur world-wide, these are unevenly distributed around the globe. Moreover, these deposits must have a certain composition and internal structure, thickness and depth range to be suitable for cavern construction and storage operation.

Conceived during the late 1940s, storage of both liquids and gases in solution mined salt caverns was reportedly first used in Canada during World War II. Storage in salt caverns of liquified petroleum gas (LPG) and other light hydrocarbons spread rapidly in the early 1950's in North America and several European countries. Storage of crude oil reportedly occurred first in England, also in the early 1950s, during the "Suez Crisis". Natural gas storage followed storage of liquid hydrocarbons by about a decade in the US and Canada.

Disposal of wastes in salt caverns began initially as a convenient on site method for discarding byproduct from nearby industrial plants that utilized brine as a feedstock. A number of waste products are disposed of in salt caverns today. However, disposal of hazardous waste has been witnessing strong environmental considerations.

Rock salt deposits are formed during stages of earth's history under dry and arid climatic conditions in restricted marine basins. Generally, rock salt exists in two principal forms: salt domes and bedded salt formations. Salt domes are thick, deep-rooted, plug-shaped or mushroom-shaped structures that have pierced the overburden sequence to ascend to a relatively shallow depth. Owing to its nature of origin, these salt domes are characterised by structural deformities such as folds and synthetic faults etc. within the salt body. Bedded salt formations with their retained original horizontal and sequential layering are usually much thinner but laterally extensive. A third, less common type of deposit also occurs in the context of large mountain ranges as a heavily deformed mixture of salt and rock fragments.

Among the largest salt deposits worldwide are the salt dome provinces of northern Germany (which actually extend into Poland, the Netherlands and Denmark) and of the US Gulf Coast and Gulf of Mexico. Both are being intensively used for cavern construction either for salt production or for storage. In India, known salt deposits are restricted to a small occurrence in the Himalayas, which is of the deformed type, and the widespread bedded salt formations of Rajasthan located in the north western region of India.

Today, this technology has been developed to high levels of perfection, capable of constructing caverns with optimal shapes, conforming to precise specifications at depths down to 3,000 m and volumes of up to 800,000 m3 and more. The leaching process is implemented by wells to be drilled from the surface down to the depths required by the specific geological and project pre-conditions. The caverns thus created are used for several purposes and one of the most important usage is inventory stock piling of crude oil.

12.2 Salt Cavern Storage Technology

Utilising the solution mining (or 'leaching') technology, caverns are constructed below ground with a very small footprint on the surface. Essentially, this is done by drilling a well down into the formation, and cycling water through the completed well. The water dissolves and extracts the salt from the deposit, leaving a large artificial cavity filled with brine. The (gaseous or liquid) hydrocarbons intended to be stored are then pumped into the cavern, thereby

displacing the brine. Salt caverns have no internal lining and are only confined by the rock salt formation itself. Rock salt are considered intrinsically tight when subject to the overburden pressure of an overlying rock column of some hundreds of meters thickness (Figure 12.1).

Much as every other underground construction, salt caverns are engineered according to rock-mechanical specifications. The storage cavern design considers the long-term mechanical stability and tightness along with

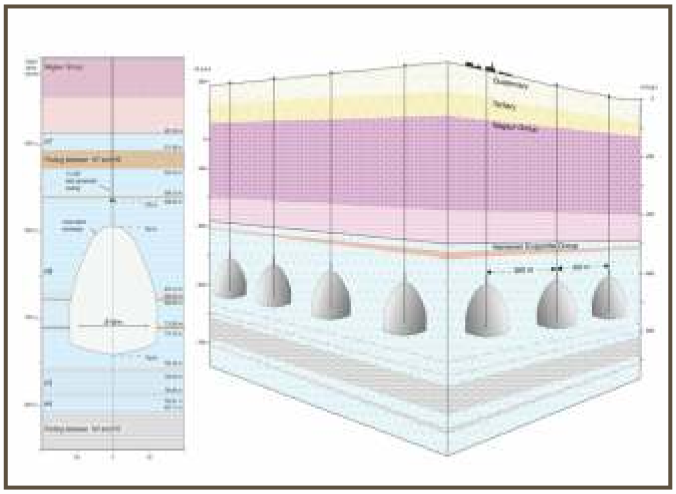


Figure 12.1: Pictorial representation of underground salt caverns

the realization of the desired volume and operational pressure (as a function of overburden pressure, i.e. depth). Thus, the starting point of the cavern design and prime constraint for cavern construction is the 'rock-mechanical envelope', i.e. the maximum theoretical boundary that guarantees the long-term mechanical stability of the cavern void. The rock-mechanical envelope is designed not to be exceeded at any time during the leaching process and cavern lifetime.

To achieve the designed cavern shape, the cavern volume is developed from the borehole by circulating water through the leaching tubings. These are concentrically installed into one another to provide different independent flow pathways for water and brine. Two different flow directions or 'leaching modes' are applied that are referred to as 'direct mode' (or 'bottom injection', i.e. injection through the inner leaching tubing) and 'reverse mode' (or 'top injection', i.e. injection through the annulus between outer and inner leaching tubing), respectively. Direct leaching secures more speedy development of the lower region of the cavern, whereas indirect leaching via the inner annulus secures a more speedy development of the upper region of the cavern. In order to protect the cavern roof from being dissolved, a fluid, non-aqueous or gaseous blanket medium which is less dense than brine (nitrogen or diesel oil) is pumped into the well via the annulus between the outer leaching tubing and the last cemented casing or wellbore. By switching between both leaching modes, changing the seating depth of the leaching tubings as well as by moving the blanket level upward according to a pre-defined leaching program, the cavern is shaped within the boundaries of the rock-mechanical envelope. The development of the cavern is monitored by constantly analyzing the produced brine and by performing echometric measurements, which provides a three-dimensional image of the actual cavern shape (Figure. 12.2).

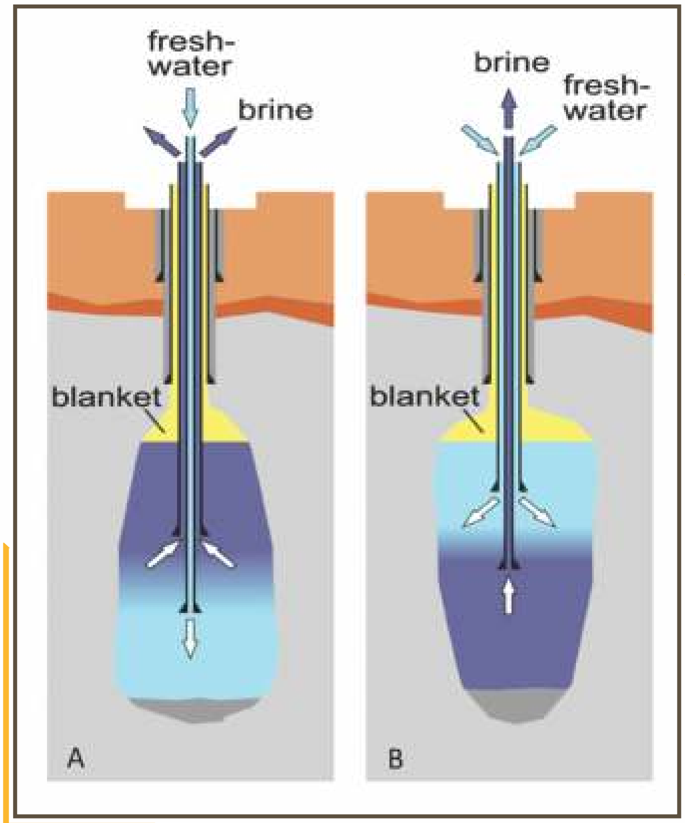


Figure 12.2 : Flow directions of water and brine in direct (A) and reverse (B) leaching mode

Typical storage caverns for liquid or gaseous hydrocarbons in domal salt are 300 to 500 m high and measure 70 to 90 m in diameter. In bedded salt, where the thickness of individual layers is limited, caverns are designed up to 100 m in diameter. Geometrical volumes of storage caverns range from 500,000 to almost 1,000,000 m^3. The construction time of a cavern of this size typically is two to four years. The resulting brine – 8 to 10 m^3 of water dissolves approximately 1m^3 of salt – is withdrawn through the inner annulus or the inner tubing depending on the method used. The outermost annulus is always reserved for the injection of the blanket.

12.3 Design Considerations

Solution mined salt caverns are primarily cavities created within the salt formations so as to use the sub surface space for storage of crude oil, natural gas and a few other hydrocarbons. Owing to its self healing properties and immiscible nature, salt caverns offer one of the best storage alternatives.

On the technical perspective, four basic parameters predominantly determine the possibility of creating salt caverns such as the geological setting of the site indicating presence of suitable salt formations, geotechnical characterisation of the salt formations, hydrogeological regime of the location as this would govern the availability of water for solution mining and fourthly the terrain of the site which has got a significant bearing on creation of these storage installations.

12.3.1 Geological Settings

Geological setting of the conceived site is one of the key considerations for any undergorund storage alternative and thus needs confirmation about a suitable geological setting for locating these installations. For the salt caverns, availability of adequate thickness of suitable salt formations is a basic necessity. The salt formations could be the most predomiant domal structures found extensively in the Gulf Coast of USA and the northern Europe or the bedded and undeformed evaporite deposits as reported beneath the arid north-west India. In addition, salt formations do occur in the form of deformed strata bound horizons with much lesser thickness. Mostly, the later formations are only used for mining of rock salt.

Information about the geology and geometry of the formation, including the overburden strata, is collected prior to commencing design work on the salt cavern storage facility. All available information on properties of the salt formation is also collected. In established cavern fields, the location, depth, diameter, and operating pressure of all caverns within 10 diameters of the planned caverns is required to be gathered. These information are often available from old drilling, cores and seismic surveys. When drilling the coreholes, log data and cores is required to be correlated with the existing information.

During the leaching program, geological (mud logging) and geophysical (core logging) camapigns are undertaken so as to monitor the development of caverns. In order to measure and ensure the cavern shape and restrict directional leaching, the cavern volume and shape is regularly surveyed by sonar measurements using specialized acoustic wireline unit. The method is based on travel-time measurements which is converted into distance when the acoustic velocity in the medium (usually brine) is known. The cavern shape is surveyed by a multitude of horizontal sections over the entire depth range of the cavern as well as by sections with tilted probe head to measure bottom, roof or any irregularities in cavern shape. The resulting digital data set of 3-dimensional co-ordinates are visualized applying special software.

Salt formations are either bedded sedimentary evaporite formations or intrusive salt formations such as domes and ridges. Bedded formations often underlie large evaporite basins, with relatively thin salt beds say about 300m thick often separated by beds of porous or impervious layers of the sediments. Domal salt formations have intruded roots in deeply buried salt beds and tend to be plugs a mile or more in diameter with great depth. The intrusion process of the displaced overlying strata causes formation of stratigraphic-structural traps on the flanks of the dome, where oil and gas

accumulations are often found. The overlying strata or cap rock and the domal salt formations often are heavily fractured and characterised by complex structure.

While knowledge of the physical location and dimension of the salt beds and any non salt layers is critical to design of caverns, in bedded salt formations the depth and thickness of overburden is also important. In contrast, domal salt geometry is often dificult as the flanks are irregular, overhung and the caprock is poorly defined owing to its complex geology. In bedded salt formations any water bearing strata must be identified and the drilling plan is developed to ensure against introducing cross connection between water strata and between the water strata and the salt formations. In terms of petrography, salt formations are best described as a rock formation composed predominantly of sodium chloride (Halite, NACL) deposits that are generally impervious to liquid or gaseous hydrocarbons. Other mineralisations include Potash minerals such as Polyhalite, Sylvinite, Sylvite, Carnallite; Gypsum or Anhydrite and Calcite & Dolomite. The salt formations are characterised by compressive strengths comparable to that of concrete. With a healing property salt formations are characterised by creep movements and thus seal fractures and voids. Being soluble in water, the salt caverns are mined by dissolution with water.

Prior to selection of a site and its suitability for developing underground salt caverns availability of adequately thick salt formation is a must. To this effect the geological studies entail following stages and assessments:

a) Study and assessment of subsurface geological setting, confirming availability of suitable salt formations;

b) Assessment of regional tectonic activity, regional and local fault zones and structural anomalies;

c) A study of available geophysical data, including seismic survey and core logging if any;

d) A study of formations from the surface to the storage zone and to a depth of 10m below the storage zone where such information is available through exploratory coreholes;

e) A study of formations and structures within a 1km radius of the subsurface perimeter of the storage zone;

f) A study of the containment properties of the surrounding formations including characterization of any potentially associated permeability zones and their impact on the proposed storage inventory;

g) A study of regional stresses and strains;

h) A study of mechanical and chemical properties of the salt and confining rock formations;

i) A study of regional dynamics of the formation, including cavern closure, subsidence, salt behaviour, and interference from neighbouring activities.

12.3.2 Geomechanical Properties

The rock-mechanical design of a storage cavern is to guarantee its serviceability and the stability of the surrounding formation during every mode of operation (leaching, oil injection, oil storage, and withdrawal). Laboratory tests on core samples give information about the geomechanical properties of the storage horizon and contiguous strata. The test results are used for numerical modelling and are the basis for the definition of design parameters such as:

a) the depth and shape of the cavern roof,

b) the thickness of the 'roof pillar', which is the portion of rock salt that has to remain above the cavern void,

c) the maximum allowable height and diameter of the cavern,

d) the thickness of the 'bottom pillar', which is the section of rock salt beneath the cavern,

e) the depth of the last cemented casing shoe,

f) the spacing of wells in a cavern field, and

g) the permissible storage operation pressures and operation modes.

Creep is the driving mechanism in such storage cavern volume loss and surface subsidence (Figure 12.3). Characterizing the creep behaviour of the storage cavern is one of the primary objectives of the geomechanical modeling Four types of geoemechanical stability criterion are considered in order to establish stability of salt cavities. These are

1. No tension criterion: Salt is brittle in tension with a very low tensile strength. The occurrence of tensile stresses in salt is therefore commonly not admissible.

2. Salt dilation/damage criterion: The modification of the initial isotropic stresses in the salt formation due to the excavation of a storage cavern induces deviatoric stresses in the salt. When the salt rock is submitted to deviatoric stresses which exceed a certain maximum value, the rock volume starts to increase due to the initiation and propagation of micro fissures. This phenomenon known as dilation is therefore an excellent indicator of rock damage and needs to be properly estimated.

3. Creep strain criterion: Being a highly ductile material, salt can sustain large deformations without failure while creeping. However, for design purposes, the creep strains are to be limited to avoid any risk of major failure likely to jeopardize the stability and integrity of the cavern wells and the salt pillar between two adjacent caverns.

4. Maximum gas pressure/gas infiltration criterion: Risk of gas infiltration is considered likely when the tangential stresses (i.e. parallel to the cavern wall) are less compressive than the normal stress (i.e. the gas pressure) acting on the cavern wall. Therefore, at the design stage, the geomechanical assessment is to verify that any potential gas infiltration zone would be of a limited extent in the salt formation around the cavern.

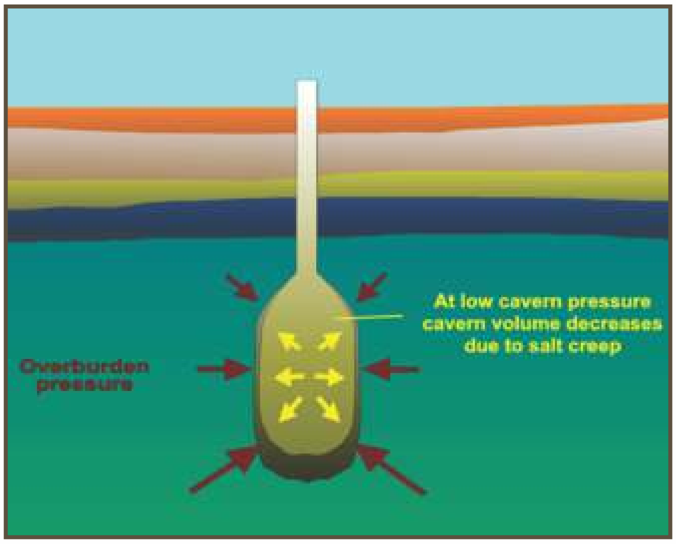


Figure 12.3 : Creep mechanism inside caverns

Thus proper design of the salt caverns requires knowledge of the in-situ stress state and the mechanical properties of the salt. In case of high purity products, knowledge of the entrained contaminants is also required. While the stress state and mechanical properties are assumed based on available data, once the project is implemented the design considerations are cross verified, when possible. The distance between two adjacent solution mined caverns in the formation is maintained in a manner conforming to the designing requirement such that the ratio S: D is not less than 2:1, where S equals the distance between the centers of the two caverns and D equals the average of the maximum diameter of the two caverns.

The following failures are prevented:

a) Fracturing the formation or casing seat by cavern operating pressure;

b) Loss of volume due to creep closure of the cavern system, with the potential for resulting surface subsidence;

c) Cavern roof collapse or sidewall slabbing with hazards to casing strings and the potential for surface

subsidence caused by rapid depressurization of the cavern;

d) Washout to the edge of the salt formation;

e) Unplanned coalescing of adjacent caverns.

The roof of the cavern is set deep enough to provide sufficient salt thickness between the cavern roof and the caprock to ensure adequate roof support of the overburden. In bedded formations, the strength of an impervious overburden layer is used to provide roof support. In domal salt, typically the cavern roof is set about 100m below the top of the salt, and the cavern bottom is not set excessively deep because temperature increases with depth and so does the salt-creep rate, therefore closure rates. Theoretically, a spherical cavern is the most stable cavern shape. An inverted cone shape and arched roof are generally considered an acceptable alternative. Shaping of the cavern is achieved through blanket material and controlled solution mining. Due consideration is also given to have additional space provided below the brine string to permit accumulation of the insoluble materials, both from initial and operational solution mining process.

12.3.3 Hydrogeological Regime

The solution mining process involves circulation of fresh water / brackish water through drilled wells at select horizons of salt formations whereby cavities are formed to be used as storage space. Therefore, availability of adequate supply of water is a prime requirement for development of salt cavern storage installations.

Sources of water could include wells, canals, rivers and local utilities. In many case scenarios, this warrant regulatory compliances. However, in consideration of the fact that adequate water could be available within the deep seated aquifers which is normally not accessible for potable usage neither have connectivity with the salt formations, the solution mining activities are often taken up after establishing the total resource base.

Based on the shallow exploratory investigation campaign, the hydrogeological regime of the selected site is evaluated and the pumping rate of the subsurface water is accessed by undertaking on site withdrawal tests. Consideration is also given to the quality of the water and its sustained source as some contaminants are detrimental to the plant, machinery and piping materials.

12.4 Brine Handling and Disposal

The brine produced during the solution mining (leaching) operations is evaporated in brine evaporation basins. Due to this, climate conditions like evaporation and precipitation are important values to define the needed evaporation basin dimensions. Therefore, the terrain characteristics of the selected site becomes crucial for brine disposal point of view.

A very minimal precipitation as in case of arid desert conditions are most preferable. The evaporation of brine is strongly dependent on factors like wind velocity, air temperature or relative humidity. Generally, these factors are difficult to determine, and changes in one of these factors have a great influence on the calculated evaporation rate. Else, in case scenarios of coastal regions the produced brine can be disposed off to the sea through a dedicated pipeline system. This in turn offers the additional advantage of not creating large evaporation basins which would have occupied large tracts of land parcels.

12.5 Aboveground Process Facilities

The aboveground process facilities are broadly divided into six main groups; the leaching / oil storage plant, the cavern pads, the water wells, the brine transport facilities, the brine evaporation basin and the brine pond (Figure 12.4).

The designed number of leaching pumps, the oil pumps, the dilution water pumps and the blanket oil pumps are arranged in compliance with the statutory regulations. The tank containing the blanket medium, mostly diesel, is installed at a short distance from the blanket pump in compliance with the design requirements. The whole tank area and the area of the oil pumps is designated as hazardous area (ex zone) as explosive gas mixtures can develop, therefore only explosion-proof equipment is installed.

The plant building houses pump stations, electric buildings and air conditioning systems including a workshop for maintenance work. The system design also includes fire fighting systems.

The cavern pads for oil storage caverns are dimensioned as 50m x 50m and are arranged in a pattern depending on the number of envisaged caverns. The foundation is designed according to the requirements of the drilling rig. Each cavern cellar is constructed with a pump sump, from which the water can be withdrawn and pumped into the sewage system of the cavern pad. The cavern area around the wellheads is designated as hazardous zone, where only fire proof equipment is installed. Within the cavern pad area, the concrete foundation for the drilling rig is installed.

In the event of leaching water sourced from the ground, the groundwater wells are arranged in a specific pattern / grid. The well locations can be partially located between the cavern pads to optimise the overall footprint and to keep the wells as close as possible to the leaching pumps to decrease the pressure losses in the water lines. Each groundwater well has a cellar, which contains the instrumentation of the field lines and the power and DCS cabinet. Submersible pumps are installed at designed depths of the ground water wells. In case of water being sourced from any different source such as a river or lake, etc. similar facilities for pumping are designed so as to achieve the requisite leaching rate.

The ground water line in the cellar must have a flow meter for discharge measurement. A check valve on the discharge side of each pump prevents backflow. Each cellar is equipped with a fluid level transmitter to detect any water leakage. In case of leakage in the cellar the corresponding submersible pump is switched off.

The brine pumps are generally located approx. 1,000 m away from the leaching / oil operation plant and preferably in proximity to the brine pond. Two solid and oil separators are installed next to the brine pumps such that the environmental considerations are addressed. A distance of 30 m is kept between the brine transport pumps and the separators. The separated oil is stored in an oil waste tank located next to separators and suitably disposed. The plant piping network connecting the topside facilities is laid in undergorund trenches.

The brine produced during leaching operation or during oil injection is discharged into the brine evaporation basin, where it is evaporated by solar energy. As a standard practice the term ‘brine evaporation basin’ refers to the area where brine is evaporated while the brine storage for oil withdrawal operation is referred to as ‘brine pond’.

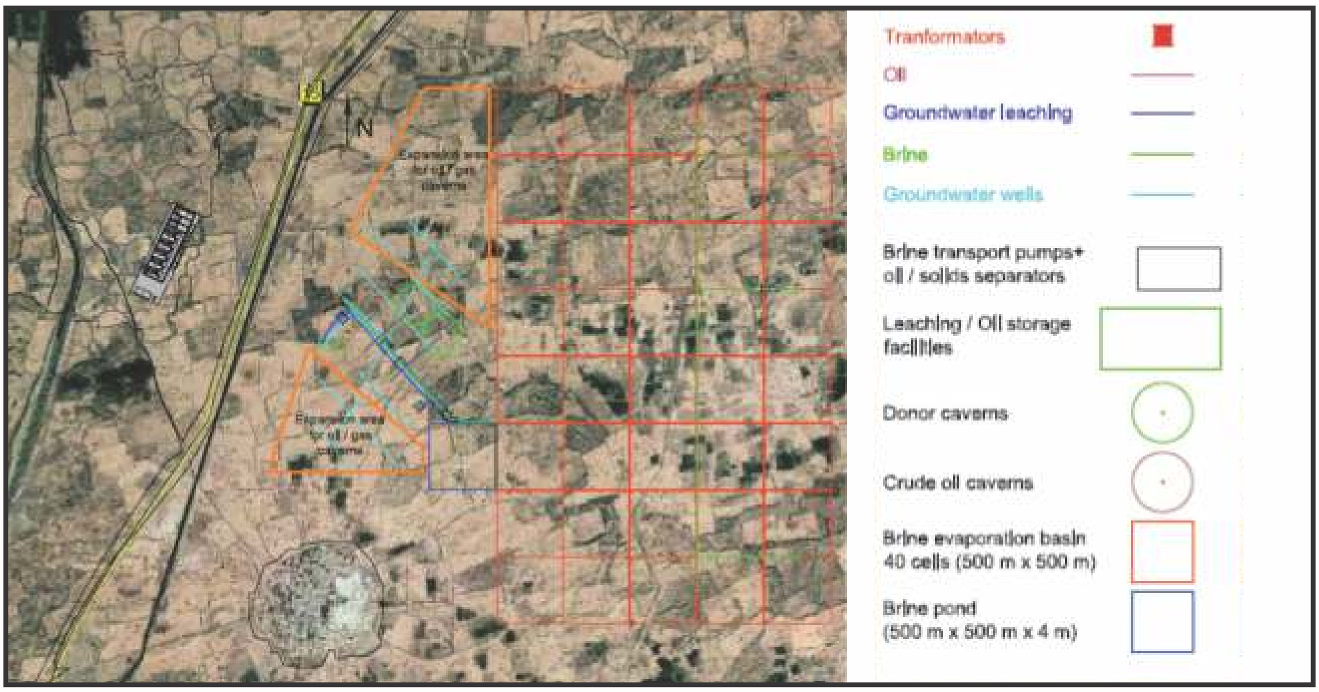


Figure 12.4. : Typical configuration of aboveground process facilities of a salt cavern installation

12.6 Leaching Facilities

The leaching pumps inject water at the required pressure through the water manifold and the field lines into the caverns. The leaching pumps are connected to the leaching water manifold, which is connected to the well heads with the field lines. If one pump fails, the redundancy pumps start up automatically and the full water flow is covered.

Depending on the leaching / solution mining campaign, the leaching pump configuration is designed with 50% redundancy capacity. The pumps are normally driven by electric motors and therefore dedicated transformers are also considered. Each pump is isolated by a hand operated ball valve on the suction side, an electric operated ball valve on the pressure side, and hand operated valves after the MOV on the pressure side of leaching pumps and is protected against unacceptable backflow by a check valve on the pressure side. On the suction side of each pump, a strainer is installed and a pressure safety valve for protection against higher pressure on the suction side. The pressure on the suction side is measured continuously and if the pressure value is lower than minimum suction pressure of the leaching pumps, the pump is switched off.

The leaching water is distributed from the discharge side of the leaching pumps through designed manifolds to the cavern pads. The same water manifolds are used during oil withdrawal for brine injection into the caverns. In the case scenarios of clustered and deferred solution mining process, often two manifolds are considered as the concept would entail simultaneous leaching and oil operation of two identified clusters of caverns.

Often, Diesel oil is used as blanket medium for controlled leaching operation so as to maintain the designed cavern shape. The diesel oil is stored in a double-walled tank and a plunger pump is considered to provide the required pressure for blanket injection. The unit is provided with a bypass in case that the blanket is withdrawn from the caverns into the blanket tank.

During leaching operation the groundwater is injected into the caverns to dissolve the salt formation. Produced brine is led to the oil and solids separators and from there via the brine transport pumps to the brine evaporation basin to be evaporated or to the brine pond to be stored. The blanket in the caverns is controlled simultaneously during this operation.

During oil operations either oil is pumped into the caverns to replace the existing brine or brine is pumped into the caverns to displace the stored brine. In both cases there is no need for blanket during this operation. Moreover, ground water is used only for the diluting of the brain during oil injection or for compensating the evaporated water from the water layer in the brine pond.

12.7 Oil Opertaions

The oil operations comprsing of oil injection and withdrawal are performed after completion of the leaching operation. In the considered case scenarios, depending on the product type and leaching program, often there is an overlap between oil operations and leaching operations.

The crude oil is delivered from the oil pipeline at the designed flow rate by frequency controlled oil pumps that increase the pressure of the delivered oil and inject it into the oil/brine manifold. As a design norm, the standby pump (s) for 50% redundancy of the rated capacity is planned. The oil/brine manifold leads the oil to the wellheads. The control valves at oil field lines control the flow rate of injected oil in each cavern. If the delivered oil has a sufficient pressure, it flows through the oil injection bypass to the oil/brine manifold without going through the oil pumps.

To avoid any crystallization of salt, groundwater is injected through the dilution string to the bottom of the de-brining string to dilute the brine. The diluting water is pumped by the diluting water pump into the diluting water manifold to the wellheads. The same manifold, which is used during leaching operation for blanket is used during oil injection operation for dilution water. The produced brine flows through the brine field manifold to the oil and solids separators and delivered to the brine evaporation basin or to the brine pond.

Since the same brine transport facilities are used during oil injection and leaching operation, the leaching process of the second cluster of the caverns are interrupted during oil injection into caverns of first cluster.

During oil withdrawal operation brine is pumped into the caverns by brine pumps and replaces the oil in the caverns. The oil withdrawal operation is either performed while other oil caverns still in leaching operation or it is done after the end of leaching operation of all oil caverns. In the first case the produced brine during leaching operation flows through the brine field manifold to the oil and solids separators, from where the brine transport pumps inject the brine into the brine transport manifold, which leads the brine in this case to the caverns. In the next case brine is delivered from the brine pond and from the donor caverns, which are leached only during oil withdrawal operation to deliver brine. Pressure of delivered brine is increased by brine transport pumps which inject the brine into the caverns through the brine transport manifold.

12.8 Concluding Remarks

The salt cavern storage is a well-established technology in north and central Europe and in the US, where storage caverns have been operated since the 1960s. The highest industry standards for operational safety and environmental compatibility have been developed in these countries. Especially in Germany, the Netherlands, France and the UK, where cavern construction and operation is done under the supervision of the mining authorities, the standards are very high. Wells and aboveground components will require safety installations such as shut-off valves, monitoring systems etc.

Due to this advanced technology, in combination with the material properties of deeply buried rock salt, salt cavern storage can be considered the safest way of storing hydrocarbons. Compared to conventional surface and underground rock cavern storages, the underground storage of crude oil in salt caverns offers significant economic advantages.

Case Study 12.1 : Underground Salt Cavern near Bikaner

The proposed site for the storage is located near Udepur, in the Ganganagar District. The site is approx. 150 km NNE of Bikaner and is accessible by NH-15 connecting Amritsar to Kandla via Bikaner as well as by the broad-gauge rail of North Western Railways from Suratgarh to Bikaner. The project site is located on either side of NH-15 and the nearest railway station is Suratgarh, located approx. 20 km from the project site.

Being located on the north-western region of the country, the storage facilities are planned to be integrated to the three refineries namely GGSRL refinery at Bhatinda, and IOCL refineries at Panipat and Mathura. The pipeline integration scheme envisages connectivity to the Mundra Bhatinda Pipeline (MBPL), Mundra Panipat Pipeline (MPPL) and Salaya Mathura Panipat Pipeline (SMPL) at Dungargarh, Sanganer and Chaksu respectively. The storage facility is planned to have access to the offshore oil terminals (SPMs) located off Mundra on the northern coast of the Gulf of Kutch for transshipment.

With the availability of a favorable geological setting, i.e. a bedded salt formation of appropriate depth and thickness, and of a shallow aquifer to provide sufficient yield of (slightly saline) groundwater, underground salt caverns is the selected storage alternative for the Bikaner area. The total envisaged storage capacity is 3.75 MM T (or 4.4 MM m^3) of crude oil. For the purpose of early commissioning, the storage facility has been designed in terms of two 'clusters' (A and B) of four caverns each. Each cavern has a geometrical storage volume of 550,000 m^3,

totaling to a capacity of 1.875 MM T for each cluster. The caverns are designed to be located at a depth of 630 to 750 m below ground with a maximum diameter of 110 m. With an objective of having an early and phase-wise commissioning, the cluster A caverns are planned for the first oil fill after approximately 45 months after spud-in of the first cavern well to be followed by the first oil fill of cluster B caverns approximately 30 months later.

While the leaching operation is planned to be carried out by sourcing slightly saline ground water from the shallow aquifer, the brine disposal has been contemplated through solar evaporation in shallow basins and removal of solid salt. However, for commercial oil operation, sourcing of saturated brine (to be used for compensation of withdrawing oil) is being secured through a combination of 'donor caverns' and above ground storage in a brine pond.

The project is envisaged to be completed in a period of 84 months, which includes 15 months duration for pre-bid engineering, tendering and award. In view of the interface requirements of the salt cavern project, the above ground facilities contract is planned to be in place by 9 months providing necessary work front. Concurrently based on supplementary investigations, the detail engineering for underground facilities will be carried out. Further the construction schedule for salt caverns is planned to be in two stages; Cluster I involving four caverns to be completed in 66 months providing a storage capacity of 1.875 MMT; followed by commissioning of the remaining four caverns under Cluster II in 84 months.

The present DPR envisaged storage scheme designed as a single purpose project with the objective to store 3.75 MMT (4.4 MM m^3) of crude oil. Due to the characteristics of the project it is deemed possible that other projects could hook up with the oil storage and could be sited in the same project area.

A possible gas storage project, which is envisaged in line with the trans-national gas pipelines would be an ideal fit and could be taken up after the leaching of the last oil storage caverns is finished. Additional investment would be limited to the completion of gas storage cavern wells and construction of the aboveground gas facilities, as the brine from the leaching of the gas cavern could be used for operation of the crude oil cavern. Further some of these caverns could also be used to store crude oil, thus increasing the total storage capacity.

A Navratna Company

REFERENCES

1. Aberg, B. (1977). "Prevention of gas leakage from unlined reservoirs in rock." Proc. Of the First International Symposium on storage in excavated rock caverns, Rockstore 77, Sep 5-8, 1977, Stockholm, Sweden.

2. ACI (American Concrete Institute). (1993). "Cooling and insulating systems for mass concrete." ACI 207.4R-93, Farmington Hills, MI.

3. ACI (American Concrete Institute). (2007). "Report on thermal andvolume change effects on cracking of mass concrete." ACI 207.2R-07, Farmington Hills, MI.

4. AFNOR (Association Francaise de Normalisation). (2005). "Design of concrete structures- Part 1-1: General rules and rules of buildings." NF EN1992-1-1 Standard: Eurocode 2, France.

5. Albataineh N (2006) Slope stability analysis using 2D and 3D methods, M.Sc Thesis. University of Akron, Akron, OH 44325.

6. Barton. N., Lien. R, Lunde, J. (1974). "Engineering classification of rock masses for the design of tunnel support, Rock Mechanics" Vol.6, N 4, pp 189-236.

7. Barton, N., Løset, F., Lien, R., and Lunde, J. (1980). "Application of the 456 Q-system in design decisions." Subsurface space, M. Bergman, ed., 457 Vol. 2, Pergamon, New York, 553–561.

8. Bieniawski, Z. T. (1984). "Rock Mechanics Design in Mining and Tunneling" A. A. Balkema Publishers, Rotterdam/Boston, 272 p.

9. Bieniawski, Z.T., (1978), "Determining rock mass deformability: experience from case histories." Int J. Rock Mech. and Min. Sc., Vol. 15, PP. 237-247.

10. Bieniawski, Z.T., (1973), "Engineering Classification of Jointed Rock Masses," Transactions. South African Institution of Civil EnQineers, Vol. 15, pp. 335-344.

11. BIS (Bureau of Indian Standards). (2000). "Code of practice for plain and reinforced concrete." Indian code IS 456, New Delhi, India.

12. BIS (Bureau of Indian Standards). (2009). "Code of practice for design of concrete structures for storage of liquids." IS 3370: Part 2, New Delhi, India.

13. BIS:1893 (1984), Criteria for earthquake resistant design of structures. Indian code IS 875, New Delhi, India.

14. BIS (Bureau of Indian Standards). (2000). "Code of practice for design loads" Indian code IS 875, New Delhi, India.

15. BS:8007 (1965), Code of practice for design of concrete structures for retaining aqueous liquids.

16. British Tunnelling Society (2006). "Code of practice for risk management of tunnel works" 2nd Edition, International Tunneling Insurance Group, London.

17. Ghee, E. H., Zhu, B. T., and Wines, D. R. (2006). "Numerical analysis of 470 twin road tunnels using two- and three-dimensional modelling techniques." Proc., 4th Int. FLAC Symp. on Numerical Modelling in Geomechanics, Itasca Consulting Group, Minneapolis, MN.

18. Hoek, E., and Brown, E. T. (1980). "Empirical strength criterion for rock masses." J. Geotech. Engrg. Div., 106(GT9), 1013–1035.

19. Kannan,G., Chakravarthy,C.P., Kumar,C., Nanda, A. and Jain, S.K. (2013). "Geological prediction of weak dyke during construction of a rock storage cavern, Indorock Solan, H.P. India.

20. Kannan, G., Usmani, A., Nanda, A. and Jain, S.K. (2013). "Stability of Caverns under high pressure ground water regime and associated grouting", Proceedings of Indian Geotechnical Conference, December 22-24, Roorkee.

21. Kulhawy, F.H. (1974). "Finite element modelling criterion for underground openings in rock", Int. J. Rock Mech. Mining Sci., 11 (12), pp. 455–472.

22. Nanda A. (2012). "Design and construction of storage caverns" Keynote Lecture, Proceedings of Indian Geotechnical conference, New Delhi, India.

23. National Standards Authority of Ireland. (2004). "Concrete — Code of practice for the structural use of concrete." IS 326 National Standards Authority of Ireland, Ireland, Dublin.

24. Palmstrom A. (1974): Characterization of jointing density and the quality of rock masses (in Norwegian). Internal report, A.B. Berdal, Norway, 26 p.

25. Slide 2.0 [Computer software]. Toronto, Rocscience.

26. Sreekanatiah, H.R. (1993) Laterites and lateritic soils of Dakshina Kannada District. Seminar on geotechnical problems in construction in lateritic formations. IGS Surathkal Chapter 1:223–1232.

27. Terzaghi, K. (1946). "Rock defects and loads in tunnel supports." Rock tunneling with steel supports. R.V. Proctor and T.L. White, eds., The Commercial Shearing and Stamping Co., Youngstown, Ohio, 17-99.

28. Terzaghi, K., Peck, R.B. (1948). Soil mechanics in engineering practice. Wiley, New York

29. Usmani, A., Nanda, A., and Sharma, K. G. (2009). "Stress and seepage analysis of underground rock caverns." Proc., 17th Int. Conf. on Soil Mechanics and Geotechnical Engineering, M. Hamza, M. Shahien, and Y. El-Mossallamy, eds., IOS Press, Amsterdam, Netherlands, 1753–1756.

30. Yazdani, M., Sharifzadeh, M., Kamrani, K., and Ghorbani, M. (2012). 492 "Displacement-based numerical back analysis for estimation of rock 493 mass parameters in Siah Bisheh powerhouse cavern using continuum 494 and discontinuum approach." Tunnelling Underground Space Technol., 28(Mar), 41–48.

APPENDIX

ANNEXURE I

LIST OF CODES

S.No.	Code No.	Part/ Section	Description
1	OISD-STD-230		Safety Standard for Unlined Underground Rock Caverns Stoarge for Petroleum and Liquid Petroleum Gas

BIS Codes

1	269		Specification for 33 grade ordinary Portland cement
2	383		Specification for course and fine aggregates from natural sources for concrete
3	455		Specification for Portland slag cement
4	456		Code of Practice for Plain & Reinforced Concrete
5	516		Method of test for strength of concrete
6	808		Dimensions for hot rolled steel beam column channel& angle sections
7	1161		Steel tubes for structural purposes
8	1239		Mild steel tubes, tabular and other wrought steel fittings
9	1343		Code of Practice for Pre- stressed Concrete for tendon
10	1489	Part 1	Specification for Portland pozzolana cement (fly ash based)
11	1489	Part 2	Specification for Portland pozzolana cement (calcined clay based)
12	1566		Hard-drawn steel wire fabric for concrete for wire mesh
13	1786		Specification for high strength deformed bar and wires for concrete reinforcement TMT
14	1892		Code of Practice for subsurface investigation for foundation
15	2062		Steel for general structural purposes (Grade A)
16	2116		Sand for Masonry Mortar
17	2131		Method for Standard Penetration test for soils
18	2132		Code of practise for thin walled tube sampling of soil
19	2140		Standard Galvanized Steel wire for fencing
20	2386	Part 1	Methods of test for aggregates for concrete
21	2386	Part 2	Method of test for aggregates for concrete
22	2430		Method of Sampling of aggregate for concrete
23	2600		Specification for Low heat Portland cement

24	2645		Specification for integral cement water proofing compounds
25	3025		Tests on water sample
26	3764		Safety code for Excavation work
27	4078		Code of Practice for indexing and storage of drill cores
28	4081		Safety Code of blasting and related drilling operations
29	4082		storage of cement at the site or work
30	4453		Code of Practice for subsurface exploration by pits, Trenches, drifts and shafts
31	4464		Code of practice for presentation of drilling information and core description in foundation investigation
32	4756		Safety Code for tunnelling work
33	4923		Hallow section or structural use
34	5313		Guide for core drilling observation
35	5529	Part 1 & 2	Code of Practice for In-situ Permeability tests.
36	5878	Part 2	Code of practice for construction of tunnels conveying water, Part 2:Tunnel Supports
37	6922		Criteria for safety and design of structures subject to underground blasts.
38	6926		Diamond core drilling-site investigation for river valley projects-Code of Practice
39	7293		Safety code for working with construction machinery
40	7356	(Part-1)	Code of Practice for installation. Maintenance and observation of instruments for pore pressure measurements in earth dams and rock filldams.
41	7526		Detonating Fuses-Specifications (Non-electric or electronic Detonators)
42	7632		Specification for detonators
43	7746		Code of Practice for in-situ shear test on rock
44	8041		Specification for rapid hardening Portland cement
45	8112		Specification for 43 grade ordinary Portland cement
46	8764		Method of determination of point load strength index of rocks.
47	9012		Recommended Practice for Shotcreting
48	9103		Specification for other admixtures for concrete
49	9143		Method for the determination of unconfined compressive strength of rock material
50	9179		Method for the preparation of rock specimen for laboratory testing

51	9221		Method for the determination of modulus of elasticity and poisons ratio of rock materials in uniaxial compression
52	10050		Method for determination of slake durability index of rocks
53	10082		Method of test for determination of tensile strength by indirect tests on rock specimen
54	10208		Diamond core drilling equipment
55	10782		Method for laboratory determination of dynamic modulus of rock core specimen
56	11309		Method of conducting pull out test on anchor bars and rock bolts
57	11315	Part 1	Method for the quantitative description of discontinuities in rock mass Part 1: Orientation
58	11315	Part 2	Method for the quantitative description of discontinuities in rock mass Part 2: Spacing
59	11315	Part 3	Method for the quantitative description of discontinuities in rock mass Part 3: Persistence
60	11315	Part 4	Method for the quantitative description of discontinuities in rock mass Part 4: Roughness
61	11315	Part 5	Method for the quantitative description of discontinuities in rock mass Part 5 : Wall Strength
62	11315	Part 6	Method for the quantitative description of discontinuities in rock mass Part 6: Aperture
63	11315	Part 7	Method for the quantitative description of discontinuities in rock mass Part 7: Filling
64	11315	Part 8	Method for the quantitative description of discontinuities in rock mass Part 8: Seepage
65	11315	Part 9	Method for the quantitative description of discontinuities in rock mass Part 9:Number of Sets
66	11315	Part 10	Method for the quantitative description of discontinuities in rock mass Part 10:Block Size
67	11315	Part 11	Method for the quantitative description of discontinuities in rock mass Part 11:Core Recovery and Rock quality
68	11315	Part 12	Method for the quantitative description of discontinuities in rock mass Part 12:Drill core study
69	11358		Glossary of terms and symbols relating to rock mechanics
70	12269		Specification for 53 grade ordinary Portland cement

71	12330		Specification for Sulphate Resisting Portland cement
72	13030		Method of test for laboratory determination of water content, porosity, density and related properties of rock material
73	13047		Method for determination of strength of rock materials in triaxial compression
74	13365	Part 1	Quantitative classification of rock mass guidelines, Part 1: RMR for predicting of engineering properties
75	13365	Part 2	Quantitative classification of rock mass guidelines, Part 2 : Rock mass Quality for prediction of support pressure in underground openings
76	13365	Part 3	Quantitative classification of rock mass guidelines, Part 3 : Determination of slope mass rating (SMR)
77	13946	Part 1	Determination of rock stress-code of practice : part 1 using hydraulic fracturing technique
78	14396	Parts 1 to 4	Argillaceous swelling Rock - Method for laboratory testing
79	14436		Guidelines on determination of resistivity of rock specimen

ASTM Codes

S.No.	Code No.	Description
1	A820	Standard Specification for Steel Fibers for Fiber Reinforced concrete
2	C1140	Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels
3	C1141	Standard Specifications for Admixtures for Shotcrete
4	C1604	Standard Test Method for Obtaining and Testing Drilled Cores of Shotcrete
5	C1609	Standard Test Method for Flexural Performance of Fibre Reinforced Concrete (Using beam with third point loading)
6	D 4050	Standard Test method for withdrawal and injection well tests for determining hydraulic properties of aquifer systems.
7	D2113	Diamond Core Drilling for Site Investigation
8	D4403-12	Standard Practice for Extensometers used in Rock
9	D4412-84	Standard test method for sulphate reducing bacteria in water and water formed deposits
10	D4428/ D4428M	Cross-hole Seismic Testing
11	D4435-0	Standard Test Method for Rock Bolt Anchor Pull Test

12	D4435-08	Standard Test Method for Rock Bolt Anchor Pull Test
13	D4630	Determining Transmissivity and Storavity of Low Permeability Rock by In-Situ Measurements Using Constant Head Injection Test
14	D4631	Determining Transmissivity and Storavity of Low Permeability Rock by In-Situ Measurements Using the Pressure Pulse Technique
15	D4879	Geotechnical Mapping of Large Underground Openings In Rock
16	D4971	Determining the In Situ Modulus of Deformation of Rock using Diametrically Loaded 76-mm(3-in) Borehole Jack
17	D5092-04	Standard Practice for Design and Installation of Ground water Monitoring wells
18	D5873	Determination of Rock Hardness by Rebound Hammer
19	D5878	Rock Mass Classification Systems for Engineering Purposes
20	D932	Standard test method for iron bacteria in water and water formed deposits
21	ISRM 1978 (EUR 4)	Suggested Methods for Monitoring Rock Movements Using Borehole Extensometers
22	ISRM 1992 (EUR 4)	Suggested Method for Blast Vibration Monitoring

INVESTIGATIONS

1.0 Geological Investigation

In general the objectives of engineering geological investigations are to:

- Establish co-relation of the geological setting of the project site intended with respect to regional geological setup
- Identification of seismicity of the site with respect to seismic zonation
- Delineate inferred layers of soil, weathered rock & fresh bed rock
- Establish characteristics of host rocks and soils
- Establish structural disposition of the litho types, lineaments, presence of igneous intrusion etc
- Geomorphology of area with respect to major geologic features & discontinuities
- Frequency, orientation and nature of joints and other discontinuities including major & minor features such as faults & folds etc
- Identify presence and extent of weathering of rocks,
- Record ground water conditions including water table quality of water
- Establish suitability of the site for the intended purpose

Some of the key geological investigation are discussed herein:

1.1 Field Reconnaissance Survey & Geological Mapping

After review of available geological maps and reports, a geological field reconnaissance of the project site and the adjacent areais made to confirm, correct or expand geological and hydro geological information collected from preliminary desktop study. If rock outcrops are present, field reconnaissance offer an opportunity for initial assessment of rock mass, geological setting & structural disposition.

Based on the results of reconnaissance survey, detailed geological mapping procedures are selected. For the purpose of detailed mapping, a sufficiently accurate base map of suitable scale with abundant spot points are used. With the basic purposes of gathering geological information, field traverses are planned where in the information such as lithology, presence of geological features such as shear zone, faults, dykes, folds etc. are collected and later transformed on to the map/sections.In case of non-availability of distinct outcrops, the litho-contacts are interpreted in consonance with the overall geologic setting, nature and type of litho units.

Major geological observations made during the mapping are as follows:

Lithology:	Rock type & mode of occurrence, such as veins, dykes, beds, lenses, etc. & sequence & thickness, Grain size, colour, and mineralogical make up
Structure:	Primary structures such as ripple marks, cross bedding, stratification, lamination, flow structures etc. Secondary structures and their attributes such as dip & strike of beds, axial planes, axes, lineation, cleavages, fractures etc. nature of discontinuities.
Metamorphism:	Kind & degree of alteration and the alteration products
Topography:	Forms & kinds, agents responsible for and relations to bed rock
Remarks:	Interpretations, tentative co-relations ideas, possibilities etc.

Detailed geological mapping procedure including quantitative description of discontinuities, rock quality etc is performed as per the respective standard codes.

1.2 Geological Mapping Results

Information collected during the field campaign shall have the following output:

- Geologic maps such as surface maps, outcrop maps, areal maps and structural maps. For detail information, the scale of map is made of 1:500 and not less than 1:1000 with 2m contour interval.
- Geologic relations, areal distribution patterns, structural attributes, sequence of strata, thickness of beds, and presence of structural discordances are established.Formation boundaries, not exposed in the field are extrapolated for completing the geological maps.
- Structural maps and the geologic cross sections. The structural attributes are represented through stereographic projection.
- Geological map supplemented with details about water table conditions of the site.
- An overall composite map with desired rock mass characterization i.e. estimate of RMR , Q and Geological Strength Index (GSI).
- Structural inferences, geological sections, block models, and 3-D models with the proposed facilities super imposed on it. The model updated with every add-on information, during construction stage.\
- Location and interaction of potential geological hazards in relation to existing & planned facilities.

2.0 Geophysical Investigation

Geophysical investigation are indirect exploration techniques for probing sub surface condition, wherein measurements made at the surface or downhole are used for interpreting the status of underground.Geophysical Investigations are also carried out to supplement borehole and outcrop information and to aid in geological interpretation / inferences. The campaign includes Seismic Refraction Survey, Electrical Resistivity Survey, Borehole Geo-Physical Logging and Cross-hole Seismic Test.

The seismic refraction survey along longitudinal and cross lines will help to delineate different subsurface stratigraphic units and assess their thickness, nature and contacts. The electrical survey comprising of Resistivity profiling and Vertical Electrical Sounding (VES) shall be used for resistivity imaging to understand subsurface lithology and ground water conditions. The geophysical bore hole or well logging will help to correlate and calibrate geophysical data with geological information obtained by studying the borehole core samples. The cross hole seismic survey will be used for rock mass characterization.Some of the main geophysical investigation to be performed are:

(i) Seismic Refraction Survey

(ii) Electrical Survey

(iii) Geophysical Well Logging

(iii) Cross hole Seismic Test

3.0 Geotechnical Investigation

3.1 Core Drilling

Major part of geotechnical investigation is the compilation of accurate borehole logs on which subsequent geologic and geotechnical information and decisions are based. A field drilling log for each borehole provides an accurate and comprehensive record of the lithology and stratigraphy of soils and rocks encountered in the borehole and other relevant information obtained during drilling, sampling, and in situ testing. Also the sample collected from core drilling shall be used for laboratory testing.

The major uses of core drilling is summarized as below:

- Define geologic stratigraphy and structure,

- Obtain samples for index testing and to determine engineering properties,
- Obtain ground water data,
- Perform in situ tests,
- Install instrumentation,
- Establish foundation elevations for structures
- To determine the engineering characteristics of existing structures

Rock Drilling both vertical and inclined is performed by rotary drilling machine and shall be carried out as per their respective standards and codes. The depth of core hole could vary from 100 to 200 m.Few additional destructive holes up to a depth of about 50 m may also be required for various hydro geological tests.

3.2 Laboratory Testing

Within the project requirements, a suitable suite of index and engineering property tests is planned both in the vertical as well as lateral direction. Selection of sample for the laboratory test from the core box is representative of the overall project facilities at various locations and depth. Preparation of rock samples and the tests are carried out as per their respective standards and codes.

The following laboratory tests is performed to investigate the physical properties of natural materials such as soil and rock and define the engineering properties in parameters usable for design of the underground rock cavern.

4.0 Test for Soil and Rock

4.1 Soils

Types of index and classification tests that are typically required for soils are listed below:

a) Water Content

b) Liquid Limit & Plastic limit

c) Sieve Analysis

d) Hydrometer analysis

4.2 Rocks

Types of index and classification tests that are typically required for rock are listed below:

a) Specific Gravity

b) Bulk Density(Wet/Dry)

c) Water Absorption

d) Water Content

e) Porosity

f) Slake Durability

g) Swelling Index

a. Thin section study and rock composition (Petrography)

h) Permeability

i) Sonic Velocity test

j) Cerchar test (Abrasivity and hardness index)

4.3 Engineering Properties Test for Rock

a) Unconfined uniaxial compressive test

b) Point load test

c) Brazilian test (Tensile strength)

d) Direct shear test

e) Triaxial test

f) Joint stiffness (Normal and Tangential)

g) Dynamic Modulus

4.4 Hardness Test

Cerchar test (Abrasivity and hardness index)

5.0 Ground Water Quality Tests

Physical, chemical and micro-biological test on water samples are carried to establish suitability of water.

6.0 In-situ Stress Measurement

The objective of the hydro fracturing techniques is to measure the state of in situ stress underground through a drill hole. The test provides the magnitude and direction of the maximum and minimum in-situ stress in the plane perpendicular to the drill hole axis. The test is conducted in vertical boreholes of minimum Ax size using diamond bit core. Condition of core hole is checked in advance, before conducting the test. Due attention is given to sub-vertical joints within the existing core hole, while lowering and raising of packer assembly.

7.0 Hydro-Geological Investigation

The purpose of hydrogeological investigation is to collect basic hydrogeological parameters required for the preparation of hydrogeological model and analysis of the proposed project site. These data are also used as background data before start of construction so as to compare it with the hydrogeological regime during or after construction. Hydrogeological investigations also corroborate/supplement the already available geological and geophysical information of the site. A lineament analysis study with the help of satellite imagery of the project area is useful before any hydrogeological investigations. Lineaments are verified at site for possible geological structural features and are intersected by core holes during geological investigations and geophysical lines during geophysical investigations. Most of the geological features are generally water bearing and are hydraulically tested.

Hydrogeological tests are conducted in core holes drilled for geological investigations/geophysical investigations. Few additional destructive wells may be required for ground water level monitoring.

The basic hydrogeological parameters collected from the site during investigation are:

a) Permeability (hydraulic conductivity) of weathered rock, massive rock, jointed rock and individual water bearing joints, dykes and other features.

b) Transmissivities of the top soil, weathered rock and the fractured rock.

c) Ground water table of the area and ground water / piezometric level of fractured bedrock and other water bearing features in the bedrock.

d) Physical, Chemical and micro-biological water quality parameters of ground water.

e) Rainfall data of the site.

f) Approximate estimation of surface runoff and rainfall infiltration from available literature studies and desktop studies of topography, geomorphology and satellite imagery of the area.

g) Source of water near by and its quality / source of groundwater nearby and its yield.

The following hydrogeological tests are conducted during the investigations stage:

8.0 Water pressure tests with packers to derive permeability i.e. hydraulic conductivity of water.

The following water pressure tests are performed:

(a) Injection fall off Test

(b) Pulse Test

(c) Lugeon Test

(d) Falling Head Test

(e) Constant Head Test

(f) Long Duration Injection Fall-off Test

Types of water pressure test are based on the ground water conditions. In addition the following are also carried out:

(a) Draw down pumping interference tests to derive transmissivity.

(b) Ground Water Monitoring

During investigation stage and subsequent construction stage, continuous on-site ground water level monitoring are carried out in all the investigation holes and other available dug wells nearby with the following objectives

a) To have an effective hydro-geological characterization of the site.

b) To have a representative ground water quality check.

c) To monitor the effect of ground water level during construction

d) To design effective water curtain and grout design during construction so as to maintain the same ground water level unaffected

Owing to site specific geological and hydro-geological regime, a conceptual hydro-geological model is developed prior to identifying and designing the ground water monitoring stations. Location and the depth of monitoring wells are recommended based above model and specific requirement. Sufficient numbers and length (at least 100m) of electrical dip meters are required to measure ground water level in all boreholes.

ANNEXURE III

PARAMETERS USED IN Q SYSTEM

The typical numerical values of parameters used in Q system proposed by Barton (1974) are given below. For details please refer Barton , 1974:

Table 1: RQD - Rock Quality Designation(RQD)

RQD	Evaluation
0 – 25	very poor
25-50	Poor
50-75	Discrete
75-90	Good
90-100	Excellent

Table 2: Joint Set Number(Jn)

Rock mass	Jn
Massive - few joints	0.5-1.0
One system of joints	2
Two systems of joints	4
Three systems of joints	9
Four or more systems of joints	15
Crushed rock/earthlike	20

Table 3: Joint Roughness Number (Jr)

Rock mass	Jr
Discontinuous joints	4
Rough or irregular, undulating	3
Smooth undulating	2
Slickensided, undulating	1.5
Rough or irregular, planar	1.5
Smooth, planar	1.0
Slickensided, planar	0.5

Table 4: Stress Reduction Factor (SRF)

Rock mass	SRF
Multiple occurrences of weakness zones containing clay	10.0
Single weakness zones containing clay (excavation depth<50 m)	5.0
Single weakness zones containing clay (excavation depth>50 m)	2.5
Low stress, near surface	2.5
Medium stress	1.0
High stress, very tight structure	0.5-2

A Navratna Company

Table 5: Joint Alteration Number (Ja)

Rock mass	Ja
Tightly healed, hard, non-softening	0.75
Unaltered joint walls, surface staining only	1.0
Slightly altered joint walls, non-softening	2.0
Silty-, or sandy-clay coatings	3.0
Softening or low-friction clay mineral coatings	4.0
Sandy particles, clay-free, disintegrating rock	4.0
Strongly over-consolidated, non-softening clay mineral filling (continuous < 5 mm thick)	6.0
Medium or low over- consolidation, non-softening clay mineral filling (continuous < 5 mm thick)	8.0
Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick).	8.0-12.0
Thick continuous zones of bands of clay	10.0-13.0

Table 6: Joint Water Reduction Factor (Jw)

Hydraulic conditions	Jw
Dry excavation or minor inflow	1.0
Medium inflow or pressure	0.66
Large inflow or high pressure (joints without filling)	0.5
Large inflow or high pressure	0.33
Exceptionally high inflow or pressure at blasting	0.2-0.1
Exceptionally high inflow or pressure	0.1-0.05

Table 7: Quality of the rock mass (Q)

Class	Value of Q
Exceptionally poor	<0.01
Extremely poor	0.01-0.10
Very poor	0.10-1.00
Poor	1.00-4.00
Fair	4.00-10.0
Good	10.0-40.0
Very good	40.0-100.0
Extremely good	100.0-400.0
Exceptionally good	400.0-1000.0

ANNEXURE IV

HYDROGEOLOGICAL MONITORING, TESTING AND MEASUREMENTS

1.0 Testing of individual water curtain bore-holes

The various procedures employed for pressure testing and temporary pressurization of the water curtain boreholes and overall system are described below.

Lugeon tests or Injection-fall off water pressure tests are carried out in all the boreholes within 5 days of completion of each hole so as to estimate hydraulic conductivity before cavern heading excavation below. These hydraulic conductivity values are taken as input in to the hydro-geological model. Based on this critical hydro-geolological areas are demarcated and incorporated in the 3D Geological model for planning of probing and grouting in the caverns below.

1.1 Temporary pressurization of the water curtain system

All water curtain bore holes are pressurised with water supply at about 3 bars and is to be maintained throughout during construction. The injection water should be clean and compatible with the natural ground water and shall receive bacteriological treatment if it contains bacteria. However, turbidity should be low and the solid contents should be less than 10 mg/l. Maximum allowable bacteria contents are: Total aerobic bacteria: 1000 bacteria per ml, Total anaerobic bacteria: 1000 bacteria per ml, Sulphate reducing bacteria: 0 bacteria per ml, Slime forming bacteria: 0 bacteria per ml. This water supply line shall not be used for any other purpose than supply of water to the water curtain boreholes.

1.2 Simple Pressure Observation Test (SPOT test) in all boreholes

Simple pressure test is performed on water curtain boreholes in order to identify any zones of grouting in cavern heading before start of cavern benching. During this test, seepage in cavern heading is monitored and its locations are mapped in a seepage map along with its quantity. Water intake in all the tested water curtain boreholes are monitored under full pressure (generally 3 bars) for 2 days and drop in pressures are monitored for another 2 days. Care must be taken not to de-saturate completely any borehole. These tests are done for 100m length at a time, in order to prevent any risk of total de-saturation. Ground water levels in monitoring wells/surface piezometers near by these tested boreholes are closely monitored for drop in water levels during this test. Sensitive boreholes with quick pressure drop and/or high water intake are identified during this test.

1.3 Pressure build-up test (PBT) in selected boreholes

Some of the sensitive water boreholes are additionally tested with pressure build-up test (PBT). The purpose is to check the hydraulic connectivity between the individual boreholes, through the water bearing geological feature. PBT involves one test borehole and generally four monitoring boreholes; two monitoring boreholes each on either side of the tested borehole. These monitoring boreholes could be nearby boreholes in the same water curtain tunnel and can also be in adjacent water curtain tunnels intersected by the same water bearing geological structure/ dyke. This extended test is called extended pressure buildup test (EPBT). During the test, water supply is cut off from all the tested and monitoring boreholes for 24 hours and drop in pressures are monitored first. Then the test borehole is opened for 30 min and water loss is measured every 5 min simultaneously drop in groundwater pressure in other monitoring boreholes are also noted. Then the valve of tested borehole is closed back, and the corresponding water pressure build-up is recorded in test and monitoring boreholes for another 30 min. Water supply to the boreholes are restored back after the test. During Extended Pressure build-up test, the test period is prolonged for 48 hours instead of 24 hours to influence monitoring boreholes further away from the test borehole. Ground water levels in monitoring wells/surface piezometers near by test boreholes are monitored closely during this time.

1.4 Water Curtain Efficiency Test

This test shall be carried out to detect unfavourable hydro-geological conditions and to decide requirements of additional water curtain boreholes. This test shall be carried out for horizontal water curtains when the top heading of the cavern is completed and no construction activity in that segment. For vertical water curtains, the efficiency test is performed when the cavern is fully excavated. The test consists of three phases according to different hydro-dynamic status. In each of the phases, the pressure on all the water curtain boreholes and the water level in the surface piezometers shall be monitored. The test duration normally last from couple of day to a week. The frequency of measurements is according to real site conditions. The test is continued until the pressure stabilizes. In the first phase, the hydro-static pressure along the water curtain is monitored by closing all the valves to the water curtain bore-holes. Water level on surface piezometers, seepage in water curtain tunnel and cavern sections are measured. In the second phase, the valves of every alternate water curtain boreholes shall be kept open and injected with water. Water flow for injected holes is measured. Pressure on all boreholes both open and closed, the water level of surface piezometers and seepage in water curtain tunnel and cavern sections are also measured. In the third phase, the valves of water curtain boreholes, which were closed during phase 2 shall be opened and the valves of those boreholes which were open in phase 2 are closed. Similar measurements as in phase 2 are taken. Care must be taken not to de-saturate any borehole completely. If during any of the phases, the pressure of the water curtain borehole drops to zero, water injection is to be resumed to keep the pressure just above zero (say < 0.5 bar) and the respective flow rate are measured. Based on the results of this testing, the efficiency of water curtain boreholes are estimated and additional water curtain boreholes are recommended.

2.0 Seepage Measurement After Grouting

Hydro-monitoring & seepage measurements help to understand the groundwater balance of seepage versus recharge (natural and artificial). The monitoring/ measurements are essential to judge requirement of grouting as well as effectiveness of grouting. It is also mandatory to evaluate residual seepage and compare it with the designed seepage levels to confirm the adequacy of design for seepage pump in shaft.

The daily hydro-geological monitoring as shown in Figure 1.0 comprised mainly of

1. Ground water level monitoring through surface piezometric wells
2. Hydraulic potential measurement from underground WCG by manometer and pressure cells. These were installed adapted to monitor identified major water bearing features.
3. Pressure and water intake measurement of all WCBH.

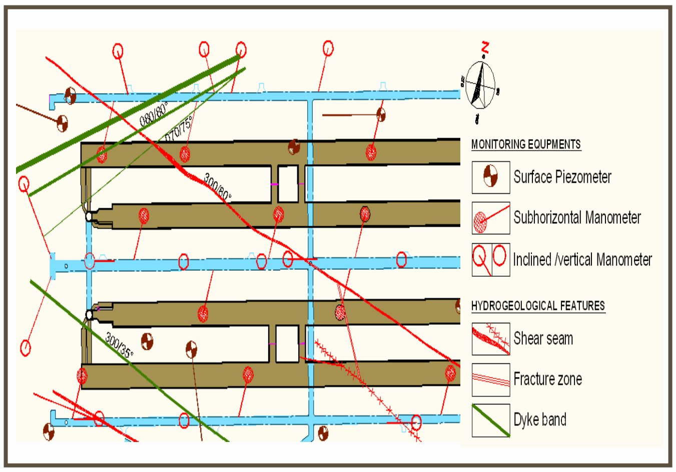


Figure 1.0: Typical hydrogeological monitoring plan.

In the project, seepage measurements were carried out through following methods:-

2.1 Indirect seepage measurement

It is the daily difference of outgoing and incoming water assessed by using flow meters. The total seepage from individual seepage points on crown and walls were mapped and measured monthly to have an idea about change in locality as well as quantity of seepage.

2.2 Direct seepage measurement

This is the total seepage measurement in isolated sections. Isolation was done by constructing concrete/clay weirs across the gallery. Management of WCBH during grouting was an important aspect. The water pressure of the boreholes in the affected area was lowered to facilitate rock bolting and grouting. However constant flow at optimum pressure was required and pressure was monitored during gout injection so that the holes do not get clogged with grout mix.

ACKNOWLEDGEMENT

A book on underground storage technologies was an idea that took shape based on the gained insight during execution of the coveted strategic storage projects. Challenges, Risks and Uncertainties were the three facets of the cited underground projects and we had an excellent opportunity to learn and assimilate the knowhow. Therefore, with an objective of knowledge sharing, this compendium was conceived and developed in the form of an edited volume.

An endeavour of this kind started with lot of enthusiasm and pushed us out of our comfort zone to commit ourselves for this niche initiative. And, this could be completed with the active support of our colleagues, associates and friends from EIL, ISPRL and other organisations. In particular we would like to thank R. Mohan, N. S. Vasudev and A. K. Maheswari for the detailed review of the volume along with all authors for their contribution.

We wish to place on record our acknowledgement through this recital of names of those well wishers who directly or indirectly helped us shape this compilation.

Anil Attavar, H.P.S.Ahuja, Neeraj Aggarwal, C. C. Babu, Mahesh Babu, Balasubramanian, N Baladhandayutapani, K.V.Chelapathi, S.Chanda, K K Chopra, Pavan Chakraborthy, Y.S.Cho, Ajay N Deshpande, Pascal Deschampes, M. Deshpande, Gosta Ericson, Sanjay Gupta, R.K.Grover, P.Gouda, Anupam Hazra, Thomas Holm, Blanca Van Hasslett, S R Hasyagar, Rajiv Jain, V Louis Joffy, K.S.Jakhar, V.K.Kaul, Gopi Kannan, K.G. Sharma, Ashiwini Kumar, Chandan Kumar, Pradeep Kumar, Florian Krenn, Sunil Kurmana, N. Kaul, U.V.Lakhane, Sophie Laurent, C.Y.Lee, J.M.Lee, Deepak Moudgil, A. Mahantta, A.K.Maheswari, Anirban Mandal, K.Y.Mallesh, R. Mohan, S.K.Mohanty, R.P. Mehrotra, S Malkani, Rakesh Nanda, J C Nakra, Rajiv Nair, A.K. Purwaha, G.Pradhan, Saikat Pal, Bhaskar Pal, Amitabh Pal, Rajan.K.Pillai, Mukesh Rohtagi, Lakshman Rao, T. Ramamurthy, Pierre Roux, P.V.Rao, N Sita Ramaiah, R Priyamvada, B Eswara Rao, P Veer Bhadra Rao, N. Rajkhowa, Dickpal Rajkhowa, Ashwani Soni, Niraj Sethi, Adarsh Saran, Charanjit Singh, Ashok Singh, N. Sitaramiah, Pawan K Singh, Oskar Sigl, Carl Soder, M.C.Singh, Vijay Shahri, Anureet Sahi, Veena Swarup, R.K. Sabharwal, Ram Singh, D Saraswat, Ashok Singh, Arpan Srivastav, Anita Sarin, A.K.Tripathi, L.K.Vijh, N.S.Vasudev, Rakesh Vineet, B Vijaykumar, Philippe Vaskou, S.Vijayananda, You.J.W.

In addition to the aforesaid recitals, experts of several organisations and professional societies have also contributed to this capacity building initiative namely G.S.I., RITES, NIRM, CGWB, CSMRS, CWC, CRRI, CBIP, IGS (Delhi), ISRM (India), ISRMTT, ISEG, IIT Delhi, IIT Bombay and Utkal Univeristy, Bhubaneswar etc.

Given the magnitude of these projects and the interfaces; with the possibility of inadvertent omissions the aforesaid list is endless and therefore we leave this acknowledgement incomplete, in their reminiscence.

Dr. A. Nanda, Dr. R. Rath, Dr. A. Usmani.

Editors

BRIEF RESUMES OF EDITORS & AUTHORS

Dr. Atul Nanda has over 30 years of academic, research and industry experience in geotechnical engineering. He has a B.Tech in civil engineering from IIT Bombay and Ph.D in geotechnical engineering from Virginia Tech USA. Presently he is Head (Technology) of the subsurface group in Engineers India Limited (EIL) responsible for the design and engineering of the stragetic crude oil storage projects in underground unlined rock caverns. He has worked on over a 100 projects covering refineries, petrochemicals, pipelines, ports, offshore and underground storage. Before joining EIL he was a senior research fellow at CERMES, ENPC, Paris, France for 2 years. He is chairman of the Indian Geotechnical Society (Delhi Chapter) and a member of the Governing Council of ISRM (India). He has published vast number of papers in international and national publications in journals and conferences and recipient of several awards in geotechnical engineering.

Dr. Ranjit Rath, an alumnus of IIFT with an Executive MBA in International Business, is currently working as Asstt. General Manager, Corporate Strategy and Business Development at Engineers India Ltd.,(EIL) a consulting organization under MoP&NG. Prior to his present assignment, as Sr. Manager with Sub Surface Projects Division, he was the project coordinator and responsible for all the geoscientific aspects of strategic storage projects involving storage of imported crude oil in underground unlined rock caverns and storage of crude oil and natural gas in solution mined salt caverns. As part of the business growth initiatives and diversifications at EIL, Dr. Rath is responsible for new investments and projects specifically in sub surface domain. He is the Operating Committee member for two upstream assets owned by EIL. An alumnus of IIT Bombay and IIT Kharagpur, Dr. Ranjit Rath is an active member of several professional societies and committees. He has published more than 30 technical papers in journals and conferences in the field of geosciences and its application in sub surface domain.

Dr. Altaf Usmani completed his B.Tech & M.Tech from AMU Aligarh and Ph.D from IIT Delhi in 2007. Since then he has been working as a Lead Geotechnical Expert in the field of design of underground structures for storage of crude oil. Within a short period of the last eight years, Dr. Usmani has published vast number of papers in international / national journals and conferences, covering both soil mechanics and rock mechanics challenges. Dr. Usmani is a recipient of "Young Geoetchnical Engineer Award 2010" from the Indian Geoetchnical Society (IGS) and "Best Paper Award on Underground Space Technology" in 2013 from Indian Society for Rock Mechanics and Tunneling Technology. Dr. Usmani has organized number of national and international conferences in Delhi region and delivered invited key note lectures at several academic institutes and conferences. Dr. Usmani is Hon. Secretary of IGS, Delhi chapter and is also the co-editor of IGS Delhi Chapter newsletter published bi-annually which provides information about the events and activities of the IGS Delhi chapter and knowledge sharing.

HPS Ahuja, a seasoned offshore engineer, started his career with ONGC in 1983 and has extensive expertise in well testing, stimulation and well completion of deep offshore exploratory wells drilled in the Bombay High oil and gas fields. In ONGC he also steered the implementation of ERP project of SAP across the entire organization located all over India and abroad. As GM (Production) of ONGC, he has been on deputation to Indian Strategic Petroleum Reserves Ltd. (ISPRL) since 2008. Currently, as Dy. CEO of ISPRL, Shri Ahuja has been involved in implementation of the Phase I strategic storage program of Govt. of India at three locations namely Vishakhapatnam, Mangalore and Padur entailing storage of crude oil in underground unlined rock caverns. He has also been involved in preparation of the DPR for Phase II storage program which will entail two new underground storage technologies viz. solution mined salt cavern and inground concrete tanks. A mechanical engineer by academics from Bombay University, he also holds a Diploma in Management studies from Bombay University, as well.

Mr. Charanjit Singh did his Bachelor of Engineering from IIT Roorkee (formerly University of Roorkee) in Civil Engineering in 1996. With an excellent academic record he is the proud recipient of "President of India Sh. Shankar Dayal Sharma Gold Medal – 1996" for outstanding academic excellence. Mr. Singh joined Engineers India Limited (EIL) in 1996 as "Management Trainee". Since then he has worked in various capacities in the fields of refineries, petrochemicals, Non- Ferrous Metallurgy, Defence etc. He has been associated with mega projects like ESSAR Oil Refinery, Jamnagar and GAIL's Petrochemical Complex at Pata. Since 2004, he is working in the field of Offshore Engineering looking after various projects for ONGC, GSPC, Middle East Clients etc. He was also associated with the recently commissioned DDW1 & PLQP Platforms for GSPC in KG Basin. He is presently working as Assistant General Manager (Offshore Engineering Department), Engineers India Limited, New Delhi.

Mr. Saikat Pal graduated from Presidency College, Kolkata in 1991; post graduated from University of Calcutta in 1993 and completed M.Tech (Applied Geology) from I.I.T., Kharagpur in 1996. From 1996 to 2002 he worked for Oil Field Instrumentation Pvt. Ltd. (OFI) as a Geologging Engineer in onshore and offshore oil and gas exploration fields of India and Bangladesh. In 2002 he joined NHPC limited and worked as Geologist and Assistant Manager (Geology) at several locations such as Arunachal Pradesh, Bhutan & at the Corporate Office of NHPC. During this period he was associated with site investigations and construction of hydro power projects at Uttaranchal, Himachal Pradesh, Arunachal Pradesh and Bhutan. He joined Engineers India Limited (EIL) in 2010 at the strategic storage project site, Mangalore and was responsible engineering geological aspects of underground crude oil storage caverns. Currently, Sh. Pal is working as Manager in Subsurface project division of EIL. He is a life member of ISEG, ISRMTT and IGS, Delhi and has presented technical papers in seminars of ISEG, ISRMTT, CBIP and NIRM and published papers in Journal of Engineering Geology (JOEG) and Journal of Rock Mechanics and Tunneling Technology (JRMTT).

Mr. Gopi Kannan is working in the Subsurface Projects Division at Engineers India Limited, as an Hydro-geologist / Engineering Geologist for the underground rock cavern crude oil storage projects. Prior to this, he has worked in National Hydro Power Corporation (NHPC) as Engineering Geologist specializing in underground structures such as Power House Cavern, Dam complex, Head Race Tunnels, etc. Sh. Kannan has graduated in Applied Geology from College of Engineering, Anna University, Chennai and received M.Tech from IIT Roorkee (formerly University of Roorkee) and is pursuing Ph.D from IIT Delhi. He has vast experiences of more than 14 years in underground excavations and has published about 20 technical papers in National and International Journals and Conferences. Sh. Kannan is a life member of Indian Society of Engineering Geology (ISEG) and its International body, and is also a member of Indian Society of Rock Mechanics (ISRM) and Indian Geotechnical Society (IGS), Delhi Chapter.

Mr. Chandan Kumar did his Graduation in Civil Engineering from Muzaffarpur Institute of Technology, Bihar 2000 and Masters in Rock Mechanics from Indian Institute of Technology, Delhi, in 2002. Presently, he is pursuing his part time PhD from IIT Delhi.

After M. Tech, he joined Consulting Engineering Services in 2003, then National Hydro Power Corporation in 2004 and presently working as a Manager in Engineers India Limited (EIL) from 2011 onwards. He has over 12 years of experience in the design of various underground structures, design of tunnels & large caverns & geotechnical works for a no of hydropower projects and strategic storage projects. He has a proficient skill in the use of various geotechnical software used in the design of underground structures. He has published a no.of technical papers in National & International Conferences. He is also involved in developing standard specifications in the field of rock engineering for EIL. He is also members of professional societies namely International Society of Rock Mechanics, Indian Society for Rock Mechanics and Tunneling Technology, Indian Geotechnical Society (Delhi Chapter) etc.

Mr. Devendra Kumar Saraswat has completed his B.Sc. (Geology) in 1991 and M.Tech (Applied Geology) in 1994 from Dr. H.S.Gaour University, Sagar (M.P). He has over 20 years of work experience in the field of Engineering Geology / Underground construction out of which more than 15 years has been spent with leading construction companies of India namely L & T, Continental Foundation JV and CCL India for Hydropower projects namely Nathpa Jhakri Hydroelectric Power Project, Kinnaur, H.P (1500MW), Kol Dam, H.P (800MW), Parvati Hydroelectric Power Project-Stage III H.P.(520MW). He joined EIL in March 2010 at the strategic storage project site, Vishakhapatnam and later moved to the project site at Padur. As Manager (Geology) he is presently working in the field of offshore and onshore site investigations related to geological, geophysical and oceanographical aspects for Offshore Engineering Department, of EIL.

Mr. N. S. Vasudev, Deputy General Manager at Engineers India Limited, joined EIL in 1986 after degree in Electrical and Electronics Engineering from Annamalai University. He was Project Manager for Mangalore, Padur & Pipeline projects. Mr. Vasudev has 29 years of multidisciplinary experience in managing construction projects at various positions and locations in India & abroad. The responsibilities include Overall Construction Management, HSE management, Progress monitoring & expediting, Cost control and Quality Assurance / Control. Also, monitoring of Engineering/Procurement activities of contractors, conducting technical audits, field tendering etc. Mr. Vasudev conversant with job related software packages and is well versed with National/International codes of practice.

Mr. Vijay Shahri, Deputy General Manager at Engineers India Limited, joined EIL after Degree in Civil Engineering from MANIT (REC) Bhopal in 1986. He was Resident Construction Manager for Mangalore, Padur & Pipeline projects. Mr. Shahri has 29 years Multi-disciplinary experience in managing construction projects at various positions and locations in India & abroad. The responsibilities include Overall Construction Management, HSE management, Progress monitoring & expediting, Cost control and Quality Assurance/Control, monitoring of Engineering / Procurement activities of contractors, conducting technical audits, field tendering etc. Mr. Shahri is conversant with job related software packages and is well versed with National/International codes of practice.

Mr. K Y MALLESH, Assistant General Manager at Engineers India Limited did his B.E.(Civil) from Karnataka University Belgaum in 1989, Post Graduate Degree in Business Administration (MBA) in 1994 and Dip. in Construction Management in 1993. He is Associate Member of IEI. Mr. Mallesh has 25 years of Experience in Projects Monitoring, Construction Management & Execution and out of which over 22 years Experience is in Oil & Gas Industry alone with Engineers India Limited. Mr. Mallesh worked as Head of Project Site for Under Ground Rock Caverns for the Strategic Crude Oil Storage Project for M/s ISPRL at Padur-Udupi, Karnataka, India.

इंजीनियर्स इंडिया लिमिटेड
(भारत सरकार का उपक्रम)

ENGINEERS INDIA LIMITED
(A Govt. of India Undertaking)

1, Bhikaiji Cama Place, New Delhi-110066
Tel.: +91 11 26762121
Website: www.engineersindia.com

Delivering Excellence through People