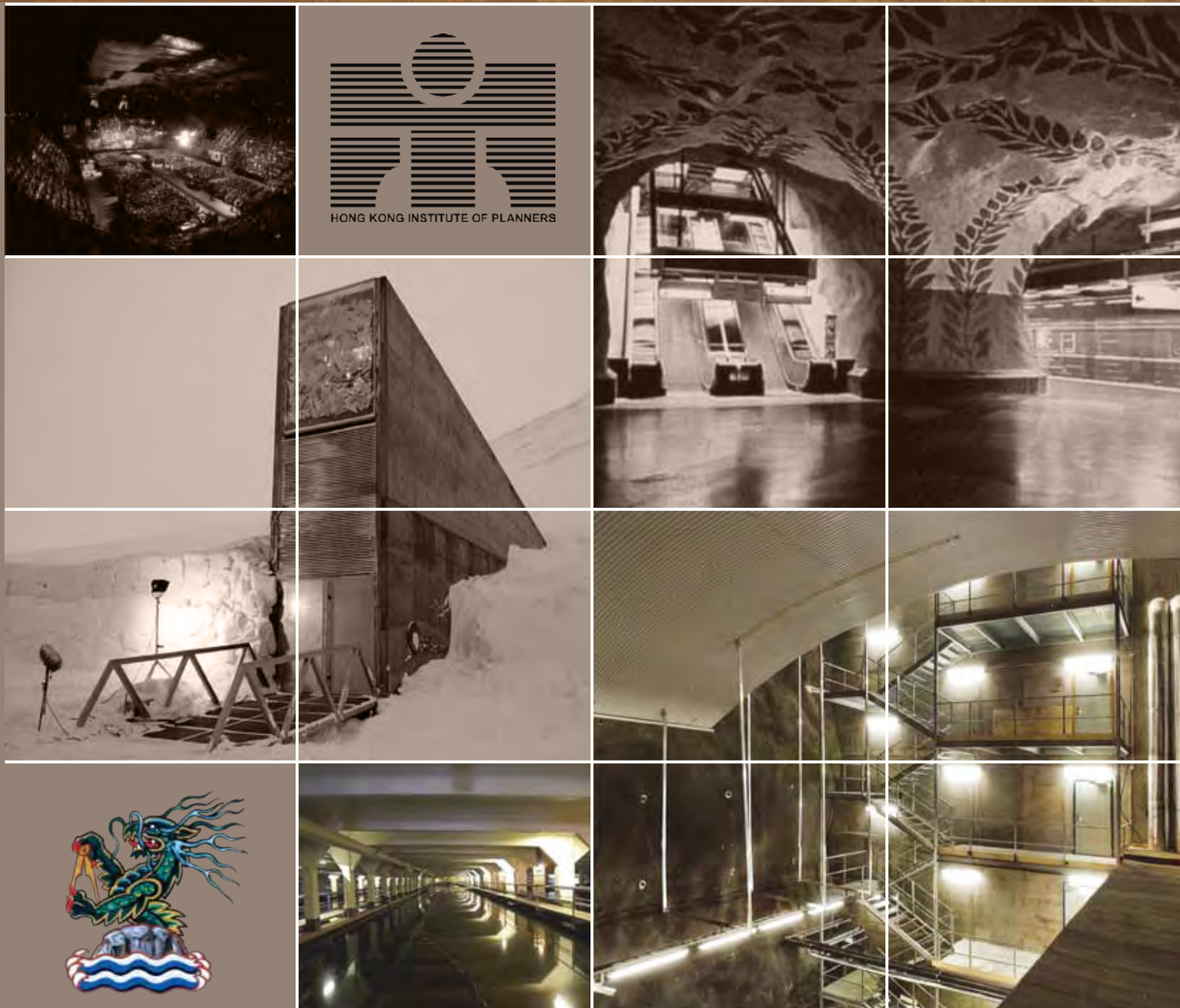


# PLANNING AND DEVELOPMENT OF UNDERGROUND SPACE

*Bridging planners, engineers and the public on innovative use of underground space*



# Proceedings of the Joint HKIE-HKIP Conference on **Planning and Development of Underground Space**

**23 – 24 September 2011  
Hong Kong**

**The Hong Kong Institution of Engineers  
The Hong Kong Institute of Planners**

## Cover photographs

Top left : The Gjøvik Olympic Mountain Hall in Gjøvik of Norway;  
Top right : The Underground Railway Station in Stockholm of Sweden;  
Middle left : The Svalbard Seed Bank, Cold Storage in Svalbard Lufthavn of Norway;  
Bottom left : The Underground Water Storage in Helsinki of Finland; and  
Bottom right : The National Archives in Oslo of Norway.

Photos courtesy of Ove Arup & Partners Hong Kong Limited.

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ISBN 978-962-7619-49-9



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# Foreword

Land is a scarce resource in Hong Kong, especially in the urban areas. The geological conditions and topography of Hong Kong offer a great opportunity for placing suitable facilities in rock caverns. The Government of the Hong Kong Special Administrative Region has recently completed a suite of strategic planning and technical studies to facilitate optimal use of underground space as part of urban planning aiming at enhancing the use of rock caverns to support Hong Kong's pursuit of sustainable development.

The Joint HKIE-HKIP Conference on Planning and Development of Underground Space aims at introducing this emerging field of urban and infrastructure development to senior practitioners and regulators with duties of longer-term business planning for their organisations. It will provide a platform for Hong Kong engineers and planners to meet and share experience with our overseas counterparts on international experiences on the planning and development of underground space, in particular on the potential of optimal use of rock caverns through creative planning in Hong Kong. To planners, the Conference will help promote the concept of systematic pursuance of opportunities for use of underground space. To engineers, the Conference will provide useful update on the opportunities and challenges ahead. The Conference will also help provide an opportunity for planners and engineers to appreciate the other profession's perspective, to produce synergy.

We want to thank all those who submitted papers for publication in this Proceedings. We also want to give a special thanks to the reviewers of the Technical Sub-Committee, whose effort and hard work reflect their commitment and dedication to the profession.

## **Ir Raymond K S CHAN**

*Co-Chairman, Conference Organising Committee  
Vice President, The Hong Kong Institution of Engineers*

September 2011

## **Mr LING Kar Kan**

*Co-Chairman, Conference Organising Committee  
Past President, The Hong Kong Institute of Planners*

September 2011

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# About the Organisers

## The Hong Kong Institution of Engineers (HKIE)

In 1947, the Engineering Society of Hong Kong, forerunner of the HKIE, was founded with an aim to bring together engineers of different disciplines for their common good. Over the next 20 plus years, the Society thrived and in 1975 was incorporated as The Hong Kong Institution of Engineers under The Hong Kong Institution of Engineers Ordinance. The HKIE is the only statutory professional engineering body in Hong Kong, currently with over 27,000 members, of which more than 12,000 are Corporate Members.

Committed to upholding the quality of membership, the Institution sets standards for the training and admission of engineers of 20 Engineering Disciplines. It also has strict rules governing its members' conduct and is dedicated to raising the ethical standard of professional engineers in Hong Kong. As a learned society, the 17 Divisions under the Institution regularly organise activities to keep members abreast of the latest engineering developments and also provide opportunities for Continuing Professional Development.

We pride ourselves as being a proactive social citizen by not only nominating our members to serve on various external boards and committees of government departments and higher education institutions, but also providing timely and constructive comments and professional advice on the government's consultation papers. In addition, the profession has one seat representing the Engineering Functional Constituency in the Legislative Council through which views of the profession can be expressed to the Government.

Enjoying a close relationship with other engineering institutions around the world, the HKIE has entered into various co-operation agreements and mutual recognition of qualifications agreements with engineering institutions in the Mainland and overseas. It is also a member of many international engineering institutions.

## The Hong Kong Institute of Planners (HKIP)

Formation of The Hong Kong Institute of Planners (HKIP) was initiated in the mid-1970s. After completion of the Constitution and By-laws and the Code of Professional Conduct, the Institute officially started operation in 1978 and was inaugurated in July 1979. In May 1991, the Institute became a statutory body with the enactment of The Hong Kong Institute of Planners Incorporation Ordinance 1991 by the Legislative Council.

The Institute is recognised by all concerned local and overseas organisations as the professional planning institute representing the planning profession in Hong Kong. The Council of the Institute appoints Members to sit on the Planners Registration Board established under the Planners Registration Ordinance. The Institute's full membership is accepted by the Government of the Hong Kong Special Administrative Region and private consultant firms for appointment purposes. There are about 600 members at present.

The Institute is one of the professional institutes in the Architectural, Surveying, Planning and Landscape Architectural functional constituency for the Legislative Council elections. It is also one of the ten professional bodies which meet to discuss matters of common interest. Its members sit on such committees as the Antiquity Advisory Board, the Land and Building Advisory Committee, the Advisory Council on the Environment and the Professional Persons Environmental Consultative Committee as well as their sub-committees. It is a member of several international professional bodies including the Eastern Region Organisation on Planning and Housing (EAROPH).

## Welcome Message from Guest of Honour Secretary for Development The Government of the Hong Kong SAR, China



I would like to congratulate The Hong Kong Institution of Engineers and The Hong Kong Institute of Planners for hosting the Joint Conference on Planning and Development of Underground Space and to welcome Conference participants.

Hong Kong's steep hilly terrain limits the supply of flat and readily available land. To support our social and economic development, there is a pressing need to supply the land by sustainable and innovative means.

The strong rocks underlying most of Hong Kong are suitable for housing a range of urban facilities in caverns. Cavern development is one of the options that could balance the economic and social need against the aspiration of having a better living environment. The Government is strongly committed to the systematic and optimal use of caverns in Hong Kong.

From putting facilities underground out of necessity in the 1980s to purpose-built caverns in the 1990s, the use of caverns is not new to Hong Kong. There is proven local experience in cavern construction as demonstrated by the various schemes built in Hong Kong. These projects have fully demonstrated the tangible and intangible benefits brought about by rock caverns in terms of cost-effectiveness, environmental friendliness, safety and security.

The two-day Conference will include topics from optimal use of underground space through planning to geotechnical and non-geotechnical aspects of cavern engineering. It will provide a platform by which planners, academics, engineers and contractors can share their vision, experience, technological advancement, as well as the challenges ahead. I am glad that I can be part of such synergy. I am sure the knowledge that will be shared and discussed at the Conference will provide us with valuable insight into ways of promoting the use of caverns in Hong Kong and around the world.

I wish the Conference every success.

**Mrs Carrie LAM, GBS, JP**

*Secretary for Development  
The Government of the Hong Kong SAR, China*

## Welcome Message from President of The Hong Kong Institution of Engineers



On behalf of The Hong Kong Institution of Engineers, I would like to extend a warm welcome to all participants and our distinguished speakers to this Conference on Planning and Development of Underground Space jointly organised with The Hong Kong Institute of Planners.

This Conference gives engineers and planners an excellent opportunity to meet and discuss future use of underground space in Hong Kong following Hong Kong Government policy initiatives and the recently completed study commissioned by the Civil Engineering and Development Department, with the support of the Planning Department, on the “Enhanced Use of Underground Space in Hong Kong”.

I sincerely thank the Development Bureau for spearheading debate in the use of usable underground space and their support for this Conference. I would also like to express my deepest gratitude to all the sponsors and supporting organisations for their staunch support and my sincere appreciation to the Organising Committee.

I wish all participants a fruitful and stimulating Conference.

**Ir Dr CHAN Fuk Cheung**

*President, The Hong Kong Institution of Engineers*

## Welcome Message from President of The Hong Kong Institute of Planners



It is a great pleasure to welcome delegates to the Joint HKIE-HKIP Conference.

The subject of this Conference on Planning and Development of Underground Space relates to an initiative in the 2009-10 Policy Address entitled “Breaking New Ground Together”. Under Chapter 1 of the Policy Agenda – ‘Developing the Infrastructure for Economic Growth’ it was proposed to launch strategic planning and technical studies to facilitate planned development of underground space aimed at promoting sustainable development in Hong Kong. By promoting the use of both purpose-built rock caverns and large excavations formed by cut and cover methods, scarce land can be provided for other uses.

Land is a scarce resource in Hong Kong. Since the 1990s the continuing demand for usable land, and concern at the growth of various environmental problems, have made this option attractive. The technology involved in this process has been well tested in Hong Kong but has as yet not been applied to the formation of underground space on as large a scale, or for as wide a variety of uses, as is already common in other parts of the world.

In recent years a number of notable studies have been carried out in Hong Kong based on general engineering and geological information. Rock cavern development has also been addressed in the Hong Kong Planning Standards and Guidelines which sets out an applicable planning process and potential candidate land uses, including the consideration of a rock cavern option in the early planning process for new government projects. Three government facilities have been purpose-built in rock caverns since the mid-1990s.

This Joint Conference establishes a platform for Hong Kong planners and engineers to discuss and share current experience with overseas counterparts. It also provides an opportunity for planners and engineers to appreciate some of the issues at stake from their dual perspectives.

I would like to thank the organisers and the Conference Secretariat for their work in bringing this important Conference to fruition.

**Dr Peter C SMITH**

*President, The Hong Kong Institute of Planners*

## Welcome Message from Co-Chairman of the Conference Organising Committee Cum Vice President of The Hong Kong Institution of Engineers



Hong Kong's steep terrain has posed severe constraint on the availability of flat land. To meet the challenge of fuelling the growth of Hong Kong, putting facilities in caverns can be one of the sustainable forms of development. In order to enhance our understanding on the issues involved and to meet the challenges ahead, a platform is needed by which issues arising from the planning, design, construction and operation can be shared among practitioners, academics and policy makers. It is with this purpose that The Hong Kong Institution of Engineers and The Hong Kong Institute of Planners have jointly hosted this Conference on Planning and Development of Underground Space.

We have come to a stage, apart from the challenge posed by the shortage of easy land, we also have to take due account of the society's aspiration for a better and sustainable living environment and our experience has shown that the use of caverns will address some of those demands.

We have invited speakers from countries, where considerable experience has been gained in cavern usage, to share with us their visions, successes and difficulties, notably on their holistic approach in dealing with the planning and associated development issues. Locally, we have amassed a wealth of experience in housing public facilities underground since the 1980s, and have considerable success in doing so especially for those environmentally unattractive facilities. I am sure the exchange of knowledge between overseas and local participants in this Conference will go a long way to better equipping ourselves for the future use of underground space.

I am particularly grateful to our colleagues from The Hong Kong Institute of Planners for their efforts in arranging the invitation for contributions, editing of submissions and the selection of speakers on planning issues.

**Ir Raymond K S CHAN**

*Co-chairman, Conference Organising Committee,  
Vice President, The Hong Kong Institution of Engineers*

## Welcome Message from Co-Chairman of the Conference Organising Committee Cum Past President of The Hong Kong Institute of Planners



The common mission of making Hong Kong a liveable and sustainable city has brought planners and engineers together to organise this Conference.

Production of developable land to sustain the growth of Hong Kong has long been our hectic challenge since the formation of this city in the mid-19th century. For the post-war generations like many of us, we are no strangers to the large-scale earth moving works of cutting development platforms at the foothills and forming reclaimed land along the foreshores of the Victoria Harbour in the 1950s to 1970s. In the decades of 1980s and 1990s, many of us not only witnessed but also participated in the planning and engineering process for development of new towns and the New Airport Core Programme which involved excavating marine fills for extensive land formation.

In the new century we withheld our conventional large-scale land formation action for a while as the community at large has puzzled on what would be the sustainable way forward for the city in producing land to meet our growing needs. This Conference can be seen as a response from planners and engineers to this puzzle.

Whilst the deliberation at this Conference would not provide a holistic answer to the wider issue of sustainable land production, I am confident that it can help putting some jigsaws in the right places for the whole puzzle. For this, we are most grateful to all speakers and participants who share with us experience and foresights in underground development as a sustainable way of creating development space within the urban fabric.

I must thank The Hong Kong Institution of Engineers (HKIE) which has shouldered not only the financial burden but also most of the organising works making this Conference possible and a success. I am honoured for having the opportunity to work with the HKIE team which is professional, dedicated and efficient. We are joining hands in the process of building consensus with the community on the sustainable development of Hong Kong.

**Mr LING Kar Kan**

*Co-chairman, Conference Organising Committee,  
Past President, The Hong Kong Institute of Planners*

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Urban Planning Society of China

# Acknowledgements

The Organising Committee would like to thank the following organisations for their support and valuable contributions to the Conference :

**科正 建築有限公司**  
Fraser Construction Company Ltd.



**SIU YIN WAI & ASSOCIATES LTD.**  
CHARTERED ENGINEERS & AUTHORIZED PERSONS  
邵賢偉建築工程師有限公司

**Tai Kam Construction Engineering Company Limited**

We are indebted to all the speakers for giving us their time and sharing their wisdom with the participants and to all those individuals whose names may not have been mentioned above but have assisted and supported us in one form or another.

# HELSINKI EXPERIENCE WITH MASTER PLANNING FOR USE OF UNDERGROUND SPACE

VÄHÄÄHO, Ilkka

Helsinki City Real Estate Department, Finland

## Abstract

*This paper examines the continued growth in the use of underground space beneath the city of Helsinki and the continued widespread lack of significant and broad planning efforts for its use. The particular focus of this paper is on sustainability issues related to urban underground space use including contribution to an environmentally sustainable and aesthetically acceptable landscape, anticipated structural longevity and the maintenance of opportunity for urban development by future generations.*

*Since the 1960s, the City of Helsinki has been adept at widely utilizing the opportunities for underground construction. More than 400 premises and over 200 km of tunnels have already been built underground. Furthermore, there are more than 200 new reservations in the register for long-term underground projects. Demand for underground facilities in central Helsinki has grown considerably and, at the same time, the need to control construction work has increased substantially.*

*The underground master plan of Helsinki reserves designated space for public utilities and important private utilities in various underground areas of bedrock over the long term. The master plan also provides the framework for managing and controlling the city's underground construction work, and allows suitable locations to be allocated for underground facilities.*

*When selecting these resources, the survey took into account their bedrock conditions, accessibility, the present and planned ground-level uses of these areas, traffic connections, land ownership and possible recreational, landscape and environmental protection values.*

## Keywords

*Land Use Planning, Underground Resources, Master Plan, Sustainability, Urban Development*

## 1. Introduction

Since the 1960s, the City of Helsinki has been adept at widely utilising the opportunities for underground construction. More than 400 premises and over 200 km of technical maintenance tunnels have already been built underground. Demand for underground space in

the central city area has grown rapidly as the 21st century was approached and begun, and at the same time, the need to control construction work has increased substantially.

As the city's structure becomes more dense, more and more facilities suited for different purposes are being placed underground. There is also a growing demand for connecting underground premises to each other to form coherent and interrelated complexes, whilst the safety of the functions and operations of the facilities has become a major factor.

While planning and carrying out new construction projects, it is important to make sure that the space reservations for public long-term projects, such as tunnels and ducts for traffic and technical maintenance, are retained for future construction. Similarly the use of the valuable and unique rock and ground must be practical and done without wasting any future resources.

The growth in underground construction and planning and the demand to co-ordinate different projects has led to a requirement to prepare an underground master plan for Helsinki. Having legal status, the master plan also reinforces the systematic nature and quality of underground construction and the exchange of information related to it. The underground master plan is a general plan that allows the control of the locations and space allocations of new large significant underground rock facilities and traffic tunnels and their interconnections [HELSINKI CITY, 2009]. The master plan will also safeguard already existing spaces because there are for example certain minimum distances for any future plans or constructions regarding those spaces.

Efforts have been made to alleviate the great demand for central city area underground space by allocating new rock resource areas suited for underground construction outside the inner city. Changing the purpose of rock facilities that have already been built is difficult, which further emphasises the importance of underground master planning. The Helsinki underground master is administrated by the Helsinki City Planning Department. The City Real Estate Department's Geotechnical Division qualified the areas and elevation levels in Helsinki which are suitable for construction of large, hall-like spaces.

## 2. Helsinki Conditions and Land Ownership Strategy

Finland has 336 independent municipalities as in 2011. Helsinki is the capital city with 591,000 inhabitants and is clearly the biggest city in Finland by population. However, the surface area of Helsinki is only 215 km<sup>2</sup> whereas the average size of municipalities is 585 km<sup>2</sup>. The population density of Helsinki is 2,755 inhabitants per km<sup>2</sup>. Helsinki spreads across a number of bays and peninsulas and over a number of islands. The inner city area occupies a southern peninsula. The population density in certain parts of Helsinki inner city area is high, reaching 16,500 inhabitants per square kilometre.

The Helsinki metropolitan area is the world's northernmost urban area among those with a population of over one million people, and the city is the northernmost capital of an EU member state. Altogether 1.3 million people, approximately one in four Finns, live in the Greater Helsinki area.

Helsinki is located in southern Finland, on the shore of the Baltic Sea. Helsinki has a climate that is between sea and continental. Owing to the mitigating influence of the Gulf Stream, temperatures in winter are much higher than the far northern location might suggest, with an average in January and February around -5 °C (23 °F). However, because of the latitude, days last less than six hours around the winter solstice. Conversely, Helsinki enjoys long days in summer, close to nineteen hours around the summer solstice. The average maximum temperature from June to August is around 19 to 21 °C (66 to 70 °F).

Helsinki's landscape is quite flat; the highest natural spot is only 60 m above sea level. One third of Helsinki's ground is clay where the thickness of clay is over 3 metres and shear strength commonly around 10 kPa. The average depth of soil material upon bedrock is 7 m, but variation is from 0 to almost 70 m. Bedrock quality is ideal for tunnelling and for building underground spaces. The average price of underground space is only 100 €/m<sup>3</sup> (including excavation, rock reinforcement, grouting and underdrainage). For the time being only the drill and blast method has been used for rock excavations. The use of Tunnel Boring Machine has not been competitive in Helsinki so far.

Finnish people are used to having lots of green areas around, even in cities. That is also a good reason for using underground as a resource for those functions that do not need to be on surface. The unofficial motto of the Finnish Tunnelling Association is "Always examine the underground alternative". Among other reasons, safety is also a major aspect for using underground instead of building infrastructure near ground surface. Earthquakes are not a threat in Finland, but in case of those, underground solutions would be advantageous. Prof. Tetsuya Hanamura studied the effects of a natural disaster which took place in Japan in March 2011 and found out that the earthquake caused little if any structural damage to the metro tunnels. Moreover, the tsunami swept off almost everything on the ground - but caused only minor leaks in tunnels.

The City of Helsinki consists of 215 km<sup>2</sup> of land and 500 km<sup>2</sup> of sea. Within its administrative boundary, the City of Helsinki owns 65% of its land (Fig.1)

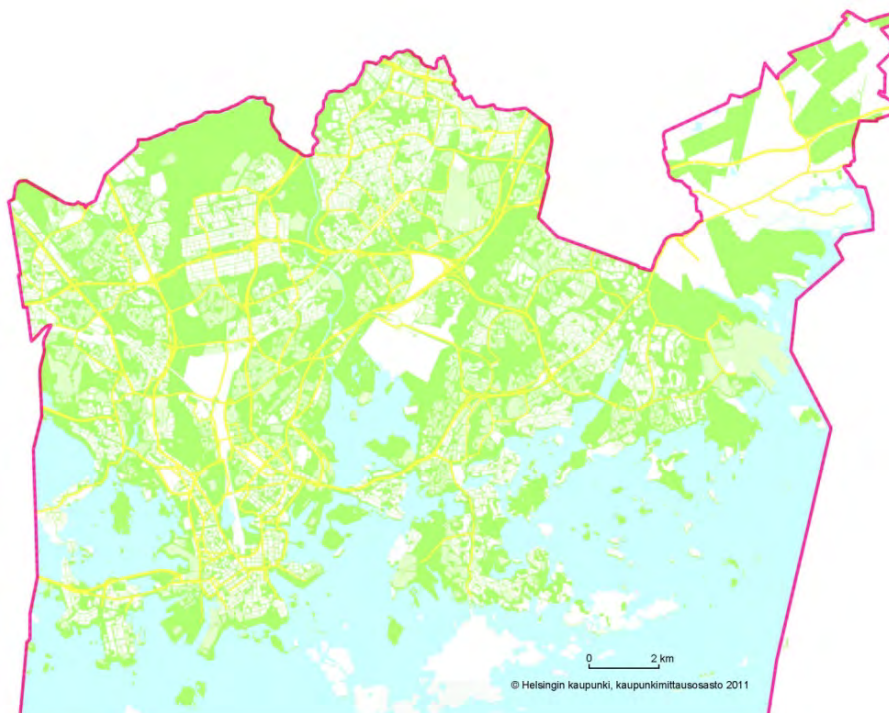


Fig. 1 The City of Helsinki  
(The green areas of land are owned by the City of Helsinki and the white areas are owned by others)

[HELSINKI CITY, 2010]. According to Real Estate Department's Land Division the city tries to buy the needed land areas as greenfield land (viz. undeveloped land either used for agriculture, landscape design, or left to naturally evolve) before city planning (zoning). As greenfield land is getting scarce, the city is, in spite of previous strategies, today more and more facing redevelopment of brownfields (previously used for industrial purposes), especially when developing waterfront areas. It is also easier to develop underground resources under your own real estate,

compared to building under somebody else's property.

### 3. Statistics of Underground Helsinki

Underground resources have an extremely important and central role in the development of the city structure of Helsinki and the adjoining areas, helping to create a more unified and eco-efficient structure (Fig. 2 & 3). Underground planning enhances the overall economy efficiency of facilities located underground and boosts the safety of these facilities and their use.

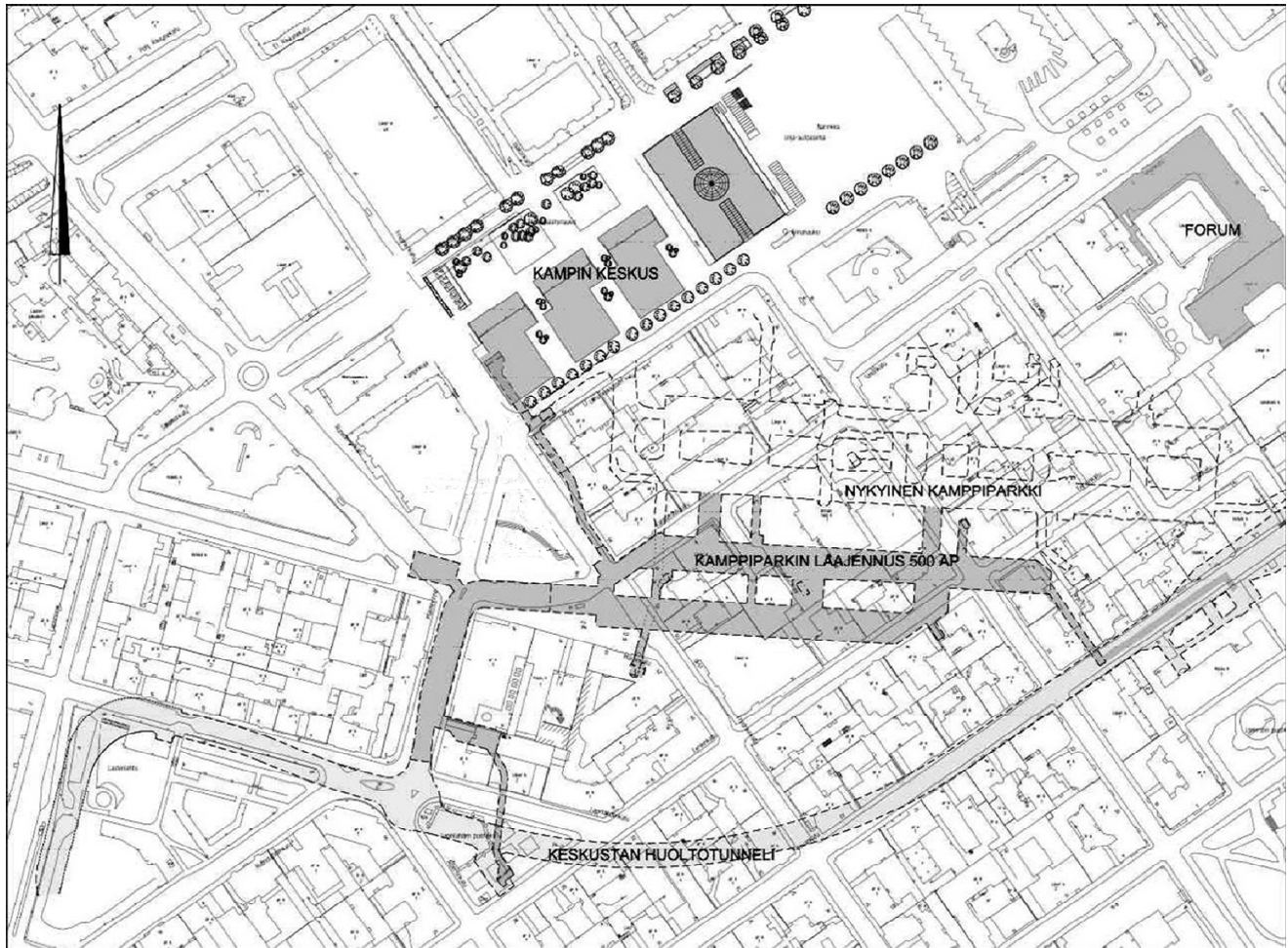
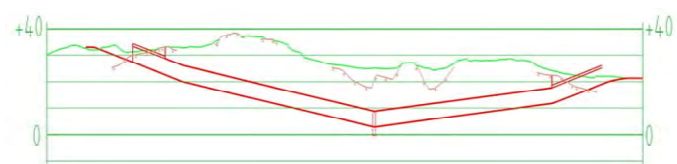


Fig. 2 An Eexample of the Development of the City Sstructure of Helsinki  
(An old car park (shown with dashed line) is connected to an extension and a new City Service Tunnel)  
(Image: Helsingin Vöylä Oy)



Aerial View (red line showing tunnel alignment)



Longitudinal Profile

Cross-section

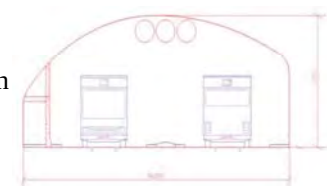


Fig. 3 Jokeri 2: Central Park Tunnel for Public Transport (Under Planning) to  
Connect Two Residential Districts

For the time being there are 10,000,000 m<sup>3</sup> underground spaces (parking, sports, oil- and coal storages, metro etc.), more than 400 premises, 220 km technical tunnels, 24 km raw water tunnels and 45 km utility tunnels “all in one” (district heating and cooling, electrical and telecommunications cables, water) [HELSINKI ENERGY, 2011]. Some quite unique examples are shown in Figures 4 and 5.

It is maybe easier to comprehend these statistics by comparing Helsinki’s surface area and the total area of underground spaces that are in use. As an average, under each 100 m<sup>2</sup> surface area there is 1 m<sup>2</sup> underground space. So, within the whole city area there still are lots of underground resources for future needs. This statistic is explained more clearly in Sections 4 and 5.

The underground swimming pool in Itäkeskus (Fig. 5) has facilities on two floors and can hold about 1,000 visitors at a time. The hall has about 400,000 visitors a year. Quarried out of solid rock, the hall can be converted into an emergency shelter for 3,800 people if necessary.



Fig. 4 Tempeliaukio Church Built into Solid Rock  
(Photo: Pertsaboy)

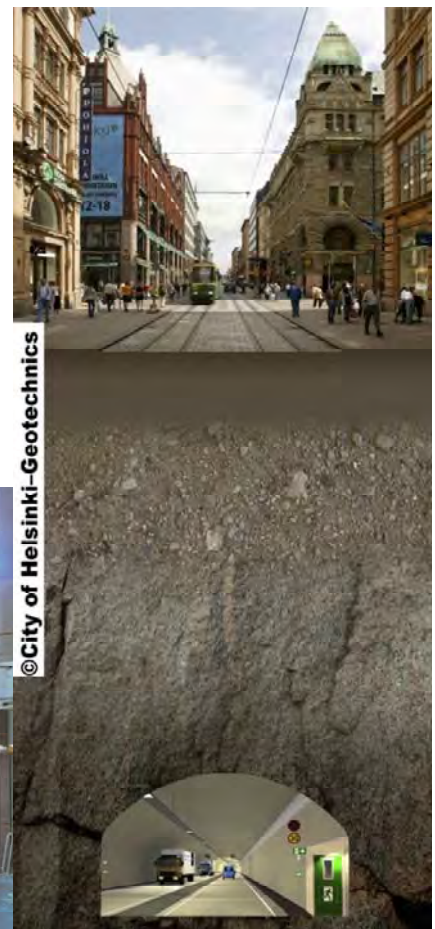
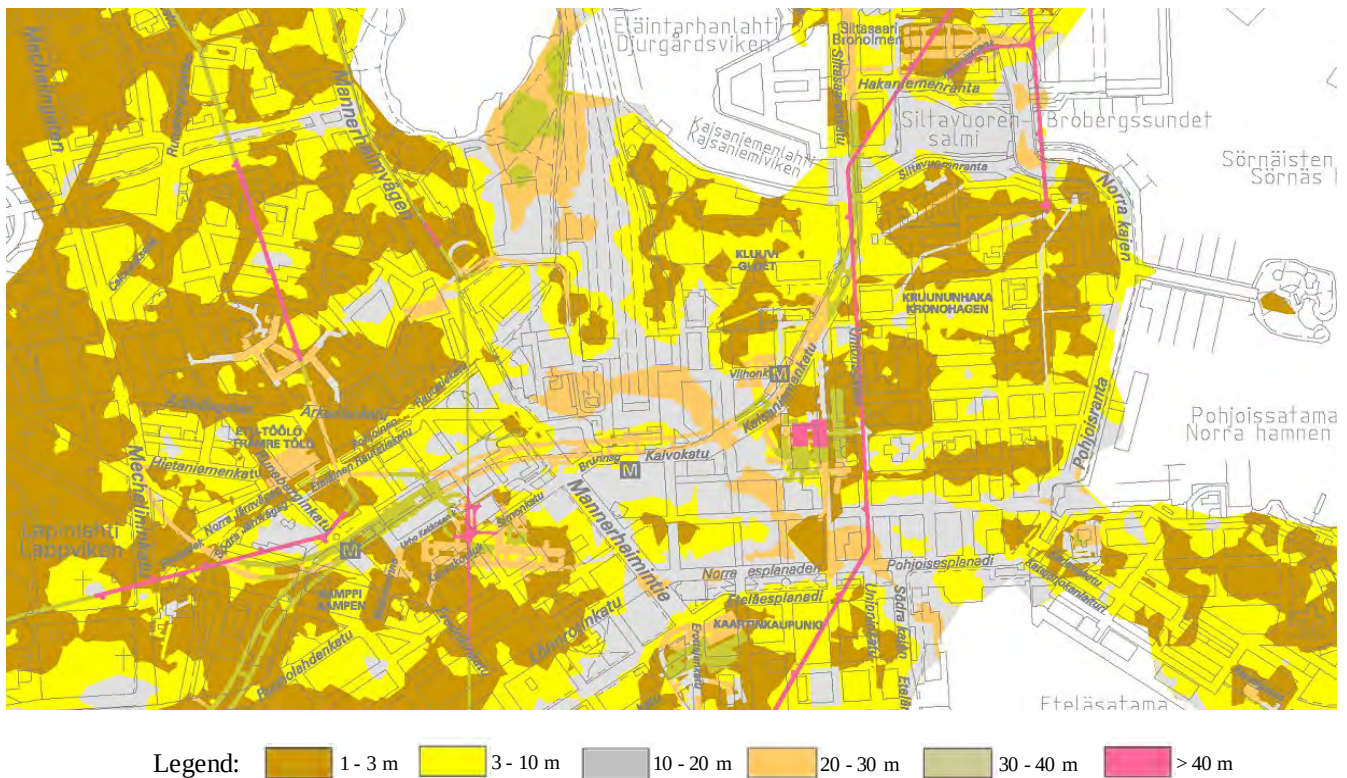


Fig. 5 Underground Swimming Pool in Itäkeskus (left) and City Service Tunnel in the City Centre (right)  
(Photo: Erkki Makkonen)

### 4. Rock Surface Model

An initial survey examined the areas and elevation levels in Helsinki which are suitable for construction of large, hall-like spaces. A model based on rock surface data was used applying a standard-sized measurement cave (width 50 m, length 150 m, height 12 m). The model of the bedrock is based on base map data for exposed rock and land surface elevations and point data obtained using drill machine borings (Fig. 6). The survey also took into account local weakness zones and rock resources that have already been put to use.

In general, it can be said that the bedrock in Helsinki is not far below the ground surface, and that there are plenty of reasonable and safe locations suitable for construction of underground facilities. Outside the city centre, the survey found 55 rock areas that had a sufficient size for accommodating large underground facilities near major traffic arteries. In many areas, future underground projects can make use of entrances to existing underground facilities, which have been marked with triangles on the master plan map. It is worth mentioning that thermal energy from bedrock is also a noticeable resource.



Definition: Deepest public underground spaces have been taken into consideration when presenting free rock resources. Estimated rock surface is based on bedrock confirmation drilling.

Fig. 6 An Extract of the Rock Surface Model

## 5. The First Underground Master Plan and Rock Resources Reserved for Unclassified Future Use

Space allocations for long-term projects such as traffic tunnels must be maintained for future construction. The same applies to those resources that are worth conserving for future projects. The exploitation of such resources must be carried out according to plan (Fig. 7). Excavating bedrock is a one-off action. Underground

master planning in Helsinki is today a significant part of land use planning process.

City of Helsinki has also reserved rock resources for unclassified future use for the construction of as yet unnamed underground facilities, with the aim of identifying good locations for functions suitable for placing underground and which would also reduce the pressures on the city centre's rock resources. The

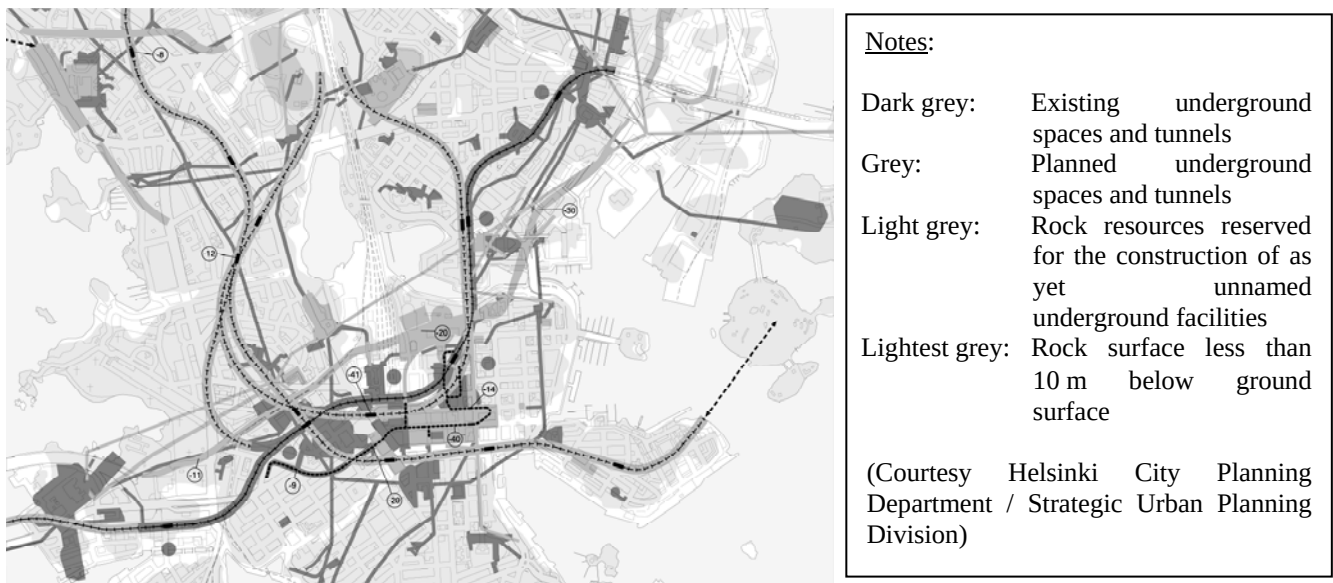


Fig. 7 An Extract of Helsinki Underground Master Plan

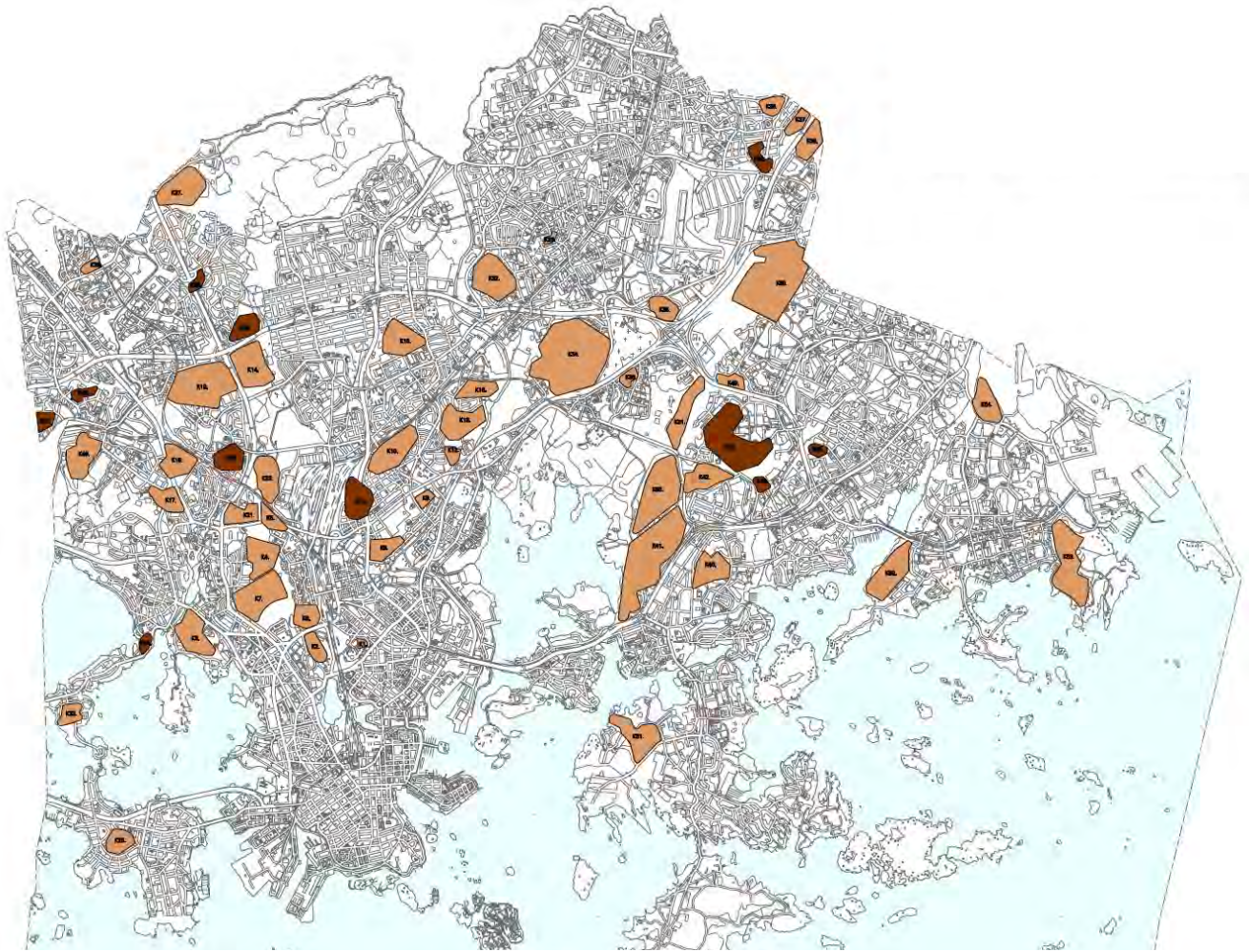


Fig. 8 Unnamed Rock Resources Reserved for Underground Construction Outside Central Helsinki as Shown in the Official Underground Master Plan of Helsinki [HELSINKI CITY, 2010]

suitability of rock areas for different purposes will be studied when preparing the town plan. There are now about 40 unnamed rock resource reservations without a designated purpose (Fig. 8). The average area of these reservations is 30 ha. Unnamed reservations have a total area of almost 1,400 ha, representing 6.4% of the land area of Helsinki. When selecting these resources, the survey took into account their accessibility, the present and planned ground-level uses of these areas, traffic connections, land ownership and possible recreational, landscape and environmental protection values. Selection of unclassified resources is purpose driven and in addition rock-resource driven.

## 6. Reliable and Optimised Large-scale Networks in Bedrock

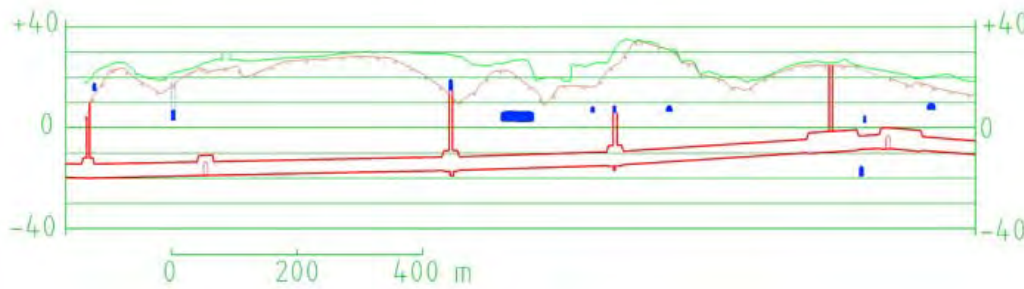
Underground facilities for municipal and other technical services (such as energy, water supply and telecommunications) are, by nature, large-scale closed networks. These facilities comprise a number of different functions together with the utility tunnels connecting them. Utility tunnels are located at such a depth that space reservations for them do not have a

significant effect on other underground facilities (Fig. 9 & 10).

The City of Helsinki has more than 200 km of technical maintenance tunnels, 45 km of which are utility tunnels used by a number of operators. Utility tunnels have been built since 1977. They house transmission lines and pipes for district heating, district cooling, electricity and water supply systems, as well as a large number of different cable links.



Fig. 9 Typical Utility Tunnel (Photo: Jorma Vilkmán)



Note:

The darkened objects are existing tunnels and underground spaces.

Fig. 10 Longitudinal Section of the Newest Utility Tunnel Contract  
(Showing the principle of locating utility tunnels at such depths that there are rock resources also for future needs)

The Geotechnical Division of the City of Helsinki's Real Estate Department has been the main designer responsible for the preliminary and construction-phase planning required for the rock construction of the utility tunnels, the underground wastewater treatment plant and the treated wastewater discharge tunnel. The facilities designed by the Geotechnical Division have included tunnel lines, halls, vertical shafts and the necessary access tunnels.

The raw water for the Helsinki region comes from Lake Päijänne, to the north of Helsinki, via a rock tunnel measuring more than 100 km. Its main investor and designer was Metropolitan area Water Company PSV. Thanks to the good quality of water reserve and constant low temperature during transport in the deep tunnel there is just a small amount of bacteria in raw water and only minimal processing is required before use. Tunnel construction started in 1972 and was completed in 1982 at a cost of about €200 million (adjusted for inflation in 2011). The original tunnel design was based on minimum reinforcement. In 1999 a small part of the tunnel was repaired due to rock falls (Fig. 11). In 2001 and 2008, the tunnel did undergo an extensive renovation. It was bolted and shotcreted in two portions in order to prevent cave-ins.



Fig. 11 Tunnel from Lake Päijänne was Repaired the First Time in 1999 (Photo: Photo Mannelin)

Waste water treatment is carried out in a centralised manner at the Viikinmäki underground waste water treatment plant (Fig. 12 & 13). Waste water arrives at



Fig. 12 Aerial View of the Viikinmäki Waste Water Treatment Plant

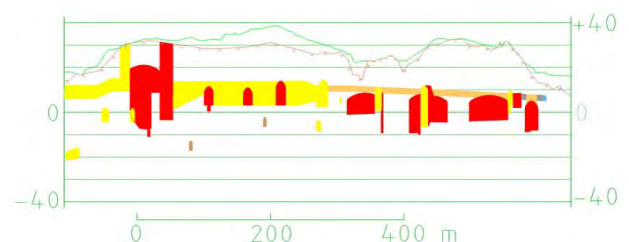


Fig. 13 Longitudinal Section of the Viikinmäki Waste Water Treatment Plant

the plant via an extensive tunnel network. The treated waste water is discharged into the sea via a rock tunnel whose discharge outlet is at a distance of 8 km from the coast. The tunnels in the treatment plant have a capacity of more than 1 million cubic metres.

The Viikinmäki waste water treatment plant is the central plant for treating wastewater from six towns

and cities. It is less than 10 km off the centre of Helsinki. The plant treats 280,000 m<sup>3</sup> of wastewater from about 750,000 people daily. Completed at a cost of approximately €180 million, the treatment plant began operating in 1994. It replaced more than 10 smaller treatment plants, all above ground, thus allowing these sites to be zoned for more valuable uses. The construction of the underground waste water treatment plant took place simultaneously with the construction of ground-level infrastructure and residential buildings. Above the tunnels is the Viikinmäki residential area with 3,500 inhabitants. There is also plenty of zoned area for future residential blocks.

Technical services and utility tunnels in Helsinki are reliable and optimised large-scale networks in bedrock which have several advantages such as:

- There is reliable energy supply via a network with multiple links (allowing alternative routes as necessary)
- Optimisation of energy generation with major transmission networks, i.e. power needs met by generating energy using the cheapest source at any one time
- Expenses are shared by several users
- Land is released for other construction purposes
- The city's appearance and image are improved as the number of overhead lines can be reduced
- Construction work carried out on underground pipes and lines has significantly fewer disadvantages than similar work carried out at street level
- Blast stones resulting from construction of the tunnels can be utilised
- Pipes and lines in tunnels require less maintenance; they are easier to maintain than pipes and lines buried under streets and the tunnel routes are shorter than those of conventional solutions
- Any breakages in pipes, lines and cables do not pose a great danger to the public
- Tunnels are a safer option against vandalism

## 7. Underground Property Ownership in Helsinki

According to the Finnish law, the owner of a property has control over the underground part of the property, though the vertical extent of ownership is not specifically defined in legislation. When interpreting the extent of ownership, the lower boundary of a property has been limited to the depth where it can be technically utilised. In practice this means the depth of 6 m from the lowest point of the building lot. This allows the owner to build a cellar under a building.

City of Helsinki charges all private and public companies using underground space. The rent of "the underground building lot" is about 50% of the corresponding ground-level rent.

## 8. Further Information

Further information and international examples of the use of underground is given by the International Tunnelling and Underground Space Association [www.ita-aites.org](http://www.ita-aites.org)

Detailed and accurate online geographic information about Helsinki City region by various maps, aerial photography, plans and real estate data can be found at [http://ptp.hel.fi/index\\_en.html](http://ptp.hel.fi/index_en.html).

Technical services and large-scale utility tunnel networks in bedrock are described more detailed in [www.geotechnics.fi](http://www.geotechnics.fi) > CaseBank.

Finnish Geotechnical Society SGY maintains website [www.sgy.fi/](http://www.sgy.fi/) for professionals that actively participate in ground engineering.

## 9. Conclusions

The underground master plan of Helsinki shows both existing and future underground spaces and tunnels, as well as existing vital access links to the underground. It also includes rock resources reserved for the construction of as yet unnamed underground facilities, with the aim of identifying good locations for functions suitable for placing underground and which would also reduce the pressures on the city centre's rock resources. The suitability of rock areas for different purposes will be studied when preparing the town plan. Thanks to the systematic planning process and good experiences among citizens, underground developing has not led to a depreciation of the property on ground. An initial survey examined the areas and elevation levels in Helsinki which are suited for construction of large, hall-like facilities. A model based on rock surface data was used applying a standard-sized measurement cave. The model of the bedrock is based on base map data for exposed rock and land surface elevations and point data obtained using drill machine borings. The survey also took into account local weakness zones and rock resources that have already been put to use. In general, it can be said that the bedrock in Helsinki is not far below the ground surface, and that there are plenty of locations suitable for construction of underground facilities. Outside the city centre, the survey found 55 rock areas that had a sufficient size for accommodating large underground spaces near major traffic arteries. In many areas, future underground projects can make use of entrances to existing underground tunnels. When selecting underground resources, the survey took into account also their accessibility, the present and planned ground-level uses of these areas, traffic connections, land ownership and possible recreational, landscape and environmental protection values.

The means of land acquisition in Helsinki are briefly the following: The primary goal of the city is to acquire the (greenfield) land needed for zoning by

voluntary transactions (purchases or land switches) before the zoning begins. Thus the value increase of the zoning is fully usable for the implementation of the town plan; i.e. building streets, parks, communal service properties such as schools, playgrounds etc. Owning the land also gives the city the power to decide on the implementation timetable (without having to negotiate with land owners). The Finnish legislation allows the city to expropriate the land if the voluntary negotiations fail. Helsinki has, however, been reluctant to use this possibility. Especially when rezoning private land, the city also negotiates "zoning agreements" with the land owners involved. These agreements stipulate how much of the public rezoning implementation costs are to be paid by the land owner. The payment is linked to the land value increase generated by the new town plan (additional and/or more valuable building rights). The land owner pays his share in form of land and/or money. The payment is due within a few months after the town plan is approved and valid.

There are several benefits of locating technical networks in bedrock such as: Energy supply is reliable via a network with multiple links (allowing alternative routes as necessary). Optimisation of energy generation with major transmission networks, i.e. power needs met by generating energy using the cheapest source at any one time. Expenses are shared by several users. Land is released for other construction purposes. The city's appearance and image are improved as the number of overhead lines can be reduced. Construction work carried out on underground pipes and lines has significantly fewer disadvantages than similar work carried out at street level. Blast stones and also construction aggregates (sand, gravel, crushed stones) resulting from excavation of the tunnels can be utilized. Pipes and lines in tunnels require less maintenance; they are easier to maintain than pipes and lines buried under streets and the tunnel routes are shorter than those of conventional solutions. Any breakages in pipes, lines and cables do not pose a great danger to the public. Tunnels are a safer option against vandalism.

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# ROCK CAVERN SPACE DEVELOPMENT IN SINGAPORE

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## Abstract

*Underground space development has become an attractive option to supplement the limited surface land resource of Singapore. With a high population density and rapidly developing economy, underground space development has become an economic imperative. In October 2007, the government, under the Ministry of National Development, set up an inter-agency Underground Master Planning Task Force that aims to map out the long-term development of the underground space, bringing the underground space development to a strategic level. In 2010, the Economic Strategies Committee made developing underground space part of the government's long-term economic strategy.*

*The first feasibility study of rock cavern construction started in 1990, and invariably led to the development of the first underground ammunition storage facility, opening a new frontier for space creation in land-scarce Singapore. This was followed by the currently on-going Jurong rock cavern project for hydrocarbon storage being constructed under the seabed. With the government push for a concerted long-term development, rock cavern development in Singapore is positioned to accelerate.*

*This paper gives a review of the history of underground rock cavern space development in Singapore, highlights the potential utilizations in Singapore, and discusses the various planning issues. A comprehensive list of references in relation to underground rock cavern development in Singapore is included.*

## Keywords

*Underground Space, Rock Cavern, Underground Space Planning,*

## 1. Introduction

Singapore consists of one larger and several smaller islands, and lies at the southern end of the Malay Peninsula with a roughly diamond shaped area between latitude 1°09'N and 1°28'N and longitude 103°38'E and 104°06'E. With a total land area of approximately 700 km<sup>2</sup> and a population of 5.0 million, the average population density stands at a very high 7,500/km<sup>2</sup>. With a high population density and rapidly developing economy, Singapore faces a severe lack of land space for development. The increasing population, coupled with the government plan for more green space for the population, means more competition for land.

Traditionally, Singapore has relied on two solutions for this land scarcity: high-rise buildings and land reclamation. In Singapore, 22% of surface land is reclaimed. High-rise housing, both private and public, is the norm for the Singapore population. However, land reclamation has become increasingly more difficult due to deeper water, lack of sand supply, and environmental and even territorial issues.

Like many other cities in the world, Singapore's initial and main use of underground space has been in the area of transport systems and some basement construction for shopping and parking purposes in the city areas [ITA, 2004]. One notable exception is the Deep Tunnel Sewage System [THE STRAITS TIMES, 2000] which has an island-wide network of tunnels connected to two major waste water treatment works to the east and west. Due to the geology of Singapore, almost all these underground projects have been constructed in soft ground. The relatively thick overburden of the Singapore geology means any rock cavern construction has to go rather deep underground.

The possibility of using deep rock caverns was first raised in 1989 [BROMS, 1989]. This was followed by several feasibility studies for cavern construction in the Bukit Timah Granite, which eventually led to the development of the underground ammunition facility (UAF) by the Ministry of Defence [THE STRAITS TIMES, 2008]. With a land savings of more 300 ha, the UAF demonstrated the significant benefits of using rock caverns to create space and improve safety, and served as a catalyst and case study for many later studies and thinking on the use of rock cavern space.

## 2. Singapore Geology and Rock Conditions

Singapore is of moderately low relief. Most of land areas range 10-30 m in elevations. The area of highest relief is at the Northern Central of Singapore, Bukit Timah area, where the highest hill rises to 163 m above mean sea level.

In regional geology, Singapore is of a southerly projection of the Geology of the Malay Peninsula, which is the southeastern extremity of the Eurasian tectonic plate [PITTS, 1984].

### 2.1 Main Geological Formations

The Singapore rocks consist mainly of, in order of geologic age from old to young, four geologic formations [PWD, 1976; DSTA, 2009]: (1) Bukit Timah granite, (2) Sajahat Formation of metamorphic

quartz sandstone and mudstone, (3) Gombak norite, and (4) Jurong Formation sedimentary rocks.

The Bukit Timah granite (BT) forms the major basement rock of Singapore and covers about one-third of the Singapore island, with distribution at central main island, Pulau Ubin island, and the surrounding sea area. The term granite is used in a general sense for the entire suite of acid rocks including granite, adamellite, granodiorite, and the acid and intermediate hybrids (mainly of granodioritic and dioritic composition) which resulted from the assimilation of basic rock within the granite.

The Sajahat Formation (S) is probably the eldest rock formation in Singapore. It is variably metamorphosed sedimentary rocks comprising quartzite, sandstones, and argillite. The Sajahat Formation (S) mainly distributes at Pulau Sajahat, Sajahat Kechilkong, along north coast of Pulau Tekong and as far to the east as Tanjong Renggam.

The Gombak Norite (GN) is a body of noritic and gabbroic rock with exposure on the western side of the Bukit Timah Granite on Singapore Island. The unit is named after Bukit Gombak where noritic and gabbroic rocks are well exposed in a number of quarries. The noritic and gabbroic rocks are coarse-grained and plagioclase-rich with varying amounts of clino- and orthopyroxene minerals appearing as interstitial grains giving an intergranular texture to the rock.

The Jurong Formation (JF) sedimentary rock is the youngest rock formation in Singapore [FONTAINE & LEE, 1993]. It overlies the above basement rock formations, and extensively covers one-third of the main island area of Singapore. It distributes at south and southwest Singapore. Extensive areas in the Jurong Formation have been affected by dynamic metamorphism resulting from tectonic activity. The grade of metamorphism is low, and it is still possible to determine the facies that has been affected.

Overlying the above basement rocks, shallow overburden deposits include residual soils, the Kallang Formation, the Fort Canning Boulder [HAN et al, 1993] and the Old Alluvium.

The Old Alluvium Formation is dominantly terrestrial deposit of early Pleistocene age and distributes at eastern part of Singapore, occupying one-third of Singapore main island area [BURTON, 1964; ONG et al, 2007]. It comprises mainly medium dense to very dense, semi-indurated, and clayey quartzo-feldspathic coarse sand and five gravels.

The Kallang Formation includes both marine and terrestrial sediments laid down from late Pleistocene to the present day. These deposits cover much of the coastal plain, immediate offshore zone and the deeply

incised river valleys. The Marine Sedimentary Unit is found in the eastern part of Singapore. It occurs over an area covering one quarter of Singapore Island.

## 2.2 Soil Overburden

The tropical climate is conducive to the erosion and weathering process of Singapore rocks. In the Jurong sedimentary formation, the weathering depth extends to a depth of 45m and is generally deeper in the faulted areas and in the mudstone region, whereas the thickness of the weathered zone for the Bukit Timah granitic formation reportedly varies from 10m to 40m [TAN et al, 2001] and 20 m to 50 m [SHARMA et al, 1999]. Residual soils occupy approximately two thirds of the land area of Singapore. The residual soils derived from the Jurong sedimentary and the Bukit Timah granitic formations are found in the western and central regions of Singapore, respectively.

The relatively flat terrain and thickness soil overburden in the Singapore geology makes it a challenge in finding good access for rock cavern construction.

## 3. A Brief History of Rock Cavern Space Development

The development rock cavern space in Singapore can be divided into five phase, as summarized in Table 1. Figure 1 shows the locations of the major site investigations for cavern feasibility studies on a simplified geology map of Singapore.

The first feasibility study of rock cavern construction started in 1990, and led to the development of the first underground ammunition storage facility, opening up a new frontier for space creation in land-scarce Singapore. Several other possible uses were also studied in this phase, including defence and civil defence, pumped storage, and warehouses [BROMS & ZHAO, 1993; WALLACE et al, 1993 & 1995; ZHAO et al, 1994 & 1995; WONG, 1993; BYE et al, 2003].

The successful development of the UAF was significant for Singapore from a land use point of view. It demonstrated the feasibility and many benefits of rock cavern use. Land savings from 300 ha to 900 ha were reported in the media. It should be noted a 900 ha land saving represents more than 1% of the total land area of Singapore. The UAF also represented a major breakthrough in overcoming a psychological barrier for many other government agencies that had been reluctant to move the first move for various reasons, and provided many good learning opportunities.

The Jurong Rock Cavern project was preceded with several feasibility studies by the Nanyang Technological University and Jurong Town Corporation [ZHAO LEE et al, 1999; ZHAO LIU et al,

| Period       | Major Development                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                |
|--------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 1990-1994    | <ul style="list-style-type: none"> <li>Feasibility Study of Rock Cavern Construction in the Bukit Timah Granite</li> </ul>                                                                                                                                                                                                                                                                                                                                                                                       |
| 1995-1998    | <ul style="list-style-type: none"> <li>Feasibility Study of Rock Cavern Construction in the Jurong Formation</li> <li>First Tasks Force on promoting use of underground space was set up and led by the Urban Redevelopment Authority. The report from this Tasks Recommends that MINDEF take the lead</li> <li>Feasibility study of the underground ammunition facility by MINDEF</li> </ul>                                                                                                                    |
| 1997-2000    | <ul style="list-style-type: none"> <li>Feasibility study of rock cavern construction in the Jurong Formation [ZHAO LEE et al, 1999]</li> <li>Feasibility Study of Underground Science City (USC) at Kent Ridge [ZHAO et al, 2001]</li> <li>Construction of the UAF started in 1999 [THE STRAITS TIMES, 1999]</li> </ul>                                                                                                                                                                                          |
| 2001-2007    | <ul style="list-style-type: none"> <li>Feasibility Study, detailed feasibility &amp; design of hydrocarbon storage caverns at Jurong Island [ZHAO LEE et al, 1999; ZHAO LIU et al, 1999]</li> </ul>                                                                                                                                                                                                                                                                                                              |
| 2007-present | <ul style="list-style-type: none"> <li>MINDEF commissioned the Underground ammunition facility in 2008 [THE STRAITS TIMES, 2008]</li> <li>Government set up inter-agency Underground Master Planning Task Force (UMPTF)</li> <li>Jurong Rock Cavern project started construction in 2009 [THE BUSINESS TIME, 2009b]</li> <li>Feasibility study on Underground Science City at Kent Ridge [ZHAO et al, 2001; ZHAO &amp; CAI 2009]</li> <li>Feasibility study on Underground Warehouse at Tanjong Kling</li> </ul> |

Table 1 Summary of Major Developments in Rock Cavern Space Use in Singapore

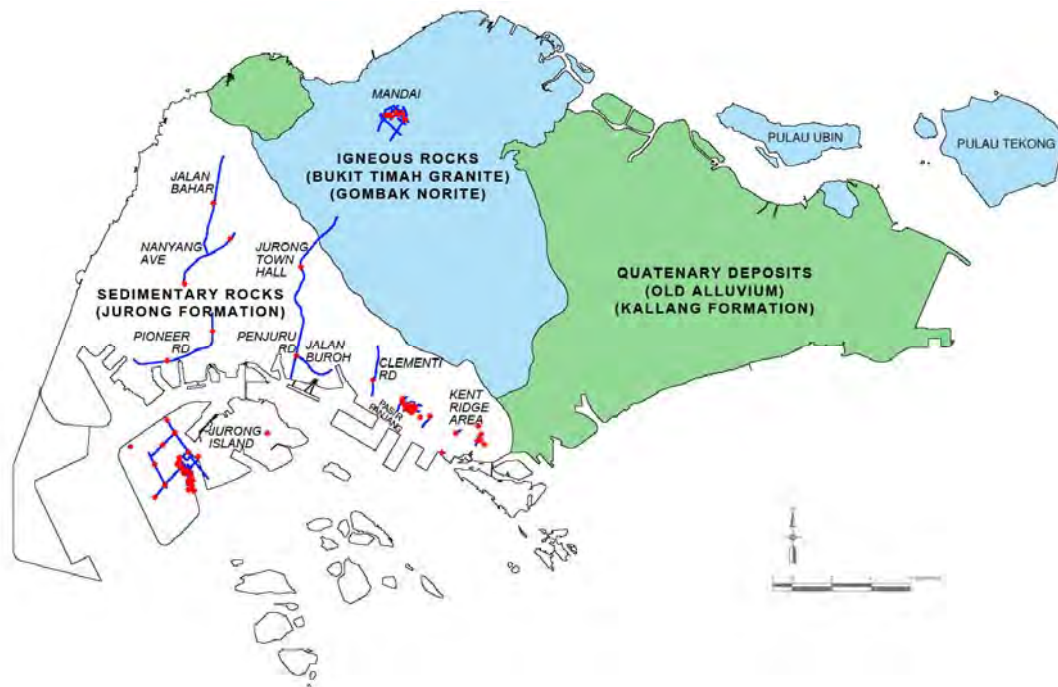


Fig.1 Simplified Geology Map of Singapore Showing Major Site Investigations

1999]. While land use was a major consideration in the JRC project, the decision for the JRC project was primarily based business viability.

Prior to the development of the UAF and JRC project, the majority of the studies was limited to specific geological formations and led by individual agencies based on their need. In October 2007, the government, under the Ministry of National Development, set up an inter-agency Underground Master Planning Task Force (UMPTF) that aims to map out the long-term development of the underground space, bringing the underground space development to a strategic level. UMPTF includes two Sub-groups. Sub-group 1 is focused on relatively shallow basement types of

underground space, while Sub-group 2 for deep rock cavern space.

The government is doing a detailed feasibility study of an underground science city [THE BUSINESS TIMES, 2008 & 2009a] and a warehouse and has commissioned rock cavern usage feasibility studies for several other applications [THE BUSINESS TIMES, 2009c]. Other potential applications being studied include: Power station and electrical substations, Incineration plants, Water reclamation plants, Data centre, Warehousing and logistics, airport logistics, R&D facilities, Landfills, and Reservoirs.

Most of these usages are land-consuming industries which have been predicted to have a significant need for more space. The preliminary conceptual design has been developed for each specific usage, and capital and life cycle costs for the URC developments were also studied and compared to the costs for corresponding aboveground developments.

In 2010, the Economic Strategies Committee [ESC, 2010] made developing underground space a part of the government's long-term economic strategy. A report of the ESC recommended that the government acts early to catalyze the development of underground space by:

- Developing an underground master plan,
- Creating basement spaces in conjunction with new underground infrastructural developments,
- Establishing a national geology office to collate underground information,
- Developing a subterranean land rights and valuation framework to facilitate underground development, and,
- Investing in underground development R&D and directly investing in cavern level test-beds.

#### 4. Planning for Underground Rock Cavern Space

Singapore, as a modern city, has well established an optimized surface land/open space master plan to achieve a balance catering the various needs for municipal & industrial developments. Following the same principle, underground rock cavern space also needs to be well planned. The need for master planning was highlighted several years ago when the DTSS tunnel and a metro tunnel met at almost the same elevation. Eventually, the DTSS had the upper hand because it was designed for gravity flow.

Demands for land space for industrial development and demands for living places are always a conflict. The master plan shall be made based on survey on industries demands and population increment. More importantly, it shall be based on the predication of long-term development trend along with the global economic tracks, while providing a framework for reforming urban areas into desirable and effective environment in which to live and work by people. Accordingly, how many industrial usages shall go underground can be projected with years in order to reserve more surface lands to population.

Optimized planning for underground rock cavern development shall be an essential precursor to the specific developments of major underground caverns. Unfortunately, it is may already be too late for the near-surface zones near the city centre. The tangled web of utilities and transit system commonly found is due to a lack of coordination and planning. However, it is till at right timing for the far-surface zones - rock caverns to have an optimized plan.

#### 4.1 The Need for Master Planning and its Benefits

From a broader perspective, the need for master planning and its benefits can be discussed from the following considerations.

(a) The underground is now considered a natural resource in many cities in the form of minerals, ground water, geothermal energy, and underground space. As a natural (and strategic) resource, the use of underground space must be planned so as to derive the maximum benefits to society in a sustainable manner.

(b) Underground space are three dimensional, and cannot be easily seen. An advantage of this 3D property is that different layers of underground space can be planned and zoned based on their functions and technical requirements.

(c) Unlike surface buildings, underground structures cannot be “taken down” once developed. Conversion of rock cavern space to other uses requires provision for the necessary supporting infrastructure. A more effective plan is required.

(d) Good space, be it aboveground or underground, is always in demand. As such there will be competition for use of underground space. The example mentioned earlier is a good example. Another example is the relatively high hills where rock mass quality is generally better and at a shallower depth.

(e) There are limited surface access and exit points for underground space. These are significant from both the points of view of construction and safety considerations (fire exits, emergency exit). The issue of surface access is especially important for Singapore due to the flat terrain and thick soil overburden. From a national point of view, it might be good to have planned access points for future construction.

(f) The geologic and hydro-geologic conditions greatly affect the types, sizes and costs of rock caverns that can be constructed. Master planning allows us to match geologic conditions and rock mass qualities to the type of rock cavern uses to achieve a more efficient use of this natural resource. For example, it is logical to reserve the good rock for applications requiring large spans.

(g) Finally, master planning makes it possible for the integration and linkages of various applications for strategic reasons.

#### 4.2 Planning Issues

##### Legal Framework and Policies

Legal and administrative restrictions may act as significant barriers to the use if underground spaces. The protection of the rights and ownership of existing

surface or underground users, the business model, pricing mechanism, the administrative control of national reserves, and the provision of personal safety and environment protection are issues that must be resolved in all cities. And this is no less critical for Singapore.

One particular interesting issue is whether the government, releasing the potential value of cavern space, should charge a space premium to government agencies or private developers for creating the cavern space. Such a premium would be effectively a penalty. On the other hand, the government could give incentives to encourage the creation of rock cavern space.

Disposal of the excavated rock from cavern construction is another tricky issue in Singapore. Good quality rock can be used construction material such as aggregates. In many countries, the excavated rock can be sold to offset the cost of construction. In Singapore, however, if the rock is disposed of as waste, it is considered a cost to the contract and there is no issue. However, if it is sold, the government charges a royalty and revenue from the sale of the rock goes back to the government. This situation gives no incentive for the optimized use of the excavated rock.

#### Safety Standards for Rock Caverns

Singapore has a high standard of fire safety. However, for rock cavern facilities, the only fire safety standards that can be used and accepted by the authority are those for basement construction. This is always a contentious issue between the developer and the authority.

Another interesting example is the safety standards related to construction. Excessive safety requirements artificially increase the cost of construction. For rock blasting, Singapore does not even have a national standard for allowable blasting vibration limits. As a result, the concerned agencies typically look for the most conservative allowable limit from other countries and divide it by half as the Singapore limit.

#### Geological Data and Investigations for Bedrock

For planning rock space development, it is important to have reliable geological data, especially concerning the bed rock. In this aspect, Singapore faces some challenges because of the lack of a central office for geological information for many years.

#### Coordination among Government Agencies

Coordination among the various government agencies is important for the planning and safeguarding of supporting surface land sites, and integration of aboveground and underground facilities, and different types of underground facilities. Dual-use or multi-purpose facilities can help reduce cost of construction

but cannot be planned without the full participation of the stakeholders.

#### Investment Time Frame

It is well known that many underground facilities, especially concerning transport systems, were built out of necessity – when there is no other choice but to go underground. Very often, the cost of such construction is very high due to the congestion and extensive existing buildings. On the other hand, the cost of construction can be much lower in areas where there is relatively little development. This takes a very long-term view on the part of the planner and decision maker – to go underground when they do not have to.

#### Cost Benefit Analysis

One of the foremost challenges for the use of underground space is the perceived high cost of construction. Large underground development may require massive investments with relatively high risks of construction problems, delays and cost overruns. While many tangible and intangible benefits have been discussed, decisions on development options are often made entirely on the basis of construction costs. It is therefore important to develop tools that allow for effective and fair evaluation the cost and benefits. Such tools can help government agencies to make decisions on a systems view, taking into consideration many other intangible benefits, including those with high strategic significance.

In one feasibility study for an underground reservoir, the author suggested that the cost of cavern construction be compared to that of water desalination or re-cycled water because with an average annual rainfall of 2.5 m, Singapore does not lack water, but storage space. Otherwise, the cost comparison would immediately rule out the rock cavern option.

#### Specific Uses and Planned Space Creation

In the Singapore, land reclamation can be done without any specific use identified for the land. On the other hand, almost all decisions on cavern development are based on identified specific uses. While such facilities have very good fit-for-purpose value, they often come with very high price tags because of lack of scale and a high percentage of preliminary work and basic support infrastructure (access, power supply, etc). On the other hand, if large underground cavern space can be developed, the cost of construction per unit space can be significantly lowered due to the economy of scale and shared common infrastructure. In areas where the rock quality is good, this can be combined with underground aggregate mining, which can further lower the cost of cavern construction. In other countries, underground aggregate mining by itself is a viable business, and the space created as a by-product is essentially free.

#### Capability Development and R&D Support

To quote Mr Quek Tong Boon, Chief Defence Scientist of the Ministry of Defence Singapore, “rocks underground are in abundance, but rock engineers are a rare breed.” Large-scale underground space development will require the necessary in-country capability and R&D support. R&D support can help develop new technologies that facilitate planning and design, allow for more efficient, economic and safer construction, and improves the performance of the facility and reduces costs of maintenance and repairs during construction.

#### Public Perception and Receptivity

One major challenge to the planners and decision makers is public perception. Traditionally, the underground has been associated with dark and dangerous dungeons. While the use of basement space for parking and shopping malls has been accepted by the general public, deep underground rock cavern space still creates some form unease. Another perception is that underground space is very expensive. This is a challenge that’s difficult to overcome if we only look at construction costs. A system engineering thinking with the ability to consider the other opportunities is necessary to change this perception.

#### **4.3 ITA Recommendations**

Corresponding to the concerns and challenges related to effective planning of the underground, the International Tunnelling Association [ITA, 1990] has recommended a direction for subsurface planning, as follows:

*The subsurface is a resource for future development similar to surface land or recoverable minerals. Once an underground opening is created, the subsurface can never be restored to its original condition and the presence of this opening can affect all future uses of the surface and the subsurface in its vicinity. These factors require responsible planning for all uses of the underground to ensure that resource is not damaged or usurped by uncoordinated first uses.*

*The awareness of the underground option among planners, developers and financiers should be increased so that subsurface planning issues are properly addressed. Subsurface planning should be an integral part of the normal land use planning process.*

*National, regional and local polices should be prepared to provide guidelines, criteria and classifications for assessing appropriate use of underground space, identifying geologic conditions, defining priority uses and resolving potential utilisation conflicts. Site reservation policies should be established for important future uses and for especially favourable geologic conditions.*

#### **5. Conclusions**

The rapid economic development in Singapore, coupled with a high population density, has meant increasing competition for land. As such, the use of underground space has become an economic imperative. Rock cavern development has become a new frontier for space creation in land-scarce Singapore. With the government push for a concerted long-term development of the underground, underground space development in Singapore is positioned to accelerate.

This paper has presented a brief history of rock cavern space development in Singapore and discussed the need for and benefits of master planning and the various planning issues that need to be addressed before the use of underground space can reach its true potential. The development of the underground ammunition facility and the Jurong Rock Cavern project are good examples of planned rock cavern space use to address specific application and land use issues. However, many planning and technical issues related to the use of underground space may present challenges and even obstacles to a rock cavern development and must be addressed in a systematic manner, treating the underground as a strategic resource whose use must be properly planned and coordinated.

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Note:

A comprehensive list of references, covering more than those cited in the paper, is provided.

# DESIGN & BUILD LARGE UNDERGROUND CAVERNS THE IMPORTANCE OF UNDERSTANDING AND UTILIZING IN-SITU ROCK STRESSES

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## Abstract

*The tunnelling industry has the means to make any "hole in the ground" almost to any size and shape that is requested. The main challenge is to integrate such solutions in the long-term development and urban planning. The underground solutions must be cost-effective to compete with surface alternatives and they must be safe and felt to be safe by the users.*

*Only by ensuring that we are capable of providing safe underground structure will underground infrastructure be perceived to provide a sound internal environment. There is public confidence that the tunnelling industry is capable of producing tunnels and caverns to the satisfaction of the clients.*

*This paper discusses the challenges of designing and building large underground caverns needed for a further developing of the underground infrastructure. One focus is directed to the understanding and utilizing of in-situ rock stresses to materialise such caverns. Some examples are also provided to visualise the beauty of underground rock caverns and their use to serve the public and thus constituting a major asset to the society.*

## Keywords

*Underground Space, Large Caverns, Design, Construction*

## 1. Introduction

The application of large underground caverns has seen a wide range of use in Norway during the last 30 to 40 years. It began with a demand arising from the development of Hydroelectric Power schemes in the post-war period. The first major caverns in Norway were built for such purpose. Later, the technology evolved and it became evident for other businesses that this technology could be utilized for other applications too. Particularly for the civil defense it became obvious that such solutions could realize combined shelters and sports facilities. During a few decades of heavily development of this field a large number of combined underground shelters and swimming pools and sports halls were built. It culminated with the construction of the most famous one, likely to be the Gjøvik mountain hall, which was large enough too room the ice-hockey rank for the Winter Olympics in Norway in 1994. The oil and gas industry realized the possibilities of storing

hydrocarbons underground and began to take this concept into use. However, the use of large underground caverns has also been materialized for use as storage of waste water, sewage cleaning facilities, cold store, storage of potable water etc., as a result of the knowledge of in-situ rock stress conditions.

A major parameter in the realization of these large span caverns is the presence of sufficient in-situ rock stress conditions. The in-situ rock stress conditions enable a confined arch to be formed above the cavern roof and thus stabilize the caverns and assisting in the realization of large span caverns to be built for a number of purposes in urban areas as well as rural areas. With no doubt the in-situ rock stress has indeed a great importance in design and construction of sub-surface opening.

This article presents some of the wide applications that large underground cavern has received in Norway as well as describing the fundamental philosophies of the design principles and the construction of these caverns.

It is interesting to quote Tunnel Engineering Handbook by Bickel & Kuesel [1992]: "The in-situ stresses present in a rock mass have an important effect on the physical properties and behaviour of rock as an engineering material. The in-situ stress is a confining pressure which affects the viscoelastic and strength properties of the rock. The intensity and direction of these stresses control the strain patterns and redistribution of stress in the rock mass remaining after an excavation has been made. Knowledge of the in-situ stresses is essential to the sound, logical design of rock reinforcement systems, excavation procedures and opening layouts, and to the interpretation of expected rock strength and deformational properties."

Taking these aspects into account it has been possible to utilize the rock mass for sustainable and environmental friendly solutions underground for the benefit of the larger public.

Not least, Mother Nature is also a major 'contractor' with respect to building large underground caverns, and to keep them stable in an unsupported state of mode. The cavern that is pictured in the figure 1 below is a major tourist attraction in Kuala Lumpur in Malaysia, the Bahtu Cave. Thousands of people visit the cave every day, still it seems unsupported to the author with naked eye.



Fig. 1 The Bahtu Cave in Kuala Lumpur in Malaysia (apparently unsupported large underground cavern; Photo: Grøvn)

Give it a second thought when next time visiting a place like the Bahtu Cave: it is a good exercise for the technical mind to figure out how is this possible?

## 2. Some General Characteristics of the Rock Mass

For the development of large underground caverns in Norway the following capacities of the rock mass has been the most important:

- stress induced confinement,
- self-standing capacity.

Secondly, for unlined gas storages, cold storage, and storages with functional requirements not limited to stability and cavern size the following capacities are important:

- thermal capacity,
- impermeable nature.

Any rock mass has a certain self-supporting capacity, although this capacity may vary within a wide range [BIENAWSKI, 1984]. The fact that there is some “stand-up” time implies that the rock mass for a certain time is not a dead load, thus it shall not be treated as if it was (Fig. 2). An appropriate engineering approach is to take this capacity into account when designing permanent support. Rock strengthening may, however, be needed to secure certain properties/specified capacities, in the same way as is the case for any other construction material. The fact that, the rock mass is not a homogenous material shall not disqualify the utilization of its self-supporting and load bearing capacity.

As large span excavations became common, particularly in connection with near surface underground sport halls, the need for more accurate stability control measures became more apparent. Also, numerical analysis is an essential tool in the design of such caverns, and thus it turned important to develop methods and test procedures for the purpose of obtaining information on the in-situ stresses in the rock mass.

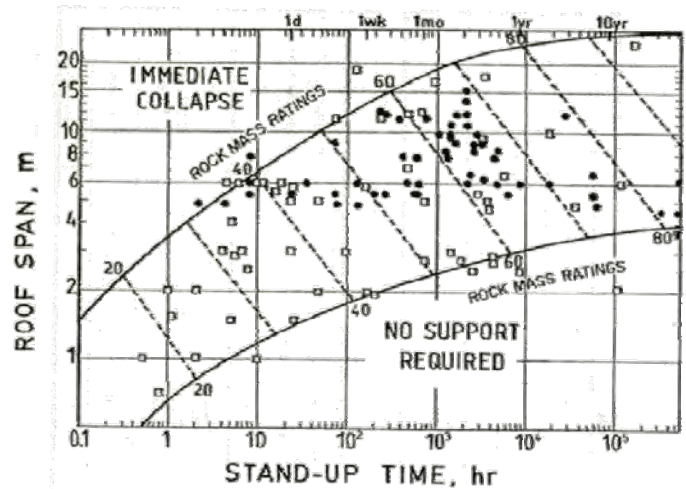


Fig. 2 Typical Stand-up Time [BIENAWSKI, 1984]

Also, experiences from the mining industry showed numerous cases where excavations of underground openings with extremely large span (60 – 80 m and more) were seemingly stable without any rock support measures at all. Research showed that a major reason for this was the existence of sufficient tectonic, horizontal stresses even at shallow depths, thus providing favourable compressive stresses in the excavation roofs, creating stable self supporting underground structures.

This called for in-situ rock stress measurements to be done preferably before the excavation started, followed by rock stress control and deformation measurements during and after excavation. Examples from the mining industry were carefully documented as the response in the surrounding rock mass was monitored following the extension of the mining openings and excavation of the pillars. Stress measurements have been carried out in connection with a number of large underground excavations to document the stability of large spans and rock pillars.

Horizontal stresses of geological origin (tectonic stresses) are quite common in Norway, and in many cases the horizontal stresses are higher than the vertical stresses, even at depths greater than 1,000 m. The majority of rock stress related problems in Norway actually originates from high horizontal stresses, rather than vertical stress due to the rock overburden. This has been the case in a number of road tunnels and tunnels connected to hydropower development, and high stresses have also caused considerable stability problems in power house caverns. This has again called for rock stress and displacement measurements.

However, the pure existence of high, or sufficient in-situ rock stresses, particularly horizontal stresses is an important condition that enable large underground caverns to maintain stability, particularly when the width of the caverns exceed 12 - 15 m which is the size of an oversized road tunnel.

The majority of such rock caverns in Norway has been built in accordance with the term “unlined tunneling”. This will be described in the following.

### 3. The Term “Unlined” Tunnelling

Using the term “unlined” tunnelling must not be misunderstood in such respect that the tunnels are constructed without any rock support at all. In general, permanent rock support consists of rock bolts and sprayed concrete, and only extremely good rock mass conditions may constitute an exception with no support measures. Water and frost protection may be taken care of by installing free-standing systems which are not load bearing, except for traffic loads, and do not contribute to the rock support. [GRØV, 2001a]

The capability of the initial, primary lining structures are utilised as permanent to the extent the quality of the rock mass requires. Such an initial primary lining is often sprayed concrete with a thickness ranging from 50 to 200 mm in combination with rock bolts, commonly fully grouted and corrosion protected, some times installed in a systematic pattern. The latest sprayed concrete technology, with alkali-free accelerators, allows greater thickness to be applied. Sprayed concrete based support structures may in many cases adequately replace massive, cast-in-place concrete structures or pre-cast segments. Such applications provide a long term use of sprayed concrete support compared to a temporary use as primary lining before a later, final concrete liner is installed.

The sprayed concrete mix design can be determined based on the actual project specific requirements that

exist for the properties of the concrete. Today, the technology associated with sprayed concrete allows the designer to select amongst a number of properties of sprayed concrete: bond strength, strength, stiffness, ductility and permeability. This indicates that the sprayed concrete holds a great variety of properties that can be customised for any specific project. The challenge for the designer is to identify the relevant and applicable quality of the sprayed concrete. Furthermore, the development of spraying robots allows efficient application, with accurate dosage of admixtures and additives to cope with situations that require particular demands. At the same time additives have been developed that allow for example; a fast setting concrete with high early strength without influencing its final strength, the possibility of applying up to a 200 mm thick layer in one spraying operation etc.

Rock mass impermeabilisation is taken care of by pre-grouting ahead of the tunnel face. The effect of improved stability in this respect is also an important momentum.

### 4. A Summary of the Importance of Rock Stress for Design

Table 1 summarizes the importance of rock stress for design and construction of various types of underground facilities. The table has been established based on the authors personal experience and may only be used as a guide.

The table indicates clearly that the presence of in situ stress in the rock mass can be used for the benefit of enabling various underground solutions. At the same

| Type of facility                                            | $\sigma_1$                                                                                                  | $\sigma_3$                                                                                                                                             | $K = \sigma_{hor} / \sigma_{vert}$                               |
|-------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------|
| <b>Rock caverns</b>                                         | Moderate to high level can enable an optimized geometry. Too high may lead to stability problems.           | Low level may produce a too small arch building in the roof/lack of confinement thus imposing stability problems.                                      | $K = 1 - 2$ is OK.<br>$K > 3$ is not OK.<br>$K < 0.5$ is not OK. |
| <b>Pressurised tunnels / caverns</b>                        | Minor influence, high level may give stability problems.                                                    | Minor principal stress component must be higher than the water pressure (or the pressure from any other confined material) as an ultimate requirement. | No particular requirement.                                       |
| <b>Facilities with particular requirements to tightness</b> | Moderate to high level can provide good confinement and stability and improved tightness.                   | Critical low level gives poor safety against leakage.<br>Grout pressure $< \sigma_3$<br>Storage pressure $< \sigma_3$                                  | No particular requirement.                                       |
| <b>Transport-tunnels</b>                                    | Moderate to high level is good for the confinement and stability. High stresses may give stability problems | No particular requirement, but should not be too low as that would reduce the confinement and reduce stability.                                        | $K = \text{appr. } 1$ is OK.                                     |

Table 1 An Overview of Requirements to Rock Stresses for Design of Various Underground Facilities

time it is important to point out that the ignorance, or lack of such stress characteristic features may lead to unacceptable stability situations or situations where the underground facility may no longer fulfill its function.

Table 1 is based on Norwegian typical rock mass conditions with a moderate to high E-moduli and UCS-values. Weaknesses occur as discontinuities in the rock mass and they can exhibit rather varying characteristics and capabilities as far as being a construction material and may transfer stresses in different ways.

The in-situ stresses are normally exposing a nature which reflects the gravitational component in the vertical direction, whilst in the upper 500m typically the horizontal component is much higher than the could be derived at by a pure theoretical approach.

## 5. Construction of Large Underground Caverns

The construction of large underground caverns is normally split in various sections, such as top heading and benching. The number of benches may vary based on the height of the caverns and the equipment chosen for the job. The top heading is normally done by horizontal blast holes, forming a full horseshoe shaped “tunnel”. In wide caverns a set up of equipment may include two drill jumbos working side by side at the same face, as one single jumbo may not cover the full face alone.

It is important to completely install all necessary support at the top heading before benching may start as it is difficult to later gain access to the roof and the upper parts of the walls for later support work, unless involving the use of large cranes, significant working platforms or scaffolding.

During the rock mass classification it is therefore required to map properly the jointing as to identify potential large wedges that may occur later during the excavation of the lower parts of the walls. In figures 3 and 4, a typical sequence of an oil and gas storage is shown. In this case it is also shown that the excavation of the cavern shall take place in a fully saturated environment, the artificial water curtain has been installed prior to allowing the tunnel face to advance.

It has been typically experienced that such large underground excavations have influenced the ground water conditions lowering temporarily the ground water level in the vicinity of the caverns.



Fig. 3 Excavation Sequence Gjøvik Hall (Photo: NGI)

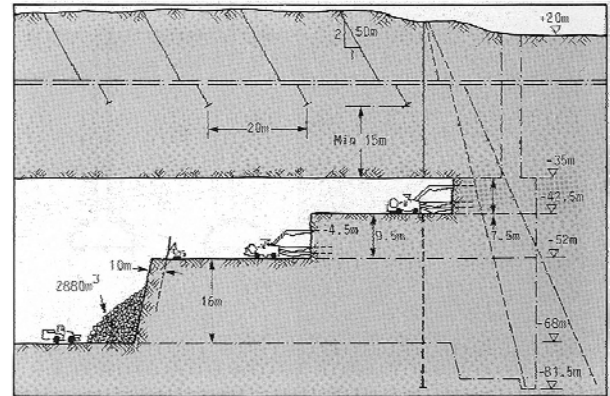


Fig. 4 Typical Excavation Sequence (Top heading and lower benches and with artificial groundwater infiltration from surface) (Ill. unknown)

Normally, there are several accesses to an underground cavern of this kind, at the top heading level and the lower benching plus intermediate levels. Mucking out is a logistical challenge as work may go on at various levels simultaneously.

The lower benches may be excavated either by horizontal blast holes or by vertical blast holes, so-called drifters. Using drifters a greater height of the bench can be excavated in each blast round. This is particularly the case when straight walls are designed for the caverns, which is the typical design in Norway. Egg-shaped caverns are normally not applied in Norway as straight walls have turned favorable with relation to cost and they do normally not impose any significant instability of the caverns.

Stability of the caverns can be followed-up by various types of monitoring systems. Typically these may include the following:

- visual inspections of exposed rock and applied sprayed concrete or rock bolts;
- measuring rock bolts;
- extensometers (rod and tape) and convergence pins.

The purpose is primarily to detect any occurrence of deformations, the results from observations and monitoring will then form the basis for additional support installations if required.

There are also advantages that can be obtained from utilizing the in-situ stress situation during the excavation, others than those described above.

Namely: related to the possibility of maintaining stable horizontal and vertical pillars for temporary use. One such situation can be exemplified by the excavation of large caverns for storage of hydrocarbon products in the Kingdom of Saudi Arabia. In competent granitic rock along the west coast of KSA the contractor choose to excavate some of the storage caverns which were designed with a width of approx. 20 m and height of 30m in a way that deviated the typical excavation sequence significantly. Instead of following the sequential top – down procedure which is shown in figure 4 above a sequence involving a temporary, intermediate horizontal pillar between the top heading and the lower bench was utilized.

This sequence was initiated due to advantages for the neighbouring works and to reduce the number of accesses to each cavern at various levels.

The sequence was developed following careful stress monitoring, numerical modelling and close follow-up of other openings in the complicated conglomerate of openings that is normally associated with such storage facilities. In cooperation with the design responsible of the project the contractor established an excavation process that allowed an unsupported, stable intermediate pillar taking advantage of in-situ stress conditions and the understanding and knowledge of utilizing these for the benefits of the project.

## 6. Development of Large Underground Caverns in Norway

During and after the Second World War, the use of the underground was given preference out of considerations to wartime security. But with the rapid advances in rock excavation methods and equipment after the war, and consequent lowering of the costs, underground location came to be the most economic solution (Fig. 5). This also tied in with the development of concrete lined, and later unlined pressure shafts and tunnels, to give the design of hydroelectric power schemes a freedom of layout quite independent of the surface topography (Fig. 6).

Underground location of powerhouses in such schemes is now chosen whenever sufficient rock cover is available. Frequently the overall project layout requires the powerhouse to be placed under very deep rock cover where rock stresses may be substantial. This requires an investigation of the stress condition in advance for finding the most favourable orientation of the cavern and the optimum location, orientation and shape of ancillary tunnels and caverns.

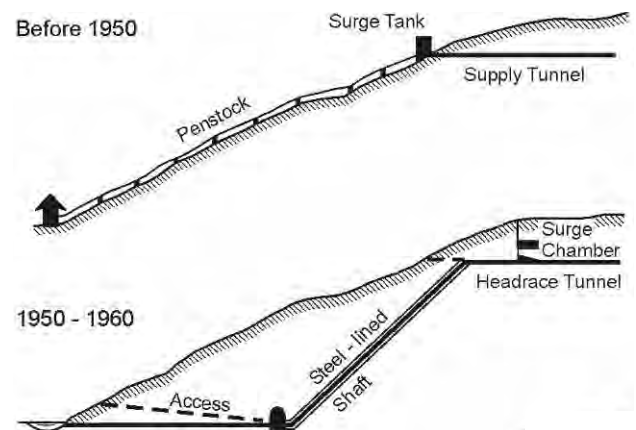


Fig. 5 Pre-1960 Development of Hydro Electric Power Stations [BROCH, 2006]

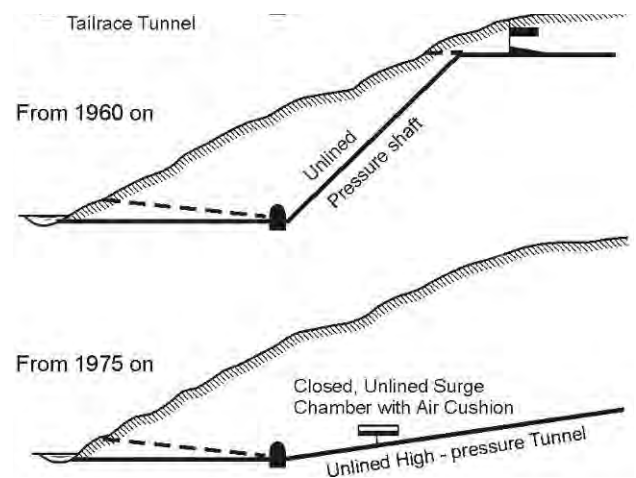


Fig. 6 Post-1960 Development of Hydro Electric Power Stations [BROCH, 2006]

The layout of these underground power stations and their adjacent facilities would typically include caverns that are 12 - 25 m wide and up to 35 m high. The width and height are depending on the size and type of the installations. The length of these caverns might be several tens of meters long, typically some 40 to 60 m long, but even up to a length of 100 m has been produced. The excavated volume would typically range from 10,000 m<sup>3</sup> of solid rock and to 50,000 m<sup>3</sup>. In

Norway some 200 such underground hydro electric power stations has been constructed.

The cost of the underground structures are tightly associated with the distance between neighbouring caverns, their size as well as the length and dimensions of the connecting tunnels. Therefore it is a must to try and optimize the layout in such a way that the caverns are placed with as short distance s possible. A typical rule of thumb would be a distance of 1 – 1.5 times the cavern width, however the local geometry of joints must be considered.

An interesting aspect of these hydropower projects is also the fact that some underground hydropower stations may as well include a transformer station being located right next to the power station itself, and with a large number of cross cuts, adits, exits etc. at various elevations surrounding the main cavern. These would of course influence the stress re-distribution in the rock mass and causing stress concentrations in the vicinity of the openings. It is therefore important to always work with a 3D-view and consider the influence and consequences of tunnel and cavern openings at various levels in the layout. Pillars may be subject to high stresses due to such stress redistribution and particular attention should be taken to the pillars themselves, their dimensions, layout, geology and support. This consideration is valid for horizontal pillars as well as vertical pillars. All in all these projects comprise complicated geometry (Fig. 7 & 8).

The location of the rock caverns are normally fixed in the design concept based on information gathered during a comprehensive pre-investigation phase, however, pending on the actual rock mass conditions

as encountered during tunnelling in the approach to the designed and planned location, changes may of course take place. Several underground hydro power stations in Norway have experienced changed locations and local optimisation to better fit the rock mass conditions. It is common to take into account such information as related to the following:

- rock types and mechanical properties;
- characteristics and frequency, spacing of rock mass discontinuities;
- in-situ rock stress;
- groundwater conditions.

During the approach to the planned location of the cavern(s) the rock mass is thoroughly mapped, joint systems are observed and characterised, weakness zones are interpreted, in-situ rock stress conditions are measured, ground water is monitored. If these conditions are not in accordance with the expected and required quality of the rock mass, it may be conclusively decided to shift the location of the hydro power station, and other adjacent caverns, or make some layout adjustments. Typically, the final layout of the caverns, their location, geometry, alignment, layout of the tunnel system and rock support design may only be decided upon when the above information is obtained from the excavation of the approaching tunnels.

Numerical analyses as well as analytical calculations are useful tools for the design and planning of the caverns. These must of course be verified during the construction phase by adequate monitoring and follow-up of the stability of the underground caverns.

The present state-of-the-art prescribes systematic bolting of the rock ceiling immediately after excavation of the top heading, followed by fibre-reinforced shotcrete from 70 to 200 mm in thickness, according to rock quality.

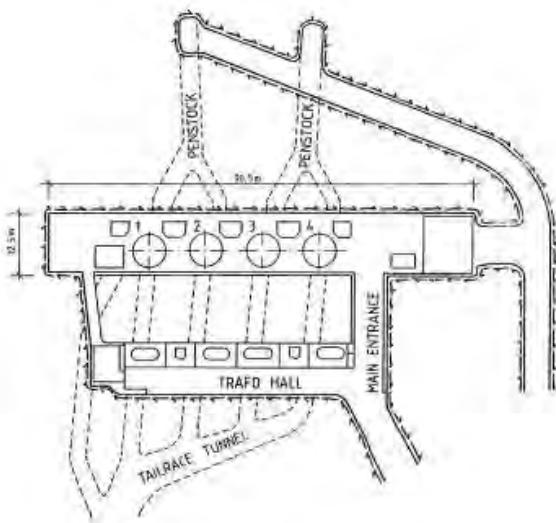


Fig. 7 Lay-out Solution (Plan) for Underground Hydro Electric Power Schemes [BROCH, 2006]

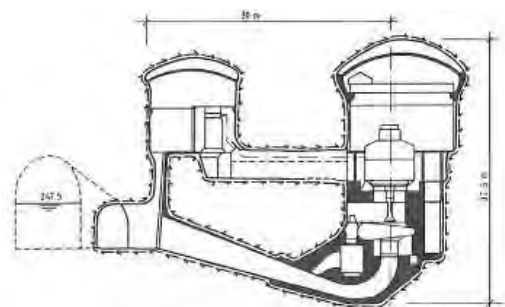


Fig. 8 Lay-out Solution (section) for Underground Hydro Electric Power Schemes [BROCH, 2006]

In a 20 years period starting from 1974 and ending in 1995 a total of some 20 combined underground defense shelters were built. These shelters serve as sports halls in peace time. Typically they were built to serve the purpose of hand ball arenas and swimming pools in accordance with international standards relevant for such arenas.

It was a particular functionality of these facilities to try and place the caverns at a shallow depth, to reduce the length of the accesses. Several of these caverns were constructed with a width of 25 m and only 15 m rock cover. The common size of these projects include in the range of 55,000 m<sup>3</sup> of rock excavation. Figure 9 shows the interior of such caverns.



Fig. 9 Combined Underground Sports Facility and Civil Defence Shelter (Photo unknown)

The main conclusion of these projects was that even at shallow depth, there is in-situ stresses in the rock mass that could be utilized for the stability of the caverns, although the gravity component of the stress situation was very small. The knowledge of this stress situation enabled the possibility of constructing such caverns.

This application of underground caverns culminated with the design, construction and realization of the Gjøvik Mountain Hall. This underground facility utilized the knowledge gained during a long period of underground construction and a rock cavern with a width of 61 m was built. The project will be further detailed below.

The utilisation of the underground openings had seen a significant growth in Norway, particularly due to the development in the hydroelectric power sector, where an increased number of projects utilised underground alternatives for waterways, pressurised tunnels and location of hydropower stations and transformer rooms. The Norwegian tunnelling industry developed techniques and methods to improve the efficiency and quality of underground works. A comprehensive experience base was established which became

important when the underground storage of hydrocarbons was introduced.

The tunnelling industry shifted from hydropower projects to oil and gas storage and then towards the current use for infrastructure purposes. In the 1970's Norway grew to be a major oil and gas producing nation with the corresponding need for larger storage facilities. It also became evident that the use of surface structures needed to be reconsidered. The solution in Norway was to excavate large rock caverns, utilising the availability of suitable rock mass conditions and the tunnelling experience obtained through the hydropower development.

In brief the main advantages of underground rock storage are:

- Utilising the variety of parameters of the rock mass
- Environmentally friendly and preserving
- Protection during war
- Cost aspects
- Operation and maintenance
- Protected from natural catastrophes

The typical size of the rock storage caverns in most recent projects indicates a cross-section of appr. 500m<sup>2</sup>. The caverns are close to 20m wide and 33m high, and this has become a typical cross sectional area for such caverns (Fig. 10).



Fig. 10 Typical Underground Oil and Gas Storage Cavern with Straight Walls (Photo unknown)

## 7. Underground Caverns for Oil and Gas Industry

In Norway, the first underground hydrocarbon storages were excavated during the Second World War, designed for conventional, self-standing oil tanks. Later, being located underground was basically for protective purposes during the cold war era.

One project of such kind is located at Høvringen, near the city of Trondheim in central Norway, where ESSO

is operating underground steel tanks, whilst one other storage is located at Skålevik, and is operated by BP. Following on from these first projects was underground hydrocarbon storage in steel lined rock caverns, designed and built in accordance with for example Swedish fortification standards. This concept implies in brief a steel lining with concrete backfill of the void space between the steel lining and the rock contour. One such project is located in Hommelvik outside Trondheim and is operated by Fina. This project provides the supply of gasoline to the nearby airport. The above described projects were commissioned almost a half a century ago, and are still being in operation. However, they represent an era and a concept which did not take into account the significant capabilities of the rock mass.

The hydroelectric power development in the sixties realised that the rock mass capabilities could be further utilised; large underground caverns, introduction of wet-mix sprayed concrete, unlined head race tunnels and air charge chambers were all contributions to an extended use of underground space. Thus the confidence in unlined tunnels and caverns grew, and the first unlined hydrocarbon storage project was initiated. Concept developments took place in other Scandinavian countries at the same time, however, in Norway unlined pressure shafts had been in use for some time in the hydroelectric power development, up to 1000 m water head, and the importance of sufficient in-situ rock stress to prevent hydraulic splitting of the rock mass was recognised as an important success criteria.

As mentioned above, in the Norwegian concept lining as a barrier had been abandoned due to the significant costs associated with such solutions. Also the techniques of pre-grouting of the rock mass to stem or reduce water leakage started to be developed during this period.

Table 2 presents a summary of the Norwegian crude

oil storage facilities and refinery caverns for hydrocarbon products. Table 3 shows a summary of the major petroleum gas storage facilities. It should be noted that all these gas storage facilities are for propane, except Mongstad commissioned in 1989 that also stores butane and Sture in 1999 that stores a propane/butane mixture. Table 4 is for the major ammonia (NH<sub>3</sub>) storage [GRØV, 2006].

Adding to this, caverns with large cross-sections and complicated lay-out geometry were already in use as hydropower stations. Thus, the Norwegian tunnelling industry was prepared and technically ready for the new challenge of unlined hydrocarbon storage in rock caverns.

In the 1970's Norway grew to be a major oil and gas producing nation with the corresponding need for larger storage facilities. It also became evident that the use of surface structures needed to be reconsidered. The solution in Norway was to excavate large rock caverns, utilising the availability of suitable rock mass conditions and the tunnelling experience obtained through the hydropower development. Underground oil and gas storages mainly utilise the following capabilities of the rock mass:

- It's impermeable nature, i.e. the actual permeability of the rock mass and associated discontinuities may vary from 10<sup>-5</sup> m/sec to 10<sup>-11</sup> m/sec.
- It's stress induced confinement, the in-situ stress situation varying from stress released rock bodies through a pure gravitational stress situation to stresses originated by long tectonic history of the rock mass.
- It's thermal capacity, i.e. the capacity to store energy over significant amount of time.
- It's selfstanding capacity, i.e. the ability of the rock mass to maintain stability even after being subject to cavities being made, man-made or natural.

| Project       | Year of Completion | Main rock type   | Width × height (m)                   | Temp. (°C) | Pressure (MPa) | Experience                   |
|---------------|--------------------|------------------|--------------------------------------|------------|----------------|------------------------------|
| Kristiansand, | 1951               | Gneis-granite    | Ø = 32 H = 15                        | 40         | 0.1            | No problems reported         |
| Trondheim     | 1955               | Quartzdiorite    | Ø = 32 H = 15                        | 40         | 0.1            | As above                     |
| Stavanger     | 1960               | Mica schist      | Ø = 15                               |            | 0.1            | Corrosion, decommissioned    |
| Ekeberg I     | 1969               | Granitic gneiss  | 12 × 10                              |            | 0.1            | No problems reported         |
| Mongstad      | 1975               | Meta-anorthosite | 22 × 30                              | 7          | 0.1            | Some water leaks             |
| Trondheim     | 1976               | Quartzdiorite    | 12 × 15                              |            | 0.1            | Water curtain has been added |
| Herøya        | 1977               | Limestone        | 10 × 15                              | 8          | 0.1            | Leak between caverns         |
| Ekeberg II    | 1978               | Granitic gneiss  | 15 × 10                              | 60         | 0.1            | Some blockfalls              |
| Harstad       | 1981               | Mica schist      | 12 × 14                              | 7          | 0.1            | No problems reported         |
| Sture         | 1987 / 1995        | Gneiss           | 19 × 33<br>~1,000,000 m <sup>3</sup> |            |                | No information               |
| Mongstad      | 1987               | Gneiss           | 18 × 33<br>1,800,000 m <sup>3</sup>  |            |                | No problems reported         |

Table 2 Norwegian Crude Oil Storage Facilities and Refinery Caverns for Hydrocarbon Products [GRØV, 2006]

| Project  | Commissioned | Main rock type | Storage volume, (m <sup>3</sup> ) | Width × height × length (m)   | Temp. (°C)                   | Pressure (MPa)       | Experience                |
|----------|--------------|----------------|-----------------------------------|-------------------------------|------------------------------|----------------------|---------------------------|
| Rafnes   | 1977         | Granite        | 100,000                           | 19 × 22 × 256                 | ~ 9                          | 0.65, tested at 0.79 | No leakage                |
| Mongstad | 1989         | Gneiss         | 3 caverns, total 30,000           | 13 × 16 × 64                  | 6-7                          | Up to 0.6            | No leakage                |
| Mongstad | 1999         | Gneiss         | 60,000                            | 21 × 33 × 134                 | - 42                         | 0.15                 | Reduced capacity          |
| Sture    | 1999         | Gneiss         | 60,000                            | 21 × 30 × 118                 | - 35                         | 0.1                  | No information available  |
| Kårstø   | 2000         | Phyllite       | 2 caverns, total 250,000          | Approx. 20 × 33 × 190         | - 42                         | 0.15                 | No leakage                |
| Mongstad | 2003         | Gneiss         | 60,000                            | 21 × 33 × 134                 | -42 (propane)<br>+8 (butane) | 0.15                 | No information            |
| Mongstad | 2005         | Gneiss         | 90,000                            | 22 × 33 × 140                 | 6-7                          |                      | Recently put in operation |
| Aukra    | 2007         | A Gneiss       | 63,000/180,000                    | 21 × 33 × 95<br>21 × 33 × 270 | 6-7                          | 0.2                  | Not yet commissioned      |

Table 3 Overview of Major Petroleum Gas Storage) [GRØV, 2006]

| Project   | Year of Completion | Main rock type      | Storage volume (m <sup>3</sup> ) | Width × height (m) | Temp. (°C)  | Pressure (MPa)     | Experience                 |
|-----------|--------------------|---------------------|----------------------------------|--------------------|-------------|--------------------|----------------------------|
| Herøya    | 1968               | Schistose limestone | 50,000 excavated                 | 10 × 12            | 6-8         | 0.8                | No leakage, decommissioned |
| Glomfjord | 1986               | Gneissic granite    | 60,000                           | 16 × 20            | - 28 to -33 | 0.1-0.13, max. 0.2 | No leakage                 |

Table 4 Overview of Major Ammonia (NH<sub>3</sub>) Storage [GRØV, 2006]

Taking into account that “Mother Nature” is not a perfect material, and that the rock mass may have a set of imperfections, it is most common that the construction process involves various techniques and methods to assist the design of a construction material that suits its purpose. In Tables 2 to 4 the various types of rock storages are listed and the local geological conditions in each of these, according to Grøv [2006]. The cross sectional areas of these caverns may not reach world record level but represent a typical level of utilisation.

### 8. An Underground Railway Stations with Large Span Cavern

The Norwegian Rail Road Authorities has just recently commenced the construction work with a new high speed rail way on the west coast of the Oslofjord connecting the cities south and south-west of Drammen with a new link to Oslo, the Vestfoldbanen (Fig.11).

The city of Holmestrand is located along this particular rail link. Holmestrand is a city which has the sea to the east and a mountainous hill to the west. Due to the stiff curvature of the high speed rail link it was decided that the rail way should be placed in a tunnel to pass Holmestrand. This means that a railway station in open

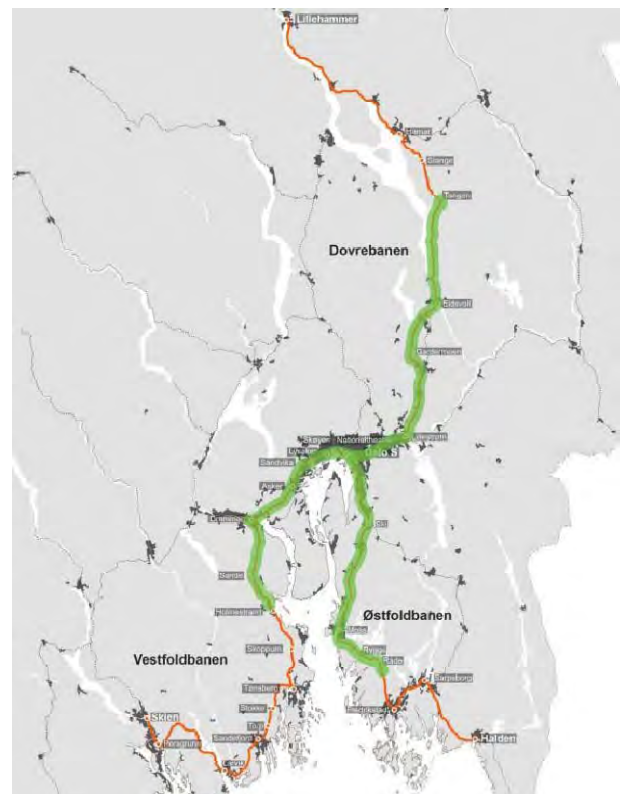


Fig. 11 The Vestfoldbanen from Oslo via Drammen and Holmestrand along the West Coast of Oslofjord (Ill. Norwegian Rail Road Authorities)

air was abandoned. However, a railway station was considered as an important link part of this railway link. Consequently, the station was replaced by an underground facility being placed in the mountainous ridge west of Holmestrand.

The speed for this rail way link is 250 km/h. The rock cavern which is designed to cater for the station area at Holmestrand is planned with a cross section of theoretically 350 m<sup>2</sup> and with a length of almost 900 m. The width of the cavern will be 36 m and with a height of 15 m (Fig. 12). This is sufficient to accommodate a total of 4 parallel tracks and platforms. The cavern is placed with a horizontal distance to the surface of 100 m.

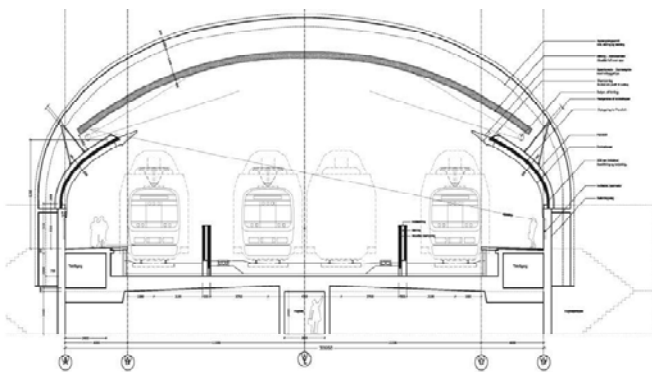


Fig. 12 Cross Section of the Underground Station (Ill. Norwegian Rail Road Authorities)

The site location has been subject to a typical pre-investigation scheme based on Norwegian standard including surface geological mapping, core drilling, structural geological analysis to mention some. The rock mass consists of basalt which has a typical UCS of approx. 240 MPa. The area is cut by several joint sets which have been identified at the surface and also by one significant weakness zone.

More important though, and the key to such an excavation is the stress measurements being conducted. These consist both of hydraulic fracturing and 3-D overcoring.

Based on these measurements it has been found that

the major stress component acts perpendicular to the cavern axis and the following magnitudes were found:

- 3D-overcoring yielded 4-11MPa
- Hydr.fracturing yielded 2-6MPa

As a comparison, the rock cover is about 70m which would produce a vertical stress component gravitationally of approx. 2 MPa and thus a horizontal component in the range of 0.5 MPa.

This example demonstrates the need of performing stress measurements and utilizing these measurements to optimize the cavern design.

### 9. A futuristic solution: cargo terminal underground

In Trondheim, the Public Railway Authorities were looking at the possibility of locating a new, major cargo terminal. The purpose of a railway cargo terminal is to receive, reload and distribute cargo and a typical layout is comprised of the following main areas: Loading area; dedicated to the handling of goods between rail wagons and other types of transport. Shunting area; dedicated to the assembly of the various wagons to make up individual freight trains. Receiving and departing area; where each freight train is recorded at the cargo terminal. A cargo terminal would necessarily occupy a large surface space, and it may therefore come into conflict with other interests and use of land. Although inter-linked, it is possible to spatially separate some of the different areas without disrupting the operational and functional efficiency of the terminal. Furthermore, the operation of the cargo terminal will inevitably interfere with and disturb the surrounding environment; not only in terms of noise and traffic from continuous operation, but also due to the aesthetic adverse impact on the terminal surroundings (Fig. 13).

Cargo terminals are traditionally considered to be surface installations. The location proposed by the authorities in Trondheim is dominated by residential, cultural and recreational areas, which has aroused hostile public opinion. The proposed site location was chosen due to its proximity to other important freight carriers. In an effort to mitigate the adverse impact on

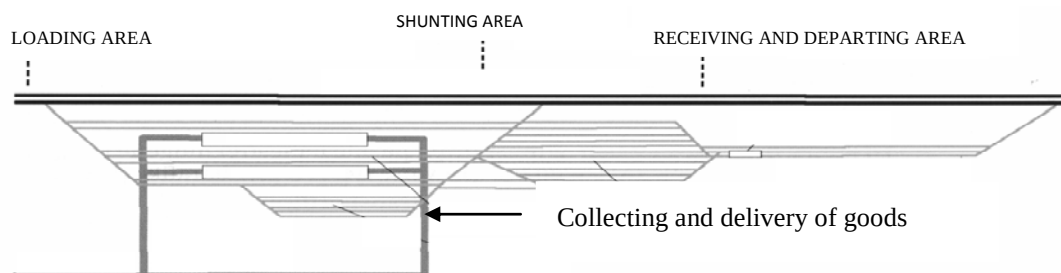


Fig. 13 Idealised Cargo-terminal Layout [GRØV, 2001b]

the existing surface structures, a study was prepared to present an alternative solution; to relocate certain portions of the cargo terminal to underground rock caverns. In discussions with the operator of the future cargo-terminal in Trondheim, the shunting area was identified as a candidate for relocation from the remainder of the cargo terminal.

The reasons for selecting the shunting area as a candidate for an underground location are the following:

- It is noisy; operating the shunting area where wagons are hooked on/off to the trains, generates a metallic, high frequency click-clacking noise together with the intense screaming from the mechanical braking of the wagons.
- It is aesthetically adverse; the shunting area comprises many parallel tracks and switches, several hundred meters in length and several tens of meters wide.
- It is an around the clock operation; the assembly of freight trains is a time consuming operation and to cope with the traffic at the terminal, an around the clock operation is required causing a continuous disturbance to the surroundings.
- It is space demanding; the total surface area required for the cargo terminal is equivalent to 25 football fields, with the shunting area including its service facilities occupying approximately 1/3 of this space.

An environmental impact assessment was conducted for the surface installations, addressing the following aspects: landscape/visual environment, cultural and recreational environment, noise and vibration disturbance, area use and real estate potential, urban development, transport and traffic arrangements and finally air pollution. The option of a shunting area in underground rock caverns is put forward with the following potential benefits:

- It allows valuable surface space to be utilised and developed for other purposes, or to be reinstated to its original use. When the land is occupied for residential, recreational and cultural use, it is important to reduce the impact that such change would have. A surface installation would require many houses to be demolished whilst a subsurface solution would reduce the impact on the current use of the area. The visual impact alone created by a 40m wide band of railway tracks with noise reduction barriers along its entire boundary constitutes a significant impetus to go underground.
- It improves the physical environment and limits awareness of and disturbance to the public. Most people prefer to keep their daily residential and recreational life apart from industrial areas. The necessity of such areas is generally accepted,

however, most people would prefer not to have them as neighbours. In this example, the most positive impact achieved by an underground installation is the reduced noise. A surface installation would probably disturb the community within a distance of several hundred meters. The airborne noise can to some extent be mitigated, but only partially. Ground vibrations created by the moving of heavy equipment will also be significantly reduced for a subsurface installation.

- It eases flexibility for future expansion. Construction of such a large, complex structure as a cargo terminal, within an area which is already extensively developed, implies a significant limitation and flexibility as regards to any future expansion. A well planned, subsurface structure could provide greater flexibility for future expansion.
- It reduces the need of construction activity at the surface and provides a surplus on the mass balance. A cargo terminal requires a large, flat surface area and could lead to a major earthmoving exercise. A subsurface solution produces a surplus on the mass balance, and the blasted rock constitutes a value in itself, which could be sold or used for other projects.
- It has a constant climate around the year that ensures reliable and regular operation. A shunting area is sensitive to weather conditions, as icy switches etc. may cause adverse effects and operational hazards, such as derailing. Particularly during cold winters in Norway the operational regularity of the shunting area would increase if located “in-doors”.
- It preserves the environment. For a subsurface solution dedicated systems for collection and handling of various types of spill was planned. With current tunnelling technology and methods, the cost estimate showed an obvious disadvantage. The subsurface alternative involved an increased investment of approximately 20% over the surface solution, when taking the cost savings on surface preparations and land acquisition into account. The installations in the rock caverns as well as the rock support measures were reasonably cost effective. However, in the bottom line the subsurface alternative has a higher cost/benefit ratio, when no price tag was put on the environmental aspects.

A railway track constructed in the early 20<sup>th</sup> century, is currently in use as a ring connection around the city. This includes a 2.5 km long rock tunnel excavated in greenschist and greenstone, mostly occurring as a quite massive rock. The rock mass conditions are fairly well known with respect to its ability to host subsurface structures, as a number of smaller caverns and tunnels have been constructed in this geological formation. The rock mass has proved to be of fairly good quality,

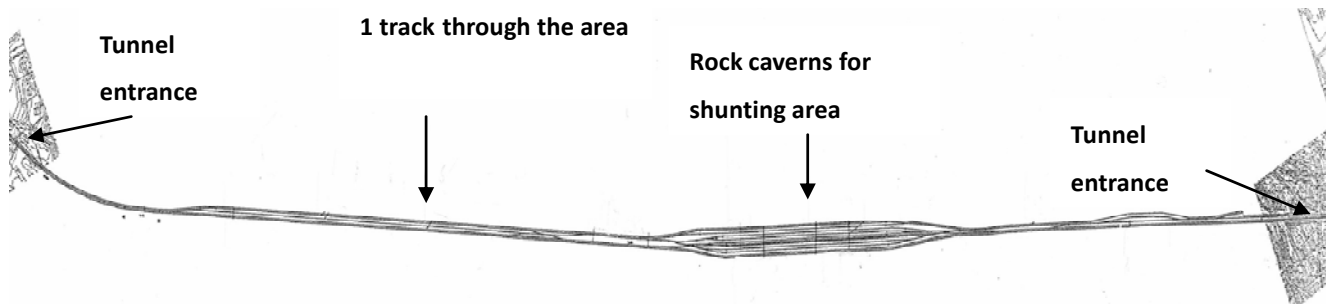


Fig. 14 Proposed Layout of Shunting Area [GRØV, 2001b]

which has resulted in a minimum of rock support and mostly dry conditions. A joint set parallel to the foliation of the schist, with a  $30^\circ$  dip is the most dominant. The feasibility study concluded that these conditions were favourable for large rock caverns. The fact that the existing tunnel is available for inspection increases the level of confidence with respect to the quality of the rock mass. Along the 2.5 km long tunnel, cast-in-place concrete lining was applied on 3 sections with significant weakness zones, constituting a total length of 110 m. Rock support is in general rock bolts. Minor spots with water dripping or moisture were observed. The rock cover above the proposed caverns is a maximum of 125 m.

The regional, in-situ stress regime indicates that the maximum horizontal stress component could be expected to be influenced by remnant stresses, producing a K-factor (horizontal stress component/vertical stress component) in the range of 1.0. The in-situ stress situation was modelled with vertical and major horizontal stress components in the range of 3.5 to 4 MPa.

The proposed solution utilised the existing railway tunnel with a major enlargement to reach the required cavern width of 42 m was studied (Fig. 14). Discussions with the operator of the cargo terminal indicated that this width was needed to allow for the required number of nine parallel tracks. Based on both analytical and numerical calculations, the feasibility study concluded that the 42 m wide cavern with a cross section of almost  $400 \text{ m}^2$  and length of almost 700 m, was feasible in the actual rock mass conditions. An alternative layout including two parallel caverns, each 28 m wide and separated by a 25 m thick pillar, was also studied and found feasible (Fig. 15 & 16). By 28m width, each cavern could hold 6 parallel tracks, or a total of 12 tracks, thus allowing a greater capacity than the option with a single 42 m wide cavern.

The subsurface proposal allows for the shunting area itself, and the various service facilities required for its operation. In addition, one separate track allows for the possibility of trains passing through the tunnel without interference with the ordinary operation of the shunting area. Consequently, together with the main cavern(s), the concept proposed a number of small enlargements of the existing tunnel, which utilise the entire length of

the 2.5 km tunnel to provide the various service facilities.



Fig. 15 Caverns and Rock Mass Structures [GRØV, 2001b]

The whole design included excavation of some  $400,000 \text{ m}^3$  of solid rock, which together with the existing tunnel would make a total underground volume of almost  $500,000 \text{ m}^3$ .

The numerical analysis, which actually modelled the two parallel caverns with a width of 40 m and a 25 m pillar, indicated that a satisfactory level of safety could be achieved by the systematic application of bolting, spacing at  $2 \text{ m} \times 2 \text{ m}$ , 25 mm diameter and 5 m long. The numerical model did not include support measures such as sprayed concrete or cast-in-place concrete. However, the design included sprayed concrete as a surface sealing.

Until this date, however, neither the caverns have been excavated nor has the above ground facilities been built. A major political decision is lacking, until then we have to wait and see before any such caverns can be built, but its feasibility and advantages for the society are obvious.

## 10. What Made the Gjøvik Mountain Hall Feasible?

The Gjøvik Olympic Mountain Hall was constructed during the years from 1991 to 93 as one of the ice-hockey venues for the 1994 Lillehammer Winter Olympic Games. With a span of 61 m, a length of 95 m, and a height of 25 m it is the largest man-made rock cavern for public use in the world (Fig. 17 & 18). The total excavated volume was  $140,000 \text{ m}^3$ . The width of

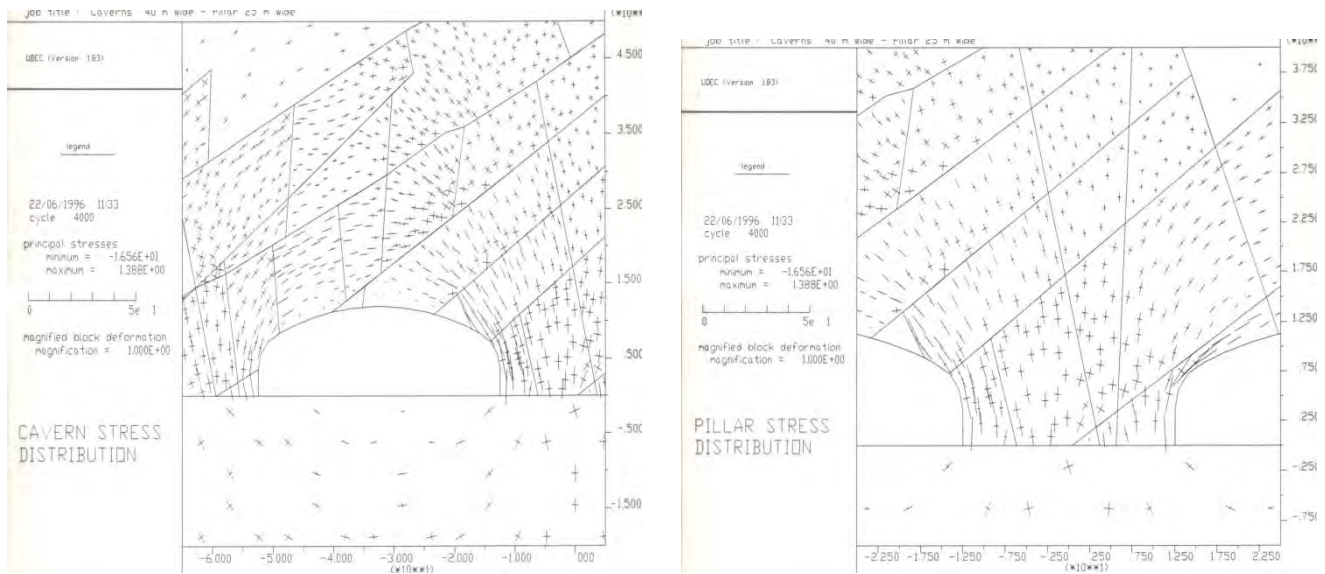


Fig. 16 Cavern Stress Distribution and Pillar Stress [GRØV, 2001b]

the cavern is not the only peculiar feature of this project, as interesting is the fact that the rock cover varied between 25 m and 55 m, i.e. the overburden throughout the underground cavern is far less than its span.

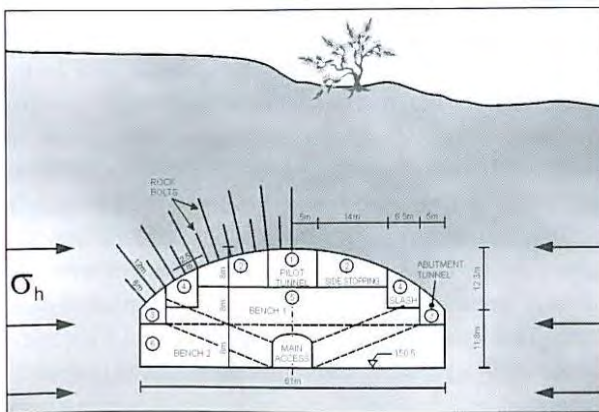


Fig. 17 Cross Section of the Gjøvik Mountain Hall [MYRVANG, 2004]

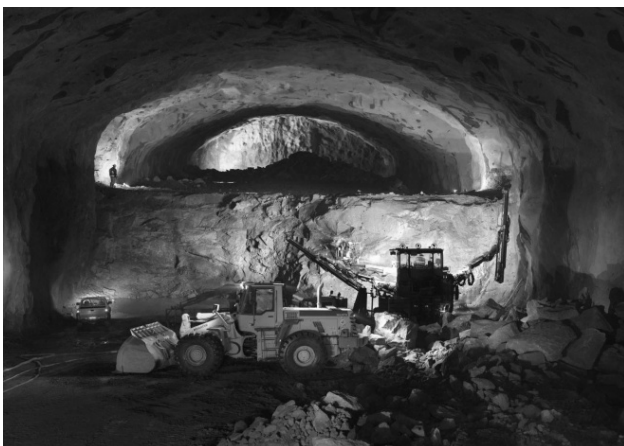


Fig. 18 Cavern Construction Sequence (Photo unknown)

The rock type at the site is Precambrian gneiss. The rock has developed a network of tectonic micro-joints, which often were filled with or coated with calcite or epidote. These rock mass conditions resulted in a well jointed rock mass with an average RQD of about 70. The typical joint character is one of persistence, moderate to marked roughness, and normally without clay filling, i.e. positive characteristics when considering large spans. The Q-value is typically 30 for the best and 1 for the poorest quality rock mass, with 12 as an average value.

Experiences from large span mining chambers in Norway indicated that a major prerequisite to obtain stable large span caverns without heavy rock support was sufficiently high horizontal rock stresses. At a very early stage of planning, in situ rock stress measurements were therefore carried out from an existing tunnel. The results showed dominating horizontal stresses in the order of 3 – 5 MPa. At a depth of 25 – 55 m the vertical stress due to gravity is less than 1 MPa, indicating that the horizontal stresses are stresses generated by geological processes (tectonic stresses). This was verified later by hydraulic fracturing tests, conducted in several rounds, including both over-coring and hydraulic fracturing in vertical boreholes drilled from the surface above the hall. Based on these findings, it was decided to proceed with the investigations.

With reliable in-situ stress values from the stress measurements, numerical modelling was carried out, using various BEM, FEM, UDEC and FLAC codes. The final conclusion was that a stable and virtually self supporting 62 m span could be constructed under the

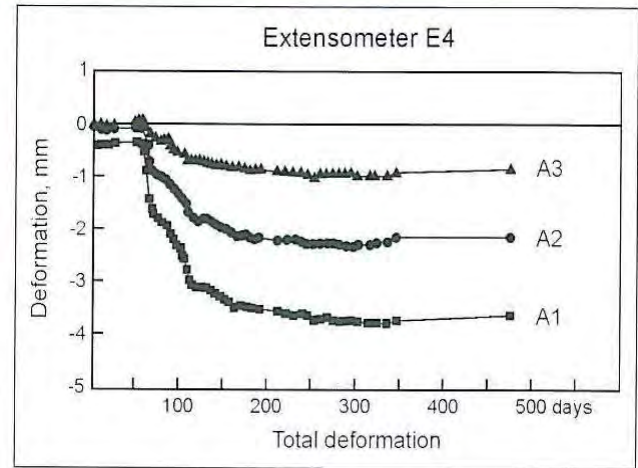
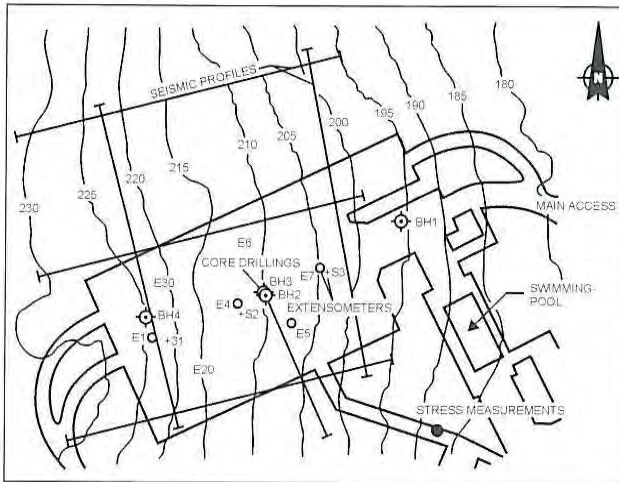


Fig. 19 Cavern Lay-out and Borehole Extensometer Locations, Typical Reading of Surface Extensometer (right hand side) [MYRVANG, 2004]

given geological and rock mechanics conditions. Maximum roof deflection could be expected to be in the 5 – 10 mm range.

A key element of this entire process is related to the in-situ stress measurements, which were Adjacent to the main cavern hall area, an underground swimming pool was constructed in 1974, thus the geology of the area was well known, and later this facility was included in the new construction. In addition, the gentle slope of the hill hosted an underground telecommunication centre in the close vicinity of the Gjøvik hall.

To monitor roof deformations a number of multiple position borehole extensometers (MPBX) were installed. Figure 19 shows the cavern layout and the position of seven, 3-anchor (position) MPBXs placed in boreholes drilled from the surface (marked E1-E7), and three placed in boreholes drilled vertically upwards in the cavern roof (marked S1 – S3). In addition, surface precision levelling was carried out on top of the three centre-line extensometers.

Readings were taken regularly throughout the construction period. Figure 19 above shows typical readings for the central extensometer E4, with A1 being the anchor close to the cavern roof. After the full span was blasted after about 100 days, the deformations show a decreasing trend until they stabilise completely after some 300 days. The maximum deformation is less than 4 mm. By adding the readings from surface and roof extensometer and the surface levelling, the maximum deflection was estimated at about 7 mm. This is well within the predicted values from the different numerical models.

To check the roof stresses, 2D in-situ rock stress measurements were carried out mid-span close to the S1, S2 and S3 extensometer locations. They all showed compressive roof stresses in the range of 2 – 5 MPa, which are good indicators of a stable, immediate roof.

The investigations clearly indicate that the roof “globally” is a self-supporting structure. The roof is, however, systematically supported by 6 m fully grouted 25 mm rebar bolts in a 2.5 m x 2.5 m grid, where every four bolt is substituted by a 12 m cable bolt. This is followed by 100mm fibre reinforced shotcrete. A series of eight rebar bolts were instrumented by strain gauges, and the load change was monitored as the span was increased from 10m, through 37 m to the final span of 61 m. Only three bolts showed any indication of load take up. This happened close to the roof surface, with very moderate load in two cases (10 kN and 15 kN), while the third one showed 87 kN, which is about 40 % of the yield load.

To check the performance of the fibre reinforced shotcrete, strain gauge rosettes specially made for concrete were installed in four locations. The readings only indicated very low tensile stresses, which probably are due to shrinkage of the shotcrete. The rock/shotcrete bond was tested by a direct pulling test on drill cores containing the bond. The average tensile strength of the bond was 0.85 MPa, which was regarded satisfactory. The main purpose of the shotcrete seems to be to bond the rock surface together, preventing smaller rock volumes to fall. Based on this, it may be concluded that the need of the systematic pattern of 6 m and 12 m bolts and cables is questionable. A systematic 2.5 m x 2.5 m pattern with 3 - 4 m fully grouted rock bolts, combined with 75 mm

to 100 mm shotcrete will do the job even for 50 m to 60 m span. However, horizontal, tectonic stresses of a certain magnitude are necessary to establish the global stability of the roof.

### 11. The Qinling Zhongnanshan Tunnel

The Qinling Zhongnanshan Tunnel, Shaanxi, China, consists of four tunnels: two railway tunnels and two road tunnels. With a length of 18.02 km the road tunnel is currently the longest double tube road tunnel in the world. The gross cross section of the road tunnels is 12.8 m x 10.5 m, accommodating three driving lanes. The tunnels were open to traffic in January 2007. For special lighting and driving safety purpose six caverns are designed for the road tunnels (Fig. 20).

The idea is when a person drives in such a long tunnel he or she may feel monotony and anxiety, and loses concentration. As a result the potential of traffic accidents increases. Special and bright lighting is designed in the caverns such that the driver will be woke-up and has a perception of being outside the tunnel. In this way, the driving safety is increased.

The tunnels penetrate the Qinling Mountain Range, where the major rock type is mainly granitic gneiss of good quality and the maximum rock cover along the tunnel route is about 1800 m. Rock bursts and spalling were frequently encountered during tunnel excavation. With a strong anticipation of high in-situ rock stress and reorganization of its significant impact on the cavern stability, a field stress measurement program by overcoring technique was carried out in two boreholes close to the cavern sites. The overburden for the boreholes is 400 and 1600 m, respectively. The measurement at the low overburden hole was successful, whilst great difficulties were encountered in the high overburden hole. Severe core dishing was observed, see Figure 21.



Fig. 21 Illustration of Core dishing: 23 Disks Observed in a 27 cm Long Core [LU et al, 2011])

Finally, the 3D overcoring was replaced by doorstopper (2D overcoring). The measurement result indicates the highest major principal stress may be as high as 45 MPa [LU et al, 2011].

The rock support was designed with empirical means and verified by 2-D and 3-D numerical analyses, and the experience gained from the Lærdal tunnel was referenced. Having realized the restrictions by the very high rock stress and the small pillar width in comparison to the cavern size (the minimum pillar width is 8 m and the cavern span is 22 m) the design was very cautious.

The designed rock support system consists of rock bolts and fibre-reinforced sprayed concrete, a clearly defined construction sequence and a monitoring system. For the caverns under extremely high stresses a flexible support is proposed which is composed of temporary support and permanent support. The idea is to allow rock deformation to partially take place before the permanent reinforcement is applied, to avoid failure of the support elements. The use of the end-anchored rock bolts with polyurethane cartridge is particularly specified. The length of the bolts was such designed such that the anchorage ends must be located at the competent rock, in other words, the bolts must be anchored in the non-yielding rock, which was ensured by the numerical analyses.

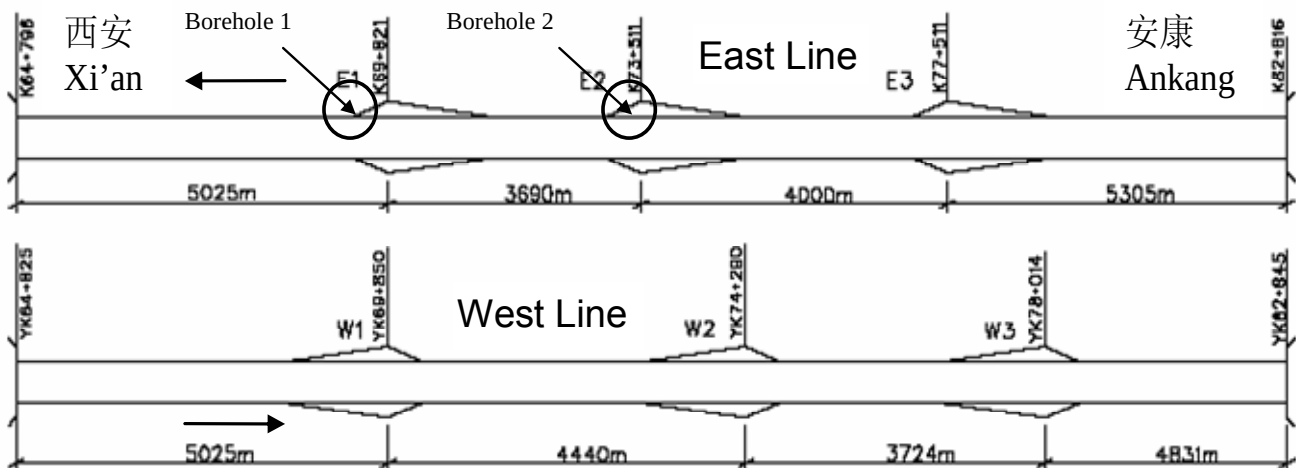


Fig. 20 Locations of Measurement Boreholes and Caverns of the Qinling Zhongnanshan Tunnel [LU et al, 2011]

## 12. Rock Caverns in Singapore

The increasing demand for surface land has driven the development of underground space in Singapore. After the first rock cavern for ammunition storage was successfully constructed in 2005 an underground cavern complex for storage of hydrocarbon products including crude oil, condensate, naphtha and gas oil is now under construction. Two site investigation programs were carried out from 2004 to 2006 to examine the geological and hydrogeological conditions of the selected cavern site at the Banyan Basin of the Jurong Island. Almost all available exploration techniques were used in the investigation such as:

- marine seismic survey (both reflection and refraction)
- land seismic survey
- horizontal directional core drilling
- vertical and inclined borehole survey on land
- laboratory tests on rock and soil samples
- in-situ rock stress measurement
- groundwater monitoring
- permeability test
- rock/groundwater compatibility tests
- other geophysical tests

In short the site investigation has confirmed the feasibility of constructing the storage facility at the selected site and provided sound basis for optimization of the cavern layout. The site is in the Jurong sedimentary formation consisting mainly competent sandstone and siltstone [TAN et al, 2011].

The facility is planned to be developed in two phases. The storage volume for phase 1 and 2 is 1.47 and 1.53 million cubic meters, respectively. Phase 1 includes 9 caverns of which 8 are in pairs. Each cavern is 20 m wide, 27 m high and 270 - 380 m long. Another distinct feature is that the caverns are not only underground, they are also under sea. The cavern roof is approximately 120 m below the sea level and 100 m below the sea bed. All caverns are excavated by drill and blast method and bolting and sprayed concrete are standard rock support. Figure 22 is extracted from an article published in a local news paper, The Strait Times, on May 26, 2010, which illustrates the layout and profiles of the caverns.

Access to underground during construction is through two vertical shafts with completed inner diameter of 18.3 m at the cavern level. All pipelines connecting under- and above ground go also via the shafts. The shafts are sole access to underground also during operation. The upper section of the shafts is situated in the reclamation fill and residual and therefore lined with heavily reinforced diaphragm wall, whilst the lower section in the bedrock is supported by bolts and sprayed concrete.

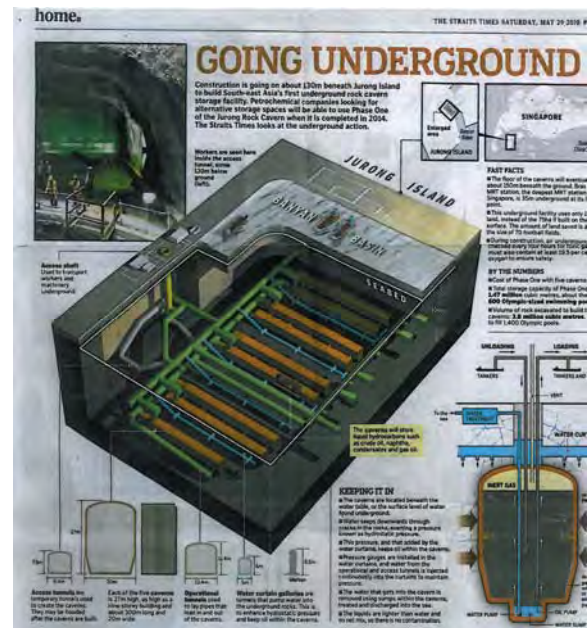


Fig. 22 Article from newspaper in Singapore

Control of oil/gas leakage from storage caverns is realized by hydrodynamic means, i.e., the groundwater pressure is maintained all time higher than the oil/gas pressure inside the cavern. Thanks to the sub-sea nature the systematic horizontal water curtain is not necessary, but vertical water curtains between the caverns and local horizontal water curtains have to be utilized.

It is estimated the Jurong Rock Cavern is able to save an equivalent of at least 60 hectares of surface land. Such land savings can be allocated for higher value manufacturing activities.

In 2007, a joint venture of several Norwegian, international and local consultants was appointed to prepare a Preliminary Master Plan associated with an increased and extended utilization of the underground for Jurong Town Corporation (JTC). This Master Plan included a review of topographic and geological conditions to identify suitable locations for underground caverns.

The Master Plan study included also strategic evaluations of access, availability, proximity to existing infrastructure and industry, and finally the cost aspects of establishing such underground facilities. In the Master Plan there are no restrictions or instructions on the development of above ground utilization within those areas which have been allocated or found suitable for underground facilities. Further, in 2008 JTC initiated a "underground rock cavern usage feasibility study" to assess how rock caverns could be used for other purposes including such as:

- Power station and electrical substations
- Incineration plants
- Water reclamation plants

- Data centre
- Warehousing and logistics
- Airport logistics
- R&D facilities
- Landfills

The study was undertaken by same group of consultant prepared the Preliminary Master Plan. Among others the study also covers the cost estimate and preliminary conceptual designs. At present few details can be released. However, it comes logical that to be able to serve the purposes listed above there is a need of establishing major underground facilities and thus large caverns would be required.

A quote from the Singaporean Prime Minister Mr. Lee Hsien Loong suggests that there is a vision to push the edge of technology and application of the underground to serve useful needs of the society:

'JTC must keep on pushing the limits of possibilities, to try out new ideas, and seek opportunities beyond known frontiers'

### 13. Conclusions

In Norway a large number of underground caverns have been built during the last 30 - 40 years. Most of these caverns are related to the development of the hydro electric power supply. It is said that worldwide there is about 500 underground power stations, out of which 200 are located in Norway. This is a rough figure but it indicates how the underground has been utilized for this particular purpose. Furthermore, several underground caverns have been constructed for the purpose of storing oil and gas, whilst a large number of underground large caverns are related to sports complexes and civil defense facilities.

The majority of these caverns is in the range of 15 to 25 m wide, some tens of meters high and up to several hundreds meter long. They are all built in accordance with a concept of unlined caverns, mainly supported by rock bolts and sprayed concrete.

One major common feature of these caverns is the presence of sufficient horizontal in-situ stresses. Another common feature is the field testing of in-situ stresses of the selected site location. Dedicated testing methods, both hydraulic fracturing and over-coring have seen great developments in Norway. Indeed, the presence of in-situ rock stress allows the possibility of opening large underground caverns. This is an important aspect that needs to be taken into account when designing underground caverns and wide tunnels in urban areas. The technology was developed in rural areas when the hydro electric power development took place in Norway, and is now available for various purposes when utilizing the capability of the rock mass.

It is required to add that the experience and knowledge gained from the mining industry with respect to the understanding of the in-situ stress conditions has been invaluable for the development of large caverns for civil application.

The examples presented in the above shows that the utilization of the underground can benefit the development of urban areas by using large underground caverns to host various activities. The use of the underground in urban areas may not be restricted to various types of transport tunnels, and even small or normal sized transport tunnels would indeed benefit from the presence of sufficient in-situ stress conditions.

## Underground, benefit of the presence of in-situ rock stress!!

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# PLANNING FUTURE CAVERN DEVELOPMENT IN HONG KONG

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## Abstract

*In March 2010, the Civil Engineering and Development Department commenced a one-year study on the ‘Enhanced use of underground space in Hong Kong’, to showcase how the planned development of underground space and, in particular, the enhanced use of rock caverns, could contribute to the effective utilisation of land resources in Hong Kong.*

*The study concluded that some 64% of Hong Kong’s land area is suitable for cavern development and has identified five strategic cavern areas with potential to accommodate multiple facilities. Over 400 existing government facilities were assessed for their potential for relocation to rock caverns, and the broad technical feasibility has been demonstrated for a few selected facilities, releasing land for other uses. Whilst further studies are required to assess the overall feasibility and merits of relocating individual facilities to rock caverns, the study findings indicate that rock cavern development should form part of Hong Kong’s future planning and development strategy.*

## Keywords

*Planning, Rock Cavern, Stock Taking, Government Facility, Strategic Cavern Area*

## 1. Introduction

Rock cavern development is in line with the policy of ‘Progressive Development’, representing the concept of sustainable, diversified and balanced development and conforms with the 2030 Planning Vision and Strategy for optimizing available development opportunities and sustained growth (Hong Kong Special Administrative Region (HKSAR) Chief Executive’s 2008-09 Policy Address).

In October 2009, the Chief Executive presented the 2009-10 Policy Address - “Breaking New Ground Together”. A new initiative was included under Chapter 1 of the Policy Agenda – ‘Developing the Infrastructure for Economic Growth’ to launch strategic planning and technical studies to facilitate planned development of underground space aiming at promoting the enhanced use of rock caverns as part of Hong Kong’s pursuit of sustainable development.

The Geotechnical Engineering Office of the Civil Engineering and Development Department, HKSAR Government commissioned a strategic study on the

‘Enhanced Use of Underground Space in Hong Kong’, to take this initiative forward. Broad aspects of the study are described by ROBERTS et al [2011]. The study covered the following key areas:

- review good practice elsewhere and identify opportunities for enhancing Hong Kong’s policy, strategy and procedures to facilitate use of underground space,
- take stock of existing and future public facilities that could be re-located/located in rock caverns, with suitable classification according to their nature and cost-benefit considerations,
- select public facilities for preliminary feasibility studies to relocate the facilities in rock caverns,
- develop a cavern suitability map demarcating areas that are suitable for rock cavern development, and
- identify priority areas and individual priority sites for rock cavern development.

The one-year study was completed in March 2011 [ARUP, 2011]. The Executive Summary of the study can be viewed at the Civil Engineering and Development website <<http://www.cedd.gov.hk>>. This paper highlights the key aspects and findings of the study, with recommendations for taking forward the initiative of cavern development in a planned manner.

## 2. Cavern Development in Hong Kong

Hong Kong’s topographical setting with steep natural hillsides pose significant constraints to development. Land, particularly in the urban areas, is a scarce resource. To support economic development and population growth, creative and sustainable approaches are increasingly required to optimize and improve the supply of land for various uses.

The abundance of strong igneous rocks in Hong Kong, which covers some 80% of the land area, offers a great opportunity for placing facilities in rock caverns. The hilly areas in the urban fringes are particularly suitable for cavern development as they allow easy access into the ground to construct the caverns. Large rock caverns up to 61 m wide, i.e. more than double the width of the largest cavern built in Hong Kong to date, have been constructed safely in areas with geological conditions similar to Hong Kong, e.g. the Gjøvik Olympic Mountain Hall in Norway.

A review of the past work carried out on cavern related activities in Hong Kong up to 2006, including

(i) technical studies (ii) institutional arrangements, (iii) technical guidelines and (iv) cavern construction was outlined by CHAN & NG [2006]. Follow up work and related studies are described in later sections of this paper.

The first caverns in Hong Kong were related to tunnel networks for water supply or railways such as the valve chamber in Western District Aqueduct in 1984 and Mass Transit Railway (MTR) stations at Sai Wan Ho and Tai Koo in 1985. MALONE & CHAN [1993] referred to these as Hong Kong's 'first generation' rock caverns, which have generally been built out of necessity to provide enlargements. Around the mid-1990s, a number of public facilities in Hong Kong were purpose-built in rock caverns (i.e. unrelated to tunnel networks) to meet the needs of the community, namely the Island West Refuse Transfer Station, Stanley Sewage Treatment Plant and Kau Shat Wan Explosives Depot. The term 'second generation' rock caverns was used by MALONE & CHAN [1993] to describe this purpose-built underground space for environmentally unattractive or security-sensitive facilities which have only been built in caverns because a suitable surface site could not be found.

Recently, the University of Hong Kong has constructed rock caverns to re-house two existing Water Supplies Department salt water service reservoirs underground to release land for the Centennial Campus development. Rock caverns are also to be constructed as part of the planned Mass Transit Railway (MTR) projects in the next few years.

There is proven local experience in cavern construction as demonstrated by the various schemes built in Hong Kong. These projects have shown that rock caverns can be a cost-effective alternative, while yielding additional environmental, safety and security benefits

for many applications. However, most of these rock cavern developments have generally been formed either out of necessity, or have been reactive and only considered where over-riding circumstances exist.

### 3. Planning Standards for Cavern Development in Hong Kong

The Hong Kong Planning Standards and Guidelines (HKPSG) have a specific section on 'Rock Cavern Development' which outlines the role of relevant Government departments/ agencies in the planning and development of rock caverns. These were first published in 1991 and revised planning guidelines were promulgated in November 2008 [PLAND, 2008]. The land uses with the potential for development in rock caverns in Hong Kong are shown in this section of the guidelines (Table 1). Following the strategic study, recommendations were made to include additional land uses and these are shown in Table 1.

### 4. Stock Taking of Government Facilities

A territory-wide stock taking exercise was undertaken to identify existing and future government facilities in the pipeline that could be located in rock caverns. The aim of the stock taking was to identify the existing and planned above-ground government facilities that have the potential for rock cavern development, and to collect information on these facilities including location, site area, capacity and plans for expansion or reprovision.

Only Government/Institution/Community Uses listed in the guidelines [PLAND, 2008] were included in the stock taking exercise. The stock taking exercise has identified over 400 government facilities could be considered for placement or reprovisioning in rock caverns (Fig. 1). The Not-In-My-Backyard' (NIMBY)

| Land Use Category                    | Potential Land Uses in the Current HKPSG (2008)                                                                                                                                                                                                              | Additional Potential Land Uses                                                                                                                                         |
|--------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Commercial                           | Retail                                                                                                                                                                                                                                                       | Food / Wine Storage<br>Warehousing                                                                                                                                     |
| Industrial                           | Industry<br>Storage / Warehousing<br>Oil bulk storage<br>LPG bulk storage                                                                                                                                                                                    | Dangerous goods<br>Data centre<br>Research laboratories<br>Science park                                                                                                |
| Government / Institution / Community | Slaughterhouse<br>Civic centre<br>Incinerator<br>Indoor games / Sports hall<br>Wholesale market<br>Refuse transfer facility<br>Sewage / Water treatment plant<br>Service reservoir<br>Transport connections & networks<br>Columbarium / Mausoleum / Mortuary | Archives<br>Bicycle park-and-ride<br>Car / Vehicle parking<br>Crematorium<br>Refuse collection point<br>Maintenance depot for rail and others<br>Underground quarrying |
| Public Utilities                     | Power station                                                                                                                                                                                                                                                | Substation                                                                                                                                                             |

Table 1 Land Uses with the Potential for Development in Rock Caverns in Hong Kong

## 5. Selection Criteria and Preliminary Ranking of Government Facilities

The selection criteria were divided into 3 grades (A, B or C) to provide a simplified basis for the ranking of the facilities. The more grade A's identified for a facility then the more suitable it is likely to be for cavern development, subject to further review. The current preliminary screening only covers basic

From the stock taking and preliminary ranking exercise, several strategic facilities with the potential for being relocated in rock caverns were identified. The facilities are distributed throughout Hong Kong Island, Kowloon and the New Territories. These facilities were broadly reviewed in terms of site suitability, land use context of the surrounding area and their relative benefits. During the review, any lower ranking facilities that were in close proximity to the selected cavern site were included in larger cavern schemes when compatible. This could lead to significant areas of developable land being released, and often the removal of potential multiple bad neighbour uses from an area. However, it should be noted that the potential opportunities offered by the cavern option have not been examined by the relevant facility owners as part of this study.

### Further refinement of the preliminary ranking

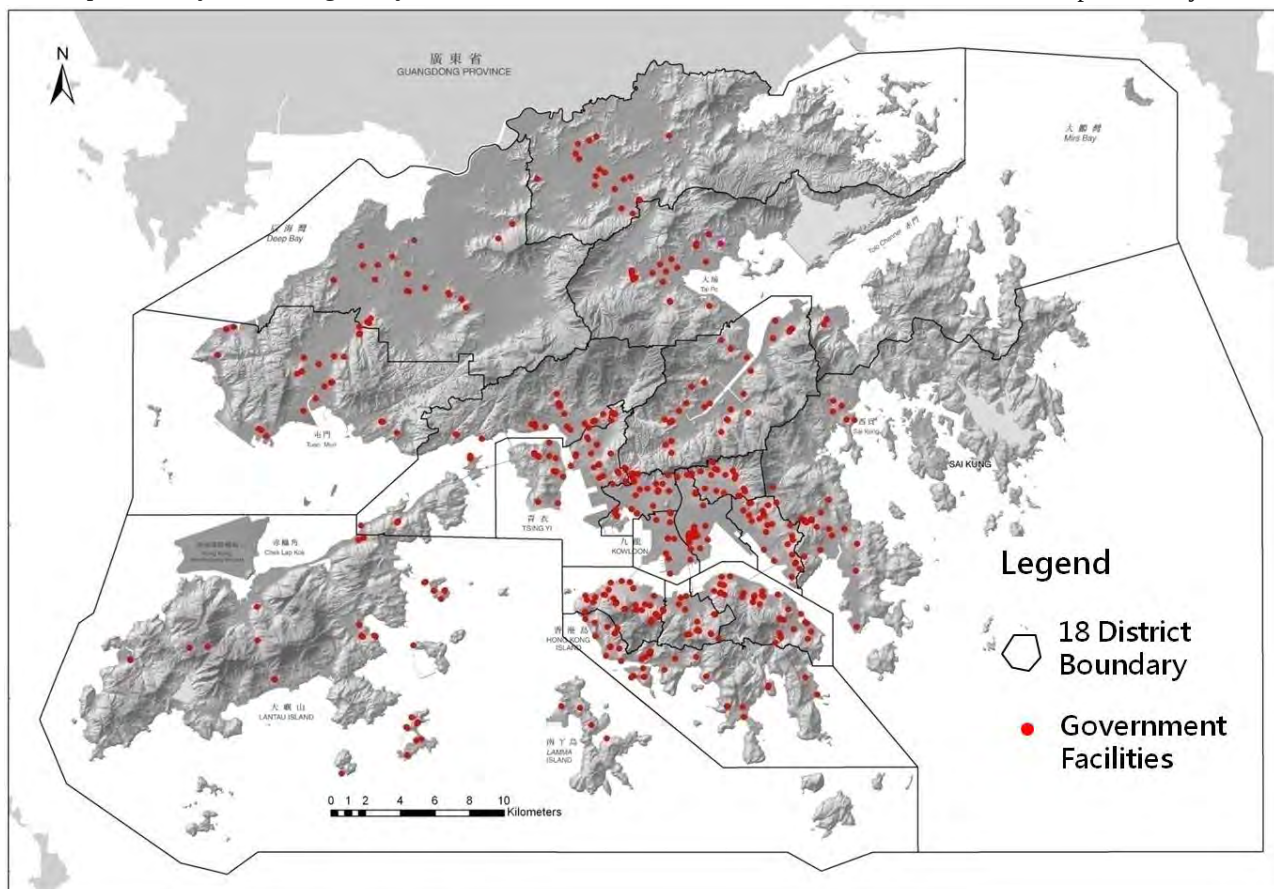


Fig. 1 Stock-taking of Government Facilities

methodology and more detailed study will be required within each of the potential cavern sites in order to confirm that the facilities are technically feasible to be relocated without significant impact to the service, community, surrounding landscape, environment and traffic network.

Future study has been recommended to further develop the criteria and the ranking and incorporate such additional factors and weighting as necessary, taking account of planning, functional and engineering issues, for the existing or other proposed criteria that are likely to be required to suit a systematic and comprehensive relocation strategy for various facilities. This will ensure that those facilities offering the highest benefits to the community as well as improvement, where applicable to the network operation will be identified for relocation in rock caverns. The potential to relocate or locate any individual facility shall be confirmed by detailed feasibility study.

## 6. Development of a Territory-wide Cavern Suitability Map

A preliminary territory-wide cavern suitability map has been developed using a Geographical Information System (GIS) based approach to aid in the future selection of potential areas for cavern development during the planning stage. The original Cavern Area Studies [ROBERTS & KIRK, 2000] outlined a series of attributes that were considered relevant for cavern

suitability and these were reviewed and additional attributes added to fit with a territory-wide study. The original and additional suitability criteria are listed in Table 2.

| Attributes considered in the original Cavern Area Studies | Additional attributes considered in the Strategic Studies |
|-----------------------------------------------------------|-----------------------------------------------------------|
| Existing and proposed surface development                 | Country park                                              |
| Existing and proposed underground installations           | Drillholes data                                           |
| Engineering geological conditions                         | Reclamation / Fill areas                                  |
| Topographic constraints                                   | Poor geology                                              |
|                                                           | Scheduled Area                                            |
|                                                           | Land fill areas                                           |
|                                                           | Impounding reservoirs                                     |

Table 2 Suitability Criteria

A broad classification for whether a cavern would be generally above or below +10 mPD was also introduced to identify direct entry caverns to those that were generally in low lying areas subject to more geotechnical constraint. The GIS analysis identified various suitability classes from high to very low and some areas that were considered to be unsuitable. The suitability classes were used to generate cavern suitability maps for the whole of Hong Kong in both detailed and simplified format and the simplified version is shown in Figure 2. As a whole, about 64% of

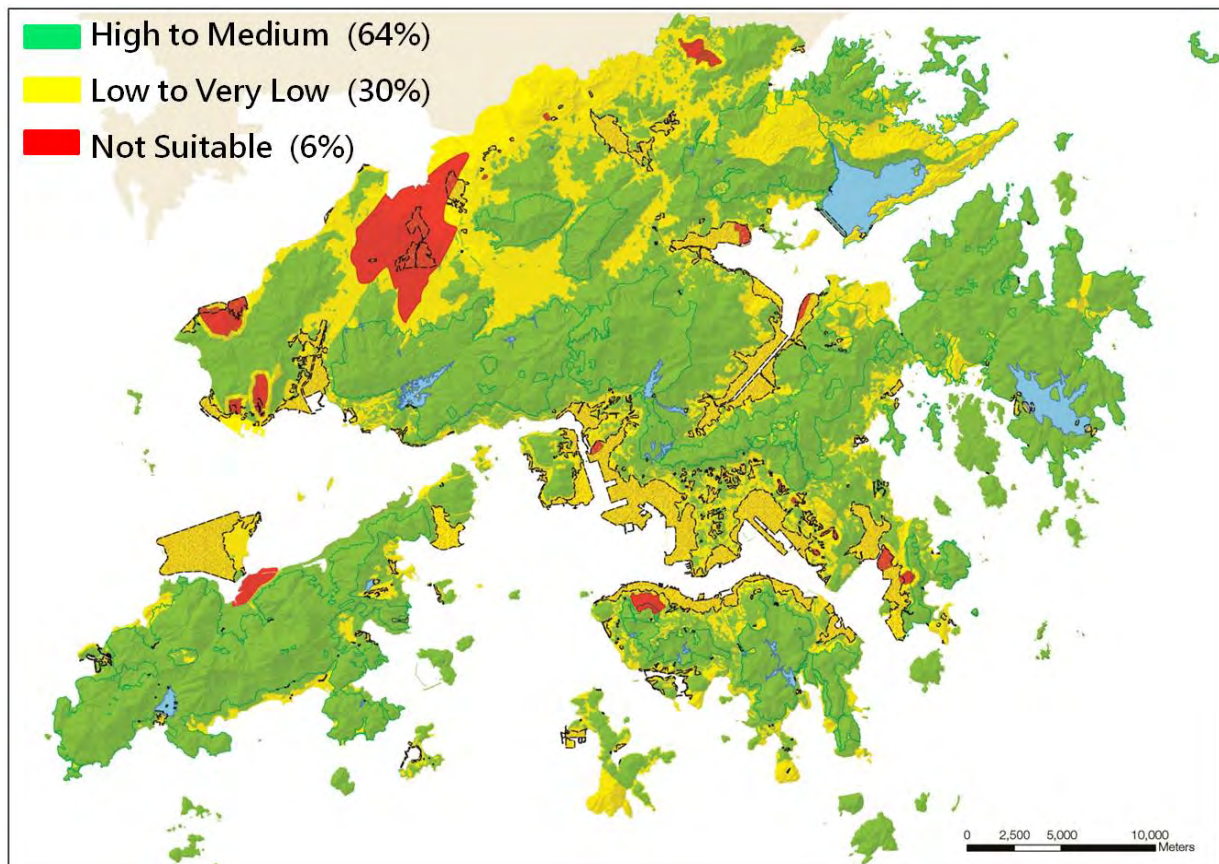


Fig. 2 Cavern Suitability Map

Hong Kong's land area is potentially suitable for cavern development, i.e. these areas are considered to be more easily developed with reduced overall construction cost.

## 7. Preliminary Feasibility Studies

Following the stock taking and preliminary ranking exercise, three facilities were selected to carry out Preliminary Feasibility Studies (PFS) to demonstrate whether a cavern scheme could be successfully implemented. The PFS were undertaken to demonstrate in broad technical and financial terms the viability of cavern development for a range of facility types and in different geographical areas as well as the preliminary technical feasibility of the cavern scheme and to identify key issues such as cavern layout and engineering and facility-specific technological requirements that would need to be addressed in future detailed feasibility studies.

By identifying the potential benefits that the cavern schemes could offer to the local community, this should also promote the merits of systematic consideration of cavern development in a planned manner in pursuit of Hong Kong's sustainable development. A brief summary of the 3 PFS is provided below

### 7.1 Sha Tin Sewage Treatment Works

The existing facility occupies a large footprint prime location at the mouth of the Shing Mun River. The tentative cavern site and the schematic cavern layout are shown in Figure 3. Some reasons for selection and key issues for the proposed facilities in this area are summarized below:

- Large footprint site area (28 ha) in prime waterfront location can be released for other

beneficial uses.

- Known similar large-scale overseas underground sewage treatment plants.
- Improved visual and environmental impacts of the area.
- Land value improved in the area.
- Future expansion can be provided for underground in a planned layout.

### 7.2 Mount Davis & Kennedy Town Fresh Water Service Reservoirs

The fresh water service reservoirs were selected at Mount Davis and Kennedy Town to illustrate the suitability of transferring the existing facilities below ground. The tentative cavern site and the schematic cross section of the two reservoirs are shown in Figure 4. Some considerations for selection and key issues for the proposed facilities are summarized below:

- Existing cavern facility and other nearby underground facilities successfully completed.
- Prime land (2 ha) can be released for other uses including potential private and public residential development.
- Traffic impact is an important aspect of any proposed construction and redevelopment taking into consideration the Pokfulam Moratorium.
- Future expansion can be provided for underground rather than extensive open-cut works.
- Improved security and integrity of the tanks.

### 7.3 Mui Wo Integrated Cavern Scheme

Various existing bad neighbour facilities are grouped along the southern waterfront of Mui Wo. Some considerations for selection and key issues for the proposed facilities are summarised below:

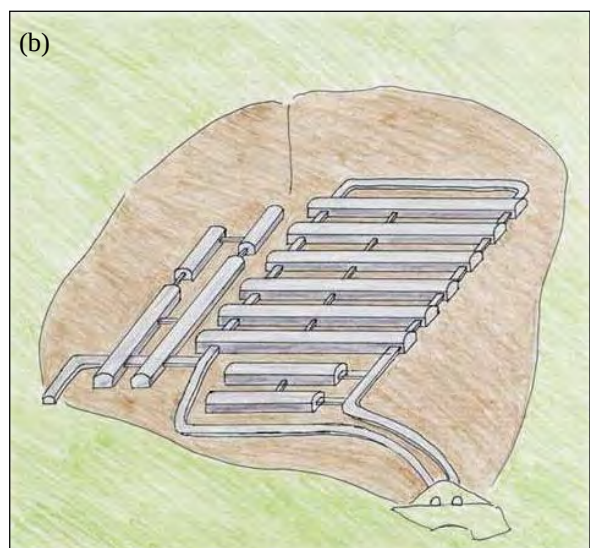
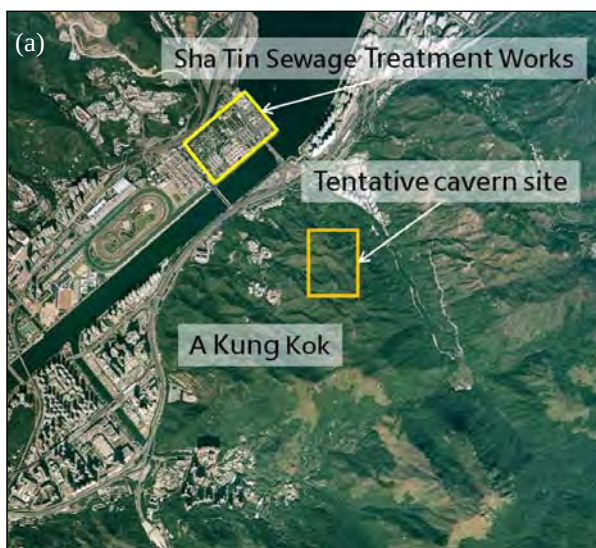


Fig. 3 Sha Tin Sewage Treatment Works: (a) Tentative Cavern Site, (b) Schematic Cavern Layout

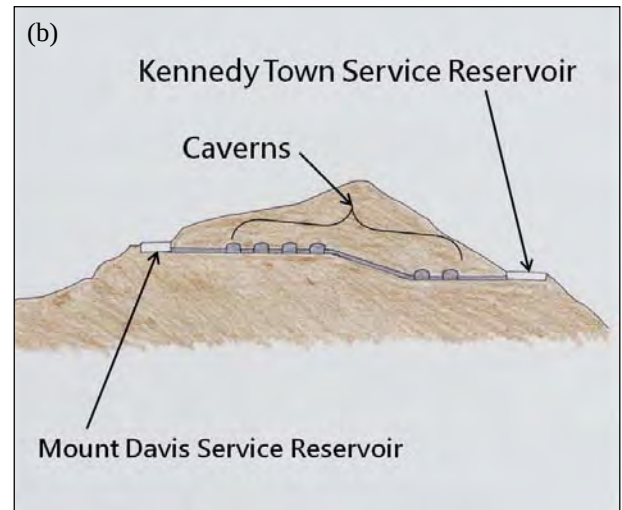
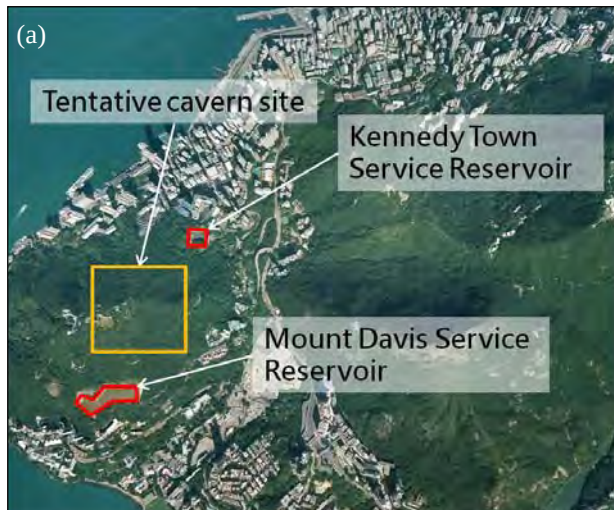


Fig. 4 Mount Davis and Kennedy Town Service Reservoirs: (a) Tentative Cavern Site, (b) Schematic Cavern Layout

- Waterfront land (2.5 ha) can be released for other uses.
- Complements Mui Wo Facelift Study by revitalizing 500 m of waterfront area.
- Existing refuse transfer station, sewage treatment works, bus depot and some other facilities can be relocated underground to provide greater amenity uses for the general public.
- Marine access needed for some of the facilities.
- Interface with some external initiatives such as mountain bike trail required.
- Improved environment for the area.

## 8. Selection of Strategic Cavern Areas

The study also identified five strategic cavern areas which were defined as being greater than 20 hectares in area and having the ability to accommodate multiple cavern sites. Relative ease of access with regard to the surrounding infrastructure network, and the ability to realise some public benefit whilst tying in with Hong Kong strategic planning initiatives, were the predominant objectives of the strategic cavern areas.

The strategic cavern areas were also required to be located in different regions of Hong Kong, namely Hong Kong Island, Kowloon and the New Territories. A preliminary screening of the areas suitable for cavern construction led initially to 12 areas that could be considered, from which, five strategic areas were selected (Fig. 5). The preferred area to be proposed for Hong Kong Island is Mount Davis while the preferred area for Kowloon is the Lion Rock area. In the New Territories the three areas of interest are Tuen Mun (Lam Tei), Sha Tin (Shek Mun) and Lantau (Siu Ho Wan). Given that over 60% of Hong Kong is considered to have high to medium suitability for cavern development, it is apparent that there are other areas suitable for consideration. A mechanism needs to be established such that these strategic cavern areas will not be compromised by future development

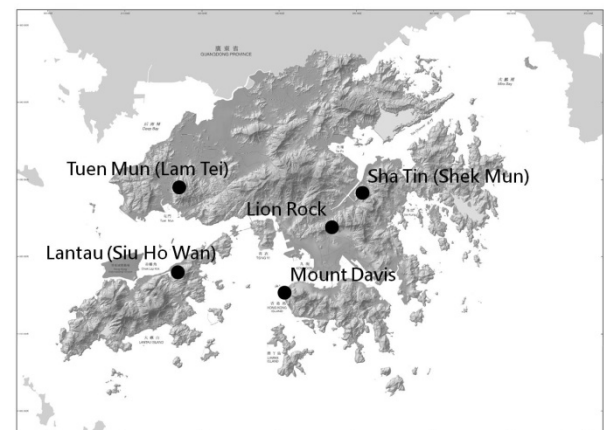


Fig. 5 Strategic Cavern Areas

projects and also not prevent or limit and future surface development [LING, 2011].

## 9. Recommendations of the Study

The strategies employed by other countries to develop underground space have been benchmarked against Hong Kong's practice. It is recommended that:

- Policy steer be provided through the issue of technical circulars to provide administrative requirements for government departments to consider the cavern option in the early project planning stage.
- The Hong Kong SAR Government formulates a strategy to systematically relocate existing government facilities underground. This will facilitate development of a comprehensive programme to gradually release land occupied by existing government facilities, for inclusion in a land reserve for other beneficial uses.
- Signature cavern schemes be implemented that would promote the use of underground space.

- A policy framework be developed to encourage private sector involvement in development of underground space. To encourage private sector involvement, incentives would be required to make the developments attractive, such as preferential land premiums, enhanced development potential or tax incentives. Another approach would be to explore more Public Private Partnerships. Suitable mechanisms should be evaluated and established to encourage involvement of the private sector in this initiative.

## 10. Conclusions

The hilly terrain and underlying geology of Hong Kong offer a great opportunity for planned development of underground space with the enhanced use of rock caverns. The ‘first generation’ rock caverns in Hong Kong were those related to enlargement of tunnel networks for water supply or railways. The ‘second generation’ rock caverns were purpose-built to house environmentally unattractive facilities, but only where over-riding circumstances had made the preferred above-ground option infeasible.

A strategic study has been commissioned that aims to enhance efficient use of land resources through the planned development of underground space and the enhanced use of rock caverns. Its outcome could help provide creative solutions in easing the pressure on land shortage in Hong Kong, particularly in the urban area, by releasing prime surface sites currently occupied by, or reserved for public facilities, for more compatible and beneficial uses.

It is hoped that the study will lead to the development of ‘third generation’ rock caverns that are formed as part of Hong Kong’s future planning and development strategy. Creating signature schemes that are world beaters in terms of scale and efficiency of the facility would promote the use of underground space and raise Hong Kong’s profile as Asia’s World City.

## Acknowledgements

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# TOWARDS AN UNDERGROUND DEVELOPMENT STRATEGY FOR HONG KONG

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## Abstract

*Hong Kong has accumulated a lot of experience and does have many successful cases in underground development. However, the use of underground space in Hong Kong in totality cannot be regarded as satisfactory, particularly in view of her geographical limitation on the availability of developable land. This paper briefly reviews Hong Kong's planning and development control mechanism and opines that this has provided little incentive and proactive guidance for the planning and development of underground space. It draws reference to the Helsinki Underground Master Plan and a development project in Guangzhou, and advocates that Hong Kong should formulate an underground development strategy to promote the systematic planning and development of underground space as a response to the land supply issue and the public aspiration for quality urban environment.*

## Keywords

*Underground Space, Cavern Development, Development Strategy*

## 1. Introduction

Hong Kong's natural hilly terrain is a challenge to urban development. This topographical setting has presented us with both opportunities and constraints in the development and use of underground space. The extensive areas underlain by massive hard igneous rocks located at the countryside and urban fringe, offer a great opportunity for underground development in the form of rock caverns. However, as large and extensive flat land is essentially not available, urban development of Hong Kong mainly concentrates in land formed by reclamation. The soft filling material of the reclaimed land has made any massive underground development within the high density and high-rise urban area costly in financing and challenging in engineering.

The Metroplan Study completed in the 1990s, which was a strategic plan for the development of the Main Urban Area of Hong Kong around the Victoria Harbour, advocated pursuing the opportunity of underground development as a response to the shortage of land in Hong Kong. To take forward this strategic planning initiative, the Government carried out the Study of the Potential Use of Underground Space (SPUN) [ARUP, 1990]. Chan & Ng [2006]

summarised geotechnical studies and institutional arrangement on cavern development in Hong Kong up to 2006. As the latest effort to promote the use of rock caverns in pursuit of sustainable development, the "Study on the Enhanced Use of Underground Space in Hong Kong" (the Enhanced Study) was completed in March 2011 [ARUP, 2011] [ROBERTS et al, 2011]. Both studies concluded that Hong Kong is suitable for the development of rock caverns from the geological perspective. The latter study further demonstrated that a number of existing public facilities can be relocated to caverns and thus releasing the surface land for other beneficial uses. It has also identified broad areas suitable for cavern development, see also CHAN [2011].

Apart from caverns, there is also a widespread use of basements below buildings/structures for parking, shopping and commercial purposes, not to mention the numerous underground spaces created for underpasses, tunnels for roads, rail and utility/services all over Hong Kong. The purpose of this paper is to explore a way forward for the formulation of a comprehensive strategy for more effective use of underground space as a source of land supply, which should also become part and parcel of the territorial development strategy of Hong Kong.

## 2. Overview of Underground Development in Hong Kong

Hong Kong is a vertical city, the skyline of which is dominated by high-rise buildings. However, it is interesting to note that, about 5 million m<sup>2</sup> of building floor area, which is about 2% of the total building volume in Hong Kong is indeed constructed underground in the form of basements. Notwithstanding, I wish to make a distinction that the widespread basement developments are in fact part of the above-ground development to optimize the use of space of individual building sites, rather than strategic development in the underground space.

Construction of the Mass Transit Railway (MTR) system started in the 1970's unveiled a new era in the development of underground space in Hong Kong. Not only most of the MTR tracks and stations were built underground, the extension of the system has provided the opportunities of developing underground pedestrian subways connecting the MTR stations with the nearby commercial development, as well as connecting stations of different MTR lines.

There are two such subways worth mentioning in view of their scale and essential function. The pedestrian subways between the Central Station and Hong Kong Station connecting the Tsuen Wan Line/Island Line with the Tung Chung Line/Airport Railway were completed in the 1990's as part of the Airport Core Programme. The subway has a length of about 220m with a daily pedestrian flow of about 120,000. Another subway system connecting the Tsim Sha Tsui Station and the Tsim Sha Tsui East Station so that the Tsuen Wan Line can be linked with the East Railway and West Railway was completed in 2000's. This system consists of two subways of length about 370m and 240m, which handles a total daily pedestrian flow of about 170,000. These subway links equipped with travelators and dotted with shopping and eating outlets and even corners for life performance and exhibition are indeed vibrant underground streets which are part of our hustle-bustle city life.

In the 1990s, purpose-built underground spaces for some bad neighbour uses were constructed. The Stanley Sewage Treatment Plant housed in rock cavern was completed in 1994, which serves a population of about 27,000 [IP & SAM, 2011]. The Island West Refuse Transfer Station completed in 1997 is located underneath Mount Davis at Kennedy Town. With a design throughput of 1,000 tonnes of refuse per day, it is a critical part of the municipal waste handling strategy [VENHOEK et al, 2011]. The transfer station and its vehicular entrance are close to residential development. With an average daily loads of about 150 refuse collection vehicles, little public complaint has been received, which indicates that the neighbourhood is quite receptive to the operation of the station.

The Kau Shat Wan Explosives Depot on Lantau Island was completed in 1997 [FRANKS et al, 2011]. It replaced the original facilities on Stonecutter Island, which was close to large-scale urban development planned on the West Kowloon Reclamation. The cavern option for these facilities was chosen for reasons of lacking suitable surface sites, substantial environmental and security benefits compared with surface option, or avoiding reclamation works in sensitive areas.

In 2009, the University of Hong Kong relocated two public salt water service reservoirs to the nearby rock caverns to free up surface space for its Centennial Campus development [CHAN et al, 2009] [YEUNG et al, 2011]. The case demonstrates the possibility for replacing existing surface facilities underground as a means to release land for development. The cavern option also preserves the heritage buildings and vegetation in-situ. Within the same district, indeed the same body of igneous rock mass, we can also find a more recent case of cavern development constructed in 2010 for the MTR West Island Line (WIL) Explosives Magazine which is a temporary facility to facilitate the

construction of the WIL. Ironically, the land allocation conditions of this site require that the cavern so created should be back-filled after its use as the temporary magazine is finished with the completion of the WIL.

Hong Kong has accumulated a lot of experience and does have many successful cases in underground development. However, the use of underground space in Hong Kong in totality cannot be regarded as satisfactory, particularly under our geographical limitation on the availability of developable land. There is a scope for more strategic use of our underground space.

### 3. Lacking a Proactive Policy for Underground Development

The development control mechanism of Hong Kong provides little incentive for underground space development. The Buildings Ordinance has provisions to exempt the floor areas for functional accommodation of building services facilities, such as plant rooms, ancillary car parks, etc. from plot ratio calculation. The exemption is applicable to such floor areas regardless they are located above ground or underground. Treated as part and parcel of the whole building structure, basements are thus fully accountable for plot ratio calculation. The same principle for counting the floor areas for control of the total permissible building bulk regardless of their location within the building structure applies also to the land lease and statutory plans (i.e. outline zoning plans and development permission plans) prepared under the Town Planning Ordinance.

At the strategic planning level, the potential of underground development is not specifically addressed in the latest territorial development strategy documented in the Hong Kong 2030 Planning Vision and Strategy published in 2007 [PLAND, 2007]. Nevertheless, the strategy does indicate the need to identify alternatives to surface development to minimize the use of land to accommodate the future growth of the territory.

In the statutory planning context, the planning control system remains "neutral" with underground development. The planning controls ascribed on the surface land on statutory plans are also applicable to the space below the ground. To develop an underground space for a user different from the land use zoning prescribed for the surface land, a specific zoning restriction may need to be devised through amendments to the statutory plan or, where appropriate, the planning permission mechanism.

Underground space development in the form of caverns, as illustrated in the above paragraphs, has largely been developed on a project-by-project basis. Many are confined to government/public facilities as

the cavern option can usually kill two birds with one stone: to overcome the overriding circumstances that suitable surface sites were not available and to “hide” the essentially bad neighbour uses such as sewage treatment works and refuse transfer stations. Construction and extension of the MTR system has been and continues to be the major driving force in the development of underground space, both for the railway operator to improve integration of the whole rail system and for private developers to enhance the value of their properties by providing direct pedestrian links to the MTR stations. These projects have been driven by pragmatic considerations and thus are ad-hoc in nature.

Following the findings and recommendations of SPUN, administrative guidelines on the planning for cavern use are provided in Chapter 12 of the Hong Kong Planning Standards and Guidelines (HKPSG) 2008 [PLAND, 2008], which include a list of land uses with potential to be located in caverns, as shown in Table 1.

| Land Use Category                            | Potential Land Use                                                                                                                                                                                                                                         |
|----------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 1. Commercial Uses                           | Retail                                                                                                                                                                                                                                                     |
| 2. Industrial Uses                           | Industry<br>Storage/Warehousing<br>Oil bulk storage<br>LPG bulk storage                                                                                                                                                                                    |
| 3. Government / Institution / Community Uses | Slaughterhouse<br>Civic centre<br>Incinerator<br>Indoor games/Sports hall<br>Wholesale market<br>Refuse transfer facility<br>Sewage / Water treatment plant<br>Service reservoir<br>Transport connections & networks<br>Columbarium / Mausoleum / Mortuary |
| 4. Public Utilities                          | Power station                                                                                                                                                                                                                                              |

Table 1 Land Uses with Potential for Rock Cavern Development (Source: Chapter 12, HKSPG, 2008)

However, as general planning guidelines, the HKPSG have not indicated priority in the planning process to explore the potential of the cavern option before resorting to the option of taking up surface land. Whether a particular facility should be located within cavern is largely left to individual facility providers.

Nevertheless, the present financial formula for considering the cost of a specific government facility has assumed that the land is provided at no cost to the facility provider. The formula has also not taken into account the opportunity cost of any alternative use of that specific site or the cost of any adverse impact on the potential use of the neighbour sites. The usually higher capital cost of cavern formation and recurrent cost for operation in caverns in terms of lighting and

ventilation render the cavern option easily an inferior choice to the facility provider in terms of financial performance. There is therefore no surprise that many land uses indicated as potentially suitable for caverns in Table 1 have never been developed in caverns.

#### 4. Inspiring Reference for Hong Kong

Hong Kong may do better in promoting the use of underground space. In my recent duty visits to Helsinki and Guangzhou, I have looked into the experience of these two cities and find some of their cases quite inspiring to us.

##### 4.1 The Helsinki Underground Master Plan

The City of Helsinki sits on a huge mass of stable igneous rock which is geotechnically suitable for creation of underground space. Underground development in Helsinki is also consistent with Finland’s civic defence policy which has obliged the authority to provide public shelter in rock caverns. Underground space also provides weather-proof activity venue under the severe winter condition of the country.

To promote and coordinate the planning and development of underground space, and to preserve the rock reserve for sustainable utilization, the Helsinki Underground Master Plan came into effect after its adoption by the City Planning Commission in December 2008 [HELSINKI CITY, 2009]. This is a statutory plan which shows the location and space allocation for the existing and planned underground space for utility tunnels, utility facilities such as treatment works and district heating plants, traffic tunnels, storage facilities, sports facilities, maintenance depots, parking and the interconnections of these facilities. The legal effect of the Underground Master Plan safeguards the existing and planned underground developments by obliging the planning and construction of any surface development to have regard to the provisions of the plan.

The City Centre Maintenance Tunnel, which was developed by the city government as an infrastructure project, provides a good example showing how the plan can work. With portals outside the city centre, this tunnel runs through the downtown Helsinki. Buildings located in the downtown are encouraged to develop underground links and loading/unloading areas to connect with the tunnel so that vehicles providing maintenance and delivery services can access the buildings through the tunnel without using the surface roads. The downtown shopping malls and shopping streets are also connected to the underground car parks through a sophisticated system of pedestrian links. The tunnel functions as an underground thoroughfare for service vehicles and access to the downtown public car parks, thus very effectively reduce the volume of

surface traffic of the downtown area, particularly by taking the heavy vehicles away from the surface roads.

In Helsinki, apart from the above-mentioned tunnel, there are many other successful cases of underground development. The underground sports and recreational facilities in Helsinki I visited were well integrated with the neighbourhoods they were planned to serve. They preserved the landscape and vegetation which would otherwise be destroyed in the case of above-ground development. The huge underground space created for retail and parking are conveniently linked with the metro system, connected to the pedestrian precincts and buildings where jobs are accommodated. In general, the Helsinki Underground Master Plan has been successful in making the underground space an integral part of the city structure.

#### **4.2 Zhujiang New Town in Guangzhou**

Guangzhou, the capital city of Guangdong Province, sits on the alluvial deposit of the Pearl River Delta, presenting a totally different geological setting as compared with Helsinki.

In 2000, Guangzhou published its master plan for strategic development which included, among others, the metro system and a new north-south development axis. These two strategic proposals have set the scene for Guangzhou's extensive underground development. Its ambitious metro system, according to recent news report, would have a total length of 666 km and 330 stations by year 2020. Many metro stations have been planned and developed as the catalyses as well as the focal points of pedestrian links and shopping streets constructed underground connecting buildings nearby. Nevertheless, in my opinion, the most remarkable example of underground development is the centre part of Zhujiang New Town (珠江新城, literally means Pearl River New Town), which is located at the prominent position of the new north-south development axis of Guangzhou.

The centre part of Zhujiang New Town is known as The Flower City Plaza (花城廣場) which consists of 4 levels. The surface level is a vase-shaped public open space of an area of 0.56 km<sup>2</sup> running north to south with a length of about 1.5 km. Four prestigious public buildings (museum, public library, opera house, and palace for the youths) as well as hotels and offices are connected seamlessly to the open space. The first and second levels underneath the Plaza provide huge underground space for accommodating sunken gardens connected to shopping malls, bus terminus and car parks. To avoid dissection of the public open space by roads, these two levels also accommodates the underpasses of two trunk roads running in the east-west direction. The lowest underground level accommodates the APM system, which has a total of

nine stations, five of which run underneath the Plaza and intersect with metro stations of Line 3 and Line 5.

The Flower City Plaza is so far the most impressive underground development I have ever seen in Mainland China in terms of scale and design. The successful implementation of the project is the result of the interplay of three key elements in the planning and development process: (i) a clear and firm planning objective of creating a iconic urban greenery on the new development axis as a strategic urban design feature and a high quality vehicle-free landscaped open space for the enjoyment of the public; (ii) a carefully formulated master plan translating the planning objective to a development proposal for implementation; and (iii) an implementation agent (in this case a development company owned by the city government) responsible for the construction of the public open space and all the underground facilities in accordance with the master plan. The agent is also responsible for disposal of individual land parcels with development conditions requiring provision of connections to the public open space as well as the underground facilities in accordance with the master plan.

#### **5. Towards an Underground Development Strategy for Hong Kong**

The experience of Helsinki and Guangzhou has brought to us a clear message that difference in geological setting is not a determining factor in pursuing underground development. Hong Kong has the igneous rock mass at the urban fringe area suitable for cavern development like Helsinki whilst our Main Urban Area largely sits on soft reclamation fills like the alluvial deposit of Guangzhou. We need an underground development strategy to capitalize more effectively the merits of underground development in our own context.

##### **5.1 A Strong Public Policy Steer to Promote Underground Development**

The experience of Hong Kong, Helsinki and Guangzhou converged at a point that the government is the main driving force in underground development. At present, the guidelines on cavern uses as a form of underground development provided in the HKPSG are not mandatory. They only confine to caverns but are generally silent on other forms of underground development, such as subways and underground shopping streets which would enhance connectivity within the compact and complicated urban fabric of the densely built-up areas.

To create a decision-making environment for taking a more proactive attitude in considering possible underground options for various public facilities, we require a strong high level policy steer. The policy

should give a bold statement in favour of the underground approach and oblige the public facility providers not to take underground option as the last resort, but should pursue proactively the possibility and potential benefits of underground option at the early planning stage. The decision making mechanism must include the consideration of opportunity costs of the surface site and the potential adverse impact on the value of nearby sites of the surface option into the financial formula in order to reflect the potential total costs of the surface site option.

### ***5.2 A Planning Process to Capitalise the Merits of Underground Development***

The SPUN, the Enhanced Study and many cases of Hong Kong and overseas have already demonstrated the merits of underground development. These generally include release of surface land for alternative beneficial use, preserve the landscape and vegetation by avoiding the "open-cut" option, more effective mitigation of the adverse impact of unwelcome but essential public facilities, and connectivity enhancement. To fully capitalize the merits in each unique circumstance, underground development should not simply be taken as an individual isolated engineering project but must be perceived as a key land use and urban design element within an integrated planning process. Any planning and engineering feasibility study for new development areas should proactively explore the possibility of underground development option at the initial planning stage.

### ***5.3 Planning Guidelines for Underground Development***

At present, HKPSG has only provided planning guidelines for cavern development which covers only a form of underground development. Consideration should be given to expanding the guidelines to cover underground development in general. For example, there are many public facilities which, though small in size (such as a salt water pump house), may occupy a prominent location along the harbourfront causing negative impact on waterfront accessibility and visual quality. There is potential to rebuild this sort of facilities underground to mitigate its adverse impact without compromising its essential function for municipal services.

The list of land uses that have potential for cavern development in HKPSG (Table 1) should also be expanded to include the additional suitable land uses proposed in the Enhanced Study. Some of them (such as those considered as bad neighbour uses) are particularly appropriate to be accommodated in caverns on space requirement and environmental grounds. For others, the benefits of cavern development are less obvious. For example, locating a sports centre within a cavern at the urban fringe area

may not be welcomed by the users due to its relatively inconvenient location. Moreover, given the high density development pattern of urban Hong Kong, a low-rise sports centre may also serve as a visual relief for the neighbourhood. In the planning of cavern development, there is a need to accord priorities to different land uses so that the cavern spaces can be utilized in an optimal manner.

In considering which types of land uses should have priority for cavern and underground development, criteria should be set up to facilitate an objective assessment. Some of the factors may include land intake, environmental and urban design benefits, safety concern, locational requirement, public acceptance, precedent case, etc. A preference list of land uses will need to be prepared as one of the components of the underground development strategy.

### ***5.4 Identification of Strategic Cavern Development Areas***

The cavern suitability map prepared under the Enhanced Study gives a broad picture on the distribution of suitable rock mass for cavern use. However, not all areas identified are deemed suitable for development when other relevant considerations such as traffic and environment factors are taken into account. There is a need to identify specific strategic cavern development areas where future major cavern developments should be directed. The selection of these strategic cavern development areas should be related to uses that the caverns are planned to house, such as bus depots, oil storage, warehousing. The creation of strategic cavern space should be considered as a strategic land formation operation to meet the long term development need of Hong Kong in a sustainable manner. The costs and benefits of creating strategic cavern space should include the tangible and intangible benefits caused by the planned after-use of the land that can be released from relocating the original uses into caverns.

### ***5.5 Formulation of Underground Development Master Plan for Selected Districts***

The Helsinki Underground Master Plan presents us a good example that a holistic approach is necessary to integrate underground development with the overall land use planning of the city. Given the complex urban fabric of the built-up area of Hong Kong, we may not be able to produce one single underground master plan for the whole territory. Instead, we should consider prepare underground development master plans for selected geographical districts, such as Hong Kong Island, Kowloon West/Tsuen Wan/Kwai Tsing, Kowloon West/Tseung Kwan O, and so on, which should not be confined by the present administrative boundaries.

The underground development strategy plans for the selected districts should show the location of the identified cavern development areas, their planned uses, the facilities that would be relocated into caverns, the location of the surface sites that would be released and the planned use of the released surface sites. To function as an inventory, these plans should also show other known and planned underground facilities and development projects other than caverns. These plans will become effective platforms for the government to communicate with the stakeholders on the underground development strategy for the selected districts.

### ***5.6 Formulation of a Relocation Programme for Selected Facilities***

The relocation of selected facilities into underground space will involve a complicated and lengthy programme. During the process of the Enhanced Study, Planning Department suggested that the relocation of the Sha Tin Sewage Treatment Plant, which now occupies a large prominent waterfront site, should be taken as a major case study to demonstrate to the community the benefits of cavern development and also the difficult issues involved. Whilst the case study has demonstrated the feasibility, it has also indicated the lengthy implementation process which includes the study and design for the cavern development as well as the new plant, statutory process to enable the relocation, excavation for creation of the cavern space, construction of the new plant within cavern, construction of off-site facilities such as access road, revamp of the sewerage system, etc., decommissioning of the original plant and de-contamination of the original site. This will take 17 years before the original surface site can be handed over for its alternative planned use.

This demonstrates the need to take a long-term strategic planning perspective to consider which facilities should be relocated underground, to where they should be relocated, and what possible alternative uses should be planned for the released sites. Not all the existing facilities will require immediate relocation to underground as each facility will have to be assessed based on certain criteria, such as designed life of the facility, any expansion programme, land use compatibility with the surroundings, beneficial after-use of the released sites, etc. As such, a comprehensive and systematic programme to select those facilities for relocation to caverns will be needed. The selected facilities should then be packaged into a comprehensive relocation programme which should form part of the strategic land production programme.

### ***5.7 Incorporation of Underground Development Strategy in Territory Development Strategy (TDS)***

The TDS is the highest tier of plans in the hierarchy of planning system in Hong Kong. It covers the whole

territory and provides a long term strategic planning framework for sustainable development of Hong Kong. The TDS is a living document which has been reviewed from time to time. The Hong Kong 2030 Planning Vision and Strategy published in 2007 is the latest version of the TDS. It however has not specifically assessed the role of underground development as an element in the long term sustainable development of Hong Kong. This paper advocates that the underground development strategy as deliberated above should be formulated by the government for incorporation into the TDS.

There are several additional strategic planning issues which should not be missed. First, as a technical backup to the underground development strategy, there is a need to produce and maintain a comprehensive inventory of the existing and planned underground facilities. The absence of such inventory at the moment reflects the unsatisfactory situation that the underground facilities have so far been treated as isolated engineering projects but not part of the overall city structure.

Second, apart from providing land for development, strategic cavern development will give the scale of economy in promoting cavern quarrying as a major source of local supply of aggregates when we have plans to close all the local open quarry sites in the near future.

Third, the feasibility of underground common utility trench should be further examined and solutions to overcome the issues that have prohibited Hong Kong from adopting this option should be suggested. It is quite disappointing that underground common utility trench has found to be an infeasible option even in new development areas which are now under planning. It is suggested that we need a change to such an unsatisfactory situation.

### ***5.8 Raising Public Awareness on Underground Development***

Since the 1990's, there have been only a few purpose-built cavern developments in Hong Kong. The general public, despite being frequent users of many underground facilities such as the MTR subways, do not appreciate the difficulties and hurdles in pursuing such options. There is a need to enhance public understanding of the benefits and issues associated with underground development and solicit their support in the context of sustainable development for Hong Kong. On this particular point, the government will soon embark a comprehensive public engagement exercise for reclamation outside the Victoria Harbour and cavern development as one of the strategic planning solutions to meet our long-term development need. Raising public awareness on the pros and cons of underground development would facilitate the

formulation of the underground development strategy together with the community.

## 6. Conclusion

Given its topographical and geological setting and the shortage of developable land, Hong Kong has the potential as well as the need for a wider use of underground space. We should have a strategic vision to enhance a more effective use of underground space and make underground development as an integral part of the city structure for the benefit of sustainable development of Hong Kong. A multi-disciplinary approach guided by a clear public policy to promote underground development and a set of administrative guidelines to deal with the technical and institutional issues is necessary to make such vision come true.

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# THE WAY TO PLAN A VIABLE “DEEP CITY”: FROM ECONOMIC AND INSTITUTIONAL ASPECTS

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## Abstract

*While more and more urban constructions are forced to go underground in order to release land surface for development, metropolises world-wide are facing challenges to plan for rational use of underground space and other valuable resources. This article tries to analyse important economic issues and institutional concerns for the development of Urban Underground Space (UUS), based on a comprehensive case study in a Chinese metropolis. In addition, a Swiss concept on sustainable management of Urban Underground Resources (UUR) is presented, in the framework of “Deep City” international research program. Our new urban territory of subsurface deserves an universal attention on spatial planning and resources management, in coordination with the urban regeneration.*

## Keywords

*Global Indicator, Demand-supply, Cost-benefit Analysis, Institutional Feasibility, Urban Planning, Resources Management*

## 1. Introduction

Facing the challenges of population growth, energy crisis, land pressure and environmental deterioration, modern urbanization is calling for governance innovation to facilitate flexible spatial transformations and to promote creative city redevelopment [UN-HABITAT, 2009]. Developing Urban subsurface as a sustainable option for renewing congested urban centers and for updating public infrastructures, should be economically viable and politically acceptable [MAIRE, 2011].

Optimization of Urban Underground Space (UUS) use has to take into accounts social-economic demand and possible supply of geo-space resources.

Firstly, from the demand side: since underground construction technologies keep advancing, construction cost as an important part of initial investment is decreasing slightly. However, for urban planning practices, the pattern and quantity of using underground space is not perfectly aligned with engineering innovation. Other dynamic determinants have to be integrated into the comprehensive economic assessment, such as the needs and preferences of the

users on spatial and environmental concerns, as well as life-cycle energy consumption concerns.

Secondly, from the supply side: since underground space is an integral part of geo-environment, its development is constrained by natural conditions and exiting built environment. Geological survey provides useful information for land administrators to evaluate potential quality and quantity of underground space to be urbanized.

In this article, a global indicator of UUS potential is put forward, taking into accounts the natural conditions and exploitation potential of the relevant area. A case study based on the pilot city of Suzhou in China is demonstrated for comprehensive evaluation. The aim is to give planners and private developers a concrete idea of how to materialize the added-value of UUS.

Institutionalization of UUS management is integrated in a multi-actor framework. Understanding the institutional dynamics of this framework is useful to guide political decision-making on urban underground projects. An organizational network of public stakeholders will be illustrated in the article.

Four main resources (Fig.1) are in urban subsurface: underground space, groundwater, excavated material and geothermal energy. A holistic approach for underground urbanization should be integrated to land use planning and urban planning. Synergetic development can be promoted, such as integrating geothermal system into underground space structures, in order to benefit a joint development of renewable energy use and UUS; in reverse, conflicting situation have to be avoided, such as groundwater contamination by tunnel construction across the aquifers, in the case where groundwater is a priority protection resource for drinking use by legal regulations.

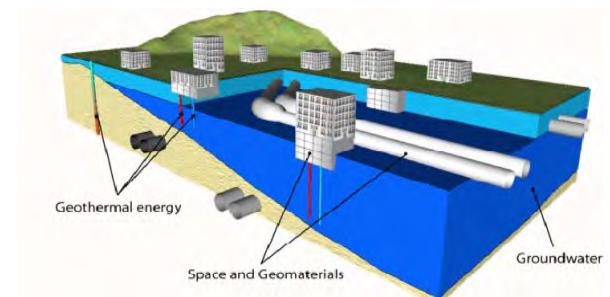


Fig.1 UUR System of "Deep City" Concept

This idea of combined-use exploitation, named “Deep City” concept [PARRIAUX et al, 2010] (Fig.1), has been put forward for this purpose, being reconsidered as part of Swiss federal law of territorial planning (in revision). It is a management methodology which is applicable to other megacities in the world, to guide their governance towards a sustainable use of UUR.

The following discussion is organized in 3 sections: the second section about economic analysis and policy research for UUS planning, the third section concerns “Deep City” concept for UUR management, and the last section with conclusions.

## **2. Planning the Development of Urban Underground Space (UUS), with a Pragmatic Approach**

### **2.1 Economic Estimation Concept Considering Investment Viability and Construction Feasibility**

#### (1) Urban demand investigation

In big cities, land resource is scarce due to saturated urbanization, high land prices in central city are caused by increasing demand for real estate development for commercial profits. In addition, population growth and urban immigration are the driving forces for excessive urban construction, including housing, transport system, public facilities and commercial spaces. Land use types, as defined in urban land use regulations, could be different above ground and below ground, because of different surface-subsurface functional requirements, as well as legal property rights. With citizens’ desire for a more convenient mobility mode, locations next to rail transit system are valuable for property development by developers.

Therefore, urban indicators such as benchmark land price, population density, land use type, and planned rail transit location, are important determinants to forecast the demand for UUS. As private sector becomes a main stakeholder to finance urban infrastructures, it will be useful to integrate their investment priorities into UUS planning practices.

#### (2) Natural supply investigation

Different from general building work and infrastructure construction, underground construction necessities various excavation techniques and supporting structures. Construction cost and project risk insurance vary substantially according to the levels of difficulty of civil engineering works. Constructions in a stable and easily excavated subsurface (such as rock foundation) will cost less than in an environment (such as alluvial soil foundation) facing a lot of geological and hydrogeological challenges. Since natural disasters are becoming more and more frequent during recent years, our concerns about urban construction safety should be on the priority agenda of public policy-making.

Evaluation of the degree of difficulty in construction needs to take into accounts various geotechnical parameters and geological data. Therefore, the more comprehensive the information is, the more precise the evaluation will be.

#### (3) Spatial restriction investigation

Beside natural conditions, a lot of UUS development projects were restricted by the exiting built environment in shallow underground layers. Users occupying most of the underground land are public networks such as utility pipelines (water, gas, waste, electricity, cable, etc). With the coming of subway projects in more and more cities of developing countries, our shallow subsurface is almost saturated [STERLING, 2007], just like the surface of many central city. In order to avoid spatial conflicts, available development volume should be clearly identified and quantified, with a view to preventing misperception of UUS resource and enabling optimized utilization.

However, to register systematically all underground infrastructure units inside the whole UUS requires a lot of administration efforts and time, for the reason that they belong to different private and public owners, and the related administrative bodies usually do not have a shared information platform. As development congestion is gradually extending to shallow subsurface, rethinking the administration mode of UUS becomes urgent for local authorities. Researchers have pointed out that an underground property market will probably appear in the near future [BARLES, 1995; PASQUAL, 2005], which also seems to become a trend in some Chinese megacities. If it is intended to leave it to the market to resolve shallow ground congestion issue, local government should devise management measures to ensure quality development of UUS without delay.

Other spatial restrictions come from the surface, such as special land use type and special building protection zones. While new development area can be intensively exploited for UUS, locations like ecological preservation area and historic heritage zones restrict strongly UUS development. All these spatial factors above also represent a high degree of difficulty for excavation.

#### (4) Global indicator of demand-supply

To conclude, before initiating development planning of UUS, several investigations and evaluations should be performed in advance. In the first place, urban demand investigation helps to evaluate the potential of creating commercial values and social benefits; then, natural space resource supply investigation helps to evaluate difficulty levels in engineering works, which implies a variation in terms of construction costs; at last, to ensure a harmonious development with conventional urban planning, spatial restriction investigation helps to

identify exploitable UUS resource and its quantity. The process provides input to subsurface land value appraisal in the future land market.

Indicator of demand-supply of UUS exploitation will be translated into monetary terms, being a cost-benefit indicator (Fig. 2).



Fig. 2 Framework of Demand-supply analysis

## 2.2 Case Study for Integral Evaluation<sup>1</sup>

The following demonstrative evaluation is based on a pilot study of Suzhou city for methodological application, under the “Deep City” international research program<sup>2</sup>.

### (1) Pilot city introduction

The city of Suzhou is a typical fast developing municipality in the east coast of China (Fig.3). Having a traditional urban center with a history of 2'500years, its new development zones (such as Sino-Singapore Industrial park) have been attracting an increasing amount of Foreign Direct Investment for its industrial and commercial business. Under land shortage pressure, the city has planned to limit industrial and commercial land supply in favor of housing land development<sup>3</sup>. Facing the constraint of commercial land provision, commercial developers are searching for alternatives, such as surface-subsurface building integration, and combined development with metro station complex.

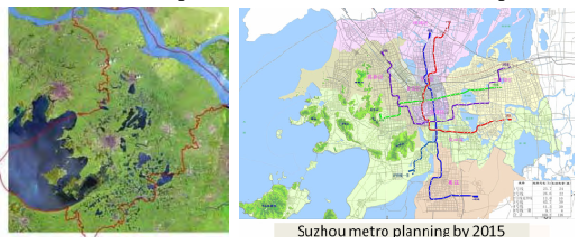


Fig. 3 Situation Maps of Suzhou City (downstream of Yangtze River, next to Shanghai) and Metro Lines

<sup>1</sup> Figures and tables are from internal report of the project in Nanjing University: “Evaluation for urban underground space exploitation potential in Suzhou city”, supported by “Suzhou urban geological survey program 2008-2011” of Geological Institute of Jiangsu Province.

<sup>2</sup> This research program is in collaboration with Institute of Underground Space and Geo-environment in Nanjing University (IUSG-NJU).

<sup>3</sup> From internal governmental reports on benchmark land price appraisal of Suzhou city.

### (2) Urban demand evaluation

Socio-economic demand for UUS is evaluated through multi-criteria analysis by Analytic Hierarchy Process (AHP) with critical socio-economic factors (Table 1). The evaluation zone covers the central city, with an area of 279.5 km<sup>2</sup>.

| Factors                          | Weight <sup>4</sup> |
|----------------------------------|---------------------|
| Zone bit                         | 0.196               |
| Population density               | 0.109               |
| Traffic Location                 | 0.191               |
| Commercial benchmark land price  | 0.112               |
| Residential benchmark land price | 0.101               |
| Land usage type                  | 0.137               |
| Civil defense need               | 0.055               |
| Historic Conservation            | 0.099               |

Table 1 Index System for Socio-economic Demand Evaluation

By integrating relative analysis maps of each factor on GIS, results can be showed on 2D plans (Fig. 4). It could be observed that high-valued zones are those near the ancient city center and near metro system (red colored).

Suzhou central city: UUS commercial value

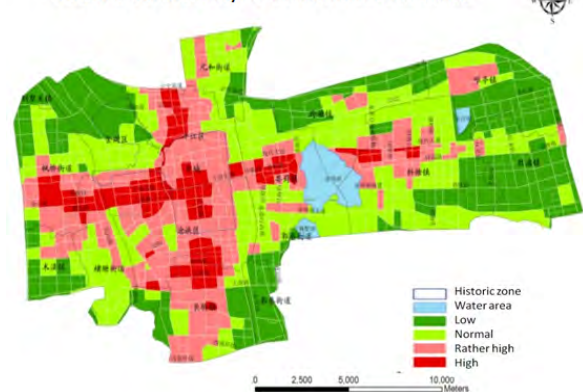


Fig. 4 Spatial Evaluation for Commercial Value

### (3) Natural supply evaluation

Difficulty levels for UUS construction are evaluated with the same method above, by selecting important geotechnical and environmental factors (Table 2). In order to forecast a long-term exploitability, each factor is given different weight on different underground development layers (Table 3).

For short-term UUS development scale, difficulty for civil engineering is evaluated on four levels (Fig.5). Brown-colored zones represent the highest level of engineering challenges. In the suburban area, it is due to thick soft soil layers, or risky geological conditions.

<sup>4</sup> The weights in Tables 1 & 3 are assigned through pairwise comparison, consulted from 10 experts from Jiangsu Provincial departments of geology, urban planning, and urban construction.

Investment on supporting structures for construction will account for a large share of total cost estimation. In the central city, the difficulty is due to over-congested underground pipelines or surface buildings conservation. Monetary compensation to owners of surrounding infrastructures or buildings will also be costly.

| Factors                               | Weight (0~15 m) | Weight (15~30 m) |
|---------------------------------------|-----------------|------------------|
| Fault activity                        | 0.0623          | 0.0679           |
| Site classification for construction  | 0.0267          | 0.0291           |
| Geomorphologic unit                   | 0.068           | 0.063            |
| Properties and thickness of soft soil | 0.0784          | 0.112            |
| Thickness of liquefaction soil layer  | 0.0336          |                  |
| Surface water                         | 0.0605          |                  |
| Thickness of semi-confined aquifer    | 0.0605          | 0.0648           |
| Karst area                            | 0.0968          | 0.1056           |
| Goaf area                             | 0.0968          | 0.1056           |
| Land subsidence                       | 0.0242          | 0.0264           |
| Ground fissure                        | 0.0242          | 0.0264           |
| Ecological sensitivity                | 0.078           | 0.072            |
| Distribution underground pipelines    | 0.092           |                  |
| Architectural protection              | 0.101           | 0.107            |
| Heritage conservation                 | 0.097           | 0.123            |

Table 2 Index System for UUS Difficulty Level Evaluation (example given for two priority shallow layers)

| Vertical Layer                | Use                        |
|-------------------------------|----------------------------|
| Shallow layer (0 - 15 m)      | Existing development layer |
| Sub-shallow layer (15 - 30 m) | Development for short-term |
| Sub-deep layer (30 - 50 m)    | Development for far future |
| Deep layer (50 - 100 m)       | Development for long-term  |

Table 3 Layered Planning Approach

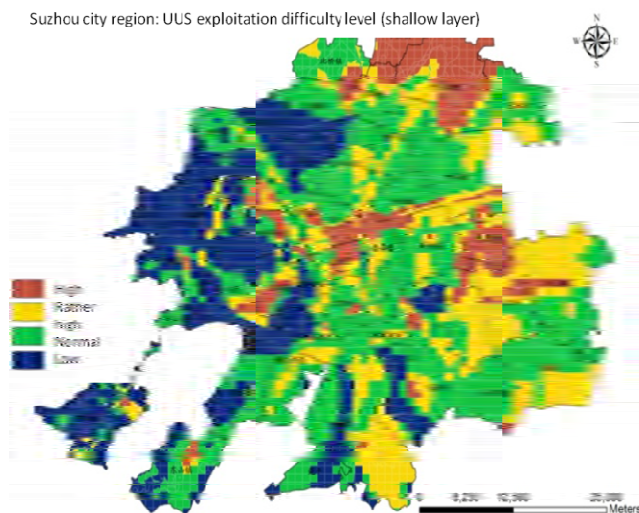


Fig.5 Spatial Evaluation for Difficulty (depth 15 – 30 m)

#### (4) Spatial restriction investigation

Exploitable UUS volume is calculated by a selective deduction procedure, taking into account the safety depth limits of existing infrastructures, as well as special protection units (ecological belts, urban heritages). Here examples are given for two shallow

layers (Fig. 6 & 7). They show locations suitable for excavation in the central urban area. (Green zones meaning “fully exploitable”, yellow zones meaning “partially exploitable”).

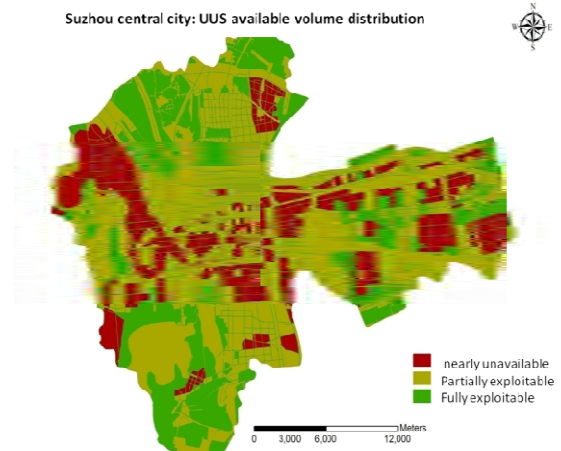


Fig. 6 Evaluation of Potential Volume (depth 0 – 15 m)

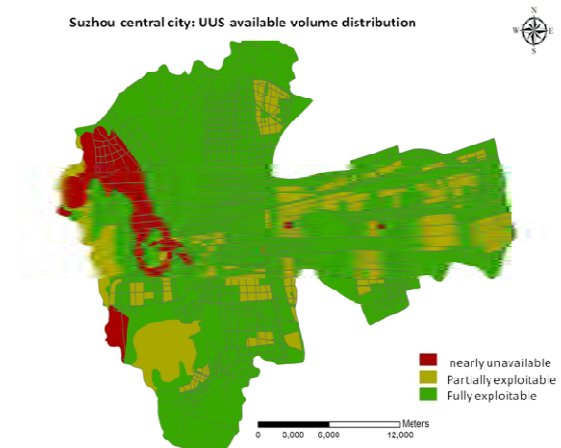


Fig. 7 Evaluation of Potential Volume (depth 15 – 30 m)

At the scale of city region, it is estimated that, for existing development layer (0-15m depth), exploitable UUS can reach  $5,533 \times 10^6 \text{ m}^3$  (61% of total UUS at that depth range); for short-term development layer (15-30 m), exploitable volume is  $8,166 \times 10^6 \text{ m}^3$  (90% of total UUS at that depth range); for the far future, exploitable volume between the depth of 30 and 100m will be about  $40,231 \times 10^6 \text{ m}^3$  (95% of the whole deep layers). This volume estimation could be expressed in available subsurface area ( $\text{m}^2$ ) to be urbanized, serving as a tool to quantify UUS floor area in UUS planning. For urban subsurface land appraisal, the pricing hierarchy should take into account the depth ranges among others. This potential volume estimation of UUS supply could be presented in 3D scale<sup>5</sup> for easy visualization.

<sup>5</sup> On-going research project is to visualize the 2D evaluation at 3D scale.

### (5) Integral potential evaluation

Urban zones with high demand for commercial development and good quality of natural resources supply (low engineering difficulty), are considered as high integrated potential for UUS use. Integral evaluation is also based on a layered approach (Fig. 8 & 9); potential is classified at 4 levels.

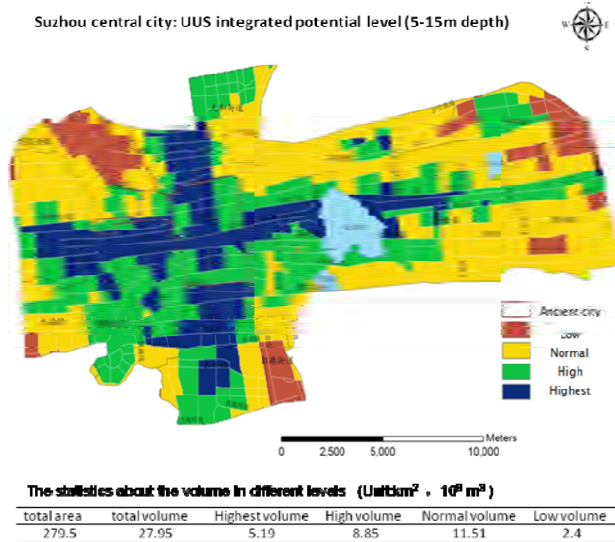


Fig. 8 Integral Demand-supply Evaluation (5 – 15 m depth, pipeline layer excluded) and Estimated Volumes

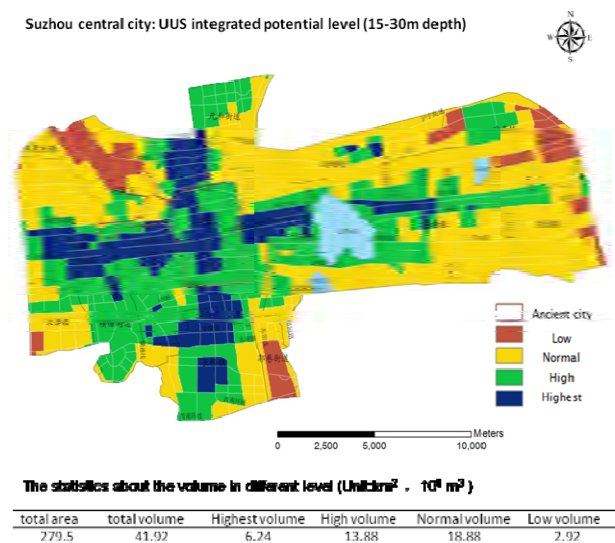


Fig. 9 Integral Demand-supply Evaluation (15 – 30 m depth) and Estimated Volumes

This comprehensive evaluation for UUS reflects the philosophy of “Deep City” concept, which advocates an urban governance innovation: **“rethinking supply capacity of natural resources before meeting urban construction needs”**. Our rapid urbanization process is aggravating the provision pressure of natural resources; a more considerate manner of space exploitation should be integrated into urban planning and land management.

## 2.3 Cost-Benefit Analysis

Demand-supply evaluation helps city administrators/professionals/business to understand the operational feasibility of using UUS. A quantitative assessment for project’s sustainability should be further developed to balance investors’ profits and citizens’ social benefits.

### (1) Benefits revealed from demand evaluation

Private benefits for investors to develop UUS in urban area with land use pressures and high development value, could be examined from several aspects:

- Gain from commercializing more floor space area  
Because of the fact that underground Floor Area Ratio (FAR) is not always considered as part of regulated land parcel FAR. Along with urban economic growth, the development of UUS will be required to change from unique use (such as parking or civil shelters), to multiple use (such as commercial complex, shopping center, cultural exhibition hall, theater, etc) [CARMODY & STERLING, 1993]. The vertical scale of UUS could also be expanded to include multiple functions. This is a trend of investment adopted by many Chinese real estate developers.
- Gain from land acquisition cost savings  
Urban land use policies seek to limit commercial land provision so as to release more land for housing. Moving towards multi-functions underground development helps save the investment in land purchase which usually accounts for a largest share of total project investment.
- Gain from customer patronage through enhanced pedestrian network  
It is a valuable opportunity to link private commercial space with metro system, by reducing pedestrian’s travel distance to a business area. In dense urban area, due to surface traffic congestion, travel time and pedestrian comfort are common concerns in daily life. The form of indoor city with underground pedestrian network is typical in Canadian cities [BARKER, 1986], serving by convenient connection under buildings and roads in a comfortable walking environment. Therefore, the reduction of intangible “travel cost” of customers and increase in patronage to shops are seen as mutual benefits to pedestrians and business.

The three factors mentioned above will increase investors’ “willingness-to-pay” for developing UUS, all contributing to the attractiveness of UUS development projects.

Public benefits to citizens could be assessed from several living quality related aspects:

• Livable city and compact city style

For land resource management, preservation of farmland for food production has to be well coordinated with commercialization of urban land. Therefore, urban expansion has to be limited. Many researchers have been advocating against urban sprawl [WISSEN HAYEK et al, 2010], because land resource is scarce and deserves a rational allocation. In addition, sprawling city requires more public investment on transport system and related urban services, which is not a sustainable way for long-term growth. With urban surface-subsurface 3D compact development, urban land will be used in a more efficient way, avoiding unlimited urban expansion.

• Gain from green open space and ecological asset

Megacities are usually described as “concrete jungles”. In order to revitalize city centers, green spaces and ecological parcels should be reintegrated into our daily life. Developing UUS helps to release more surface land, which could be planned for urban parks, serving citizens as leisure places. The intangible value of green space was estimated based on hedonic pricing models of housing market, revealing an increase in rental price due to proximity to green space [BARANZINI & SCHAERER, 2007].

(2) Costs revealed from supply evaluation

Based on the natural supply evaluation of geological conditions and engineering difficulty, costs comparison could be made on land parcels with different quality to reveal the differences of initial UUS construction investment. An example of cost estimation under three scenarios is prepared for a commercial building project based on Swiss context, including geological context and economic context.<sup>6</sup>

• Commercial building model as showed in Figure 10

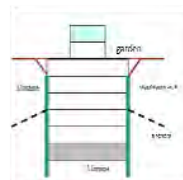
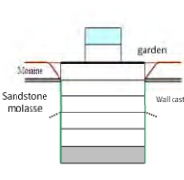
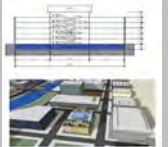
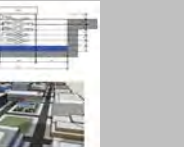
| Scenario 1                                                                                                              | Scenario2                                                                           | Scenario3                                                                           |
|-------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|
| Surface building (with Swiss Minergie standard)                                                                         | Underground building in bad geological condition                                    | Underground building in good geological condition                                   |
| Basic dimensions:<br>30 m x 50 m,<br>Floor height: 4 m,<br>6 floors in total,<br>with 1 floor for technical maintenance |  |  |
|                                      |  |  |

Fig. 10 Building Model for Cost Comparison

<sup>6</sup> Analysis was performed in the scheme of “Deep City” project, supported by Swiss National Fund.

• Construction costs as showed in table 4

| Scenario1  | Scenario2  | Scenario3  | S2/S1 | S3/S1 |
|------------|------------|------------|-------|-------|
| 30'464'000 | 37'563'000 | 33'437'000 | 1.23  | 1.11  |

Table 4 Construction Costs Comparison, with Land Price Excluded (in CHF, estimated by Economic Institute of Construction, SA)

It is estimated that construction on land parcel with higher difficulty level costs 23% higher than construction on the surface.

• Energy consumption during 50 years is showed in Table 5.

|             | Scenario 1 | Scenario 2 | Scenario 3 |
|-------------|------------|------------|------------|
| Lighting    | 192.3      | 192.6      | 192.6      |
| Ventilation | 20.5       | 20.5       | 20.5       |
| Devices     | 22.5       | 22.5       | 22.5       |
| Heating     | 47         | 25.7       | 29.7       |
| Cooling     | 15.9       | 12.3       | 9.5        |
| Auxiliaries | 1          | 1          | 1          |
| Total       | 299.2      | 274.6      | 275.8      |

Table 5 Energy Consumption of the 3 Scenarios (MJ/m<sup>2</sup>/year)

Life-cycle energy consumption is much lower in the underground building scenarios than in the surface building scenario. This is a long-term benefit for investors.

• A comparison of total costs (4<sup>th</sup> row) composing construction costs (1<sup>st</sup> row), life-cycle energy costs (2<sup>nd</sup> row) and land acquisition costs (3<sup>rd</sup> row), is showed in Table 6.

|                   | S1         | S2         | S3         | S2/S1 | S3/S1 |
|-------------------|------------|------------|------------|-------|-------|
| Construction cost | 30'464'000 | 37'563'000 | 33'437'000 | 1.23  | 1.11  |
| Energy cost       | 2'988'200  | 2'852'400  | 2'866'500  | 0.95  | 0.96  |
| Land price        | 15'465'000 | 15'465'000 | 15'465'000 | 1     | 1     |
| Total cost        | 48'917'200 | 55'880'400 | 51'768'000 | 1.14  | 1.06  |

Table 6 Total Life-cycle Costs Comparison (in CHF)

Taking into account the capital costs and maintenance costs during the building's life time, the total cost ratios of underground scenarios to surface scenario becomes much lower. If underground construction is on a good geological condition, the ratio is approaching equal, meaning only a slight difference with surface building.

This comprehensive life-cycle cost estimation is an important input to adjust UUS investment. According to engineering experts, construction cost is more or less stable in the world market, while energy cost varies a lot along time, which should be considered in advance for project evaluation.

## 2.4 Institutional Feasibility for Developing UUS

In order to succeed in implementing underground construction project, no matter for large scale

government-invested ones or small private-invested ones, different components as “hardware” and “software” have to be integrated. The “hardware” includes land purchase, engineering feasibility, capital, labour and operating bodies, while the “software” here includes regulatory guidelines, legal permission and management tools. Government as the city administrator, should play a leading role in steering all major urban projects. Intra-governmental interests of different hierarchies should be coordinated to provide necessary regulatory system and stand ready to guide. Structure of the institutional network and related objectives and regulations are showed on Table 7.

Similar to conventional surface construction projects, UUS development should go through all related management bodies within municipal institutional structure, to avoid conflicts with existing infrastructures and buildings, as well as to avoid incompatibility with the natural environment. Since the last decades, many countries have been trying to establish planning laws and related regulatory framework for development of underground space. Harmonizing the intra-governmental interests within the framework could help to facilitate construction projects and promote sustainable subsurface utilization. Interviews were conducted with officials and professionals from Geneva Canton in Switzerland and from Jiangsu Province in China<sup>7</sup> to examine politico-economic constraints in implementing UUS development. Both administration regions playing leading role at national level indicated that, they face different issues to manage UUS as shown in Table 8.

The urban areas of these two regions are facing strong land supply pressure, due to increasing housing demand, as well as redevelopment restrictions due to heritage preservation in city centers.

### 3. Integrated Governance of Urban Underground Resources (UUR) to Attain the Goal of Sustainability

This section emphasizes environmental protection planning and energy planning in the process of developing urban subsurface. Since UUS development have not been given much attention in the past decades, it is necessary to take this opportunity to include it into the whole sustainable city transformation practice [BOBYLEV, 2009].

As presented at the beginning, there is a natural environmental system beneath our urban land. Each of these natural resources in the system contribute significant role to support our urban life. Placing infrastructure and developments underground challenges the natural state of these resources. To have a comprehensive understanding of the development

| Stakeholder parties                        | Main objectives                                                                                                                                                                                                                                                                                  | Related regulations                                                                                                             |
|--------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------|
| <b>Economic Planning Commission</b>        | <ul style="list-style-type: none"> <li>Promotion of urban economic growth by delivering approval for potential projects to qualified developers.</li> </ul>                                                                                                                                      | Macro-economic plan, National Economic agenda, Public finance plan, Investment regulations, etc.                                |
| <b>Land Management Department</b>          | <ul style="list-style-type: none"> <li>Allocation of land resources, emphasizing on farmland protection.</li> <li>Management of property rights under land transaction and registration system.</li> </ul>                                                                                       | Law of Land Administration, Civil Code on property rights, Regulation on Land property market, etc.                             |
| <b>Urban Planning Department</b>           | <ul style="list-style-type: none"> <li>Promotion of spatial growth by allocating urban functions (commercial, industrial, residential, public space) on designated area.</li> <li>Establishment of detail planning and design guidelines for construction projects to ensure quality.</li> </ul> | Law on spatial planning, Urban Planning standards and guidelines, etc.                                                          |
| <b>Urban Construction Department</b>       | <ul style="list-style-type: none"> <li>Management of construction works by establishing operational guidelines for civil engineering.</li> <li>Standardization of urban construction project management for contractors and owners.</li> </ul>                                                   | Law on urban construction, Building standards, Building energy consumption standards, Law on Fire Protection and Security, etc. |
| <b>Real Estate and Housing Department</b>  | <ul style="list-style-type: none"> <li>Management of real properties by establishing obligations and rights.</li> </ul>                                                                                                                                                                          | Real Estate Law, etc.                                                                                                           |
| <b>Urban Infrastructure Department</b>     | <ul style="list-style-type: none"> <li>Provision of public facilities, including transport system, water and sewage system, energy utility, and communication networks.</li> <li>Standardization of public infrastructure project design to ensure social benefits.</li> </ul>                   | Law on urban infrastructure, Regulations on concessions for public infrastructures, etc.                                        |
| <b>Environmental Protection Department</b> | <ul style="list-style-type: none"> <li>Control environmental impact assessment for construction projects.</li> <li>Promotion of natural resources protection by imposing related standards according to natural carrying capacities.</li> </ul>                                                  | Law on Environmental Protection, Law on Waste Management, Regulations on Environmental Impact Assessment, etc.                  |
| <b>Civil Defense Department</b>            | <ul style="list-style-type: none"> <li>Provision of basic shelters and evacuations during disaster and emergency.</li> </ul>                                                                                                                                                                     | Law on civil defense, etc.                                                                                                      |

Table 7 Public Stakeholder Structure and Regulations

| Geneva Canton (CH)                                                     | Jiangsu Province (CN)                                                        |
|------------------------------------------------------------------------|------------------------------------------------------------------------------|
| Construction costs and related remediation costs for field cleaning-up | Construction costs and related compensation costs for nearby infrastructures |
| Social resistance for excessive constructions                          | Property rights of land ownership                                            |
| Low public attraction of using UUS                                     | Lack of knowledge and useful information                                     |

Table 8 Politico-economic Constraints for UUS Use

<sup>7</sup> Interviews in China were performed in June 2011; Interviews in Switzerland were performed during 2010.

impacts on individual components and the interrelationship of the impacts enables city administrators and project owners to work jointly for synergy effect in developing UUS (such as combining basement and foundation construction with energy geostructures<sup>8</sup>), as well as to avoid engaging in conflicting and incompatible utilization (such as damaging drinkable groundwater during tunnel excavation). The concept of “Deep City” was put forward in the framework of Swiss national research program NRP54 “Sustainable development in the built environment”. This project named “Underground resources and sustainable development in urban areas” developed a management methodology to optimize multiple exploitation of these underground resources in city perimeters, with detailed case study for the city of Geneva for methodological validations. Methodological scheme with multi-use approach to manage urban underground resources is showed in Figure11.

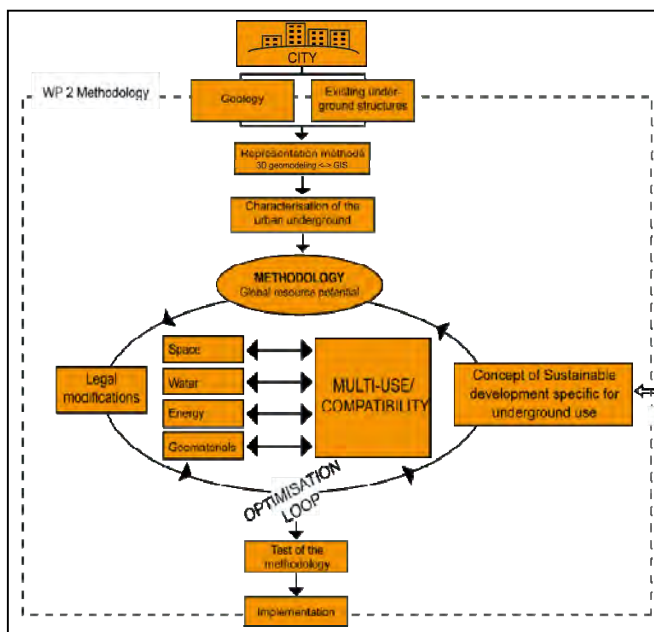


Fig. 11 Multi-uses Evaluation Methodology

In order to widen the applicability of this concept to much larger cities around the world, an ongoing project named “Deep City China” was launched with Nanjing Institute of Underground Space and Geo-environment (whose contributions are presented in section 2.2). This comparative research between Swiss and Chinese contexts in terms of urban underground resources (UUR) management will have a complementary focus on economic assessment of subsurface development. It will also explore the feasibility of UUS development under a coordinated administrative and regulatory system. In order to guide national application of the concept, territorial underground typology will be classified to suit the needs of various urban governance and resources management.

<sup>8</sup> Energy geostructures: <http://lms.epfl.ch/energy-geostructures>

#### 4. Conclusion

Research significance of this paper is to show a comprehensive methodology on the evaluation of demand-supply potential of Urban Underground Space (UUS) and resources (UUR). Potentials illustrated in the framework are then translated into economic terms as cost and benefits in order to justify project investments. Further work will be to complete the evaluation of benefits based on a real project scheme. Institutional feasibility investigation plays an essential role to facilitate future implementation of UUS planning and management. Finally, the holistic approach of UUR exploitation, “Deep City” concept, is presented with the aim to raise political awareness for optimal use of natural resources.

#### Acknowledgements

The work presented in this paper is supported by the Sino Swiss Science and Technology Cooperation (SSSTC 2009-2012) and National Natural Science Foundation of China (40872171).

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# SPACE SYNTAX IN UNDERGROUND SPACE'S SAFETY ASSESSMENT SYSTEM

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## Abstract

*In recent years, facing an increasing amount of pedestrians and more complicating spatial configuration in underground spaces, people call for higher design requirements to prevent disaster in these places, making it very urgent to establish an integrated safety assessment system. Shanghai has undertaken an experimental research on the safety assessment system towards different uses of underground spaces [SHU, 2009] but most of its indexes are subjective and descriptive, thus making the result not entirely precise and credible.*

*Space Syntax, as a theory of analyzing the relation between human behavior and spaces, is suitable for assessing human reaction in hazardous situations in places with much human activities, like in metro station and underground shopping mall. Three indexes are introduced to the assessment system from Space Syntax parameters, including Integration/Width, Gate Counts and Intelligibility, standards are established by comparing with related researches. A trial assessment was carried out on the East Nanjing Road Metro Station with improvement suggestions in reference to the assessment outcome.*

*This paper enriches the indexes of safety assessment system in underground spaces, making the result of it more credible and comparable with those of other cases, and contributing to improve both POE (Post Occupied Environment) and newly designed underground spaces.*

## Keywords

*Underground Space, Space Syntax, Safety Assessment*

## 1. Introduction of Existing Assessment System

Tongji United Underground Space Research Center, a pioneer in the field, has established an underground space safety assessment system in 2009, but with an obvious drawback in that as most of its standards are qualitative than quantitative, thus the assessment result becomes less precise and non-comparable with other similar projects. To set the background of the system, a brief introduction of it is highlighted in the following parts.

### 1.1 Concept and Objective

To implement the requirement of 'Safety First, Precaution As Centre' during the planning and construction stage of developments in China, the existing underground safety assessment system was established. Through assessing the general and crisis management, emergency equipment installation, building structure and spatial environment, improvement suggestions and redesign plan were provided. Among them, the assessment of the environment is closely aligned with the direction of research project presented in this paper. The assessment parameters include the location and design guides/requirements, legal restrictions on the fire-resistant structure design, entrance and stairway planning, etc. The main part of the assessment is made by way of answering '0-1' questions, which could be very subjective. In the end of the assessment, different weights will be assigned to each parameters and a score will be given.

### 1.2 System Structure

The existing restriction structure of assessment system is shown in Figure 1. All restrictions are qualitative in reference to the existing laws and standards, results of which are only 'Conform' or 'Not Conform'.

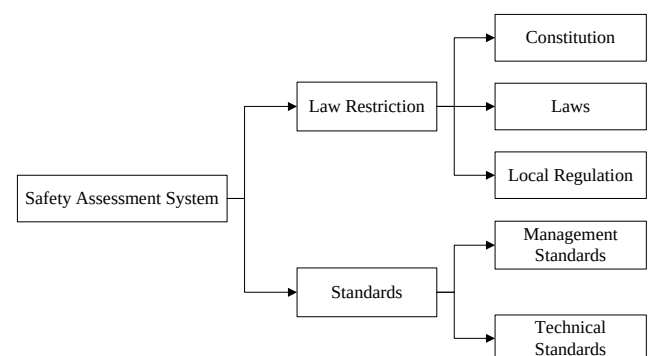


Fig. 1 Existing Restriction of Assessment System

### 1.3 Indexes Introduced

All indexes (Fig. 2) are divided into three main classes: Safety Management, Equipment and Environment, and each has a secondary class index. A marking system is

set for each index from 1 to 20, and different weight will be given. In the end of the assessment, a total will be calculated automatically.

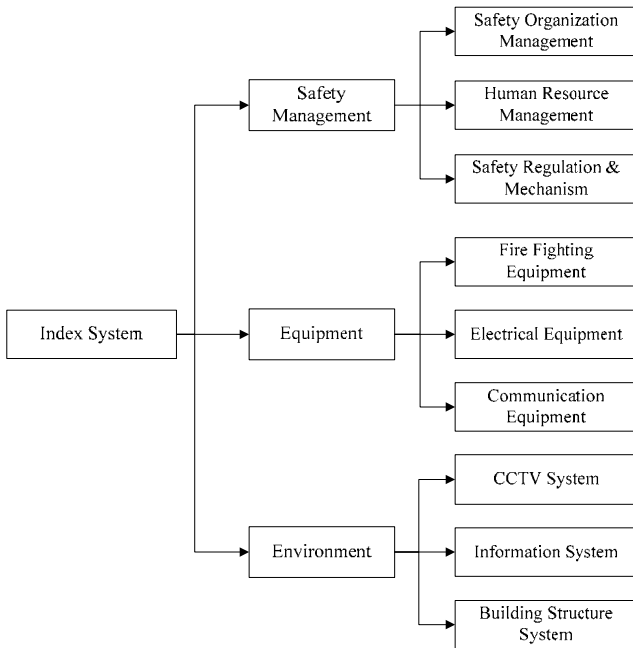


Fig. 2 Existing Assessment Indexes

#### 1.4 Needs for Improvement

As a safety assessment system, requirements under laws and regulations are essential items that need to be complied with. To ensure the safety of underground spaces and to make it possible for comparison with other cases, adoption of more quantitative indicators is necessary. Among all indexes, the 'Building Structure System' is most suitable for incorporation into the Space Syntax, as it is a widely used method in building design.

## 2. The Method of Space Syntax

### 2.1 Introduction of Space Syntax

#### (1) Basic Theory

Space Syntax is a set of techniques for the analysis of spatial configurations of all kinds, especially where spatial configuration seems to be a significant aspect of human affairs, as it is in buildings and cities. Originally conceived by Professor Bill Hillier and his colleagues at the Faculty of the Built Environment at University College London (the Bartlett) in the 1980s as a tool to help architects simulate the likely effects of their designs, it has since grown to become a tool used around the world in a variety of research areas and design applications. It has been extensively applied in the fields of architecture, urban design, planning, transportation and interior design. Over the past decade,

space syntax techniques have also been used for research in fields as diverse as archaeology, information technology, urban and human geography and anthropology

#### (2) Use in Underground Space

In recent years, since more and more attention has been drawn to the underground spaces, Space Syntax, as a useful tool in analyzing space configuration, has already been widely used.

- Alain Chiaradia [CHIARADIA et al, 2005] chose the London Underground as a case study for the exploration of configurational effect in a global public transport networks, help to prioritise variables and disentangle demand forecasting difficulties.
- Jumpei Ueno [UENO et al, 2009] applied the theory to the Shibuya Station, Tokyo, analyzed the relation between pedestrian flow and various indexes of Space Syntax and estimated the number of pedestrian flow within the target space according to the Space Syntax theory.
- Alper Ünlü [ÜNLÜ & EDGÜ, 2007] introduced the Space Syntax theory to enlighten currently emerging design problems of the underground railway stations of Istanbul Metropolitan Municipality, found that the theory is very useful for architects to make comparisons of design options such as design strategies in metro station.

All these researches have proven that the Space Syntax theory is applicable for analyzing many kinds of underground spaces like metro station and underground shopping mall, but none of them have ever concluded a useful index system for evaluating and assessing the safety of space, and this paper will choose this as our main research field.

Two main software packages have been developed to calculate those indexes by Space Syntax Limited, CONFEEGO is a software well integrated with GIS systems, and DEPTHMAP is the other widely used software for its good manipulation. Since GIS is not involved in this research, DEPTHMAP is chosen.

### 2.2 Indexes Introduced

#### (1) Integration/Width

Many previous researches have proven the close relation between Integration and pedestrian amount. Integration is a normalized measure of the mean shortest path from the point to all other points in the system. Generally, places with higher Integration attract a larger number of pedestrians, which will possibly bring about hazard of collision, especially when those places are designed with a narrow passage. So, it is very reasonable to include Integration and

passage width as the parameters in the underground space safety assessment system. Through comparing the value of the ratio between Integration and passage width with the real crowd condition on site, standards can be established.

### (2) Intelligibility

Intelligibility is calibrated by the ratio between the parameter of Integration (Rn) and Connectivity (Cn), the Connectivity is used to discuss the number of lines connected to another, the ratio shows the possibility or ability of 'reading' the whole space through observing the neighboring spaces, which in another word is the explanation power of regional spaces. Pedestrian in better intelligibility spaces are easier to find route to their destination and the special design is more humanity.

### (3) Gate Counts

Space syntax deals with the configurational properties of environments. In the software, spatial agents use vision to assess the configuration, and move towards open space by a stochastic process: choosing a destination at random from the available space, and walking towards it. The rules are: walk 3 steps, look around and choose a new destination, walk 3 steps, and so on. Trails will be shown in the image, which can be considered as the simulation of the real pedestrian trail in the spaces.

This function is called Gate Counts in Depth map. The agents walk over 1m grid squares they lay trails behind them. Black areas represent few agent trails and colorful areas many agent trails. In one research made by Space Syntax Lab, in the city, the correlation coefficient of  $R^2 = 0.67$  with actual people gate counts and in the Tate Gallery  $R^2 = 0.76$ . Although there is some reason unexplainable for the space capture mechanism of agents, the Gate Counts is still applicable in explaining the pedestrian flow.

## 2.3 Standards Establish

Among the three indexes introduced before, the standard of Gate Counts is the most complicating one, because not only the picture itself is very intuitive to tell the pedestrian situation, some quantitative indexes could also be calculated as well, so the standard of Gate Counts consist two parts.

There are mainly two methods to establish the standard of this research: One is to investigate and follow pedestrians, find the relation between the index value and pedestrian movement; another is to conclude from relating papers to find commonly used standards, this paper introduces the latter method.

### (1) Integration/Width Standard

Limited by the function of Space Syntax software, the value of Integration is more plausible when making comparison with one case. For comparison amongst cases, the value varies and become less predictable. The ratio between Integration and Width, however, does not have such problem. In the research 'Space Syntax in Pedestrian Safety Analysis' [ZHU et al, 2011], an index of Integration/Width has been calculated. The above research findings were employed in deriving the standard of Integration/Width (Table 1) in this paper.

| Integration/Width | Level  | Remarks                        |
|-------------------|--------|--------------------------------|
| >1.5              | Low    | Low accessibility and poor LOS |
| 1~1.5             | Medium | Little crowded in peak hours   |
| < 1               | High   | Appropriate width design       |

Table 1 Integration/Width Standard

### (2) Intelligibility Standard

Intelligibility, a very commonly used parameter in Space Syntax and widely adopted in researches, is mainly used for assessment in areas with very high pedestrian flow. Jui-Sung Huang and Szu-Yu Tzeng did a study on way-finding path in an out-patient area [HUANG & TZENG, 2009] and Yu Qianbai analysed the space organization of a shopping mall by using Space Syntax [YU, 2005], etc. After examining their research results which set standards for their studies, a standard that suits our research purpose is derived as shown in Table 2.

| $R^2$ of Intelligibility | Level  | Remarks                                                    |
|--------------------------|--------|------------------------------------------------------------|
| < 0.3                    | Low    | Hard to read the space, need guidance to finding their way |
| 0.3~0.5                  | Medium | The space can be read and people can find their route      |
| >0.5                     | High   | Well-designed space with smooth pedestrian flow            |

Table 2 Intelligibility Standard

### (3) Gate Counts

Compared with the other two indexes, Gate Counts is newly developed, very few researches used its quantitative result. Most of them just use the pedestrian flow pictures to evaluate the level of service (LOS) condition. The quantitative result of Gate Counts includes the value of maximum, average and minimum; the higher the value of maximum and average, the greater the chance of collision of pedestrians. Through collecting and analyzing some cases which adopted Gate Counts in their studies, the standard is derived as shown in Table 3. We find the value of maximum covers a wide range, as long as the total number of agents released in the space is 50, the maximum value will not be exceeded. This makes the setting of maximum value quite rough.

| Value of Gate |       | Level  | Remarks                                  |
|---------------|-------|--------|------------------------------------------|
| Ave           | Max   |        |                                          |
| > 5           | >40   | Low    | Chaos of pedestrian flow, high collision |
| 3~5           | 20~40 | Medium | With several pedestrian flow             |
| <3            | <20   | High   | Smooth pedestrian flow, little collision |

Table 3 Gate Counts Standard

## 3. Trial Assessments

### 3.1 Background

East Nanjing Road station lies in a well-established pedestrian street in the downtown with a high pedestrian density (Fig. 3), attracting large number of tourists and locals. It is a terminal station connecting Metro Line 2 and Line 10, in which, 7 exits link to the ground. Metro Line 2 was built in 2000 while Line10 was recently finished in 2010. The distance between the two stations is essential in consideration of the safety control during construction. It makes the space and pedestrian flow inside the station complicated at the same time.

The whole spaces of station hall (Fig. 4) are included in the research, besides those columns, other installations, which affect the pedestrian movement, are also taken into consideration. For instance, temporary fences, though not a visual barrier, is treated the same as concrete wall, as they in effect obstruct people movement. (Adjustment is made, different from Space Syntax's 'visual decides behavior' theory).

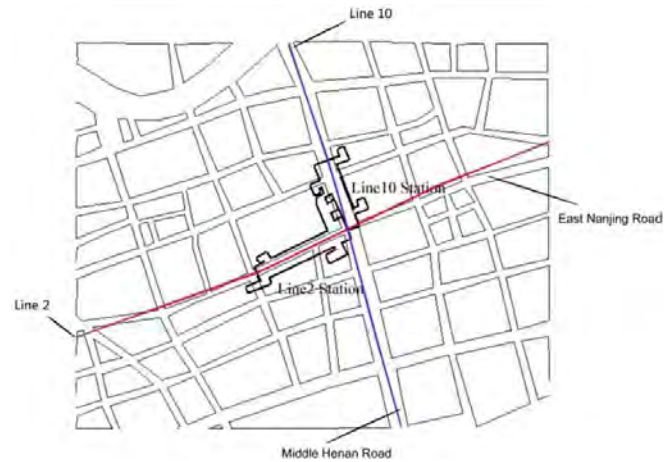


Fig. 3 The Location of Station

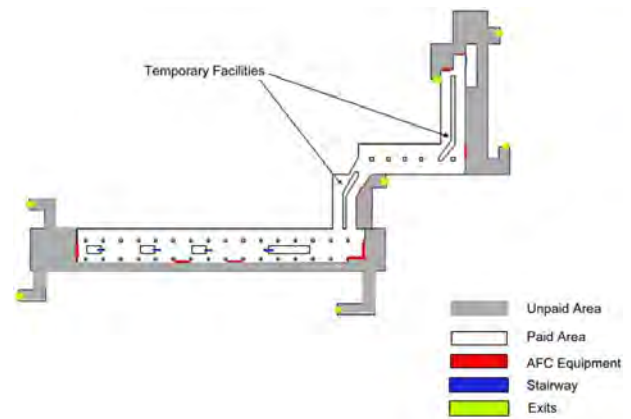


Fig. 4 Floor Plan of Station

### 3.2 Assessment Calculation

After importing the DXF pictures drawn by AutoCAD into DEPTHMAP, the value of parameters Integration, Gate Counts and Connectivity are calculated. 2m grid size is chosen and during the VGA analysis, agents are released from each entrance and stairway according to the pedestrian investigation on site.

#### (1) Gate Counts

The picture shows the flow condition is quite smooth, especially in the hall of Metro10 and at two main meeting points (Fig. 5) in the hall of the Metro Line 2, one is at the intersection place of the hall before reaching the exits of two stairways, another is in the entrance area where pedestrian from paid zone and unpaid zone meet. After adding temporary facilities in the link passage as ZHU Yuning suggested [ZHU et al, 2011], the pedestrian condition has been much improved.

The calculation shows that the maximum value is 21 and the average value is 4.11. According to the standard established above, it ranks Medium, which conforms to the picture result.

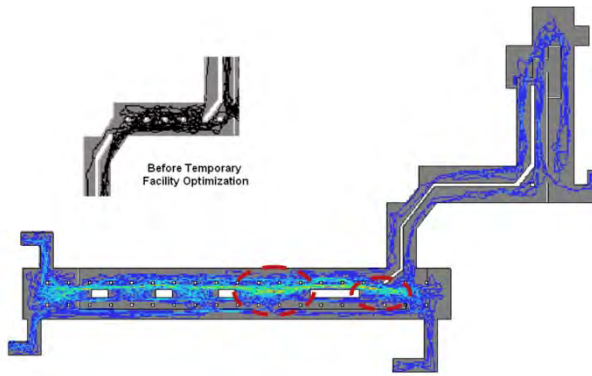


Fig.5 Gate Counts

## (2) Intelligibility

After calculating the Integration and Connectivity of the space, the correlation between two parameters is drawn as shown in the scatter diagram in Fig.6. The assessment result shows that the  $R^2=0.302$  which is of “Medium” rank in accordance with the standards established before.

Considering that the study area is a transportation hub, most pedestrians are destination-based and have a good visual picture of the whole space, low intelligibility may not bring serious impact on their way-finding, especially where the place is well equipped with information boards. With a view to achieving a better  $i$  value of  $R^2$  of Intelligibility, we could further optimize the space design to make the station more human in scale and more readable by pedestrians.

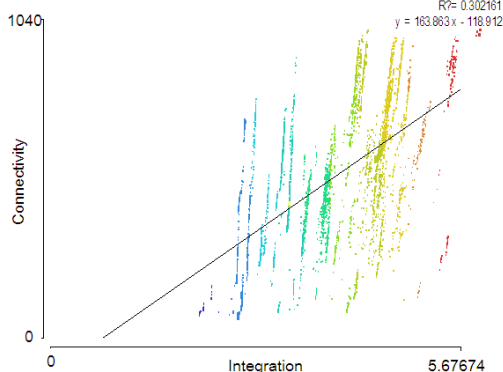


Fig.6 Correlation between the Integration and Connectivity

## (3) Integration/Width

After adding temporary facility (removable fences) in the link passageway, the pedestrian flow in the hall of Metro10 is smooth during peak hours, so the trial assessment will mainly focus on the hall area of Metro Line 2. In consideration of the situation on site, 5 areas are chosen to calculate the index (Fig. 7): Areas 1, 2

and 4 are entrances with high pedestrian flow in peak hours; Area 5 includes the main intersection (Compared with the result of Gate Counts above); Area 3 is a narrow passageway.

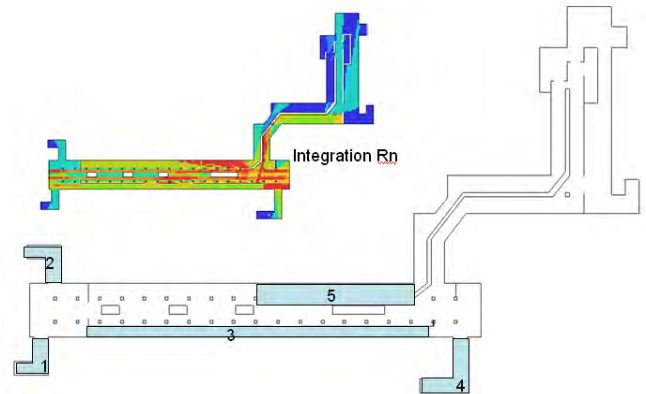


Fig.7 Assessment Areas and Integration Result

Assessment result (Table 4) shows that the accessibility and LOS in East Nanjing Road Station is satisfactory, but in the circumstances when the pedestrian flow surges to an exceptional level in real situation, the ranking need to be adjusted downward to ensure the safety.

| Area | Integration | Width | Integration/<br>Width | Level  |
|------|-------------|-------|-----------------------|--------|
| 1    | 5.20        | 10    | 0.52                  | High   |
| 2    | 4.83        | 10    | 0.48                  | High   |
| 3    | 8.63        | 10    | 0.86                  | High   |
| 4    | 5.03        | 7.64  | 0.50                  | High   |
| 5    | 12.13       | 9.98  | 1.21                  | Medium |

Table 4 Integration/Index Assessment

## 3.3 Assessment Result

### (1) Safety Level

After integrating the assessment result of three indexes in Table 5, in sum, the East Nanjing Road Station ranks ‘Medium’ in this trial assessment, which means that the safety condition of the place is satisfactory, the organization and management status is good, but in emergency situation, there is still high possibility of hazards.

|                          | Level  | Value          |     |
|--------------------------|--------|----------------|-----|
| <b>Integration/Width</b> | Medium | 1.21 (Highest) |     |
| <b>Gate Counts</b>       | Medium | Ave            | Max |
|                          |        | 4.11           | 22  |
| <b>Intelligibility</b>   | Medium | 0.302          |     |

Table 5 Assessment Result

## (2) Suggestions

In existing places with high pedestrian flow, there are always improvement measures through management means. Before this assessment, some temporary facilities have already in place in the East Nanjing Road Station to smooth the pedestrian flow, but drawing on the assessment results, more improvement opportunities are identified, and each of these opportunities could be tested by the assessment indexes to achieve optimized design.

- Equip automatic fare collection (AFC) units in western and eastern ends of the paid zone could reduce the level of cross flow in Area 3 with a view to improving the index of Integration/Width.
- Optimize the capacity of stairways by separating the upward and downward flow, equip more temporary facilities, with a view to improving the 'Gate Counts' value and smoothing the pedestrian flow.

Provide more information boards and signage in the station to assist way-finding, with a view to improving the Intelligibility index.

## 4. Conclusion

Taking the lead to establish quantitative assessment of the safety of underground space, this paper introduces the theory of Space Syntax, including the adoption of three indexes, i.e., Integration/Width, Gate Counts and Intelligibility for assessment. Instead of assessing the management side, which has been researched before, this paper focuses on the safety of pedestrian assessment. The three indexes are capable of analyzing the risk of collision and the rationality of passage width design. A trial assessment is made on the East Nanjing Road Metro Station, the results of which show that the safety level ranks 'Medium'. With reference to the application of the indexes, suggestions to improve the pedestrian safety condition are also proposed.

This paper leads the way to better organize and assess the underground spaces. The assessment process could be referred to in refining and optimizing the design of space. With the benefit of this scientific approach, assessments are made in other districts of Shanghai,

including underground space with five uses (restaurant, commercial, parking, hotel and entertainment). As this paper presents a pioneering research with only one method introduced, and thus only limited new indexes are produced. It is expected that other methods and quantitative indexes could be established in future. After optimization of the assessment system, the underground spaces will be undoubtedly safer and more user-friendly.

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# SPACE SYNTAX IN PEDESTRIAN SAFETY ANALYSIS: A CASE STUDY IN WANDA UNDERGROUND SHOPPING MALL

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## Abstract

*With the high speed of urbanization in China, land in big cities has become more and more in short supply, which makes the underground spaces a prime choice for urban development. Meanwhile, safety concerns arise as huge amount of people are being attracted to underground spaces. In recent years, much attention has been given to the underground space safety management, especially the pedestrian safety.*

*Previous researches focused mainly on the evaluation of emergency reaction and evacuation performance. The project presented in this paper chooses to research on the pedestrian daily safety management by analyzing the impact of space configuration on pedestrian behavior with reference to the theory of Space Syntax, and using Wanda underground shopping mall as a case study. After comparing the results of Space Syntax with the real condition data, we find the variables 'Integration', 'Isovist' and 'Connectivity' can well explain the pedestrian behavior in the underground shopping mall. This research finds that places with low value of 'Isovist' are more vulnerable to criminal activities, the ratio between 'Integration' and passage width can be taken as an indicator to evaluate the Level of Service (LOS) for pedestrian, and 'Integration' and 'Connectivity' can be used to forecast the distribution of pedestrian. Optimization design and management advices are given in reference to the research, the redesign outcomes are measured by the improvement of variables.*

*This paper not only provides a more quantitative method to evaluate the safety management in those underground POE (post occupied environment) but also broaden the scope of future underground space design idea.*

## Keywords

Underground Space, Space Syntax, POE, Safety Management

## 1. Introduction

### 1.1 Background

In recent years, the high speed of subway development in Shanghai has spurred many other uses of underground spaces simultaneously, among which, multi-use underground spaces that integrate the

function of shopping, transportation terminals and parking lot are most common. This kind of complex environment makes people inside of it difficult to appreciate their relative locations. As a result, pedestrian safety hazard may increase. At the same time, as those mega underground spaces always attract a huge amount of people, the situation there are even worse. So it is very urgent and essential to research on the pedestrian safety in the underground spaces.

The case of Wanda Underground Shopping Mall (Fig. 1) is introduced in this paper, as it is one of the biggest ones in Shanghai and locates in the sub-center Wujiaochang. The whole Wanda Shopping Mall has a total floor area of 340,000 m<sup>2</sup> (including 260,000 m<sup>2</sup> business areas) for restaurants, shopping and entertainments uses, with an average of 300,000 customers per day. There are 40,000 m<sup>2</sup> shopping area on the first level of underground space.



Fig. 1 Floor Plan of Wanda

Nine elevators provide connection between the first underground level and the ground, with the southern entrance linking with the underground square and the northern entrance leading to the Metro Line No. 10. After the opening of the Metro Line, the amount of customers hugely increased. This makes the safety of pedestrian seriously threatened in peak hours, criminals and crowds chaos happen from time to time.

### 1.2 Previous Methods

To improve the pedestrian safety condition, a lot of methods have been introduced, among which, following methods are most commonly used for underground spaces.

#### (1) On-Site Analysis

"Gate Points" are set in places where spaces change, pedestrian data both in peak and low time are collected on site, including speed, distribution and density.

#### (2) Multinomial Logit Model

The logit model was introduced by Joseph Berkson in

1944. The discrete choice model is most commonly used by researchers to analyze the pedestrian in different spaces. It statistically relate the choice made by each person to the attributes of the person and the attributes of the alternatives available to the person.

### (3) Pedestrian Model Analysis

In recent years, there have been several attempts to model pedestrian flows. The Highway Capacity Manual [TRB, 1985] is dedicated to modeling pedestrian flow and describing macroscopic techniques for capacity analysis. Blue and Adler [1998] applied cellular automata micro-simulation to model uni- and bi-pedestrian directional walkways. Helbing and Molnar [1995] have worked toward developing pedestrian flow models. The most widely used pedestrian modeling software based on these theories includes Exodus, Legion, STEPS, etc.

Methods used before mainly focus on the pedestrian himself and analyze the pedestrian condition through human behavior theory, the results of them are very direct and obvious, but most of them neglect another important factor on human behavior, the **SPACE**.

### 1.3 Space Syntax

Surrounded by all kinds of spaces, no matter big or small, complicated or simple, people can ‘read’ the information given by the space unconsciously. After receiving the information, people will decide their movement, which forms the pedestrian flow. In this sense, analyzing the spaces where movement take place is very essential in optimizing the pedestrian condition, so this paper choose to analyze the pedestrian safety by introducing the theory of Space Syntax, which is used by architects very often in space design.

Space Syntax is a set of techniques for the analysis of spatial configurations of all kinds [Space Syntax Limited, 2008], especially where spatial configuration seems to be a significant aspect of human affairs, as it is in buildings and cities. Originally conceived by Professor Bill Hillier and his colleagues at the Faculty of the Built Environment at University College London (the Bartlett) in the 1980s as a tool to help architects simulate the likely effects of their designs, it has since grown to become a tool used around the world in a variety of research and areas and design applications.

### 1.4 Purpose and Features

The purpose of the project presented in this paper is to research on the pedestrian safety in a large underground shopping mall by introducing the Space Syntax theory to analyze the space, provide general underground space design suggestions for both POE and newly built spaces in the end of this paper.

This project has three features: First, it investigates the correlation between pedestrian and underground shopping spaces. Second, it provides a new method in designing the underground shopping mall. Finally, it develops some possible assessment indexes, which will be useful in establishing the assessment system in the future.

## 2 Method

### 2.1 Study Area

This project took the whole accessible space, except the storerooms, on the first floor under the ground as study area, and it simplified the entrance A, disregarded the linkage with the underground square. Because the elevators are the only access for pedestrian to go upstairs, it has great effect on the pedestrian condition and human behavior in the underground shopping mall, so in the process of the software analysis, special arrangements were made to the points in the first floor level and ground level of the elevators are linked (Fig. 2).

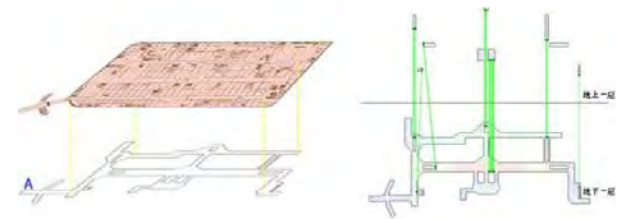


Fig. 2 Study Area

### 2.2 Visual Graphic Analysis

Three main expressions are included in Space Syntax: Apex, Axial Lines and Isovist (Table 1). This paper will mainly focus on the Visual Graph Analysis (VGA) and the results are expressed through Connectivity, Integration and Isovist, the software Depthmap was used as the main tool in analysis.

| Index        | Meaning                                                                                                                                                                                                                                                                                           |
|--------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Connectivity | Visual Connectivity, for neighborhood size or the approximate Isovist area, is used to discuss the number of lines connected to another.                                                                                                                                                          |
| Integration  | Integration is a normalized (inverse) measure of the mean shortest path from the point to all other points in the system.                                                                                                                                                                         |
| Isovist      | Isovist expresses the amount of vision area of a point, its polygons represents the visually accessible area from a location, along with measures of those polygons (such as perimeter, area and so on), or further join a dense grid of Isovists into a visibility graph of intervisible points. |

Table 1 Parameters of Space Syntax

### 3 Safety Analysis in Wanda Underground Shopping Mall

Occurrence of street crime is one of the most common and biggest problems in shopping mall, since the theory of 'Defensible Space' [NEWMAN, 1996] first proposed by Newman in 1970s, many research attention has been drawn into it, but most of them are qualitative. Therefore, the authors chose to analyze the crime prevention with a more quantitative method.

Meanwhile, overcrowding is another severe problem inside of the underground shopping mall, too many pedestrians converge in a narrow passage with unsatisfactory spatial design will trigger a chain reaction and cause disaster eventually, so this paper proposes a proper passage design based on the Space Syntax analysis.

#### 3.1 Anti-Criminal Safety Analysis

Underground space with complicated design, many blind spots and low Isovistis vulnerable to crime. It is necessary to optimize the space design to make it safer. People and space are two main factors in this research, Connectivity and Integration are taken as the parameters to calibrate the amount of pedestrian flow, while Isovist is taken to calibrate the amount of vision area.

##### (1) Connectivity and Integration

We separately calculated the index of Integration (Fig. 3) and Connectivity (Fig. 4) in Wanda Underground Shopping Mall. Two indexes decrease with the color changes from red to blue. The courtyard area shows the highest Integration and Connectivity and the passages linked with the axis also show a relatively high value. Previous research findings have proved that pedestrian intensity has a positive correlation with the value of Integration. We have the same observation on site. Apparently, these two places are the focus of pedestrian flow and where potential crimes occur. Besides, the two elevators in the courtyard linking to the ground and the southern end of the passage are the only entrance leading to the underground square, provide easy escape for the criminals. Having regard to the above reasons and examining the real situation, we identify the main criminal spots as shown in Figure 5.

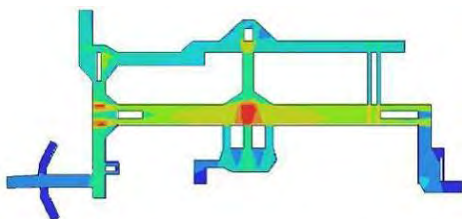


Fig. 3 Integration

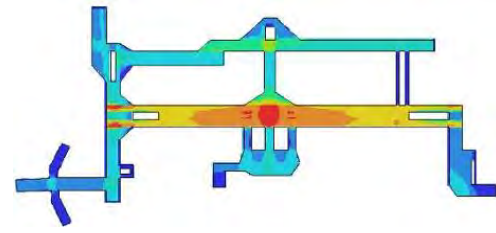


Fig. 4 Connectivity



Fig. 5 Criminal Spot Distribution

##### (2) Isovist Analysis

Besides the high pedestrian flow, bad vision scope which deprives places of enough "Street Eyes" [JACOBS, 1961] also makes them more vulnerable to crime. The result of the visual analysis (Fig. 6) shows that the courtyard has the best Isovist (which is the same to our direct feeling), while the places A, B, C, D, E have low Isovist. Among those places, A, B, C, E are cul-de-sac and D is a passage severely blocked by an elevator (Fig. 7).

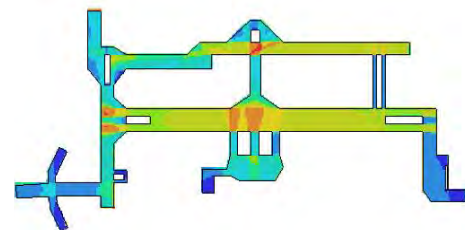


Fig. 6 Isovist Perimeter

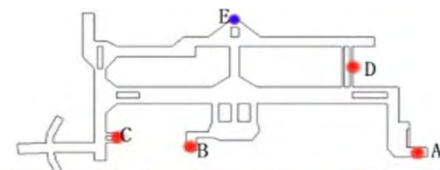


Fig. 7 Low Isovist Locations

#### 3.2 Improvement Suggestions

With reference to the analysis results, the following suggestions are proposed for improving the anti-criminal design in Wanda Underground Shopping Mall:

- To Deploy Additional Security Guards  
It provides positive surveillance to address the lack of “street eye”, prevent criminals and protect customers as well. Two hot spots, the courtyard area and the passages linking the axis, are suggested to be mainly the places to station security guards.
- To Install Additional Surveillance Cameras  
If the existing surveillance cameras do not cover all area in the Underground Shopping Mall, more cameras are suggested to be installed at suitable locations. Meanwhile, improve the through-view of department stores can also strengthen the effect of “street eye”.

### 3.3 Walking Safety Optimization Design

#### (1) Current Condition

During the peak hours in Wanda Underground Shopping Mall, some narrow passages with high pedestrian flow tend to cause accidents of collision and jostle, so it is essential to optimize the passage design in accordance to the pedestrian flow (Fig. 8). The authors used the index of Integration (Fig. 9) to measure the amount of pedestrian flow, and ratio between the width of passage and Integration (Table 2) to identify passages with problems.

The result shows that passage 2 and 10 have the highest value, and so is to the passage 6, 17, 13, considering the limit use of Space Syntax, the function of the spaces and real condition (Fig. 10) on site should be taken into consideration.

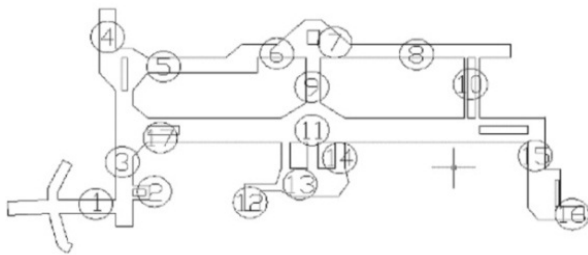


Fig. 8 Passage Number

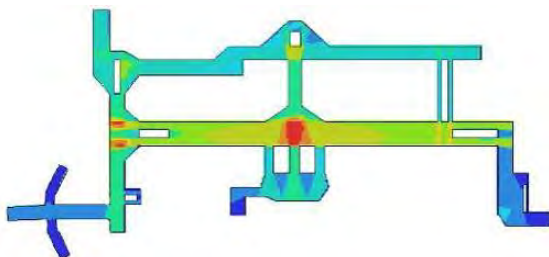


Fig. 9 Integration

| Passage Number | Integration  | Width (m)  | Integration/width |
|----------------|--------------|------------|-------------------|
| 1              | 4.832        | 8.0        | 0.604             |
| 2              | <b>3.332</b> | <b>2.0</b> | <b>1.666</b>      |
| 3              | 7.788        | 8.8        | 0.885             |
| 4              | 7.156        | 9.0        | 0.795             |
| 5              | 6.470        | 9.0        | 0.718             |
| 6              | 6.684        | 6.5        | 1.028             |
| 7              | 5.412        | 6.5        | 0.832             |
| 8              | 6.521        | 7.5        | 0.869             |
| 9              | 7.540        | 7.5        | 1.005             |
| 10             | <b>6.105</b> | <b>3.7</b> | <b>1.650</b>      |
| 11             | 12.557       | 14.2       | 0.884             |
| 12             | 3.316        | 7.7        | 0.430             |
| 13             | 6.506        | 6.0        | 1.084             |
| 14             | 6.306        | 12.0       | 0.525             |
| 15             | 5.411        | 8.7        | 0.621             |
| 16             | 3.654        | 7.7        | 0.474             |
| 17             | 9.248        | 9.1        | 1.016             |

Table 2 Value of Ratio

#### Passage 2

Passage 2 is a cul-de-sac under an elevator where a store is set, compared with the result of Space Syntax, the actual pedestrian flow and collision chances is much less.

#### Passage 10

Passage 10 is a space severely blocked by an elevator. As it is the link between the main and auxiliary axes, pedestrian flow is huge according to the real condition on site. The condition makes it very vulnerable to accidents (the same as Passage 17).

#### Passage 6

Passage 6 is a turning point of the space, where pedestrian flow changes, the pedestrian intensity is quite big there (especially after the launch of the stores along the axis, more customers will be attracted), so the passage 6 is also vulnerable to accidents.

#### Passage 13

Passage 13 is the space connecting to the main axis and restaurants area. According to our observation, in peak hours, pedestrian flow is low, maybe it is related to the attractiveness of stores. And because the floor plane is of irregular shape, it is difficult to measure the width of the passage, so the results of Space Syntax are less accurate in this area.



Fig. 10 Passage Condition on Site

## (2) Optimization Passage Design

Since space configuration has a strong relation with the pedestrian flow, optimization design for Wanda Underground Shopping Mall is discussed in this section in reference to the case of Bartlette Museum Newly Design [DURSUN, 2007]. Passage 10 is chosen as the example of optimization design target.

Because the space of Passage 10 is mostly occupied by an elevator, there is limited scope to make major changes, so two measures in the following are considered:

(a) Widen the passageway by 1 meter along both sides of elevators (Fig. 11).

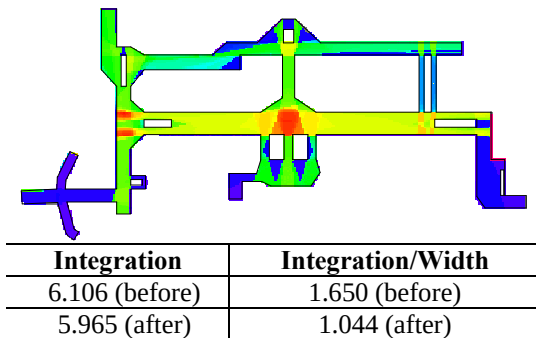


Fig. 11 Before and After Passage Widening

(b) To move forward the north end of the auxiliary passage (Fig. 12).

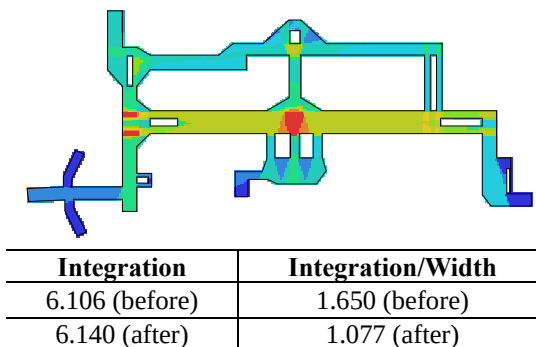


Fig. 12 Before and After Modification of Auxiliary Passage

Results showed that both designs are capable of reducing the ratio between integration and width, which means the overcrowdings eased and improved. But as the first measure involves major design change of the original construction, it is more useful for adoption in a newly designed project, while the second design is more useful to the POE (pre-occupied environment) like Wanda Underground Shopping Mall.

## 4. Conclusion

This paper presents a project that analyzed the safety of the underground spaces by researching on the

effects of different space configurations on pedestrian. The method of Space Syntax was introduced to address two main safety issues: crime and collision. Through our research, we demonstrated the possibilities of applying the Space Syntax into the design of underground spaces, and analyzed the close relationship between indexes and space configuration. After analyzing the Wanda Underground Shopping Mall, which is taken as our case study, redesign and improvement suggestions were given.

In the future, further researches are needed to optimize the use of Space Syntax in the planning of underground spaces in following areas:

- In the analysis of the security against criminal activities in underground spaces, more criminal data need to be collected. A correlation calculation between the relevant parameters and the data should be made.
- CCTV installations and their vision area need to be analyzed and compared with the result of Isovist analysis.
- Pedestrian simulation software should be used and comparison should be made between results of Space Syntax and Pedestrian software to optimize the spatial design.

After introducing a more scientific research method into the spatial design of underground spaces, the POE redesign and newly planned spaces will be safer to the users, which will enhance the attractiveness of the underground space and make its future development more inviting.

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# RESEARCH OF URBAN UNDERGROUND SPACE UTILIZATION DEMAND FORECASTING UNDER THE BACKGROUND OF “EXTRAORDINARY DEVELOPMENT”

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## Abstract

*Over the past decade, China has rapidly grown up to a major promoter and user of urban underground space (UUS) in the world. How can UUS demand forecasting play an effective and practical role in controlling and guiding the “extraordinary development”? It has become a prime concern in planning practice. On the basis of examining and analyzing the key issues in research and practice, it puts forward the basic system framework of UUS demand forecasting, taking account of developmental needs. The framework is divided into two planning levels: underground space master planning and detailed planning of special areas. Separately, it defines the substantive purpose, analyzes the demand characteristics, and then adopts different ways and methods specifically. It aims to further improve the scientific-basis, adjustability, effectiveness and standardization of demand forecasting.*

## Keywords

*Extraordinary Development, Urban Underground Space, Demand Forecasting, System Framework*

## 1. Introduction

From 2000 to 2010, cities with rail transit increased from 4 to 12, and a further 28 cities have obtained State Council approval to build rail transit. The total length of the operating line will increase to over 6000 km from 1270 km (up to October 2010) in the next decade. Over the past decade, with the great opportunity of urban rail transit construction, large-scale urban renewal and new city projects, China has quickly become a leading country in the underground space development and utilization. Most cities pay more attention to the underground space development and utilization, and take it as an effective way to ease city syndrome and achieve sustainable urban development [SHU, 2002].

Urban underground space (UUS) development in China thrives as a trend of “Extraordinary Development” on the whole. On the one hand, most cities have great enthusiasm for developing UUS, with increasing scale, growing functional uses types, accelerating construction process and involving more related fields. On the other hand, there are constraints of traditional concepts, institutions and laws. The

supporting techniques, theoretical researches and planning also failed to keep up. The comprehensive benefits and overall quality need further improvement. There are so many prominent contradictions in construction, operation and management.

The then Ministry of Construction enacted the Regulations of Urban Underground Space Development and Utilization in December 1997, and revised it in 2001. Its article 6 states that the main contents of UUS development and utilization planning include: the status and development forecast of underground space, the underground space development strategy, the development levels, contents, period, scale and layout, and the implementation steps as well. The then Ministry of Construction promulgated the Measures for Formulating City Planning in 2005, and brought the UUS planning into the system of city planning. The Urban and Rural Planning Law coming into effect in 2008 also involves the contents of UUS planning. In domestic UUS planning process, it is generally acknowledged that the UUS demand forecasting is an important basic work [CHEN et al, 2007].

## 2. Investigation and analysis of the status quo

### 2.1 Overview

Generally, underground space is an extension of urban development on the ground. As UUS development is complicated, having long term impact and even irreversible consequences, it may need stronger unified planning to ensure rational, orderly, secure and efficient development than what above ground development requires. But comprehensive underground space planning is still at the exploratory stage worldwide. Overseas researches and practices on underground space development focus on the planning and design of special and single underground projects more, not involving the forecast of the underground space demand and the capacity of the whole city [CHEN, 2010].

From the 1980s, domestic researchers began to carry out basic researches on underground space planning. Chen Lidao, Shu Yu and Zhang Dongshan et al forecasted the underground space demand of Shanghai from the perspective of the urban population and on per capita in 1990. Chen put forward views that the

UUS master plan could adopt system engineering method, and the demand forecasting of each unit could be assessed by mathematical models [CHEN et al, 1997].

## 2.2 Summary of demand forecasting methods

There are still few achievements and researchers on UUS demand forecasting in China and abroad. To date, the representative researches are: analysis method of demand model based on land use, demand forecasting based on harmonious development approach, subsystems and single index calibration method, as showed in Table 1.

| Method                                                      | Concept                                                                                                                                                                                                      | Detailed contents                                                                                                                                                                                           | Values                                                                                                                                             | Defects                                                                                                                                                   | Overall evaluations                                                                                                                                                                                                                                                                                                                                                                                           |
|-------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Analysis method of demand model based on land use           | Aiming at the urban central area, it takes their blocks as the unit. It analyzes and summarizes several factors impacting the underground demand, and establishes the demand forecasting model based on them | (1)Location dividing;<br>(2)Demand classification;<br>(3)Intensity assignment;<br>(4)Intensity adjustment combining analysis of factors.<br>(5)Adjustment with status, calculation the actual demand scale. | Full consideration of various factors' affect to the demand.<br>A more scientific and rigorous forecasting process comparing to the other methods. | The operational process should simplify further;<br>The intensity table lacks of sufficient basis;<br>It doesn't reflect the demand feature on functions. | (1)Those methods analyze the major factors impacting the demand for underground space from a overall framework of urban development, and take it as the method's basis;<br>(2)Land-use types and functional types are the two basic directions of demand forecasting. Each type has its special features;<br>(3)The difficult of each method lies in the determination of the core indicators and parameters. |
| Demand forecasting based on harmonious development approach | Establishing the indicator system from the perspective of city system. Respectively applying appropriate forecast method in coordination with the urban planning system.                                     | (1) The overall development: forecasting the functional types and scale of major facilities;<br>(2) The local development: forecasting the functional types and scale of each.                              | Fully considering the relationship between the scale and form of functions;<br>Clear layers and dominant content.                                  | The specific forecasting process needs to be improved a lot in depth.                                                                                     |                                                                                                                                                                                                                                                                                                                                                                                                               |
| Sub-systems and single index calibration method             | Based on the sub-system partition, forecasting each unit and then aggregating.                                                                                                                               | The forecasting of each sub-system adopts mathematical model. And the model's parameters are chose by calibration of the individual indicators.                                                             | It reflects the demand features of functions well.<br>And there is strong practical significances.                                                 | Reasonable determination of the indicators is still a big challenge.                                                                                      |                                                                                                                                                                                                                                                                                                                                                                                                               |

Table 1 Summary of Major Demand Forecasting Methods [CHEN et al, 2007; XU et al, 2008; LIU, 2009; LU, 2009]

## 2.3 Analysis of key issues

### (1) Inadequate essential exploration, missing the key objective

It has been a consensus of the necessity to do UUS demand forecasting. But when it comes to why, the explanations mostly limit to macro level objectives, such as rational, orderly and sustainable development. Compared with the demand forecasting in special plan of population, transport and municipal facilities, UUS demand forecasting apparently lacks substantive purpose with risk of accuracy associated with forecasting results especially at the level of the whole

city, compared with the urban population forecast, the traffic demands forecast and the housing demands forecast in urban planning.

In planning, forecasting is just a means, not the purpose. But in practice, due to lacking of in-depth understanding of the substance and purpose of UUS demand, it leads to simple and isolated forecast. The forecast cannot be effectively translated to the subsequent planning and implementation. It often results in putting the cart before the horse, and to do it just for doing.

### (2) Insufficient basic data and empirical study

It is the basis of demand forecasting to have continuous, complete and reliable underground statistical information, relevant land use types, land prices and FAR. Presently, most of the UUS developments are still at the primary stage without compilation of empirical data. Although there have been much information in relatively developed cities, such as Shanghai and Beijing, it is difficult to access to them by the researchers. It is also very hard to find related data in overseas cities with developed UUS. Due to the shortage of basic data for reference, researches carrying out demand forecasting mostly infer the influencing factors from theory, rarely cover empirical and quantitative analysis of relevant indicators.

### (3) Defective research concept, difficult to cope with the uncertainty of the development environment

The UUS demand forecast is carried out on the basis of an overall development demand of the urban area. Its specific concepts mainly include: making up the ground shortage with underground space, coordination of above and below ground, subsystem or local forecasting for aggregation. But this sort of demand forecast exercise usually put emphasis on regularities of forecasting methods under certain conditions, neglecting the fast-changing uncertain background of specific project in hand at the initial stage. The quantitative forecast results (largely depending on limited experiences) usually lacks practical relevance and corresponding flexibility to meet changing circumstances So it is likely that the results could not represent the actual demand, leading to frequent adjustments and revision to the plan .

### (4) Irregular forecasting method, too many subjective factors

For many planners, it's a common wish to have a forecasting method being rational and scientific, convenient and practical, not only easy, but also helpful. The current methods seek to explore some objective indicators in underground space development. Any sound methods may fail by its flawed indicator system. Whether the indicator is properly calibrated would directly affect the rationality of the forecast results. The indicators are mostly biased by personal experiences as the above restrictions. And in order to

enhance the results' rationality, it introduces the fuzzy form of interval value (sometimes the interval range is very big). This also leads to that the forecast results lack objective criteria and effective validation, and its practical significance is greatly reduced.

### 3. Basic system framework of demand forecasting

#### 3.1 Basic strategies

##### (1) Basing on the four-level underground space demand of urban development

Abraham Maslow's Hierarchy of Needs provides rich inspiration to understand the underground demand mechanism of urban development. As a special kind of physical space, the underground construction needs clear motivation. And as we become more familiar with and accept the UUS, the motivation to develop UUS will be strengthened continuously [LIU, 2009]. The demand for developing underground resources can be understood from four levels: Survival demand, Security demand, Growth demand and Quality demand (Table 2). They range in order from primary to advanced, but are greatly interdependent between levels.

| level | concept         | contents                                                                                                                                                                                                  | features                                                                                                                                                                                                                                |
|-------|-----------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 1     | Survival demand | Municipal infrastructures; transport infrastructures, such as rail transit, underground parks, underground walkways, underground roads and tunnels                                                        | The transport and municipal infrastructures are the foundation for urban survival and development. Enhancing their function to provide healthy and comfortable environment for urban everyday life.                                     |
| 2     | Security demand | Air/disaster defense facilities; storage facilities for energy and materials                                                                                                                              | Make use of the protective features of underground space, improve urban ability to prevent disaster, guarantee urban safety                                                                                                             |
| 3     | Growth demand   | Underground commercial streets, underground public facilities of culture and sports, underground complex, factories, storage and logistics                                                                | The underground space is the extension of ground development. It should make full use of its characteristics, blending and coordinating with the development of ground functions, in pursuit of comprehensive benefits                  |
| 4     | Quality demand  | Shift the municipal pipelines and stations, road and railways from ground to underground, utility tunnel; system of underground walkways, public activity and roads, underground energy, underground city | Shift most functional facilities to underground; show the development trends of deep, systematic, comprehensive and network underground facilities; the ecological and sustainable development at inner space and with the ground city. |

Table 2 The Four-level Underground Space Demand of Urban Development

##### (2) In coordination with the characteristics of planning content, taking account of local conditions

Most of the underground facilities have a strong public characteristic. The government is the main promoter of

underground space development, being responsible for the construction of underground backbone infrastructure. In this regard, the UUS demand forecasting should mainly emphasize the control of public underground space development. While for non-public underground space driven by the market, the focus should stress on setting regulations and providing guidance. The underground space planning includes two levels: the master planning and detailed planning. And the underground development areas include two types: the old city and new district. Correspondingly, the demand forecast method should be different.

##### (3) Focusing on the practical results, combining qualitative with quantitative forecasting

Marx once realized intensively that "There is always reason, but it doesn't always present in the rational form." Whether the demand forecasting method is rational and effective, it must be tested by the practical effects it plays.

The several methods currently used are not entirely perfect, and each has its own characteristics and scope of application. At present, the underground space demand forecasting methods are still mainly qualitative analysis, relying on experience. For cities having reached certain maturity, more comprehensive statistics are compiled and relatively stable trend is formed, it is suitable to adopt quantitative forecasting analysis relying on mathematical models (including time prediction, causal prediction).

#### 3.2 The basic system framework

Urban planning aims to set the direction for future actions and make specific arrangements and organization. The demand forecasting is future-oriented essentially, analyzing and projecting future developments. Its objective is to manage the possible situation in future.

The concept of constructing the basic system framework (Fig. 1) of underground space demand forecasting is to set a systematic approach in carrying out demand forecast. In the current context of Extraordinary Development stage of underground space, the basic system includes two planning levels: underground space master planning and detailed planning of special areas. The framework starts with defining the substantive purpose of the demand forecast to enhance the forecast's actual value, then analyzing the demand characteristics at each level by identifying the forecast's dominant contents. Different forecasting ways and methods could then apply, as appropriate, to further improve its scientific-basis, adjustability, effectiveness and standardization.

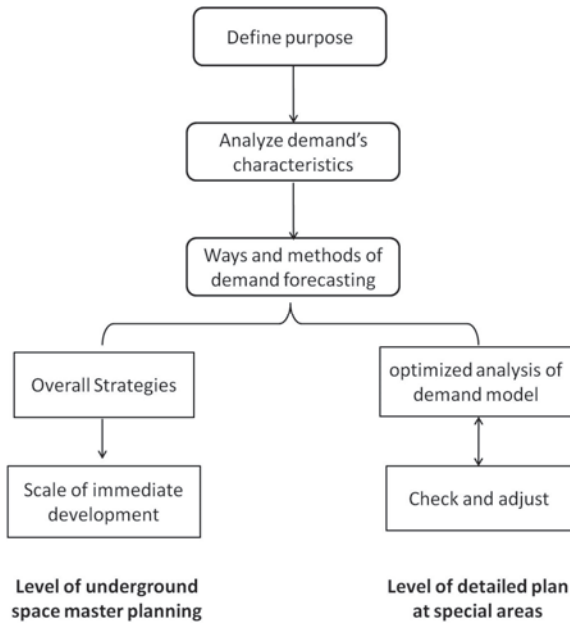


Fig. 1 The Basic System Framework of UUS Demand Forecasting

### 3.3 Level of underground space master planning

#### (1) Purpose of demand forecasting

It is generally recognized that the demand forecast of development scale is a practical problem relating to the plan's feasibility [TONG et al, 2009]. With proper demand forecast, wastage of underground space resources arising from blind development could be avoided so as to achieve sustainable utilization of the resources [XU, 2006]. However, according to the comparative analysis of six cities with better-developed UUS (Table 3), the understanding of demand forecast and demand scale in respective plans are not unified yet. It reflects vague purposes, different methods, diverse contents and messy relationship with demand forecasting. And judging from these cities' practical development situation of UUS, not much importance have been attached to the effects and significances of development scale forecasting which is just by way of a simple quantitative method to express a specific demand figure. The demand forecasting at this level should break through its limitations.

As an important foundational work in planning, the demand forecasting's essential objective is to coordinate the development relationships between above and below ground in the urban area, to guide the rational and orderly utilization of underground space resources, and to provide guidance and basis for the follow-up planning in the future development.

| city            | Beijing                                                                                                         | Shanghai                                                  | Guangzhou | Shenzhen                                                                                                   | Xiamen                                                                                                                                            | Wuhan                                                                                                                                        |
|-----------------|-----------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------|-----------|------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------|
| plan            | Underground space development and utilization plan for the core area of the central city of Beijing (2004-2020) | Underground space conceptual plan of Shanghai (2004-2020) | void      | Underground resources plan of Shenzhen (2006-2020)                                                         | Underground space development and utilization plan of Xiamen (2006-2020)                                                                          | Underground space comprehensive utilization special plan for central city of Wuhan (2007-2020)                                               |
| Demand forecast | Scale of 90 million m <sup>3</sup> in 2020                                                                      | void                                                      | void      | Overall strategy and development goal in short/long term; underground ratio 10~15% of new projects in 2020 | Scale of 21.3~26.3 million m <sup>3</sup> in 2020; underground ratio not less than 10% in short term                                              | Scale of 20 million m <sup>3</sup> in 2020, not including facilities below the roads and municipal pipelines.                                |
| method          | Use the averaged FAR and "underground / ground" ratio from experience                                           | void                                                      | void      | Value from experience                                                                                      | Independent demand between Land-use types and 11 functional subsystems; applying indicator calibration from experience; sum the units forecast up | Analysis method of demand model; demand classification and intensity analysis of land with ground building and no ground building separately |

Table 3 The Underground Space Planning and Its Demand Forecasting in Six Cities

#### (2) Analysis of demand characteristics

##### • Judgment of development stage

The actual development situation of UUS indicates that the economic development level is only a necessary condition to UUS development and utilization. The judgment of underground space development stage should base on external opportunities and internal regularities. The development of underground space is an organic process of growth, from scattered and dispersed development to systematic and comprehensive development, to full development eventually. The development process is like "S" curve (Fig. 2). There are four stages, namely, initial start, propelled development, quick promotion and matured perfection, and each stage has different conditions and characteristics of development.

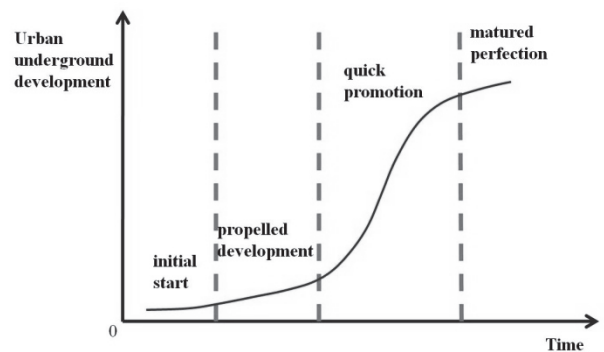


Fig. 2 The "S" Curve of Underground Space Development

- Analysis and forecasting of demand contents

At this level, the underground space can be separated into several independent units according to types of functional facilities, land use and special areas (Table 4). As each unit's demand for underground space being also independent, we can analyze and forecast it separately, and the core of which is to define the contents of various demand.

|                              |                                                                                                                                                              |
|------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Functional facilities</b> | <b>Facilities of subways, underground garage, underpass and tunnels, underground commercial establishments, municipal infrastructure, air defense et al.</b> |
| <b>Land-use types</b>        | <b>Types of residential district, public facilities, roads square &amp; green land, industry and warehousing.</b>                                            |
| <b>Special areas</b>         | <b>Areas of business &amp; offices, commerce &amp; shopping, culture &amp; sports, transit hub.</b>                                                          |

Table 4 Classification of Underground Space by Contents

At the present stage, the underground space demand forecasting should be based on the principle of prioritizing key points, those contents leading the underground space development at specific stage in different cities. It focuses on analyzing the demand's characteristics of the major functional facilities, typical land use types and key development areas, providing basis for latter planning work. The paper will not analyze the demand's characteristics of each unit in detail for lack of space.

### (3) Demand forecasting method

- Overall Strategy

Based on the above, for the main purpose of UUS development and utilization, and according to its general rules, the demand forecasting puts forward the overall strategy for short-term and long-term planning, focusing on solving the problems in urban development status and development process.

The short-term strategy starts with addressing the inconsistencies between urban status and development needs. It makes clear of the general ideas of UUS development and utilization, and identifies the principal development areas and their development directions, and proposes supporting measures as well. The long-term strategy consolidates historical experiences and projects future trends of UUS development and utilization in China and abroad. Based on the state-of-the-art of urban development and the stage of UUS development, it sets panoramic picture of UUS development and utilization.

- Scale of immediate construction

The plan of immediate construction identifies locations suitable for underground development and major constructional projects in the short-term. It pays more attention to the implementation of plan, and seeks to

solve practical construction problems in upcoming projects. In terms of projection, most short-term social, economic factors and trends are more stable than those of long-term, so it is possible to make quantitative demand forecast with greater degree of reliability.

In the principle of controlling the key units, the forecasting bases on the construction plan of dominant functional facilities and special areas. And taking into account of the capacity of the urban area, the development scale forecasting adopts the mind of aggregate of forecasts of individual sub-systems or sub-areas.

### 3.4 Level of underground space detailed planning at special areas

#### (1) Purpose of demand forecasting

As places of concentration of urban activities, special areas are also places rife with development problems. They are in great need of developing and utilizing underground space. However, the underground space of these areas is usually subject to difficult constraints, some of them may be prohibitive on development. As such, detailed plans should be made to develop and utilize the underground space in these areas.

Qualitative and quantitative indicators are the key to detailed planning. Underground space detailed planning must set quantitative limits to accommodate functional facilities. The scale indicator and intensity indicator in developing underground space are closely related to local environmental capacity and infrastructure capacity, with reference to technical and economic indexes, including estimate of project quantities, investment estimate, and evaluation of comprehensive effects and benefits.

#### (2) Analysis of demand characteristics

The underground space can be divided into public and non-public (Table 5) according to the nature of ownership. The demand scale of public underground space is the core of demand forecasting. The public underground space is more closely related with urban life, playing the role of integrating non-continuous urban space and enhancing local efficiency and vitality. In public underground space, the government assumes the responsibility of constructing the non-profit-making part, and the private capital can participate actively in the profit-making part.

|                                     |                                                                                                                                                                                                    |
|-------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>Public underground space</b>     | <b>Commercial space such as shopping malls, supermarkets, shops, restaurants; Transport space such as subway station, entrances, walkways, atrium, nodal square; underground public car parks.</b> |
| <b>Non-public underground space</b> | <b>Basement of building used as car parks, equipment rooms, storage and air-defense; Municipal pipelines for water, electricity, gas, heat and communication.</b>                                  |

Table 5 Classification of Public and Non-public Underground Space

### (3) Demand forecasting method

The demand forecasting method at this level is mainly an optimization analysis of demand model. It should determine the proportion of different functional types when forecasting the scale. Then we can check and adjust the scale with statistical analysis, analogical analysis or cost-benefit analysis. If the results differ greatly, we have to interpret the reasons in detail, and return to re-forecast when necessary.

#### • Optimaization analysis of demand model

The analysis of demand model bases upon land use, and considers a variety of important factors. Its forecasting process is more scientific and rigorous. In particular, it is suitable for detailed planning in special areas. The core of the method is the rational determination of a demand intensity table by land use types. It introduces experiential assignment of expert system as an ad hoc arrangement, and needs more rational improvements. It is feasible to make a large-scale investigation research on underground space of major land-use type in cities having extensive underground space development. It aims to get the reference indexes of underground space development classification and intensity of different land-use type by statistical analysis.

There has been a study of quantitative analysis of underground space development intensity (the ratio of underground built-up area to developed land area) in four central districts of Tokyo. The correlation analysis shows that there is a strong correlation and objective rule between the land price and FAR and underground space development intensity. And analysis of variance shows that the land-use type has a significant influence on the underground space development intensity [LIU, 2009]. When the author makes a special research of demand forecasting in planning a city's underground space, the results of a small-scale quantitative analysis show that the ratio of above ground built-up area to underground built-up area of high-rise residential land reaches an average of 0.24, and its underground space development intensity reaches an average of 0.7. The city's underground space in commercial land usually extends to 1~2 floors, and correspondingly the two indicators are 0.3 and 1.

A primary value of demand scale can be calculated by the method. The scale should reflect the actually functional demand, taking account of building form. In order to enhance its practical use as guidance, the demand forecasting needs to set out the proportion of demand by functions. According to the statistical analysis of the built-up area of various functions in Japanese underground complexes, the proportion of walkways, car parks, shops and others is 24:37:24:15 in average [XU, 2010].

#### • Methods of checking and adjusting

Statistical analysis, analogical analysis or cost-benefit analysis could be used to check and adjust the development scale value. The former two are the most commonly used in practice.

According to the scale data of 13 special underground space development areas (Table 6) collected by the author, the statistical analysis showed that the development ratio (underground built-up area to above-ground built-up area) is 0.31 in average, ranging from 0.2 to 0.5, and the development intensity is 0.63 in average, ranging from 0.35 to 1.

| nom | district                            | Land-use area(km <sup>2</sup> ) | Ground built-up area (million m <sup>2</sup> ) | Underground built-up area (million m <sup>2</sup> ) | Underground d'ground | Develop ment intensity |
|-----|-------------------------------------|---------------------------------|------------------------------------------------|-----------------------------------------------------|----------------------|------------------------|
| 1   | Hangzhou Qianjiang CBD              | 4.02                            | 7.3                                            | 2.8                                                 | 0.38                 | 0.7                    |
| 2   | Hangzhou Chengdong new city         | 9.31                            | 14.26                                          | 3.6                                                 | 0.25                 | 0.387                  |
| 3   | Zhengzhou Zhengdong CBD             | 1.32                            | 4.1                                            | 1.05                                                | 0.26                 | 0.8                    |
| 4   | Chongqing Jiangbei CBD              | 2.4                             | 5.1                                            | 1.12                                                | 0.22                 | 0.47                   |
| 5   | Qingdao CBD                         | 2.1                             | 4.08                                           | 1.27                                                | 0.31                 | 0.61                   |
| 6   | Wuhan Wanguan CBD                   | 7.41                            | 13                                             | 2.62                                                | 0.2                  | 0.35                   |
| 7   | Beijing Chaoyang CBD                | 3.99                            | 10                                             | 2.5                                                 | 0.25                 | 0.63                   |
| 8   | Beijing Lize CBD                    | 2.92                            | 6.5-7                                          | 2                                                   | 0.3                  | 0.68                   |
| 9   | Beijing Fengtai Technology district | 1.81                            | 3.3                                            | 1.4                                                 | 0.42                 | 0.77                   |
| 10  | Tianjin Yuliapu core area           | 3.86                            | 9.5                                            | 2                                                   | 0.21                 | 0.52                   |
| 11  | Beijing Zhongnancun west area       | 0.5                             | 1                                              | 0.5                                                 | 0.5                  | 1                      |
| 12  | Tangshan airport core area          | 3.1                             | 3.766                                          | 1.2                                                 | 0.32                 | 0.387                  |
| 13  | Shanghai Jingansi                   | 0.69                            | 1.35                                           | 0.6                                                 | 0.44                 | 0.87                   |

Table 6 the Statistical Analysis of Scale Data in 13 Special Underground Space Development Areas

The analogical analysis firstly selects the cases having similar features, such as layout and development goal to the one under study in the planning area. Then it checks the forecast's rationality through comparative analysis of the functions, proportion and intensity of underground space development. Meanwhile, it should illustrate the impact of key factors on different cases.

The development and utilization of underground space need to be supported by adequate economic incentives. As majority of public underground constructions are invested by the government, it is essential to evaluate the comprehensive benefits of the forecast results. Cost-benefit analysis is often used to make economic evaluation for those public projects required to quantify indirect benefits. It can evaluate the achievement of the goal of development by quantifying and comparing all costs and benefits in planning terms.

The cost of underground projects is very high, but the indirect benefits are also remarkable, including social benefits, environmental benefits and disaster preventing benefits. However, the broad indirect benefits are so complex that they may not be easily reflected in quantitative and comparable terms. The Underground Space Planning of Beijing has organized a preliminary study on the comprehensive benefits evaluation of underground space development and utilization. There are also other researches on the subject. But the substantive progress is rather slow, and it is still a big research challenge ahead.

#### 4. Conclusion

Facing the great pressure and challenges of large-scale underground space development and utilization, the UUS planning has made considerable progress and will continue to develop in depth and breadth. The study of underground space demand forecasting is the right approach and should be in focus. The method needs innovation, and the idea needs breakthrough even more. The paper mainly focuses on examining and analyzing the status of research and development, and puts forward the basic system framework and methods of underground space demand forecasting. It brings out several new points on status analysis, levels of demand, developmental stages, characteristics analysis and selection of methods. Some arguments of the contents are required to be further discussed.

The author applies some results in specific research on demand forecasting in a city's underground space master planning. The principle and idea of stressing the importance of purpose and effectiveness is to enable the strategy and method being flexible to address the fast-changing characteristic of the "extraordinary development" at initial stage. The author sincerely welcomes more communications with scholars both at home and abroad.

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# ECONOMICS OF UNDERGROUND SPACE DEVELOPMENT IN HONG KONG

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## Abstract

*Strategic planning and technical studies pertaining to development of underground space have been initiated to promote the enhanced use of rock caverns as part of Hong Kong's pursuit of sustainable development. Investigation of potentially viable uses for large manmade underground spaces in rock caverns is not a new idea with several studies previously undertaken. These studies confirmed that the development of underground space in Hong Kong was a viable alternative to conventional above-ground development and one which could offer significant economic, social and environmental benefits. With many kilometres of tunnels already constructed in Hong Kong for the use of road and rail traffic, the technology involved in this process had been well tested. However this had not yet been applied to the formation of underground space on as large a scale as was already common elsewhere in the world.*

*Underground options are typically associated with higher capital and operating costs relative to an above ground option due to factors such as additional ventilation, energy, lighting and safety requirements. Furthermore there are the capital costs for the construction of the cavern itself. Alternatively there are also substantial benefits which provide real incentives to locate infrastructure underground. Key sites that are occupied by utility or government uses can be re-provided in rock caverns to free up valuable land in densely populated areas. The revenues from land sale can be used as an important financing mechanism for cavern development.*

*The release of high value land for other purposes also have numerous non-monetary benefits including minimising environmental, landscape and visual impact and social benefits through provision of open space and low income housing. In some cases, these types of benefits can outweigh the quantified monetary benefits. If the costs for land are properly considered, the underground option can be a viable alternative to an equivalent above ground option.*

## Keywords

*Caverns, Infrastructure, Economics, Costs, Benefits*

## 1. Introduction

The Development Bureau of the Hong Kong SAR Government has put forward a new initiative to launch strategic planning and technical studies to facilitate planned development of underground space aimed at promoting the enhanced use of rock caverns as part of Hong Kong's pursuit of sustainable development.

Investigation of potentially viable uses for large manmade underground spaces in rock caverns is not a new idea. It was investigated as part of the Study of the Potential Use of Underground Space (SPUN) undertaken by consultants of the Geotechnical Control Office (GCO) of the Hong Kong Government in 1988 to evaluate whether rock cavern development could help ease the demand for usable land, the supply of which is severely limited in Hong Kong Island and Kowloon [ARUP, 1989a-d, 1990a-d]. The SPUN study confirmed that the development of underground space in Hong Kong was a viable alternative to conventional above-ground development and one which could offer significant economic, social and environmental benefits.

Following SPUN, Cavern Project Studies (CAPRO) were initiated by the GCO in 1990 including ground investigations for two schemes [ARUP, 1991a,b]. The studies included the Refuse Transfer Station in Mount Davis and the Government Warehouse in Chai Wan.

Several Preliminary Engineering Geology Studies (PEGS) were carried out by the GCO and Geotechnical Engineering Office (GEO) between 1990 and 1992 to assess the potential for relocating various existing and planned facilities to rock caverns, including sewage treatment, fuel storage, abattoir & service reservoir facilities [CHOY, 1990; GEO, 1992, FRANKS, 1992; ROBERTS, 1991a, b, 1992]

Concerns about the growth of various environmental problems made options to contain essential but less desirable neighbouring land uses such as waste and wastewater treatment facilities underground even more attractive. Many kilometres of tunnels had already been constructed in Hong Kong for the use of road and rail traffic, and caverns had also been formed as part of the Mass Transit Railway (MTR) system and for water supply. Thus the technology involved in this process had been well tested in Hong Kong, but had not yet been applied to the formation of underground space on as large a scale, or for as wide a variety of uses as was already common elsewhere in the world.

## 2. Existing Cavern Schemes in Hong Kong

Four cavern schemes and their above-ground equivalent are present in Hong Kong.

### 2.1 DSD Stanley Sewage Treatment Works

Drainage Services Department (DSD) Stanley Sewage Treatment Works is the first example of underground works of its kind in Hong Kong. The treatment works was constructed to treat sewage from the Stanley Peninsula, Tai Tam, Chung Hom Kok and Redhill areas. Part of the original proposals drafted in 1989 was to build and house the facility underground into three caverns, each about 120 m long, 15 m wide and 17 m high. There is also 450 m of access road, ventilation tunnels and shafts [DSD].

The original design was to provide secondary treatment for an average flow of 11,600 m<sup>3</sup> per day of effluent. Construction began in 1990 and was completed in 1995 at a total cost of HK\$410M.

### 2.2 EPD Island West Transfer Station

Environmental Protection Department (EPD) Island West Transfer Station is a cavern facility located in Mount Davis, Kennedy Town. It is a centralized collection point for Central, Western and part of Southern areas of the Hong Kong Island for transfer of waste to the strategic landfill. Major plant and equipment include container vessels, live floor system & compactors, tractors, container handling units, road sweeper, containers, computerized weighbridge system, exhaust ventilation system with odour removal unit, waste water treatment plant and vehicle washing system.

The design capacity of the Island West Transfer Station is 1,000 tonnes of waste a day. The current average throughput is 500 tonnes of waste per day and the cost of processing refuse is HK\$290 per tonne. It was commissioned in 1997 and cost HK\$640M.

### 2.3 CEDD Kau Shat Wan Explosives Depot

The Civil Engineering and Development Department (CEDD) Kau Shat Wan Explosives Depot was completed in 1997. The facility was relocated from a smaller site on Stonecutters Island and included up to eight new magazines, tunnels, utilities, safety equipment and M&E services. Following consultation with Mines Division (CEDD) the final project cost of the development was around HK\$500M.

### 2.4 Western Salt Water Service Reservoir (HKU)

To provide space for the development of the new Hong Kong University (HKU) Centennial Campus an existing salt water service reservoir was relocated into

caverns within the Lung-Fu hillside. The construction protected the environment, released land for the use of HKU and avoided local disturbance as compared to the open-cut option. Important land resources have also been preserved for alternative use by the HKU. The caverns were designed to maximise underground space without disturbing the local environment or causing a visual impact on nearby residents.

The total plan area of the caverns is 4,000 m<sup>2</sup> with a total capacity of 12,000 m<sup>3</sup> for the salt water service reservoirs. Construction of the service reservoirs and associated works commenced in 2007 and took two and a half years to complete, at a total cost of HK\$500M.

## 3. Comparison with Above Ground Schemes

### 3.1 Transfer Stations

Island East Transfer Station is an above-ground facility which was commissioned in 1992 and cost about HK\$410M to build. This cost included two vessels for the transfer of refuse to the landfill as well as pushpit / compactors, yard tractors, landfill tractors, container handling units, road sweeper, containers, computerized weighbridge system, exhaust ventilation system with odour removal unit, waste water treatment plant, vehicle wash system and a recycling facility.

Island West Transfer Station is an underground facility which was completed in 1997 and was constructed for about HK\$640M which included two vessels for transfer of refuse to the landfill. The operating costs were noted to be around HK\$56M per year when it was commissioned in 1997.

The Island West Transfer Station cavern processes about 500 tonne of refuse each day while the Island East Transfer Station processes about 800 tonnes per day. The Island West Transfer Station is roughly equivalent to the Island East Transfer Station in terms of the station area and so a reasonable comparison of the above and below ground facilities can be made.

Island West Transfer Station design capacity is about 1,000 tonnes per day and is running at about half of actual design capacity. The Island East Transfer Station has a capacity of about 1,200 tonnes per day and is operating at around three quarters of capacity. The cost of processing refuse per tonne for Island West Transfer Station and Island East Transfer Station are HK\$290 and HK\$190 respectively. The cost for the Island West Transfer Station is higher than the Island East Transfer Station and partly reflects the additional operating costs for the underground facility.

### 3.2 Sewage Treatment Works

The underground Stanley Sewage Treatment Facility is a secondary treatment plant. In consultation with DSD, it is considered that Sai Kung Sewage Treatment Works is similar to Stanley Sewage Treatment Plant in terms of population served, amount of sewage treated and the level of treatment required. The Sai Kung Sewage Treatment Works and Stanley Sewage Treatment Works currently handle 8,800 m<sup>3</sup> and 8,200 m<sup>3</sup> per day and their design daily capacities are 8,000 m<sup>3</sup> and 11,600 m<sup>3</sup> [DSD, 2009].

The Stanley Sewage treatment facility was built in 1995 for a capital cost of around HK\$410M and is operating at a cost of around HK\$3.5 per m<sup>3</sup> of processed sewage. The plant can process up to 11,600 m<sup>3</sup> dry weather flow of sewage per day. While a comparison of these two sewage treatment works can be made it should be noted that the process employed for the two plants is different. The process used in Stanley Sewage Treatment Plants was selected to reduce the space required.

The Sai Kung Sewage Treatment plant was built in 1988 and covers an area of about two hectares. The operating cost of the sewage treatment process is around HK\$2.0 to 2.2 per m<sup>3</sup> of sewage and is lower than the Stanley Sewage Treatment process. Information regarding its capital costs is not readily available.

## 4. Economics of Cavern Schemes

### 4.1 Overview

The economics of cavern schemes are driven by the costs for their construction and operation relative to the financial, economic and social benefits they produce in comparison to an above ground scheme.

### 4.2 Costs

The costs for the underground scheme include costs for caverns and adits within which the facility will be contained, civil works and the facility itself. Other relatively minor costs are incurred for M&E and design fees. The capital and operating costs are higher for the cavern scheme relative to an equivalent above-ground option due to additional requirements such as ventilation, lighting, access and safety.

There is significant variation in cavern construction costs with current contract estimates suggesting an upper cost range of HK\$1,500 to HK\$2,000 per m<sup>3</sup> for the excavation and temporary support. With a final concrete lining installed the total cost of the construction is likely to range from around HK\$2,500 to HK\$3,500 per m<sup>3</sup>. These costs are for excavation and support only and do not include all the necessary

associated preliminaries and other costs associated with an underground cavern scheme such as mobilisation, access, ventilation, M&E, etc.

Operation and maintenance costs (O&M) are derived from basic cost elements such as power, maintenance and labour and can be further divided into fixed and variable components. O&M typically starts in the year after construction is complete and lasts over the life of the facility. There may be a ramp factor as the facility initially commences and less labour, chemicals, energy etc is consumed while operation is less than full capacity. Engineering estimates indicate that the additional operating costs for an underground facility are some 30% to 50% higher than an equivalent above ground facility.

### 4.3 Benefits

Benefits from the cavern option are often derived from the potential release of the surface land for alternative use which has real economic and/or social value.

In densely populated cities such as Hong Kong, there are increasing pressures on land supply with land costs becoming expensive and prohibitive. In recent years, the number of land sales by the Government has declined as a result of the cessation of periodic public land auctions and sale of land sites under the Application List System (Fig. 1).

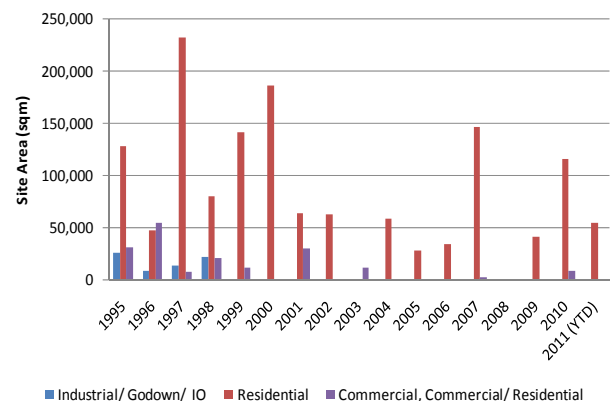


Fig. 1 Hong Kong Annual Land Sales  
(Source: Lands Department & CB Richard Ellis)

New supply of private housing has been declining since 2003 following the change to the land sales system. The formation of new households has outstripped the net addition of private housing for the past seven years and this is expected to remain for the short to medium term (Fig. 2).

Hong Kong residential property prices have increased by 15% per annum since the property market hit its lowest point in 2003 as a result of SARS. Supply constraints and strong market demand for residential property particularly from mainland investors have fuelled high property prices (Fig. 3).

The price of retail space has also significantly increased since 2003 which reflects in part the strength of the local economy, inflows of foreign direct investment and the number of regional headquarters and offices (Fig. 4).

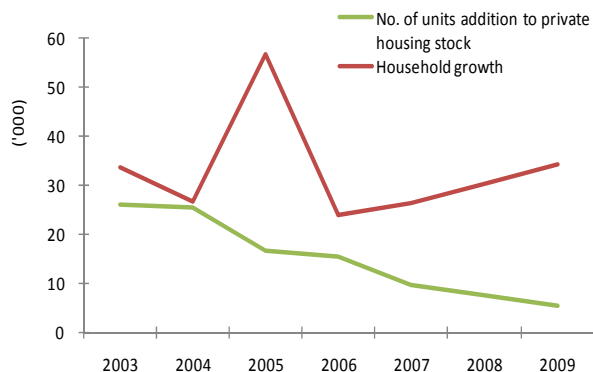


Fig. 2 Net Supply of Housing and Household Growth  
(Source: Lands Department & CB Richard Ellis)

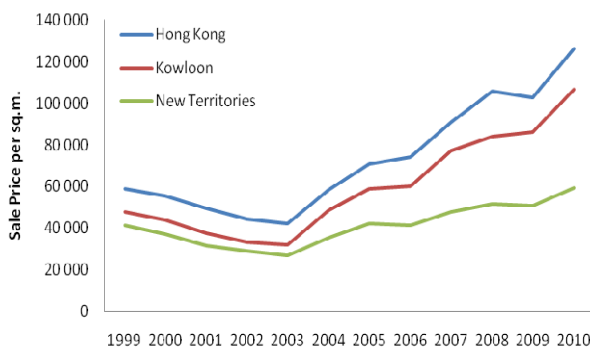


Fig. 3 Average Residential Price  
(Source: Rating and Valuation Department)

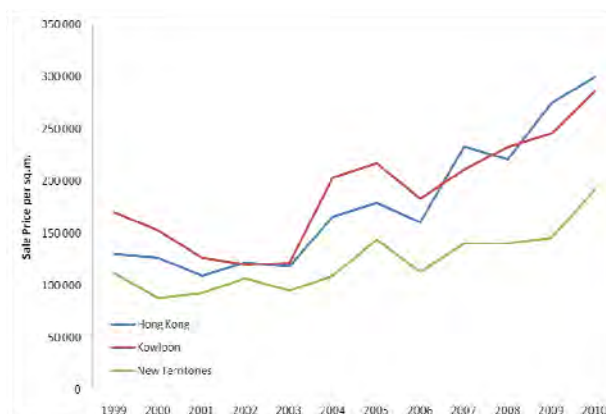


Fig. 4 Average Private Retail Price  
(Source: Rating and Valuation Department)

The strong upward pressure on property price drives land value and it is this factor which makes underground cavern schemes attractive from an economic and financial perspective. Land sale records for selected areas on Hong Kong Island and Kowloon published by the Hong Kong Lands Department indicate that price per square meter of residential Gross Floor Area (GFA) range on average from around

HK\$47,000 per m<sup>2</sup> to HK\$158,000 per m<sup>2</sup> depending on location.

Assuming an average land sale value of HK\$103,000 per m<sup>2</sup> of GFA, a residential development of 40,000 m<sup>2</sup> GFA would fetch a land sale value of around HK\$4 billion. Similarly, freeing land for a development of 500,000 m<sup>2</sup> or more of GFA would generate a land sale value of around HK\$20 billion or greater assuming HK\$40,000 per m<sup>2</sup> GFA.

These are substantial revenue streams generated by Hong Kong's unique property market which provide real incentives to locate infrastructure underground. Underground space offers an alternative method of land supply for certain uses. Key sites that are occupied by utility or government uses can be re-provided in rock caverns to free up valuable land in densely populated areas. Land that is currently occupied by low density development that does not allow multi-level development e.g. service reservoirs and sewage treatment works could also be relocated in caverns to ensure better land utilisation.

In addition to the release of high value land for other purposes and indirect benefits on the value of planned adjoining housing,<sup>1</sup> the utilization of underground space has many non-monetary benefits in comparison to the above-ground scheme including:

- Minimizing landscape and visual impact.
- Minimizing modification of existing landform.
- Minimizing ecological impact.
- Reducing noise and odour impact.
- Providing open space.
- Providing opportunities for low income housing.<sup>2</sup>

In some cases, these types of benefits can outweigh the quantified monetary benefits. For example while low income housing does not generate substantial land sale revenues, it does offer real social benefits to low income households who in Hong Kong have a real need for affordable housing. Similarly provision of open space yields low economic return yet its social benefits to the community are significant.

## 5. Assessment methodology

### 5.1 Overview

A number of factors need to be considered when assessing the economic feasibility of underground cavern schemes.

<sup>1</sup> For example, removing a waste treatment facility would cause property price and hence land value in the surrounding area to appreciate.

<sup>2</sup> Assuming this was the intended use. The main point is that low income housing generates little, if any, land sale revenues. However its social value is significant.

## 5.2 Timing

The economic feasibility of a cavern scheme needs to compare the full whole of life costs for construction and operation to an equivalent above ground scheme. Cavern schemes will have higher whole of life costs and it is important this additional costs is identified to determine whether revenues are sufficient to offset this additional cost.

Individual proposals will have differing time frames to others and this time frame must be varied to reflect the actual proposal with a maximum of 50 years.<sup>3</sup>

## 5.3 Costs

The time profile for capital expenditure needs to be clearly defined so that it can be allocated in the correct period. Typical construction periods range over three to five years and the timing of when expenditure occurs will have an impact on the result due to discounting and inflation factors.

Similarly operational costs are allocated as they occur which is usually on an annual basis. For longer assessments, expenditure for major overhauls or sustaining capital needs to be included at the correct period.

## 5.4 Benefits

Quantified benefits are included and compared with costs to estimate the overall feasibility of the proposal. There will be other social and environmental benefits which can be more difficult to quantify but which should be included wherever possible. One of the main project benefits are derived from the potential release of the surface land for alternative use.

The land value is determined by (i) plot ratio (ii) current property prices in the surrounding locality and (iii) the construction costs associated with the proposed development, which can include site preparation and infrastructure costs. The residual method of valuation is one approach to estimate land value based on estimates of the Gross Development Value (GDV), which is equivalent to the current market value of the development. The land value is represented by the difference between the GDV and the total costs involved (e.g. construction costs, professional team fees, finance costs, marketing costs, developer's profit). Comparisons with similar properties are used to benchmark residential property prices. Other variables such as construction period and void period are also taken into account.

<sup>3</sup> Discounting tends to make any effect too small after 50 years to have a significant effect. If the asset has a longer life span than say 50 years, this value could be captured by allocating it as a residual value.

Similar to costs, the time profile for benefit accrual needs to be clearly defined and allocated in the correct period due to the effect of discounting.

## 5.5 Discounting and Inflation

Whole of life costs and benefits are assessed using a discounted cashflow model where costs and benefits are allocated at the appropriate time at which they occur. A discount factor which reflects the opportunity cost of capital, risk and inflation depending on whether the analysis is undertaken in real or nominal terms, is used to convert future monetary values into Net Present Value (NPV) terms. The formula for the calculation is shown below:

$$NPV = \sum_{t=1}^T \frac{TC_t}{(1-i)^t} - K \quad (1)$$

where:

T = Total time period

t = time at period 1

TC = Capital plus operational costs

i = Discount rate

K = Initial capital cost

From a public sector point of view projects are generally evaluated using a lower discount rate, for example 4%, which reflects the long-run cost of government borrowing. For the private sector where the commercial returns, borrowing costs and risk influence the cost of capital, a higher discount rate is used.

The assessment can be undertaken in real (inflation is excluded) or nominal terms (inflation is included). If inflation is excluded, the discount rate must be in real terms. Alternatively if inflation is included, a nominal discount rate must be used.

## 5.6 Findings and Sensitivity Tests

A project is potentially feasible if the NPV is greater than zero, and the proposal with the highest NPV is preferred. Sensitivity tests are undertaken on key parameters such as costs, land value, GFA, plot ratio and discount rates to test the robustness of the findings.

## 6. Example – Sha Tin Sewage Treatment Works

### 6.1 Background

The Oslo VEAS sewage treatment plant in Norway is located underground. A comparison between the existing above ground Sha Tin Sewage Treatment Work (STSTW) and the Oslo VEAS sewage treatment facilities is summarised below (Table 1). The scale and treatment level for both facilities are similar. In addition, STSTW occupies a prime location in Sha Tin new town with residential developments in the opposite bank of the Shing Mun River at Ma On Shan,

for which occasional complaints on odour have been received.

| Facilities        | STSTW (Hong Kong)                                                                                                                                      | Oslo VEAS (Norway)                                                                                                                                                    |
|-------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Operational since | 1982                                                                                                                                                   | 1982                                                                                                                                                                  |
| Population        | 600,000 (current)<br>830,000 (design)                                                                                                                  | 650,000                                                                                                                                                               |
| Design Capacity   | 340,000m <sup>3</sup> /day                                                                                                                             | 303,000m <sup>3</sup> /day (80mgd)                                                                                                                                    |
| Sewage Treatment  | Screening, Grit Chamber, Primary Sedimentation, Secondary Treatment + Nitrification, Secondary Sedimentation, UV Disinfection Effluent to THEES Tunnel | Screening, Grit Chamber Sedimentation Nitrification Filter, Denitrification Filter Marine Outfall                                                                     |
| Sludge Treatment  | Sludge Thickening Anaerobic Digestion Sludge Dewatering Landfill Disposal                                                                              | Sludge Thickening and Heating Anaerobic Digestion Filter press Biosolids production                                                                                   |
| Footprint         | 28ha (above-ground)                                                                                                                                    | 400,000m <sup>3</sup> (cavern volume)<br>max span 20m, height 16m, 8 caverns total<br>6 out of 8 caverns rebuilt from 1991 to 1995 to meet new discharge requirements |

Table 1 Sha Tin and Oslo Sewage Treatment Works

The STSTW currently occupies approximately 28 hectares of land that forms part of the west bank of Shing Mun River. Surrounding the site are a large variety of land uses including the Chinese University of Hong Kong and the Sha Tin Racecourse. The Hong Kong Science Park is also located within proximity of the site as are a large variety of low, medium and high density residential developments.

The site enjoys excellent visual amenity with uninterrupted views of the Sha Tin Hoi and Tolo Harbour to its north and Shing Mun River to its east. Connectivity is good with immediate access to both Tolo Highway and Tate's Cairn Highway which are part of the Hong Kong Strategic Routes No. 9 and 2 respectively. The University Station of East Rail is also located within proximity of the site.

## 6.2 Capital Costs

It has been assumed that the current STSTW will be rehoused underground to achieve a similar capacity to the current treatment works. The facility is intended to cater to an increase in the population due to the

proposed residential land use of the site that becomes available from the transfer of the treatment works underground. The underground facility has been sized accordingly to meet that capacity and is a like for like transfer of the current facility underground with minor increase in capacity.

Operating STSTW in an underground facility will incur higher capital costs for the facility as well as additional costs for the formation of the cavern, adits and shafts in which it will be contained. These underground structures will also attract some costs in terms of maintenance and operation including periodic inspections. The caverns, adits and shafts for the underground facility to contain the STSTW are assumed to be 545,000 m<sup>3</sup>, 133,000 m<sup>3</sup>, and 20,600 m<sup>3</sup> respectively. The current design capacity is assumed to be 350,000 m<sup>3</sup> to serve a population of up to 840,000.

The general assumptions in the cost estimate include for the following major items in the facility including effluent pumping station, new central laboratory, regional workshop, administration building, transformer room and switch room, chemical storage house and emergency outfall are included under the cost heading "STW Facilities".

The supporting sewage infrastructure for the site has also been considered and there will be some additional items required and these are likely to be associated with the following:

- Possible additional costs of improvement to Tai Po Pumping Station to convey treated effluent water to the new Sha Tin facility.
- Shatin sewerage catchment upgrading works - including pumping, diversion, reprovisioning, etc.
- Re-provisioning of aged sewers and rising mains.
- Construction of new diversion mains from existing to cavern STSTW, upgrading of existing Shatin Main Pumping Station and existing STSTW Inlet Works, and other ancillary works.
- Reconnection or provision of additional outfall to Tolo Harbour.

These costs are also included in the overall costs for the proposed relocation of the facility underground. The costs of any clean up of the site are contained in the contingency allowed for in the cost of the scheme.

The capital cost of the transfer of the STSTW into underground space is estimated to lie within the range of about HK\$14 – HK\$16 billion. The costs of upgrading and reprovision of pipes, pumping stations, site cleanup etc has been included as an additional cost (Table 2).

| Item                                        | Lower Bound        | Upper bound        |
|---------------------------------------------|--------------------|--------------------|
| Cavern Construction                         | 1,600              | 1,600              |
| Access Adits                                | 440                | 440                |
| Shafts                                      | 200                | 510                |
| STW Facilities                              | 10,100             | 11,800             |
| Additional Costs for Underground Relocation | 1,200              | 1,200              |
| <b>Total</b>                                | <b>HK\$13,540M</b> | <b>HK\$15,500M</b> |

Table 2 Capital Cost Estimates (2010 Prices)

Reference:

1. Cavern Construction Cavern - construction cost per m<sup>3</sup> with permanent cast in-situ concrete lining from the SPUN studies and a recent MTR Project
2. Access Adit - The cost of permanent lining per m<sup>3</sup> from a recent MTR Project.
3. Shafts - The cost of forming and lining the shaft per m<sup>3</sup> from a recent DSD project involving raised bore shaft formation.
4. STW Estimate based upon outline costs of MBR facility from known overseas facilities and allowances for local increases and variations in cost suggested by DSD.

It should be noted that the range in cost is based upon a spread of CAPEX that has degrees of uncertainty associated with it. The uncertainty relates to the final sizing and space proofing provisions of the options considered as well as the potential technologies available at the time of the final design that influence the process and facility operation. In addition fluctuations in the cost of materials and construction may also be included within the range associated with the proposed scheme. There are also some uncertainties about the final layout with respect to operational and maintenance provisions that need to be fully developed with the various government departments and this may change the overall size, complexity and layout of the cavern schemes as well. All these contribute to the estimated range in the costs of the scheme.

### 6.3 Operational Costs

There are typically higher operational costs for underground facilities than above-ground equivalents due to the need for greater ventilation and odour control as well as potential accelerated wear and tear of E&M equipment. For this assessment, an additional 50% in operational costs has been applied to underground estimates compared to above-ground facilities to account for these additional requirements.

Estimated operational costs per year of the cavern scheme is estimated at around HK\$540M to HK\$690M per year which comprises HK\$360M to HK\$460M per year for an equivalent above ground scheme plus an additional 50% increase of HK\$180M to HK\$230M per year to account for ventilation, odour control and increased repair/maintenance inside the cavern (Table 3).

| Item                                                    | Lower Bound     | Upper bound     |
|---------------------------------------------------------|-----------------|-----------------|
| Operational costs for above ground facility             | 360             | 460             |
| Additional operational costs for cavern scheme          | 180             | 230             |
| <b>Total operational costs for cavern scheme (/ yr)</b> | <b>HK\$540M</b> | <b>HK\$690M</b> |

Table 3 Operational Cost Estimates (2010 Prices)

Reference:

1. By reference to operational costs for primary and secondary treatments in Advisory Council of the Environmental Paper (ACE 16/94) and taking various local elements into account.

### 6.4 Land

It has been assumed for this assessment that the plot ratio would be 3.0 which takes into account the surrounding existing residential areas and is in line with the New Town Residential R3 standard as set out in the HKPSG. It corresponds to a moderate density development.<sup>4</sup>

The neighbourhood around the site is predominately made up of G/IC and green space. Private residential developments are located in the vicinity and any future development is expected to overlook the racecourse to its southwest, Shing Mun River Channel to its southeast and Shatin Hoi to its northeast. As such this might attract a high premium and therefore a luxury development might be proposed. It is expected from nearby comparable developments that similar property values could be achieved for the site.

Assuming the proposed site will be developed as medium rise luxury residential property with a total GFA of around 420,000 m<sup>2</sup> and with reference to the current sale prices of comparable properties in the surrounding area and appropriate deduction for construction cost and fees, the land value, in broad terms is assessed to lie within the approximate range of HK\$18 billion to HK\$26 billion with an Accommodation Value (AV) at roughly HK\$43,000 – HK\$62,000 per m<sup>2</sup>.

### 6.5 Whole of Life Costs

The estimated whole of life costs over a 3 year construction period and 30 year operational period, discounted at 4% real and presented as present values together with estimated and revenues from land is shown below (Table 4). The additional operational costs of the underground facility are around 50% equating to approximately HK\$2.6 billion – HK\$3.3 billion in present value terms.

<sup>4</sup> Higher plot ratios up to 5.0 are present in nearby residential developments such as Mountain Shore and Sausalito Tower in Ma On Shan.

| Item                                                          | Lower Bound      | Upper bound      |
|---------------------------------------------------------------|------------------|------------------|
| Present Value Capital Costs [a]                               | 12.5             | 14.4             |
| Present Value Additional Operational Cost for Underground [b] | 2.6              | 3.3              |
| <b>Total Present Value Costs [a]+[b]</b>                      | <b>HK\$15.1B</b> | <b>HK\$17.7B</b> |
| <b>Land Sale Revenue</b>                                      | <b>HK\$18B</b>   | <b>HK\$26B</b>   |

Table 4 Revenues and Whole of Life Costs

In total the costs for the underground scheme are estimated to range between around HK\$12.5 billion to HK\$14.4 billion in present value terms. Allowing for estimated additional operational costs over a 30 year assessment period the total cost is likely to range from HK\$15.1 billion to HK\$17.7 billion. No allowances have been made for the cost of additional development for G/IC and other landuses as well as transport connections for the future development.

The results of the analysis indicates that the financial returns from land sales, based on the assumptions adopted, is sufficient to cover the capital and additional operational costs for locating the sewage treatment work (STW) underground and the proposed cavern scheme is potentially feasible.

These findings are from a purely financial perspective however there are additional wider benefits to constructing public facilities underground as described previously. These include the ability to use the land for alternative uses which benefits the public, and the benefits to surrounding property prices and the community by removing the STW to a more appropriate location.

## 7. Discussion

It can be seen that underground options are typically associated with higher capital and operating costs relative to an above-ground option. These higher costs can be a strong deterrent to implementation. However, it can also be seen that if the costs for land is properly considered, the underground option can be a viable alternative to an above-ground option. It is therefore important that full costs of a development are properly considered during the feasibility stage, including the land which could be developed for another use. This provides a real benefit that can be used to offset the higher costs for the underground option.

Underground schemes need to be also considered in the context of the other benefits they provide and not just in land savings. These include the broader benefits to adjoining land and property values in the surrounding area where bad neighbour facilities are transferred underground, the opportunity to provide low income housing, open space and environmental factors. When non-monetary benefits and land costs

are also taken into account, the rock cavern schemes become more attractive.

There are several challenges facing cavern scheme development in Hong Kong that need to be overcome from an economic perspective. These include:

- Ensuring the feasibility stage includes an accurate comparison of above-ground and underground options, particularly the alternative land use values that arises through rock cavern development options.
- Highlighting savings in land and the ability to use it for some other use that benefits the community, which may have a social value and not necessarily an economic one.

A further challenge is encouraging private sector uptake. Although it has been confirmed that underground space has its own benefits in many circumstances and while the private sector generally does not dismiss the possibility of locating underground, they do have concerns in relation to the operation and construction cost of underground spaces which could reduce their profit margins.

Ultimately, the existence of the private sector is to earn a profitable return. If the government were to promote the use of underground space by the private sector, there is a need to provide some type of incentive to do so. This could include allocating a proportion of the land revenues generated through alternative use to fund favourable infrastructure for below ground operation by the private sector.

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# THE ROLE OF ENGINEERING GEOLOGY IN THE PLANNING AND INVESTIGATION FOR ROCK CAVERN DEVELOPMENT IN HONG KONG

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## Abstract

*Various caverns have been constructed in Hong Kong since the 1980's. In all of those projects the importance and input of good engineering geology assessment has been recognised as a key factor for the success of the project. Engineering Geological input for underground space requires an in-depth knowledge and understanding of the geology, ground condition, rockmass and hydrogeology in order to predict and manage the risks associated with cavern and underground development. Early identification and prediction of poor ground conditions can greatly influence the cost and successful implementation of the underground schemes. As a result, appropriate site selection and adequate input from experienced engineering geologists in collaboration with tunnel engineers is fundamental to reducing the risks associated with underground space development. This paper outlines some of the key considerations that need to be explored when engineering geology is used in the assessment and implementation of cavern development in Hong Kong.*

## Keywords

*Cavern, Engineering Geology, Ground Risks, Rockmass*

## 1. Introduction

Engineering geology has a significant role to play in engineering and infrastructure projects in Hong Kong. It is a science that links the geology, including its variability, properties and characteristics, processes, formation and decay, with the changes that man wishes to make to meet an engineering objective. This engineering objective usually means increasing or reducing some load or stress on the ground by means of construction or excavation. The load or stress can be transient or sustained and needs to meet short term as well as long term objectives. The engineering geologist (EG) needs to be aware of the scale of the engineering and the types of excavation that are being considered for the project so that he/she can understand what the implications and risks may be from the geology.

In order to understand the objectives that need to be met in engineering geology we need to consider the definitions that have been assigned to the science. Several definitions are outlined below:

- 'Engineering geology is the science devoted to the investigation, study and solution of the engineering

and environmental problems which may arise as the result of the interaction between geology and the works and activities of man as well as to the prediction and of the development of measures for prevention or remediation of geological hazards.' [IAEG, 1992].

- 'Engineering geology is more than geology that is useful for engineers, it is a type of applied geology in which the practitioners have training and experience in ground problems encountered in engineering and in the investigation and characterization of ground conditions and ground performance for engineering purposes' [FOOKES, 1997].
- 'The application of geological knowledge in engineering analysis, planning, design and construction. Within this field geology is the focus' [PALMSTROM & STILLE, 2010].

These definitions clearly demonstrate that the interaction, as well as the cause and effect, between what is to be constructed and the real world is the key element of understanding in the science. It is also a science very much based upon practical experience, sharing of knowledge, prediction and uncertainty.

How we approach projects and the risks associated with them depends on who is assessing the ground and the experience that they have. Normally a geologist or EG will undertake the initial site characterization. How this person develops and translates the ground model into something meaningful depends on many factors, such as:

- The quality, coverage and relevance of the ground data available, either project specific or archival.
- The EGs experience of that type of ground, either locally or elsewhere, and how this pre-disposes their expectations for a certain type of ground model, be that correctly or not.
- Whether or not the person visits the site and/or whether they review the Aerial Photograph Interpretation (API) themselves.
- The risk appetite of key parties involved, such as Contractors / Clients etc., and how this influences the assessment to be more or less conservative, and how this influence fits in to a Geotechnical Baseline Report (GBR) or Ground Reference Condition report.
- The time, budget and manpower available (in terms of both man-hours and relevant experience) to complete the ground model.

- The software / geology tools available to the person making the assessment.

Where there is limited data the assessment will tend to be conservative and assume a worst case scenario. Where the data is reliable, better quality and at a higher density, the scenario may result in a more realistic assessment.

It is important to understand the differences in training and approach between the pure geologist and the engineer and where the engineering geologist fits into the approach. Generally the geologist is trained to make observations and conclude on those observations by preparing speculative and interpretive models. He should be able to think in 3D and apply geological principles and data in both subterranean and surface contexts. The engineer typically prefers to be analytical and defines the world in terms of numerical models with known material properties and applied factors of safety to account for variations and differences in the real world. The engineering geologist generally falls somewhere in between these two realms (Fig. 1), although even they can have a more analytical or observational bias depending on their training and experience.

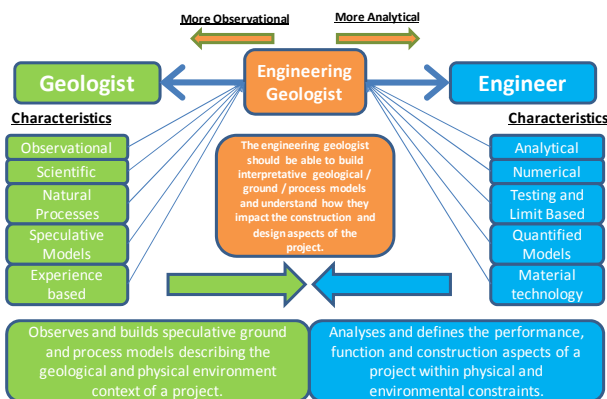


Fig. 1 The Engineering Geologist

Engineering geology has been well embedded in the approach to engineering projects for many years in Hong Kong and, in recognition of this, the Geotechnical Engineering Office (GEO) of Civil Engineering and Development Department (CEDD) published a guidance document on Engineering Geological Practice in Hong Kong [GEO, 2007]. This document provides a number of good examples of how engineering geology has been applied in Hong Kong for various aspects of engineering. Fundamentally EG input is recognized as good practice around the world and, if a project does not embed this in the planning, design and implementation of works then it may be at risk of significant losses if litigation ensues following any ground related problems.

The development of project specific geological, geomorphological and hydrogeological models is

essential good practice in order to adequately assess the impact and interactions an excavation will have with the surrounding ground and terrain. The important thing to note is that the ground model should be developed to accurately reflect the truth in the ground based upon the data provided, whilst the interpretation of impacts may vary depending on the type of excavation and ground support to be employed. In order to adequately review these variations, the EG should also be aware of the limitations and fundamentals of different excavation techniques, as well as the design requirements and performance requirements of the ground.

There will rarely be a definitive model that the EG can present as there is always an element of potential variance in the ground model that needs to be considered. In the case of three boreholes with differing rockhead levels there are numerous potential interpretations that could be developed. One interpretation could be straight lines along rockhead but this would be very unusual in nature, which generally produces non-linear features that wave and undulate, change aspect and angle sometimes abruptly, within the ground. The EG should attempt to develop and fit a typical 3D model of what the geology and ground is like from the available factual data. They should also suggest how it has been formed and how it might continue to evolve or change when impacted by construction and/ or ongoing natural processes.

The EG must be able to give more than one credible scenario to a ground model. The types of scenario explored may be different depending on the type of facility to be constructed. However, a variety of potential conditions must be reviewed in order to test the robustness of the engineering design and analysis. In this respect the engineer should always ask the EG whether there could be a different interpretation or scenario. Indeed all geologists and engineering geologists should be able to provide differences in the model and understand what impacts that might have on the proposed development be it either for a cavern or any other surface or underground development.

FOOKES [1997] in his Glossop lecture further describes the variances in model definition from different experience levels and perspectives, including those from geologists to geomorphologists to engineers, and presents good examples of how geological models and interpretations should be considered for different environments and geological domains.

An example where the interpreted ground model did not realistically represent actual conditions is the Tai Lam Tunnel case presented by GEO [2007]. Despite the as-built geological cross section appearing to present significantly worse ground conditions than the pre-construction geological cross section, the estimated amount of temporary rock support at the design stage was about three times higher than that actually provided.

Indeed, on many other tunnelling projects it seems that the amount of temporary support installed is in the order of 2 to 3 times less than estimated by Consultants during the design. This conservatism could arise from various reasons, such as limited data availability during design, the scale effects of assessments based borehole core runs as opposed to the full face excavations during construction, summation of joint sets from boreholes that are offset and not coincident within actual area of ground to be excavated or simply as a result of pessimistic interpretations to avoid claims.

The geological model incorporates various attributes that need careful consideration. The rocks, their defects or discontinuities, structure, groundwater etc and the various geological processes and hazards all combine to formulate the model. The EG needs to synthesise and relate these elements together in order to provide an understanding of the mutually contributing effects that bind them together. Figure 2 below shows how the model interacts between the various elements.

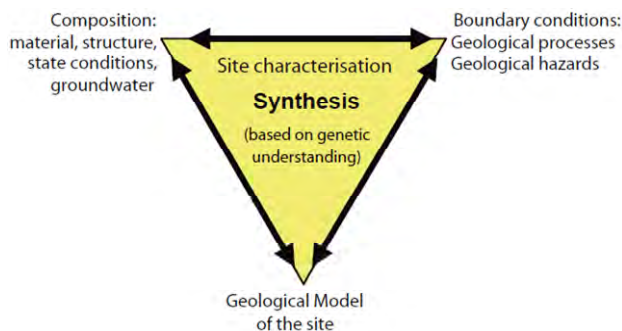


Fig. 2 Key Elements of the Geological Model  
[BOCK, 2006]

## 2. Elements of a Cavern Scheme

The project brief needs to be fully understood by the various parties involved in the design and the EG needs to liaise closely with the design engineers to understand the scope and layout so that they can fully develop a model encompassing the various geological and ground risks that could be encountered. In order to ensure this they need to work closely with the tunnel/ cavern engineers at all stages of the planning and design process.

### 2.1 Location

The location and layout of the cavern should be reviewed and optimised by the EG in order to identify areas of high quality rock mass away from major faults and photo-lineaments. This will allow the designer to minimize costs, improve the construction programme and result in a higher degree of confidence on the ground conditions to be encountered. However, the recommendations from the EG will need to be balanced against various other drivers controlling site location, which may have a higher priority in terms of dictating

where an underground space could be housed. These may include access requirements, land ownership, environmental impacts etc.

An example where the EG assessment enhanced the location and layout of a development is the Island West Transfer Station built by 1997, where a weakness zone was identified towards the end of the initial cavern site. While it was important to ensure the size of the cavern adequately fitted the end use process, the design also recognised that the cavern should not extend too close to the weakness zone.

When planning a cavern layout, the EG and engineer should also consider, identify and include areas for possible future cavern expansion to meet the Clients long-term expectations, where necessary.

### 2.2 Purpose of the Underground Space

The type of facility to be placed underground may necessitate different considerations to be made on the hydrogeological or ground models developed. For instance, caverns developed for underground storage of LPG, oil/gas or nuclear waste will require careful consideration of the hydrogeology to ensure effective containment and isolation of the facilities whereas other underground facilities may require less stringent control of groundwater inflow. The EG, in conjunction with the engineer, needs to identify whether there are any elements of the facility that require specific attention and consideration when developing the geological and hydrogeological models.

### 2.3 Entry to the Underground Space

Efficient and convenient access is a vital component in the development of underground facilities. The portals providing access and egress should therefore be located in safe accessible areas with good links to the existing transport network, where possible. Due consideration of future land uses, as well as provision for emergency and construction traffic should also be accounted for when determining a convenient entry point to the hillside.

The EG needs to develop the geomorphological model as well as the morphogenetic model to understand the risks associated with landslide threats to the portal. Rockhead levels and soil and rock conditions at the portal are also important to define as these will dictate the type and amount of site formation works required.

### 2.4 Scale and Size

The EG should consider the scale and size of the facility with the ground model, its rockhead, rockmass and discontinuity conditions. Layouts should be optimized in terms of their orientation in order to minimise the impact of weaknesses in the rockmass, improve the

overall stability and reduce costs associated with rock support.

The benefit of going underground is the ability to plan and develop 3D layouts with different levels, different stacking arrangements, including vaulting and various other configurations that can be considered to maximise the space available. The type of facility and process within it may dictate the layout but the EG should be able to advise on the merits of utilising the space in a three-dimensional manner to make it as efficient as possible.

### **2.5 External Impacts – Above and Below Ground**

Whilst the main focus of an engineering geology study for an underground development will be on the ground conditions within the excavation itself, the study should not overlook the impact that construction activities could have on above-ground features. The surface area overlying a cavern may include boulders or landslide susceptible terrain and construction using drill and blast methods could result in ground vibrations that induce instability. The EG should determine the risks of instability and any potential stabilisation measures necessary.

### **2.6 Lining and Support**

The EG should review and advise on the most likely rockmass quality distribution along the alignment and advise how that can be reasonably optimised and rationalised into a series of appropriate classes for the assignment of temporary rock support measures. This should extend beyond the investigation and design stage and the EG should work with the Contractor to assess the ground conditions as the excavation proceeds to verify and check the ground models and geology to ensure that the design assumptions are valid.

In this respect the EG should be involved in rockmass mapping and assessment of probe hole drilling during construction. The ground model should be seen as a live document and the EG should be taking the opportunities afforded during the construction works to develop and improve the models and the potential impacts this may have on the construction works.

Drained tunnel linings, which are commonly employed in cavern schemes, need to have limiting inflow criteria defined for the underground space such that they will not significantly impact surrounding areas. To achieve this the hydrogeological models developed need to show both transient and steady state groundwater impacts and the EG should explore near field and far field potential impacts from the underground scheme. The EG should also advise on those areas where groundwater inflow needs to be more tightly controlled, such as fault zones and open potentially transmissive rock joints. To identify such areas they will need to look

closely at the rock core and televiewer data and delineate zones of rock mass where groundwater flow may be concentrated within the rockmass.

### **3. Initial Engineering Geology Planning for a Cavern Development**

Detailed examples of the process for developing a site for potential cavern usage and producing an Engineering Geology model are outlined in detail in the various Preliminary Engineering Geology Studies (PEGS) produced by GEO for a number of end-uses such as GEO [1992]. These identify the basic elements that need to be considered during the initial review of sites in order to determine their feasibility for cavern development. The specialist input required from the EG includes the identification of the key geological and ground model elements through reviews of existing data, existing maps and publications, field reconnaissance and mapping and the evaluation of the surrounding terrain and how the development will fit within this. Further reference on this matter can be found in the aforementioned PEGS reports and the Engineering Geology Practice publication [GEO, 2007].

Some engineering geology principles can be used at the regional planning stage to aid the selection of where cavern development can be carried out more easily. These were recently applied under a consultancy agreement (No. CE66/2009) with the GEO/CEDD in the study of the Enhanced Use of Underground Space in Hong Kong, where general engineering geological principles were used to develop a Cavern Suitability Map for HKSAR.

Experience in Hong Kong and globally has identified that faults and shear zones frequently result in poor rockmass quality and adverse construction conditions for underground developments. Under Agreement No CE66/2009 a territory wide search of such features was undertaken through reviews of existing data and vetting of large numbers of drillhole records. Where identified, such features were buffered to represent zones in which higher development costs could be anticipated. Photo-lineaments were also identified as they too comprise potentially adverse geological features, such as faults and shear zones. Whilst it was recognised that they may also comprise geological contacts, dykes, mineral veins, etc., at a regional scale it was deemed that they should be less suitable areas for cavern development.

Poor ground and weak rocks also present difficulties for cavern development and it was concluded in Agreement CE66/2009 that the weaker sedimentary and metamorphic rocks in Hong Kong had characteristics that would tend to increase the cost of underground development, so were less favoured than the stronger and more resistant granitic and volcanic rocks.

- Carry out or review aerial photographs in detail and visit the site to identify any key geological or geomorphological features that will have an impact on the portal or cavern layout. The aerial photograph (Fig. 3) below shows a number of rock exposures where the rock structure can be easily seen. This can be used to assess the rockmass and the orientation/location of any major weakness zones that may intersect any underground facility located nearby.
- Review existing ground information and identify locations with minimal geological and ground related constraints and threats for the proposed project. The relevance of existing GI data to an underground scheme will vary as many drillholes may not reach rockhead or will only include minimal (5 m) drilling into rock. Careful planning of additional GI would be needed with EG input.
- Develop preliminary geological and hydrogeological models devoted to identifying the variations in the ground and what impact it will have on the project and what impact the construction will have on the ground. The hydrogeological model shown in Figure



- Identify areas where there is significant uncertainty that needs to be explored in more detail or declared as potential areas of risk?

- Consult with the Hong Kong Geological Survey on the current geology and ground conditions associated with the site.
- Plan and implement phased GI to reduce the ground related risks to confirm and test the geological model in stages, all within time and budget constraints, use horizontal directional drilling, use angled and vertical core holes to recover samples for lab tests, plan in-situ testing to confirm permeability profiles, rock modulus, in-situ rock stresses, joint orientations, ground response and condition. The EG should review the data to ensure that it is consistent with the observed geology. For example when rock strengths are described from the cores is that consistent with the UCS results and point load test results.
- Ensure all aspects of the Engineers design intent and requirements are known with regard to above ground space planning in terms of scale and extent.

#### 4.2 What Ground Conditions Have to be Anticipated in the Design?

##### Portal / Vent Structures

- Place the site into some geomorphological context to identify potential ongoing processes and risks. The geomorphological model in Figure 6 for a typical hillside in Hong Kong shows where the various hazards could exist for a portal. It may also provide clues to the preferred location to target thin soil cover.

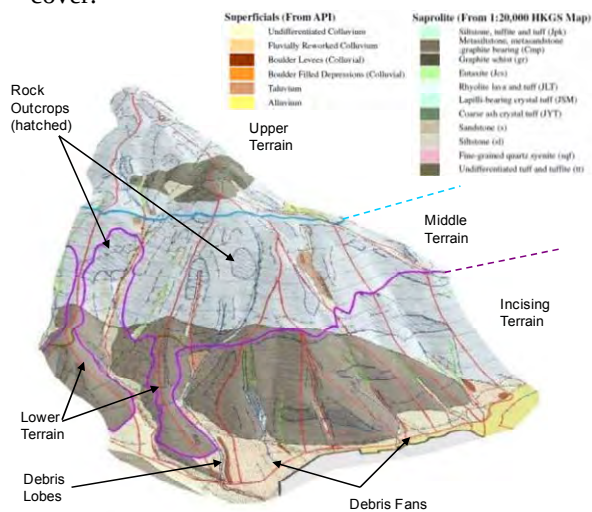


Fig. 6 A Typical Geomorphological Model for a Hillside Developed by the EG from the API and Field Mapping

- Identify interfaces in the ground such as base of fill, colluvium to saprolite, saprolite to Rockhead etc and their variability and extent.
- Ensure that correct definitions of rockhead are used and the use of computer software to develop contours of the rockhead surface have the correct method and analysis parameters with limited search radii (equivalent to 3 to 5 times GI density) and number of points (limited to try to triangulate) to

constrain the contours. Include data that does not confirm rockhead but is very deep, reference this data for context? Add data where the model suggests that a deep weathering trough will occur but note the data speculation? Figure 7 shows a rockhead profile with a deep trough running through it. Some boreholes through this trough may have not encountered rockhead yet still penetrated below the contours. The EG should review and decide how this data should be presented, included in the model referenced to ensure that extrapolation of the rockhead is noted. The EG should check that the contours developed by the engineer do not extend or extrapolate too far from the actual data. The second plot shows limited search radius where the data provides a better representation of the potential reliable rockhead model extent for the project. Statistical models and variance can usually be output from the computer software and the EG should check the accuracy of the data and the model with the interpreted geological model.

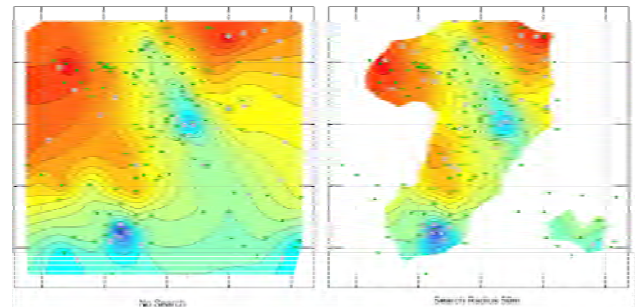


Fig. 7 Rockhead Contours and Model Showing Potentially Reliable and Unreliable Extent of Data

- Assess whether man has altered the hillsides nearby that may have changed the surface drainage and left loose fill deposits that could affect the site formation?
- Consider past known problem ground at portals such as faults intersecting the portal, quartz monzonite weathering producing instability. High clay content and unfavourable sheeting joints leading to potential instability, refer to GEO Reports and TGN.
- Where are the seasonal groundwater levels for the portal design, need to ensure adequate monitoring and performance of the slope to satisfy the design.
- Avoid locating caverns close to zones where large groundwater inflow could occur, consider soil pipes at interfaces and larger seepage / groundwater flows at rockhead. If it is in rock then is it really a problem and what seasonal groundwater variations exist? Will it become a problem later, so do I need to grout now and what control measures do I need to define?

##### Caverns and Adits

- Identify rockhead interface, similar to portal approach to ensure adequate cover to the cavern. It is usually accepted that a minimum of one times the largest span or height, whichever is the larger of rock cover is required above the crown of the cavern

or adit. The EG needs to consider rockhead in all dimensions and should draw geological cross sections through the narrowest cover zones or nearest points to the ground. Where streams and valley features cross and project towards the underground excavation the EG should check rockcover and visit the streams to identify exposed rock for mapping and whether there are any clues to why the stream or valley was there in the first place.

- Identify extent and thickness of any mixed ground and zones of corestones to define areas of higher ground risk for adits and cavern cover. Rock stress should have been substantially lost in those areas and normal soil loading conditions will tend to dominate any numerical modelling in those zones.
- Use surface mapping as much as possible, looking in stream courses, existing rock cuttings and steeper rock terrain (subject to safe access) to characterise the rockmass. Figure 8 shows a rock slope photograph with jointing reference marks to link recorded data from the EG with the rockmass on the slope. Jointing should be displayed on stereoplots to allow further assessment of portal and cavern kinematic analysis. The EG should take note of joint and geological feature persistence (typical ranges shown in Figure 9 for main geological features / discontinuities) which is important in assessing the scale and size of potential impacts of the jointing on the underground works. Remember that the infill and jointing character may be different below ground, at rockhead and within fault / shear zones.

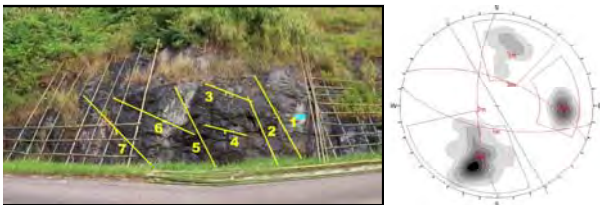


Fig. 8 Typical Exposed Rock Mapping with Overlay Data Display of the Photograph and Stereoplot (Safe access and/or rope access may be required for very steep slopes)

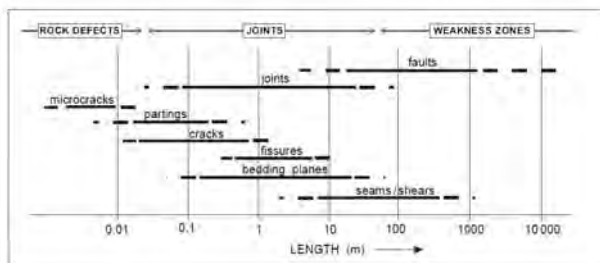


Fig. 9 The Main Types of Discontinuities According to Typical Persistence. [NBG, 1999]

- Identify weakness zones such as faults / shear zones that intersect the underground space and characterise the feature for rockmass condition and groundwater flow and inflow potential. Has the fault had various phases of movement and/or rehealing? Is jointing

nearby in the rockmass very closely spaced, when does the influence of the fault diminish, is jointing on one side of the fault different to the other? A good example that shows how within constraints of alignment for the West Island Line that engineering geology input has redefined the construction and rock support model at Sai Ying Pun Station [CHIU, 2011]. A fault zone was noted to intersect and run across the end of the station cavern with the result that the main cavern location as well as the layout of the end of the station was changed to minimise the risks associated with this feature. Provision has also been made in the construction phasing to use exploratory adits to check and confirm the scale and integrity of the fault zone as well as provide as necessary ground treatment and pre support prior to opening up the whole cavern excavation.

- Assess joint orientation data in detail and the stability of the rockmass in the underground excavation. Consider nearby surface rockmass and look to see what might fail if it was excavated, visualise the problems in the nearby surface rocks if available. Look at the moulds of rock and what is missing from the rockmass as those were probably the easiest pieces to remove and hence most unstable. Consider how those rock moulds may interact with an underground excavation, the working face sidewalls and roof. Identify typical block sizes within the rockmass and whether it varies or is concentrated in locations near faults or shear zones.
- Provide joint orientation and property data to input to wedge stability software to check the stability of the excavation, especially complex zones of cavern geometry where rock stresses concentrate and allow more movement and greater potential instability. Figure 10 shows potential rock wedges and support requirements. This allows the EG to verify and check the rockmass on site for wedges and features that need to be bolted and supported.

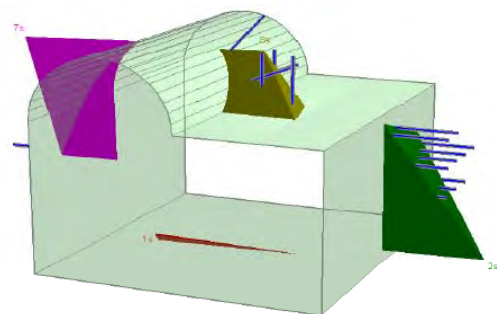


Fig. 10 Wedge Geometries at Various Portions of the Underground Excavation

- Look at weathered and ironstained joints to see whether groundwater has moved vertically or horizontally within the rockmass, use this to assess hydrogeology / joint fracture flow models. Assess whether these features cross the cavern or adit and what might happen when uncovered. Remember seasonal groundwater flow differences might occur.

- Consider rock properties such as mineralogy, strength, weathering grade, flaws and discontinuities.
- Consider how the drilling may be affected by jointing orientation and how jointing may affect blasting design fragmentation and overbreak. Only use material failure UCS results to describe rock strength. The EG should check to ensure that the physical descriptions of the core match the test results and that spurious data is rejected. Figure 11 below shows the significant difference that can result from different modes of crushing failure with unreliable partial or joint controlled failure shown.

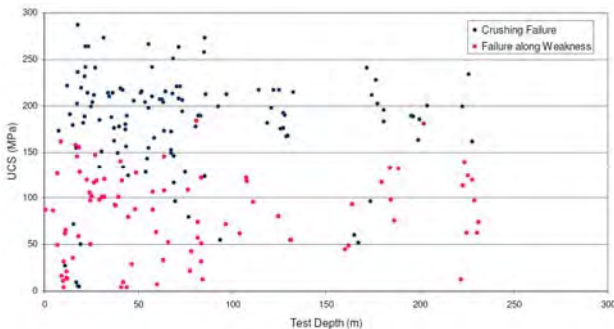


Fig. 11 Comparison of the Differences in UCS Failure Mode for a Set of Rock Data

- Physically look at the rockcore in detail to identify and assess the jointing condition and parameters, if no core is available at the preliminary stage then use the core photographs with combined summary plots to undertake the review. Prefer not to use only borehole logs, as descriptions are usually too general.
- Confirm how the in-situ stress regime relates to the joint orientations within the rockmass and for the orientation of the underground excavation. Input stress information to the numerical modelling. High horizontal stresses are likely to be found in general rockmass zones that aid the arching effect for arched excavations but may be problematic for flat joint derived roofs. Near major faults the in-situ stresses need to be considered in more detail.
- Look at geological interfaces, Dykes and contacts may or may not be different, basalt dykes can have smoother joints or different joint infill and hence more unstable blocks, they may also have deeper weathering at surface and sheared margins so need to consider them in more detail for stability assessments and groundwater inflows along contacts.
- Identify and consider past known documented problem ground in tunnels and caverns such as collapses and instability in faults, flat jointing that develops flat slab roofs needing more support, high groundwater inflows in sub-sea tunnels, refer to GEO Reports and TGN.
- The EG should provide rockmass sketches / zoning and parameters to the engineer to carry out the numerical modelling model in 2D, especially UDEC analysis. The EG should remind the Engineer of the 3D impacts of the rockmass and any out of sections joint set impacts. Rockmass domains and any

weakness zones should be modelled to present the ground and geological model as accurately as possible. The model should be developed to look like the anticipated rockmass zoning and layout of joints and anticipated discontinuity persistence's. The geometry of the joints and discontinuities are outlined below in Figure 12. The EG should prepare and verify this model.

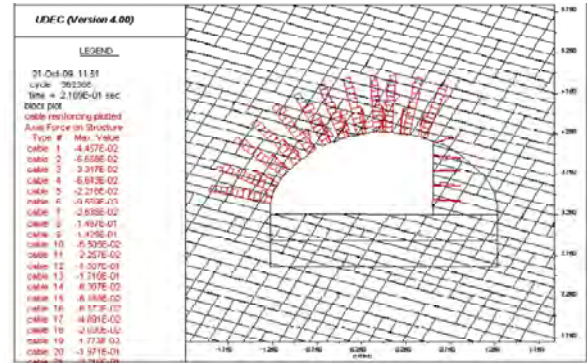


Fig. 12 Typical UDEC Model Showing the Jointing Geometry with Support Model Included

- The size of the rock block is important to consider in developing the rockmass model. Agree with the Engineer what rockmass assessment model is applicable, Q for temporary support, RMR/GSI for modulus etc and understand the parameter input variance and differences. Document the variation in rockmass for the alignments. The Q or rockmass assessment should be assessed and displayed on the long geological section of the underground alignment and related to the geology and the potential hazards that the project will encounter.

#### 4.3 What Surface Engineering Geology Risks Need to be Considered?

- Natural terrain hazards versus location suitability versus cost to mitigate the location to connect to transport networks.
- Consider scale, size, direction and flowpath of potential hazards, can they be designed out for minimal cost / impact or better to move the portal? The digital terrain model (DTM) as shown in the example below (Fig. 13) can provide information on potential landslide areas and provide more optimal placement of a portal. The DTM can also identify steep terrain that might be rock or shallow thin soil cover. LiDAR can aid this assessment process and provide more accurate modelling.

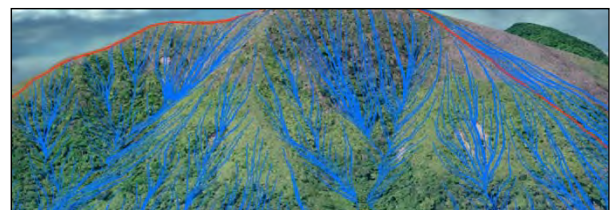


Fig. 13 Landslide Trajectories Simulated on a Hillside

- Avoid locating cavern adit portals, where possible, near drainage lines due to increased seepage from the ground due to seasonal saturation.
- Avoid locating cavern adit portals, near geological poor ground and deep weathering to reduce soft ground excavation / tunnelling and landslide hazards.
- Boulders that lie close to portal cuts and above the underground blasting works need to be assessed for their stability. For blasting vibrations, the boulders need to be very close to falling over with their centre of mass and rotation point almost coincident to each other to be potentially unstable. They must also lie on steep slopes > 40degrees or have significant falls to provide enough energy to slide, roll or bounce.

#### 4.4 What Underground Engineering Geology Risks Need to be Considered as Important?

- Weak ground, low stress zones such as faults, shear zones are important to identify or anticipate. These have a much larger impact on the excavation when combined with significant groundwater inflow. The size of the underground excavation will be affected differently. Detailed investigation of the feature prior to underground excavation is essential to cater for contractual risks as well as anticipated excavation methods. Exploratory pilot tunnels can be used to map out the condition of the feature prior to full excavation of the cavern. Mapping and probe drilling assessment should be carried out with EG input to define the character of the fault at the incident depth for the underground excavation.
- The properties of wide fault zones with potentially weak ground may be difficult to confirm. Full characterization should be attempted but may not be possible due to depth of cover and location during the planning and design stages. Flexibility in specifying the temporary underground support measures to deal with the poor ground during excavation is often the best option. Ground treatment of localised poor ground zones from tunnel level during excavation is quite often the best option. Advance probing must be carried out with multiple fan probes to characterise the fault zone. The EG should be combining all relevant observations from surface mapping, existing boreholes, probeholes, coreholes and underground mapping to provide a model of the fault. Advice on the potential severity of the ground should be made for risk considerations / sharing but should be reviewed continuously during construction.
- Sub-horizontal or shallowly inclined persistent transmissive joints may be close to rockhead and controlled by sheeting joints.
- Sub parallel persistent jointing and weakness zones running along the alignment of a tunnel or cavern will have greater influence on stability, so consider orientation of the underground space to reduce risks and improve cost effectiveness. Figure 14 below shows the main discontinuity trends of data for a

site. If the underground space was parallel to the main trends the potential joint weaknesses and instability could be increased. Grouting performance can be affected so the EG should be careful in assisting the Engineer in choosing the correct orientation for the underground space.

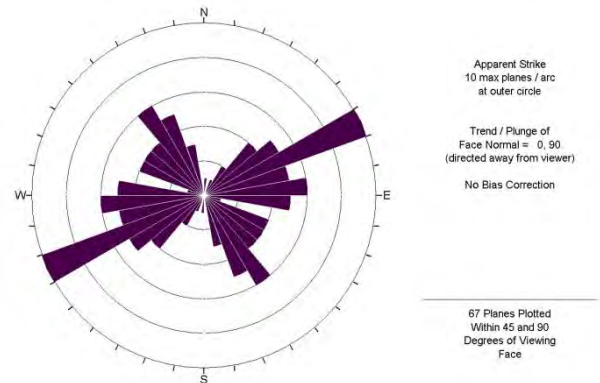


Fig. 14 Discontinuity Rosette Plot for Comparison with Underground Space Orientations

- The EG should assess potential kinematic instability from joint orientations and anticipated persistence of the discontinuities relative to underground geometries and directions. He should carry out mapping and confirmation of the rockmass / discontinuities and “design as you go” temporary support as construction proceeds. He should optimise the temporary rock support classes with the engineer to match the anticipated range of classes for the rock along the alignment.

#### 4.5 How Will the Main Excavation Techniques be Affected by the EG?

##### Portal / Vent Structures

- How will the ground react to excavation in soil? Are there any unfavourable weaknesses, clay seams etc. that could be present, are they persistent and widespread? Will they be daylight in the excavation?
- What physical properties do the soil horizons have, soft, hard, weak, brittle, elastic, erodible, etc?
- Is there unfavourable jointing that will need extra support or produce instability in the excavation and portal area? How persistent and what orientation?
- Will rock excavation leave the rockmass damaged and how will that need to be treated?

##### Caverns and Adits

- Consider what drill and blast will do to the rockmass and how broken it becomes around the excavation, its secondary fracture development and instability.
- Rockmass rating systems should be used with experienced EG / Tunnel Engineer input to the support of the underground space.
- Excavation phasing and approach should be confirmed by numerical modelling for large excavations with 2D and 3D modelling where appropriate. The EG needs to check and confirm that

the anticipated excavation and support sequence is realistic and fits the ground model and how the rockmass deforms and stresses redistribute, the model needs input from in-situ stress regime, how much breakage of the rock occurs during excavation, joint orientation and persistence.

#### 4.6 What Temporary and Permanent Support Needs to be Considered?

##### Portal / Vent Structures

- What soil or rock needs to be supported, how variable and extensive is it?
- Have the geological interfaces been identified to ensure that the slope / site formation design safely matches those interfaces eg. Where rock cut is anticipated it should be present instead of soil?
- What rockmass will be found around the portal?
- Where is the groundwater, in the soil, perched, in the rock or in a fault zone?

##### Caverns and Adits

- Identify rockmass zones that will be encountered within and around the underground excavation.
- Always consider the ground conditions and working face orientation and stability and how kinematically, rock blocks might present a danger to the tunnellers?
- Where is the groundwater and where are the voidage and conduits within the rockmass that need to be grouted to meet the hydrogeology constraints?
- Provide advice on the geology and ground conditions and overall classes of rock support that will be required for the specification and the contract.
- Provide input to the Ground Reference Conditions Report in terms of overall geological and ground conditions uncertainty and acceptable limits for the expected ground conditions of the underground excavation. Use reasonable approaches and quantified assessment of the variance in the ground conditions based upon “most reasonable ground models” although this can be influenced by external factors such as Clients appetite for risk sharing.

#### 5. The Engineering Geologist and Ground Risk

The engineering geologist should develop the initial register of ground related risks and hazards for a cavern project. In order to ensure that relevant and realistic risks are identified the risks assessment should be made by someone with experience in applying engineering geology assessment to slopes, site formation and underground projects.

The engineering geologist should also identify what level of variance and confidence there is within his/her model based upon the available factual data that has been used to develop the model for the project. Descriptions and statements should be made of anticipated problem ground to inform the engineer and contractor of the scale and magnitude of the geological

features that are likely to be encountered. Statistical approaches can be provided, where appropriate, to substantiate the variance that could be encountered.

In particular he/she must be involved in the preparation of the Ground Reference Conditions Report or Geotechnical Baseline Report where the Contractual statements and baselines are made on the predicted and estimated ground conditions.

#### 6. Conclusions

Consideration of Engineering Geology in rock cavern development is fundamental to the adequate assessment and understanding of the ground related risks associated with underground development. Implemented well with experienced Engineering Geologists it can provide risk avoidance and strategies to manage the risks associated with underground excavation.

Various key issues in connection with the Engineering Geology assessment have been outlined in this paper that provide pointers for consideration of future engineering geology assessments for underground cavern development in Hong Kong.

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# GEOTECHNICAL INVESTIGATION FOR THE SECOND AVENUE SUBWAY IN NEW YORK

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## Abstract

The Second Avenue Subway Project is a major capital expansion project of the New York City subway that will provide a dedicated line for the east side of Manhattan. The project is approximately 13.7 km long including 16 stations, and its estimated cost is about \$17 billion. Under the current design of the whole subway route, 10 stations will be cut-and-cover and 6 mined caverns. The excavated diameter of the bored tunnels is 6.6 m and the caverns span ranges from 12 m to 21 m. As the geology of Manhattan varies along its length, the subway will pass through both hard rock and soft ground and there will be multiple rock/soil interfaces along the alignment.

The philosophy behind the construction methodology was to minimize the impact at street level on neighboring communities and businesses during the construction period. With most of the work being done within the right-of-way of Second Avenue, the largest impact is related to the maintenance and protection of traffic and street restoration. Because of the nature of the work, close proximity of high rise buildings, critical nature of adjacent utilities, the characteristics of the ground along alignment, and the visibility of this project, strict performance criteria and limitations were imposed and comprehensive instrumentation and monitoring programs were designed to ensure compliance with action and trigger levels to protect third parties for noise, vibration, subsurface movements and protection of overlying utilities and structures.

A comprehensive subsurface investigation has been carried out that included archive searches, rock exposure mapping, test borings, in situ soil and rock testing, instrumentation, monitoring and laboratory testing over the full length of the alignment. This paper presents the geological techniques that were used to investigate soil-rock-groundwater systems in order to advance the understanding of the fundamental structural geology of central Manhattan. It also briefly discusses some details of planning, design and construction of the first phase in the very dense urban environment of Manhattan.

## Keywords

Geotechnical Investigation, Folded and Faulted Rocks, Tunneling in Dense Urban Environment, Shallow and

*Large Caverns, Drill and Blast Method, Sequential Excavation*

## 1. Introduction

The Second Avenue Subway Project is a major capital expansion project of the New York City subway that will provide a dedicated line for the east side of Manhattan with a link to the existing subway network [NASRI, 2008]. The proposed alignment runs from Harlem in the north to the financial district in the south with possible extension to Brooklyn. The project is approximately 13.7 km long including 16 stations, and its estimated cost is about \$17 billion (Fig. 1). Under the current design of the whole subway route, 10 stations will be cut-and-cover and 6 mined caverns which will be constructed through vertical shafts within the right-of-way of Second Avenue. In addition, there are numerous multi-track tunnels, crossovers and connections that will be constructed in caverns. The excavated diameter of the bored tunnels is 6.6 m and the caverns span ranges from 12.0 m to 21.0 m. All caverns have rock cover less than their span. As the geology of Manhattan varies along its length, the subway will pass through both hard rock and soft ground and there will be multiple rock/soil interfaces along the alignment.

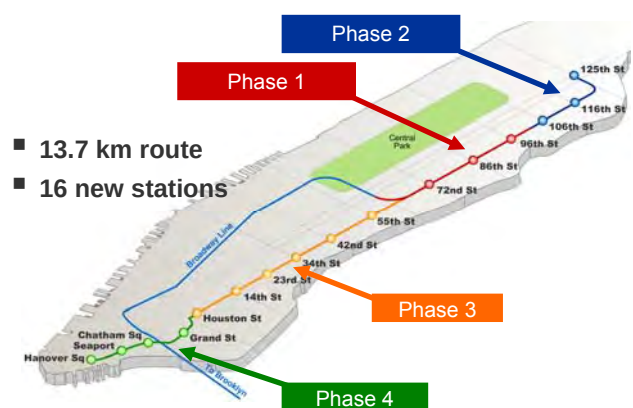


Fig.1 Second Avenue Subway Alignment and Construction Phases

The philosophy behind the construction methodology was to minimize the impact at street level on neighboring communities and businesses during the construction period. With most of the work being done within the right-of-way of Second Avenue, the largest

impacts will be related to the maintenance and protection of traffic and street restoration as travel lanes are reduced from six to four during the construction. Because of the nature of the work, close proximity of high rise buildings, critical nature of adjacent utilities, the characteristics of the ground along alignment, and the visibility of this project, strict performance criteria and limitations were imposed and comprehensive instrumentation and monitoring programs were designed to ensure compliance with action and trigger levels to protect third parties for noise, vibration, subsurface movements and protection of overlying utilities and structures. The final engineering was undertaken for the New York City Transit Authority by an AECOM led joint venture and the construction is currently underway by a tri-venture of Shea, Schiavone and Kiewit.

The Second Avenue Subway project has been broken into four construction phases, which could potentially overlap, to make funding of this mega project more manageable. The project is being funded by a combination of State and Federal contributions. The budget for phase 1 is \$4.5 billion, in year of expenditure dollars, and it is scheduled for completion by the end of 2016. Phase 1 includes 3.9 km of twin TBM rock tunnels, double-track 21.0 m span mined rock cavern stations at 72<sup>nd</sup> Street, and 86<sup>th</sup> Street, and a double-track cut-and-cover station at 96<sup>th</sup> Street. The overall configuration of the stations aimed to achieve as shallow a cavern as feasible to minimize passenger access time between entrances and platforms and to avoid interaction with existing subway underground structures. This had to be balanced against the need to provide an adequate rock cover for the caverns. Phase 1 of the Second Avenue Subway provides early revenue service, with ridership expected to be over 200,000 weekday riders when operational. The 3D models of two mined stations and their connecting tunnels are shown in Figures 2 and 3.

## 2. Geological Settings

The project area mainly consists of the Manhattan schist rocks, calcareous rocks of the Inwood marble

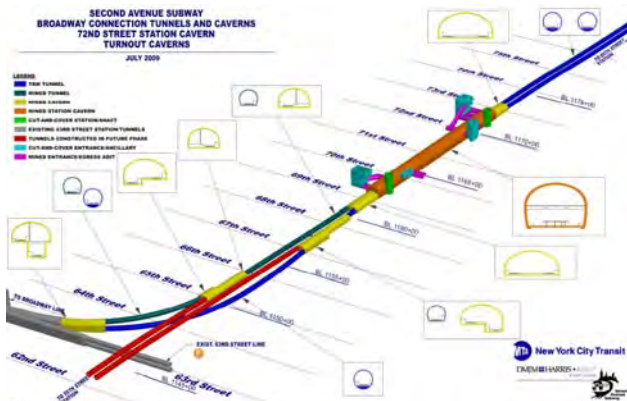


Fig. 2 3D model of 72<sup>nd</sup> Street Station and Connecting Tunnels

and Fordham gneiss. Manhattan schists are typically crystalline variations of essentially quartz and mica composition with quartz and feldspar rich zones, garnetiferous biotite and muscovite mica schist, quartz-hornblende-mica-garnet schists, and chlorite schists. Numerous pre and post to late thrust kinematic pegmatite intrusions of varying size have been emplaced within these schists typically along and occasionally across the foliation and along other fractures.

The underlying bedrock geology of New York City is highly complex. The rock types encountered ranges from Precambrian to Devonian in age. The crystalline rocks of New York City are divided into two major units separated by Cameron's thrust fault. This regional feature has been classified as a suture of the proto-american plate. The Cameron thrust faulting has affected both these units and imparted various structural features such as faults, shears and joint systems. The rocks of Manhattan area have undergone multiple deformation events causing three identifiable foliations. The rock mass is characterized by three principal joint sets with sub-sets and the dominant joint set is parallel to the foliation.

## 3. Subsurface Investigations

At a very early stage in the design process the significance of the fundamental geological structures were recognized as being a key to understanding the rock mass behavior. The exploration program included geological studies from microscopic to regional in addition to conventional geotechnical methods to advance this understanding. The geological investigation of the site started with collection and assimilation of existing information. This database included more than 600 historic borings which were sufficient to develop a preliminary conceptual geological model for the alignment.

An exploration program was designed to obtain data to current standards, to check, correlate and enhance existing boring data, to transform the conceptual geological model to a definitive model, and to compare

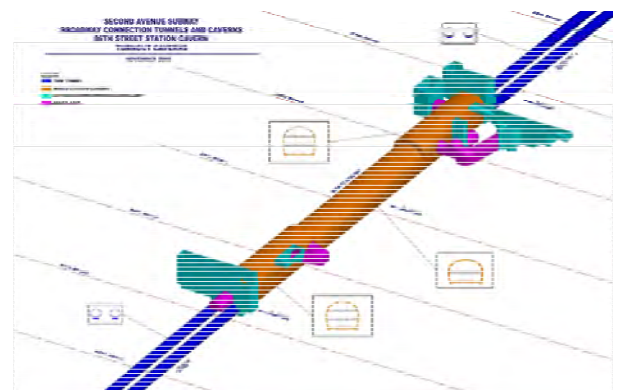


Fig. 3 3D model of 86<sup>th</sup> Street Station

this with the published geological model. Historic maps showing geomorphology, geology, land use and progressive development from early colonization of Manhattan were used to make initial interpretations of structural geology, particularly the location of major fault trends because these were postulated to be a significant influence on the natural drainage pattern of Manhattan Island before its development. This preparatory work was used to plan the exploration program with clearer focus on geological zones of importance and their relevance to design of the project.

During preliminary engineering over 350 new borings were taken along the Second Avenue Subway corridor to determine and/or verify ground conditions. In addition, over 200 environmental borings were taken in the soil overburden at locations where present or prior activities may have resulted in hazardous or industrial soil contamination. The investigation included not only the basic soil sampling and rock coring for laboratory testing and classification, but also oriented core drilling, cone penetration tests, geophysical surveys of boreholes, installation of monitoring wells and vibrating wire piezometers, observation wells, packer testing in bedrock, cross-hole seismic testing, seismic refraction testing, and in-situ stress testing. Total number of borings along the Phase 1 alignment during preliminary and final engineering was more than 180.

The primary source of geological information was rock core, although nearby excavations for deep basements and tunnels were mapped as part of the program. The rock cores were logged in general accordance with ASTM and ISRM. The approach that was developed and operated for the Second Avenue Subway project had three basic components: fabric and petrographic logging of rock core in a field laboratory, fracture and joint set orientation and classification by core logging, core scribing and borehole geophysics, and petrographic analysis by thin-section.

It is very difficult to distinguish between folded, faulted and unfaulted ground using conventional methods of core logging even if the full range of ground types and rock mass conditions are intercepted by the boring. It is possible to make general interpretations of the structural geology with rock outcrops to supplement the borings but these are rare in Manhattan. The conventional methods of fracture logging provide basic spacing and dip angle data but it is not possible to make direct interpretation of these data into structural groups or joint sets because the dip direction is unknown. Therefore, the detailed fracture logging of the core was enhanced by imaging the borehole wall with an acoustic televiewer. This data was used to approximate the thickness of the faults and shears and their relative orientation to the proposed tunnel alignment.

The significance of the acoustic televiewer is that the image is oriented so the dip and dip direction of the features, such as fractures can be determined mathematically from the data. Combining these methods produces the engineering characterization of planes such as joints, faults, fracture zone boundaries, their spacing, dip and dip direction and provides spatial control of the data.

Fabric and petrographic logging was used to classify the rocks by stratigraphy, genesis and deformation event. Petrographic analysis by thin section helped to determine the proportion of hard minerals, the degree of alterations and decomposition, and the extent of mineral segregation. These are critical concepts for understanding the engineering properties of the intact material where anisotropy may influence the behavior of the specimen under load. This examination can identify alteration and weathering associated with the faulting and hydrothermal action.

Because the quality of the rock can change dramatically in a very short distance, major features can be overlooked (Fig. 4). This image presents a major fault with hydrothermal alteration. It shows a zone of extremely fractured and degraded rock. The rock has been reduced to silty sandy gravel in places and the intact pieces are friable. There is alteration in the form of secondary mica and distortion of the schistosity. These are characteristics of faulted ground. If there is three or more borings in close proximity that intercept this feature then it is possible to estimate the orientation of the fault.

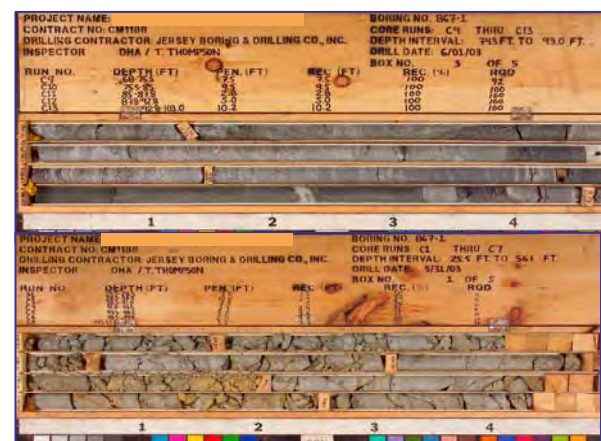


Fig. 4 Variability in Rock Quality

Groundwater generally follows the interface between soil and rock or stands approximately 4.6 m below ground surface. The rock mass permeability is generally very low with local high permeability associated with fracture zones, faulting and alteration.

A solid model was needed for the orientation of discontinuities, their properties, discrete features that may have a local influence on behavior and the potential risks from major failure. To achieve this goal,

subsurface exploration and testing program included orientation and frequency of fractures, shear strength properties of fractures, abrasivity of rock, faults and shear zones, intrusions and alteration, rock material properties, rock mass properties, and soil-rock interface profile and condition.

The variable quality of the rock mass along the cavern alignment required the development of multiple models representing the zonal differentiation of the rock mass in terms of foliation, jointing, and the presence of joint swarms and fractured zones. The methodology adopted, which can be described as “deterministic” was based on: the geometrically exact projection of the main rock mass features (e.g. shear zones, etc.) found in adjacent boreholes onto the section of analysis, and the inclusion of the sets of joints onto the plane of analysis on the basis of statistically derived spacing and dip angles as determined from adjacent boreholes to the section of analyses. The low bound spacing values were selected in all cases, whereas all joints in the sets projected were inferred to be through-cutting.

#### 4. Design Approach

Large excavation spans, low rock cover, variable geotechnical conditions, relatively large and complex intersections, and dense urban environment characterize the design challenges of the Second Avenue Subway caverns (Fig. 5). The mined cavern excavation sequence and support system were designed to ensure the stability of the rock mass and adjacent structures. Therefore, maximum allowable vertical ground movement in crown was limited to 50 mm and maximum allowable differential settlement for historical buildings near cavern to less than 1/1000.

Large cavern sections require multiple drill and blast drifts. The design of drift sizes and shapes was governed by excavation rate, different drifts and cavern

stability, and ground settlement and vibration concerns. Various possible cavern excavation sequence including center out drift, side in drift and their combination was considered and their pros and cons were studied through numerical modeling. The analyses show that given the nature of the rock mass (generally competent) and the tendency for gravity induced rock mass stability mechanisms, a center out sequence of excavation may be potentially more beneficial than an equivalent side in approach (Fig. 6).

The center out sequence involving the opening of a central heading followed by lateral extension and excavation of the side headings to form the full top heading through a three stage sequence for the larger cavern, will allow continuous dissipation of the induced stresses away from the excavation profile and will facilitate the gradual formation of a rock arch over the crown. The side in sequence can be considered to initiate an increasing concentration of stresses in the central pillar, which will add to the gravity loadings released upon pillar removal during the final development of the top heading. The critical top heading excavation drifts need to be separated longitudinally to allow optimum stress redistribution to occur as well as to facilitate parallel excavation and stabilization activities in the different headings. A minimum distance of one cavern span would be appropriate in our case.

In addition to excavation sequence and support system impact on the cavern stability, the size of various drifts (cross section and round length) was adjusted in order to limit the amount of charge per delay for each blasting cycle to satisfy the strict vibration limit of 12.5 mm/sec peak particle velocity under the historical buildings. The Phase 1 construction schedule requires that the TBM tunnels be excavated prior to drill and blasting of station caverns, which imposes some restriction on the excavation sequence configuration and mucking process.

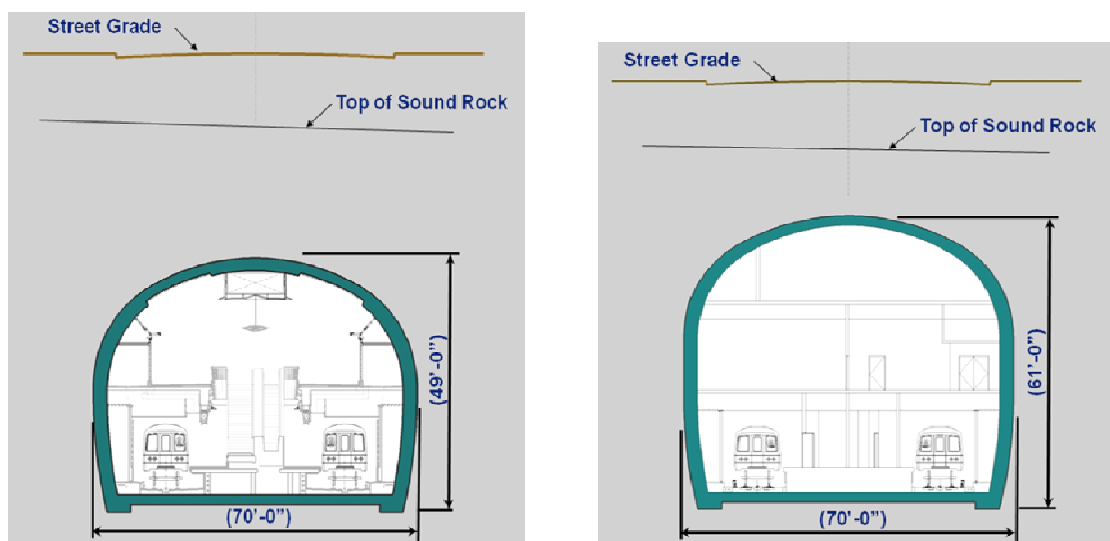


Fig. 5 Double Track Station Caverns at 72<sup>nd</sup> and 86<sup>th</sup> Street Station

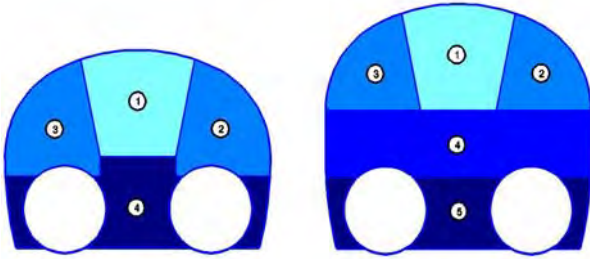


Fig. 6 Excavation Sequence Analyzed

Empirical data shows that there is a breakdown of the natural arching concept below some minimum cavern rock cover to span ratio. Underground rock engineering practice sets a limiting cover to span ratio of  $\geq 1/3$ . To avoid heavy support requirements and allow conventional construction methods in hard rock, the cover to span ratio over all of the cavern length was kept above  $1/3$ .

#### 4.1 Design Based on Q Empirical Method

The cavern design features represented by large, shallow openings, jointed rock masses, random shear zones, and variable rock covers required a robust design procedure including a combination of empirical methods, continuum and discontinuum analyses. Barton's Rock Tunneling Quality Index empirical method, Q, was employed to ensure that the designed support system was compatible with successful existing and similar rock caverns.

The raw Q values were developed for each core run from more than 50 deep borings encompassing a zone that extended at least  $1/4$  cavern span above and below the crown. From these raw Q values, the weighted average over the crown zone was taken to obtain representative Q values. Using these representative values, along with the northing and easting coordinates for each of the borings, an input file was generated to plot Q contours across the cavern plan and the centerline Q values were obtained by cutting a longitudinal section across the contours.

In zones influenced by penetrations and portals, the centerline Q values had to be reduced by a factor of 3.0 and 2.0, respectively, after the contours had been generated and the longitudinal section had been cut. For this analysis, the Q reduction was taken over a zone of the station cavern roughly equal to one-half the width of either the penetration or the portal, as appropriate.

#### 4.2 Discontinuum Analysis

The existence of low rock cover within a jointed rock mass led the designers to consider a block interaction problem rather than a stress strength one [NASRI & BERGESON, 2010]. Discontinuum analysis was used

to ensure that the presence of joints and faults in the rock mass around the cavern does not result in unacceptable bolt loads or displacements in the cavern structure. The Universal Distinct Element Code, UDEC, was employed to perform the discontinuum analysis and calculate the ground response, and rock bolt and shotcrete forces. Basic UDEC input parameters including cavern geometry, rock cover thickness, joint pattern, rock mass and rock joint parameters, and material properties of shotcrete and bolt structural elements were determined by the geotechnical investigation program or through literature review.

The first step in the design process was to divide the cavern into different ground class zones. For each ground class zone two deterministic jointing patterns (expected worst condition and expected typical condition) and a support class obtained from empirical methods were assigned and the available data for intact rock and rock joints properties were interpreted and best estimate and lower bound values were determined. UDEC was used to evaluate the global stability of each excavation drift and the entire cavern after each drift excavation and before and after its support installation. The analysis aimed at optimization of the design in terms of excavation sequence and type and quantity of support. Intrinsic stability mechanisms of the caverns were studied by excavating each drift and the entire cavern without installing the support. This was critical to the identification and interpretation of the range of rock mass responses.

For intact rock the Hoek-Brown criterion, for the foliation and cross foliation joints the Barton-Bandis joint behavior model, and for the shear zones the Mohr-Coulomb shear failure criterion were used. The convergence-confinement analysis method was applied to account for the three dimensional effects of the excavation face and a relaxation coefficient of 50% was applied after the excavation of each drift and prior to the installation of the initial liner. Three different shotcrete strengths (1, 7, 28 days) were used according to the timing of different excavation stages.

The main modeling steps consisted of: development of a rock mass model representing physical and mechanical characteristics of the ground, which was the principal factor controlling the structural behavior, initialization of the primary stress state through model consolidation under rock, soil and buildings gravity loading, excavation of various drifts and entire cavern without support to assess intrinsic stability state, and installation of the primary support in line with the appropriate excavation sequencing.

The evaluation of the results included: evaluation of the principal stability mechanisms, review of the induced stress-displacement fields, assessment of

supporting function of various rock reinforcement systems in terms of tunnel profile deformation control and their load capacity requirements, and overall engineering evaluation of the modeling results.

In the shallow discrete structure model, global stability may be associated with the unique geometrical combination of the joints. In addition, adverse through-cutting structure can be likely, while the crown arching capability is very limited.

#### 4.3 Continuum Analysis

Continuum analysis was used to ensure that the design does not result in adverse stress strength condition in the rock mass around the cavern opening. Rock mass parameters were determined using RockLab and the excavation sequence and support installation of the cavern was modeled using Phase<sup>2</sup>.

RockLab was used to determine the Generalized Hoek-Brown strength parameters as well as the rock mass deformation modulus. The input parameters comprise uniaxial compressive strength, intact rock parameter ( $m_i$ ), geological strength Index (GSI), disturbance factor (D), and the intact rock deformation modulus.

The uniaxial compressive strength and the deformation modulus of the intact rock were obtained from rock core laboratory tests. The disturbance factor (D) was assumed to be 0.8 based on the expected rock blasting quality. The intact rock parameter ( $m_i$ ) was considered to be 10 as recommended for schist. The structure of the rock mass was expected to be blocky with fair to good joint surface conditions. The expected typical condition assuming good joint surface condition resulted in a GSI value of 60 while the expected worst condition assuming fair joint surface condition resulted in a GSI value of 50. Based on these input parameters, Hoek-Brown strength parameters and rock mass deformation modulus were calculated using RockLab for the expected typical condition and the expected worst condition.

Two 2D continuum models with 1.8 m x 1.8 m bolt spacing for the expected typical condition and 1.5 m x 1.5 m bolt spacing for the expected worst condition and their corresponding sets of rock mass properties were developed. The caverns were excavated in 3 top heading drifts and one or two benches and the corresponding support systems were installed after each excavation stage. A relaxation coefficient of 50% was applied.

Based on the analyses performed, it can be concluded that this type of continuum analysis for jointed hard rock cases results in very small deformations and a very low level of stress in bolts and shotcrete. Therefore, continuum analysis in this kind of jointed rock cases fails to detect the local and global failure

mechanisms generated by the joint sets and cannot be reliably used in the design or verification of the design of excavation sequence and support systems.

#### 5. Construction Considerations

Station cavern construction through Manhattan schist can only really be efficiently accomplished using traditional drilling and blasting excavation methods (Fig. 7). Roadheader has been tried in recent past and have only achieved marginal success – largely due to the compressive strength and the considerable abrasivity of the rock mass.

Current construction methods utilize a modern approach to well-established rock excavation operations. These include drilling and blasting using computer controlled and guided multi-boom drills, powerful scoop-trams and robotic shotcrete spraying equipment. Careful excavation control is accomplished with frequent laser scanning equipment that is considered an integral part of the day-to-day excavation operations. Additionally, custom-designed muck handling systems have been developed to work well in a dense urban environment resulting in minimal impact to the local community (Fig. 8).

Blasting operations are generally carried-out on a daily basis and in a manner that is least disruptive to the surrounding neighborhood that is largely residential. Dust and noise controls have been adapted to suit the strict controls on emissions that are constantly monitored. Muck haulage to off-site landfills, for example, is performed on a restricted time basis during the day and along designated haul routes – to minimize impacts to traffic.

Considerable traffic control provisions are required in order to fulfill commitments made to the local community while still allowing the Contractor sufficient space to perform the work, store materials and have space for large equipment units needed for the nature of the work. Generally, the traffic control provisions are closely linked to the construction sequence and schedule and are designed to provide safety to the public while maintaining a constant and reliable flow of vehicles and pedestrians through and around the work sites.

The 2<sup>nd</sup> Avenue Subway projects, for example, have little space to perform the work; hence all designated areas must be well utilized for each successive construction operation. The work sites generally include several city blocks along 2<sup>nd</sup> Avenue with the notable distinction that these work sites are not interconnected and are really independent areas requiring fencing, security and appropriate traffic control provisions – unique to the size and location of the site(s).



Fig. 7 Excavation of the Center Drift of the Top Heading

The continuous and reliable handling of tunnel and station cavern muck (blasted rock and other excavation spoils) is essential for a successful operation. The urban environments require more attention to this system in order to maintain scheduled progress rates while performing the work in a safe and tidy manner. All muck is hauled off-site to approved land fill sites. Owing to the properties of the rock, it is not suitable for processing into aggregates. None of the muck will remain in Manhattan and will be hauled to sites in New Jersey and elsewhere.

The muck produced from the 2<sup>nd</sup> Avenue site is shot rock which is for the most part smaller than 12" although a fraction of the muck may exceed 36" on occasion. None of the muck is crushed or screened in the excavation area. This would not be practical owing to the nature of the excavation and the schedule and quantities for the work. Dust control is accomplished with water, with only small quantities of muck remaining at the site in excess of 24 hours.

Owing to the location of the project, careful and reliable noise and vibration controls are essential for all blasting and construction operations. Noise from blasting is generally difficult to muffle whereas continuous construction equipment noise can be addressed with engineering controls and the proper design and application of the equipment. Noise monitoring from blasting is measured in dB for each blast. Noise from other construction operations is measured on a time-weighted average basis in accordance with the Contract requirements and local ordinances.

Blast vibration controls are measured by numerous local seismographs placed in a manner to record ground energy (not building responses). Blast energy (Peak Particle Velocity – PPV) measured in inches per second (ips) is recorded for each blast and must remain below preset limits per the Contract requirements. These requirements are assigned relative to the proximity of buildings that have been previously inspected and classed into the following categories:



Fig. 8 Custom-designed Muck Handling System

Historic (and/or Landmark status), Fragile, Sensitive, and Normal.

The Contract provides guidance for the design of blasts and, therefore, the prediction of vibration levels as a function of the charge weight per delay and distance to the seismograph. The Contract also provides the design for the Initial Support system that is linked to the anticipated ground conditions in various areas of the station caverns, connecting tunnels, adits, ancillaries and entrances.

In New York City, the primary labor force for the excavation and concrete lining of all tunnels, shafts and caverns is the Sand Hogs. The Sand Hogs have a long tradition (since before 1900) of accomplishment under difficult and often dangerous underground conditions. They have considerable experience in the construction of soft ground and rock tunnels under Manhattan as well as some of the most world-renown subaqueous tunnels ever constructed. The Sand Hogs are well skilled and have a structured organization for the staffing of an underground project.

Owing to the quantity of large and complex underground construction projects currently underway in New York City and nearby, the Sand Hogs and their skill and dedication to the work is in demand.

Partnering on large, difficult projects is essential for success. The specifications and drawings together with the site conditions and local community issues require frequent and abundant communications between the parties. Trust and mutual understanding are the foundation to a successful relationship that must endure unexpected difficulties throughout the course of the work to meet the overall goals of the project.

Modern drilling, blasting and shotcreting equipment were chosen for the project due to efficiency and reliability considerations. Other factors included the size and speed of the equipment for drilling and muck handling. Shotcrete was a key component of the Owner-designed Initial Support system and would

involve several successive layers applied at considerable heights as the excavation advanced.

### 6. Conclusions

The Second Avenue Subway Project is one of the largest and complex construction projects in the United States and a critical part of the success for the project was the safe and optimum design of its large and shallow rock caverns. Empirical method and two and three dimensional continuum and discontinuum analyses were used in designing the excavation sequence and initial support system. Understanding each method's limitations in comparison of their results provided a comfortable margin of safety compensating for the unknowns in the design process.

Another critical part of the success for the project was the definition of the fundamental structural geology. This was achieved by adopting conventional and specialized geological techniques. The conventional methods have been enhanced successfully by applying fabric and petrographic logging and combining fracture logging of the core with reconstructed acoustic images from the borehole wall. The use of these combined geological techniques improved the ability to predict and interpret fundamental geological structure for the tunnel alignment.

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# ENGINEERING GEOLOGY AND ROCK ENGINEERING

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## Abstract

*Engineering geological input is critical in rock engineering projects, such as tunnels and caverns, where it can have significant positive impacts. Ground information for such projects is often limited and the engineering geologist is ideally placed to assist. However, the engineering geologist must have the correct skills and experience and these are discussed in the paper. Through discussion of case studies the paper then examines common failings in rock engineering projects where engineering geological input is lacking, and provides recommendations for future best practice.*

## Keywords

*Geological Models, Rock Characterisation, Kinematic Assessment*

## 1. Introduction

Timely, high quality engineering geological input is critical to the success of any undertaking in rock engineering, particularly those involving tunnels and caverns.

The aim of this paper is to show, through case studies and discussion, that engineering geologists can have significant positive impacts on the safety, performance, serviceability, cost, programme and maintenance of underground projects involving rock engineering. Conversely, the omission of, or provision of poor quality engineering geological inputs will likely have negative effects including over-design, cost overruns, unnecessary maintenance requirements or even failure.

Typically, only limited ground data is available for the tender and design of tunnels and caverns. Consequently, the engineering geologist is theoretically ideally placed to assist with understanding and characterising the rock mass. This is because they have expertise in evaluating and more importantly predicting multidimensional geological variations including rock formations, tectonic modification, overprinting of weathering and hydrogeology.

This paper first provides a description of an ‘ideal’ engineering geologist, with particular regard to the attributes required for rock engineering projects. Observations on current practice are then made, with reference to case studies, which highlight the common problems that occur where there is insufficient engineering geological input.

## 2. The Engineering Geologist in Rock Engineering Projects

The IAEG Statutes [IAEG, 1992] define engineering geology as ‘the science devoted to the investigation, study and solution of the engineering and environmental problems which may arise as the result of the interaction between geology and the works and activities of man as well as to the prediction and of the development of measures for prevention or remediation of geological hazards.’

However, as discussed in depth by Parry *et al* [2011], to make an effective contribution to projects an engineering geologist must have the correct training, knowledge, philosophy, approach, understanding of their responsibilities and appropriate skills. So, what attributes should a good engineering geologist have?

Firstly, they should have the right philosophy, i.e. that they are first and foremost scientists.

Secondly, they should use engineering geological models in their approach to refine their understanding of a particular project. The scheme proposed by the IAEG [2011] that divides models into *conceptual*, *observational* and *analytical* engineering geological models is one such approach.

Thirdly, they must have a correct understanding of their responsibilities. Baynes [2003] considers that these are:

- Observation and investigation of the geology in engineering projects
- Engineering geological model development
- Establishing standards and scope for the engineering geological activities
- Engineering geological information management
- Communicating the geology to engineers

Fourthly, the engineering geologist must have the required technical knowledge and skills to carry out their role. These will naturally vary from practitioner to practitioner although a list of the baseline knowledge and skills might include the following (modified after GSL, 2008):

- Undergraduate level geology
- Ability to collect and collate the relevant existing engineering geological data
- Engineering geological mapping
- Ground investigation

- Soil and rock description
- Soil and rock mechanics
- Interpretation of engineering geological data
- Production of engineering geological models
- An appreciation of geo-engineering design
- An understanding of construction methods
- Recognition and management of uncertainty
- Recognition and management of geo-engineering risk
- Effective communication with engineers and other professionals

Critically, given that geology is possibly the most observational of all the sciences, it is difficult to imagine a good engineering geologist who does not have extensive experience of geological fieldwork. In particular, the engineering geologist should have skills in factually recording and subsequently interpreting engineering geological information from a wide variety of sources including exposures and core.

To make a successful contribution to a rock engineering project, the following particular attributes will also be required:

- Experience in rock engineering projects
- Good understanding of the principles of rock engineering design
- Ability to carry out rock mass characterisations appropriate to the feasibility, investigation, design and construction stages of a project
- Experience in GI methods important to rock engineering projects (including orientated core, televiewers, plate bearing tests, dilatometers, hydrofracture tests, borehole jack tests, cross-hole tomography, petrographic description, index tests, unconfined compressive strength, point load tests, slaking tests, direct shear tests, full-scale loading tests, geophysics etc).

The term *engineering geologist* as used for the purpose of this paper describes an individual who consciously strives towards the level of professionalism that has been discussed. The reason this definition has been provided is that it is an implicit assumption of this paper that efficient rock engineering design and construction in tunnel and cavern projects requires that an engineering geologist has significant involvement.

### 3. Case Studies

The following case studies illustrate common problems that occur without or with only limited use of engineering geological input. Recommendations are made where appropriate.

#### 3.1 Case Study A

##### Introduction

This involves a deep excavation for a large cut and cover underground space with near vertical sides,

predominantly in competent plutonic rock in Hong Kong. The project was part of a larger tunnelling project that was at the tender design stage. The tendering contractor was looking to optimise elements of the design, including the rock support.

This section examines a number of issues that became apparent during the attempted optimisation and how these affected the project. These issues are related to desk studies, site investigation, rock mass assessment, discontinuity strength, kinematic analysis, block sizes and communication. Where appropriate observations are made on wider rock engineering practice, recommendations are made for future improvement.

##### Desk study issues

The desk study for this project should have included the consultation of existing tunnel records, but this was not done.

Whilst it is a requirement for construction tunnel logs to be provided to the Hong Kong Government, the accessing of this data is not straightforward and a Geographical Information System (GIS) system, similar to United Kingdom Highway Agency Geotechnical Data Management System (HAGDMS), with a link to a scan of the tunnel records would be highly beneficial. Incidentally such a system could be used to manage all geo-engineering information in the Special Administrative Region (SAR) including ground investigation, registered slope and Landslip Preventative Measures (LPM) and Natural Terrain Hazard Assessment (NTHS) reports. Such a system would help to make this information much more accessible, allowing more useful desk studies for underground projects.

##### Site investigation issues

In Hong Kong, in addition to recently constructed tunnels where recorded data exists, there are numerous disused tunnels throughout the SAR relating to mining and civil defence from the Second World War, and these provide extremely valuable large-scale exposures. Despite this these are rarely inspected and were not for this case study.

In addition to existing tunnels there were also nearby cuttings with significant rock exposures. However, similarly the opportunity to inspect these exposures was not taken.

As with the majority of projects, the emphasis was on ground investigation (GI) rather than site investigation (SI). Unfortunately, in common with the majority of drillhole logs examined by the authors, the discontinuity descriptions were inconsistent, lacking in detail and no attempt had been made to attribute the discontinuities to individual sets.

### Issues related to the assessment of the rock mass

Acoustic televiewers were used to obtain discontinuity orientations. However, no assessment of aperture and roughness was made from this information. Optical and acoustic televiewers have a number of benefits, including the speed with which discontinuity data can be obtained and prepared for analysis. However, they also have a number of shortcomings, chief of which are:

- The number of discontinuities identified by core logging and by a televiewer can differ significantly. Therefore caution must be exercised in assigning values such as Rock Quality Designation (RQD) on the basis of televiewer information alone [MAYBEE et al, 2002].
- Often it is not possible to tell the difference between features such as a joint, a dyke, an incipient discontinuity or a clay infilled joint from a televiewer.
- It is not possible to obtain discontinuity information other than orientation, aperture and a broad sense of roughness from a televiewer.

As was the case on this occasion, there appears to be a worrying trend of using televiewer data as a substitute for detailed rock mass characterisation, such as detailed discontinuity descriptions of exposures and core. This approach results in an enormous amount of discontinuity orientation data and an impressive looking (and easily produced) stereonet, but does not provide sufficient information for a full rock mass characterisation.

In contrast, where the investigation is based on drillholes alone it is important to note that there are a number of discontinuity characteristics that cannot be reliably recorded in core such as persistence, aperture, wavelength and infill. This is why exposures such as natural outcrops, cuttings and unlined tunnels are preferred.

When detailed discontinuity descriptions from core and televiewer logs are used in conjunction, then this is valuable information. However, if the project engineering geologist does not log the core and inspect the televiewer logs, there is no opportunity to undertake a sense-check of the data and develop a 'feel' for the rock mass.

Drillhole core for rock engineering projects should be orientated wherever possible. Failure to do this means the possibility of linking detailed discontinuity descriptions of core to those from exposures, televiewer and impression packer data is much reduced. In this case study, the core was not orientated and no attempt appears to have been made to record detailed discontinuity information from the core, let alone relate it to the televiewer data.

With respect to GI and discontinuity evaluation, the standard approach for rock mass description and the logging of core for rock mass assessment is that they are undertaken by GI contractors. However, despite the often extensive GI that is undertaken, the core is logged by individuals with little or no engineering geological training or rock mechanics expertise and who are provided with no background to the project. It is the authors' opinion that engineering geologists should undertake this work. Whilst the initial cost may be higher, the improvement in the quality of the data will facilitate the selection of the most suitable design - easily offsetting this cost later.

At the tendering stage, the time constraints meant that the tenderers had to accept the discussed inadequate data for their tender designs. Consequently this meant that conservative assumptions had to be adopted for the analysis and design of the excavation. However, even with the quality of data provided, it is considered that improvements with respect to the parameters adopted for tender design could have been made. However, rather than evaluating the existing data from an engineering geological perspective to determine which parameters could be adopted based on experience, the design engineers adopted conservative values for each assumption. This resulted in a highly unlikely ground model. For example, a conservative estimate of the angle of friction of the joints was made based on shear box testing of core. However, such testing already gives very conservative values due to the limited size of the sample. Also, no normalisation of results was undertaken and no allowance made for joint roughness, waviness, amplitude etc. The assessments of joint persistence and spacing were also absent or inadequate.

### Issues related to the assessment of discontinuity strength

The angle of friction of joints should include considerations of joint roughness, infill, wall strength etc. As noted for plutonic rocks in GEO [2007]: *'Where shear box testing of a joint is undertaken the natural roughness of the surfaces should be taken into account by normalising the data to account for dilation during testing (Hencher & Richards, 1982). A basic friction angle of 40° has been proposed by Hencher & Richards (1982) regardless of the decomposition grade'*. Hencher & Richards [1982] note that a roughness angle should also be applied, and an example of this approach is provided by Richards & Cowland [1982]. This suggests that a joint angle of friction of 40° could be considered as conservative for Hong Kong granites in the general case, yet angles of friction of much less are regularly seen in rock engineering designs for Hong Kong granite.

Every effort should be made to select appropriate parameters for analysis and design. An example of an appropriate basis for selecting these is the definition of a *characteristic value* as defined in Eurocode 7 [BSI,

2004] which defines it as a ‘cautious estimate of the value affecting the occurrence of the limit state’ or statistically as ‘the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%.’

The discontinuity orientations were also analysed in very simplistic terms by assuming that a concentration of poles to the planes (derived from televiewer data) plotting within a failure envelope on a stereonet equalled the risk of failure, rather than being used as a prompt for further analysis.

### Issues related to kinematic analysis and block size determination

Kinematic analysis is only the first step in block stability analysis [WYLLIE & MAH, 2004]. The plotting of a pole, or the intersection of two planes, within a failure envelope merely indicates the possibility of failure. This is a prompt for further detailed investigation and analysis, not an immediate indication of risk.

Unrealistic assessment of joint persistence and spacing for this case study meant that the block sizes adopted in the analysis were generally smaller than those expected to exist in reality and were unrealistic geometrically. The assessed block sizes required less force to support than the actual likely block sizes, but in some cases were assessed as more likely to fail, as less friction was mobilised on the joints. Therefore, the assessment was simultaneously over and under-conservative. Whilst blocks of this size and shape might exist, they were unlikely to represent the general case.

Ranges of realistic block sizes, taking into account joint persistence and spacing are required. Whilst it is difficult to estimate this from drillholes alone, it can be done with detailed work. This could have been undertaken relatively easily if the available exposures were inspected.

Unfortunately in this case, due to the shortcomings in original GI and consequent lack of the required basic data, it is unlikely that significant optimisation of the tender design could have been carried out effectively.

### Communication issues

The case study also involved two significant failures of communication. The first was that the designer did not provide a quantification of the hazard posed by potential rock instability, meaning the tenderer found this aspect difficult to account for. The second was that the designer failed to explain their design to the tenderer in a way that was understood. This meant that there was a lack of confidence in the design, for reasons not simply attributable to the shortcomings already discussed. Clear communication of the

potential geological hazards and risks is critical otherwise misunderstandings and errors can occur.

However, it is difficult to put values onto geological features and risks. It is one of the main responsibilities of the engineering geologist that they quantify that which sometimes seems unquantifiable, by putting numbers to soil and rock masses and assigning percentages to hazard and risk [HOEK, 1999]. Without this the engineering geological input becomes greatly limited.

An effective way of communicating the key points to other geo-engineers is through the use of uncertainty and risk registers [KNILL, 2001]. An uncertainty register provides a rigorous, transparent and documented method of managing, investigating and reducing geological uncertainty.

A geo-engineering risk and opportunity register is a similarly transparent, rigorous and documented tool with which to record, manage and communicate risks with other geo-engineering professionals. This is sometimes done but there are problems including failure to keep the register current and poor communication of the risks to the project team.

One additional impediment to communication seen in this case study, and many others, was the misuse of effort on summarising data that has no or minimal relevance to the project. This is an issue that has been discussed elsewhere [PARRY & HART, 2009] and rock engineering projects involving underground space are no exception to this. This is not to say that sources of potential useful information should not be consulted. Rather, that only the important points are summarised, otherwise the key information is swamped and communication is impeded.

### Consequences

These missed opportunities, misconceptions and errors meant that any chance of developing an efficient rock support design prior to construction was effectively lost. This resulted in three possibilities:

- (a) A safe, but considerably over-conservative design.
- (b) A design that has missed a critical hazard, such as a large, weak, through-going discontinuity.
- (c) An efficient design that is not necessarily based on the provided data, but is safe and appropriate due to the application of the engineering geologist's knowledge and experience.

Generally it is (a) that is the most likely result. However, (b) continues to occur in engineering projects, both large and small with regularity. Outcome (c) is rarely seen in today's litigious culture. Therefore the importance of the engineering geologist providing

the data that the designer needs to produce an efficient design is clear.

In this case study the tender design of the rock support adopted for the excavation consistently used conservative parameters. This was partially as a result of the data provided and partly due to of the approach of the tenderers' design team. The result was that the likelihood of failure was probably overestimated and the consequent support was likely overdesigned. As a result the tender was probably overpriced.

The most notable aspect of this case study is that the information required to carry out the analysis properly was available, or could have been acquired relatively easily. Certainly the modest additional cost of these measures would be saved many times over in efficiencies on the rock support design or significant failures avoided. It is suggested that many of these problems could have been avoided or reduced had an engineering geologist been involved throughout the project.

### 3.2 Case Study B

#### Introduction

A contrasting example is provided by a tunnel and cavern construction project through tectonised volcanic rock. This project was at the early stages of construction when discussed.

This section discusses issues related to engineering geological models, *unforeseen* and *unforeseeable* ground conditions, faults and quartz veins. Again, reference to broader practice and recommendations are made where appropriate.

The original investigation appears to have been undertaken without an adequate geological model and a nearby existing tunnel was not inspected. The contractor's designer realised that a considerably improved rock mass characterisation was required for a proposed optimisation of the design, to replace two parallel tunnels with a single cavern, and in particular two key issues arose:

- Were features logged as faults in drillholes really faults?
- Would large inflows of water occur in association with the 'faults'?

To address these issues the following work was carried out by engineering geologists:

- Re-logging of drillcore.
- Engineering geological mapping for rock mass characterisation in the nearby tunnel.
- Improvement of the engineering geological model.

#### Engineering geological models

For all engineering projects the development, from the desk study stage, of a robust, defensible and evolving engineering geological model is of paramount importance. This is the foundation on which all the geo-engineering aspects of the project should rest and where there is the most scope to make savings and reduce risks to the project. Unfortunately many engineering geological models are overly-simplistic or simply incorrect.

The Total Geological History approach [FOOKES et al, 2000] and published geological data suggests that the following geological conditions could be applicable to this case study:

- Tectonic model – convergent plate boundary – magmatic arcs.
- Geological model – igneous – acid volcanic.
- Geological model – structural – strike-slip faults.
- Geomorphological model – hot wet climate.

These models provide a 'check list' of the potential geological conditions, geohazards and engineering challenges that may face a site and consequently the risk of unforeseen ground conditions is reduced or eliminated. In this case study these models predict the following geological conditions that were subsequently encountered during the project:

- Acid volcanic (rhyolitic) rock.
- Pyroclastics and tuff.
- Hydrothermal mineralisation/alteration.
- Faulting.
- Aplite/pegmatite dykes.
- Faults and joints displaying consistent pattern.
- Extensional pull-apart features.
- Deep weathering profiles.

These are discussed further below.

#### Unforeseen and unforeseeable ground conditions

Related to engineering geological models, there is a general misunderstanding and misuse of the terms *unforeseen* and *unforeseeable* with regards to ground conditions. It is often the case that these ground conditions are perfectly foreseeable to a competent engineering geologist. As noted by Baynes [2010].

*'There is very little geology or geomorphology that will be unforeseen on a site if the investigation is carried out properly. This means that if a project is managed in a way that implements all of the established techniques developed to mitigate geotechnical risk, and the work that is carried out is high quality, then the probability of an 'unforeseen' condition being encountered during construction should be reduced to negligible proportions.'*

*‘Nevertheless, there are some geological conditions that are ‘unforeseeable’, and when those conditions are encountered (they will have been anticipated) there will inevitably be some undetectable variations in the geology that can never be completely investigated within practical limits; for example, cavernous ground as a result of karst may be recognized but it may be impractical to attempt to investigate the details of every single cavity; the details are ‘unforeseeable’.*

#### Quartz veins and faults

In this case study, features originally identified as ‘faults’ were reinterpreted as hydrothermal quartz veins on the basis of the re-logging and the consideration of an appropriate engineering geological model. As such, unlike faults, they were not formed by shear with an associated reduction in strength, but were formed in tension and subsequently the discontinuities were largely ‘healed’ by hydrothermal minerals, in this case quartz. Some faults were encountered but these were very minor and might have been more appropriately described as joints associated with accommodation movements.

Whilst the mechanical strength of such zones is considerably higher than faults, in certain locations related to this case study the veins are associated with significant groundwater flow. This is because the metalliferous minerals in the quartz veins are easily decomposed, forming a preferential flow-path for groundwater.

One of the reasons for the misidentification of the quartz veins as faults was poor descriptive practice. It is the authors’ experience that the description of weak zones, such as faults and shear zones are generally inadequate for rock engineering purposes. The United States Department of the Interior method for describing such zones is recommended as a suitable descriptive method [USDOI, 2001], and was adopted for the re-logging in this case study.

Had an appropriate engineering geological model been developed at an earlier stage of the project then mapping could have been carried out at the surface. A focus of the mapping would have been establishing the location, orientation and nature of faults and hydrothermal veins. The from mapping of nearby exposures such as natural outcrops, cuttings and quarries information could have been used in conjunction with the drillhole data to estimate the spacing and location of these features and to begin to build a 3D model. This would have allowed prediction of where these features would be encountered in the proposed tunnel. Mapping allows early access to detailed and potentially valuable information and is a much more efficient use of project budget than relatively costly ground investigation. It is noted that surface mapping is particularly underused in Hong Kong practice.

### **3.3 Case Study C**

This case study involves a tunnel that is approximately 150 years old, driven through limestone. There had been several small rockfalls in the tunnel and a number of potentially unstable blocks were observed in the tunnel walls. An investigation of the tunnel was carried out which included a desk study, tunnel mapping, ground investigation and interpretive reporting.

However, there were a number of significant shortcomings in both the tunnel mapping and ground investigation. Firstly, the discontinuity descriptions were not sufficiently detailed. The joint descriptions only included orientation data (a common problem) along with general comments of varying use. Critical aspects such as persistence, spacing, roughness, aperture, infilling and wall strength were not recorded. Merely having orientation data only allows a rudimentary analysis, generally leading to conservative and occasionally unsafe design. Furthermore, the drillholes were not orientated and no televiewer, impression packer or *in-situ* tests were carried out.

In addition, standard rock engineering classification and analysis approaches were used inappropriately. For example, it is good practice in rock engineering to compare the findings of empirical design tools (such as Q, RMR and RMi) with calculations (such as the results of hand calculations and software packages including UNWEDGE and UDEC) to check that the approaches used provide a similar result. Generally the more conservative result is chosen as the basis of design. However, such classification systems and analytical methods were formulated to assess new excavations in rock. These methods are appropriate for such purposes provided that the engineering geological input is of high quality.

However, such an approach may not be appropriate for analysing and designing rock support for tunnels that have existed for many decades, as many of the failures that are likely to occur in these tunnels will have already occurred or been mitigated since construction. Ongoing failures will likely be due to rock mass deterioration.

In this case Q and RMR classifications were compared with the results of an UNWEDGE analysis. The result was an inappropriate and extremely conservative design which proposed pattern bolting for extensive sections of the tunnel.

Whilst this may have been appropriate for a new tunnel, a more appropriate design for this old tunnel would have been spot bolting of those blocks assessed to be potentially unstable combined with light scaling and mesh in areas where ravelling might occur. This work, if guided by an engineering geologist, would have been much cheaper, quicker and appropriate.

One factor that may have led to the over-design in this case is the relative lack of guidance on the assessment, maintenance and design for existing unlined rock tunnels and caverns. The vast majority of current guidance and standards relates to new and proposed works.

### 3.4 Case Study D

This final case study involves a cavern and several related tunnels and adits in plutonic rock. This project was at the construction design stage and the contractor wished to optimise the design. The preliminary and tender designs were based on a combination of empirical design methods and computer modelling and further investigation of the assumptions, parameters and analytical models used in the design was required.

An increasingly common approach in rock engineering projects, including this case study, is that rather than obtaining data to characterise the rock mass, the investigation focuses on obtaining data simply to input directly into rock mass classification systems such as the Q and RMR systems. This may be in an attempt to save time, but the authors consider such an approach to be technically constrained. If instead a rigorous rock mass characterisation is carried out, then the data required for the classification systems will have been collected as a matter of course. Furthermore, such an approach has the added benefit that a model of the rock mass will be developed and understood, against which the recommendations derived from the classification system can be evaluated.

A second issue related to the Q system is that inadequate rock mass characterisation often leads to significant underestimation of the Q values and hence underestimation of the rock mass quality. It appears that the Q value had also been underestimated for this case study, allowing significant potential for optimisation of the design. However, this could have been incorporated at an earlier stage of the project.

Evidence of this is given in GEO Publication No. 1/2007 [GEO, 2007], Table 6.7.3 which is reproduced below.

| Total Length of Bored Tunnel (1,609 m) |       |           |       |
|----------------------------------------|-------|-----------|-------|
| Q-value range                          | < 0.3 | 0.3 – 4.0 | > 4.0 |
| Pre-construction estimate              | 14.7% | 60.6%     | 24.7% |
| As-built records                       | 7.5%  | 43%       | 49.5% |

Table 1 Sha Tin Heights Tunnel Q-value Ranges Estimated at the Pre-construction Stage and From As-built Records

What this table and several others in the source document make clear is that Q-value is being significantly underestimated at pre-construction stage in Hong Kong. While some underestimation is to be expected, the size of the difference is of concern. This suggests that the rock support designs upon which tenders are won are likely to be overdesigned and hence overpriced. It is considered that this discrepancy could often be significantly reduced if an engineering geologist carries out a thorough rock mass characterisation at the pre-construction stage and then refines this throughout the project.

Finally, numerical analysis software such as FLAC, PLAXIS and UDEC are increasingly being used and both were adopted in this case study to assess movements related to the excavations. However, the analytical models that were adopted for this project were, from an engineering geological perspective, very unrealistic. The following points were observed:

- PLAXIS and other finite element codes (such as FLAC) only realistically model massive unjointed rock masses. These are relatively rare, and these conditions did not apply to this case study. For instance there are no such rock types in Hong Kong. That is not to say that these methods should not be used but that their limitations should be realised.
- UDEC and other discrete element codes are more appropriate for jointed rock masses, such as the rock found at the case study site.
- Discrete element modelling will not help analysis and design if an unrealistic discontinuity model and parameters are used. It was considered that the parameters used for joint friction in this case study were overly conservative.
- The discontinuity model should include realistic discontinuity lengths, spacing and variations of orientation. The joints modelled for this case study were quite unrealistic as they were analysed as having great persistence and very close spacing, which did not reflect the predicted geological conditions. This results in unrealistic and conservative results.
- Factored design parameters should not be used in numerical models unless an extremely conservative result is sought.

## 4. Summary and Conclusions

It is considered that there is still considerable scope for improvement in engineering geological practice and its application to rock engineering projects, particularly in the development of realistic engineering geological models, collection of rock mass data, rock mass characterisation and kinematic stability analysis. It is suggested that if an engineering geologist meeting the description provided in Section 2 of this paper was sufficiently involved in these case studies then the

majority of these issues would have been removed or their impact reduced. The four case studies illustrate common problems in underground rock engineering that lead to inappropriate and overly conservative designs.

The key factors that underlie any analysis and design are the assumptions. A single conservative assumption may be defensible. However, the cumulative effect of many conservative assumptions can lead to a significant divorce from reality. Rock is a structural material and in many cases will be stable or meta-stable. If this is accepted, then it is logical that less conservative design should follow. The tendency in Hong Kong to over-design may also be influenced by insufficient incentives for efficiency, particularly for large well-funded projects. As a result there is little incentive to optimise design in a market where funding is ample, and where there is little desire to accept responsibility and even the lowest levels of risk.

Many of the recommendations made in this paper are in line with guidance contained in Geoguide 4 'Guide to Cavern Engineering' [GEO, 1992] and so these are not new issues.

The range and scale of the underground rock engineering projects taking place in Hong Kong means that there is an excellent opportunity to establish our engineering industry at the forefront of international practice. Good engineering geological input will help us achieve this.

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# GEOTECHNICAL ENGINEERING FOR AN UNDERGROUND RAILWAY STATION, NORWAY

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## Abstract

*The Norwegian National Rail Administration has recently (June 2011) completed the detailed design and tender documents for a new railway tunnel. Included in the tunnel is a 34.5 m wide, 16 m high and 250 m long underground railway station in the town of Holmestrand, located to the southwest of the capitol of Oslo. The station hall is to be constructed in basalt lavas that contain three sub-vertical, mutually orthogonal, joint sets, and one sub-horizontal joint set. Upon completion, in 2016, it will be one of the largest underground railway stations in the world. To the north and to the south of the station, the large span is continued, resulting in a 34.5 m spanned cavern with a total length of roughly 500 m. At both ends of the cavern, the 34.5 m span is reduced to a normal double-railed tunnel of 14.5 m. In the area, to the south of the railway station, a 30 m wide tunnel section will be excavated through a major fault. The central part of the fault is expected to influence the stability and excavation methods over an axial length of up to 100 m.*

*This paper addresses the main challenges in the project, the design of rock support in the large spanned cavern, and excavation through the fault. Numerical modeling using 3DEC has been carried out in order to investigate the modes of failure, and modeling using Phase2 has been completed to assess the expected necessary rock support in this project. As well, the design of excavation method through the major fault is described.*

## Keywords

*Underground Railway Station, Numerical Modeling, Rock Cavern, Large Span*

## 1. Introduction

The Norwegian National Rail Administration is modernizing the railway system in the central part of Norway. One of the routes carrying a significant percentage of traffic is from the capitol of Norway, Oslo, to the county of Vestfold, to the south of Oslo. The railway section between Oslo and Vestfold was opened in 1881, a renovation of the railway is essential for transportation of people in an efficient and modern way.

One of the sections currently being modernized is between Holm and Nykirke (Fig. 1). This section will include a 12 km railway tunnel, which will host an underground railway station in the town of Holmestrand. The station hall is planned to host a four-track rail, two of which are designed to handle high-speed trains with a velocity of up to 250 km/hr. In order to accommodate these criteria, a 34.5 m wide and 250 m long railway station will be constructed.

The Norwegian National Rail Administration has recently completed in June 2011 the detailed design and tender documents for this project. The area above the railway station is a residential area, called the "Holmestrand plateau". Excavation and grouting must be carried out carefully. Construction work will start in November 2011 and the main geological challenges in this project are believed to be:

- Unfavorable jointing in the rockmass leading to large potential unstable wedges in the roof of the large-spanned railway station.
- Excavation through a major fault to the south of the railway station, where the tunnel span is 30.5 m. Stress measurements have indicated only gravitational stresses in the area.

This paper addresses these challenges and describes ground investigations and design in order to ensure successful excavation of one of the world's largest underground railway stations.

## 2. Design

The railway station will host four-tracks; the high-speed trains (250 km per hour) will be located on the two middle rails, and normal-speed suburban trains on the outside rails (Fig. 2). The design takes into consideration high-speed trains which will be implemented in the future (planned in 2030). The high-speed trains will not stop in Holmestrand, only the suburban trains will stop here.

This design has been stipulated by the Norwegian National Rail Administration. Design considerations include noise, wind and emergency preparedness. The railway station will have a span of 34.5 m, and length of 250 m. The design was completed by the architects Public Arkitekter Denmark in cooperation with the consultants Rambøll Norway and Norwegian Geotechnical Institute (NGI).



Fig. 1 Overview of the Section between Holm and Nykirke in the Southern Part of Norway

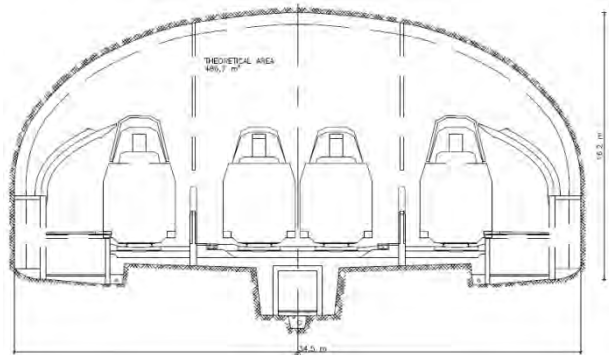


Fig. 2 Profile of the Railway Station in the Town of Holmestrand (The span is 34.5 m, the height is 16.2 m and the area is 486.7 m<sup>2</sup>)

To the north and to the south of the station hall, the large span is continued, resulting in a cavern with a span of 34.5 - 30.5 m and a total length of roughly 450 m. The large span narrows to an ordinary two-rail tunnel with a span of 14.5 m over a section of 150 m to the northern and to the southern extents of the cavern (Fig. 2). Two entrance tunnels from the town of Holmestrand are planned in conjunction with the railway station, both with a length of approximately 150 m (northern and southern entrance tunnel are shown in Figure 2).

### 3. Geology

The bedrock in the area consists of basaltic lavas from Permian with underlying sediments from Ordo-carbon and sandstones from Devonian. The railway and the station hall in this project will be excavated in the basalt lavas from Permian (Fig. 3). The jointing in the basaltic lavas consists of three sub-vertical, mutually orthogonal, joint sets, and one sub-horizontal joint set. The basaltic lavas are partly covered by marine sediments of thickness greater than 20 m.

The basaltic lava along the tunnel consists of several independent lava flows, with an estimated thickness of 5-10 m [LARSEN et al, 2008]. The top of the lava flows holds fragments of solidified lava which has been thorn off by lava which has still been liquid. Gas bubbles in the top layer of the lava have now been filled by carbonate minerals. Between the different deposited layers, time breaks are visible. These time breaks are evidence of disintegration of the rockmass. This is especially in the top layer, at the same time as foreign material was added by water and air.

Logging of cores from five boreholes drilled from surface above the railway station hall show that the rockmass is characterized by four distinct joint sets, three steeply dipping joints with varying strikes NW-SE (set I), NE-SW (set II) and N-E (set III) in addition to a sub-horizontal joint set. The joint sets form what is termed “columnar basalt” (Fig. 5).

The rock overburden is between 9-65 m, with the lowest rock overburden at the section where a major fault crosses the tunnel, 40 m to the south of the railway station. Average fracture frequency and spacing for the joint sets I-IV, logged in two inclined boreholes (KH4 and KH5), 137 m and 120 m in length, respectively is shown in Table 1. Other two inclined boreholes have been drilled from the surface through the fault (KH6 and KH7 in Figure 3). Logging of the cores indicates a central zone of the fault consisting of around 15 m of weak material/gravel, possibly with clay. The fault crosses the tunnel with an angle of 30°, influencing the tunnel stability for as much as up to 100 m along its axis.

### 4. Stress Conditions

No data on the stress conditions existed in the area prior to the design study. Stress measurements were therefore carried out north and south to the major fault located to the south of the railway station hall. Investigation of the stress conditions included 3D-overcoring from an existing road tunnel in the area (Holmestrandtunnelen), from the rock slope to the east of the railway station as well as hydraulic fracturing in two of the vertical boreholes drilled from the surface over the railway station. The measurements indicate:

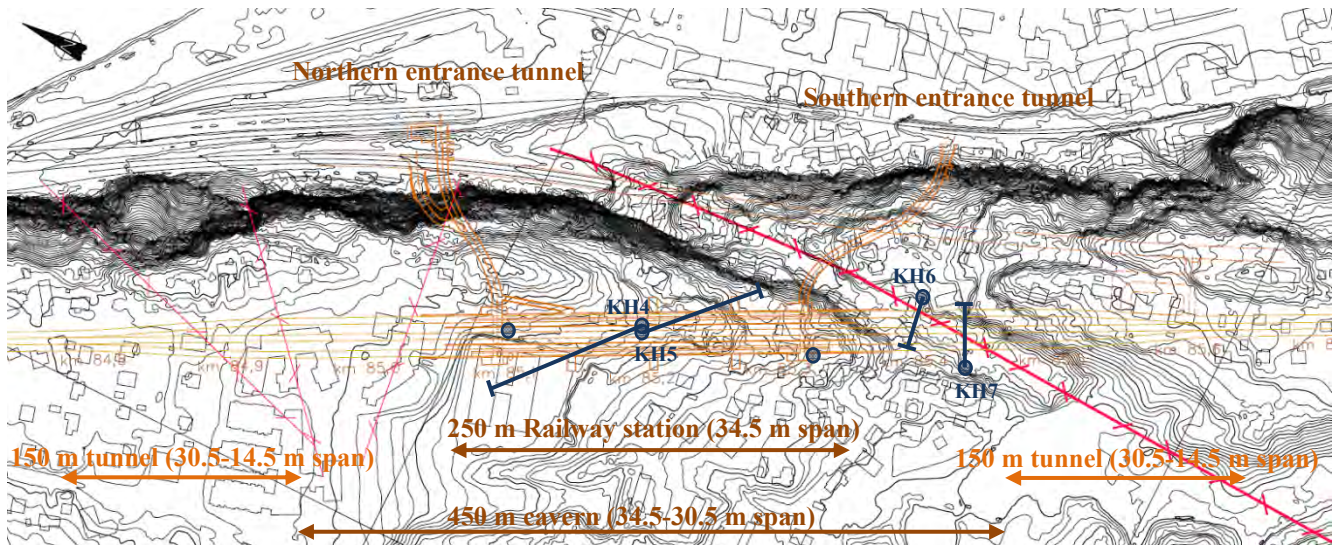


Fig. 2 Overview of the Railway Station and Associated Tunnels to the North and to the South, including Northern and Southern Entrance Tunnels (Boreholes for obtaining drill cores are shown (blue circles), including two boreholes drilled through the major fault to the south of the railway station (KH6 and KH7))

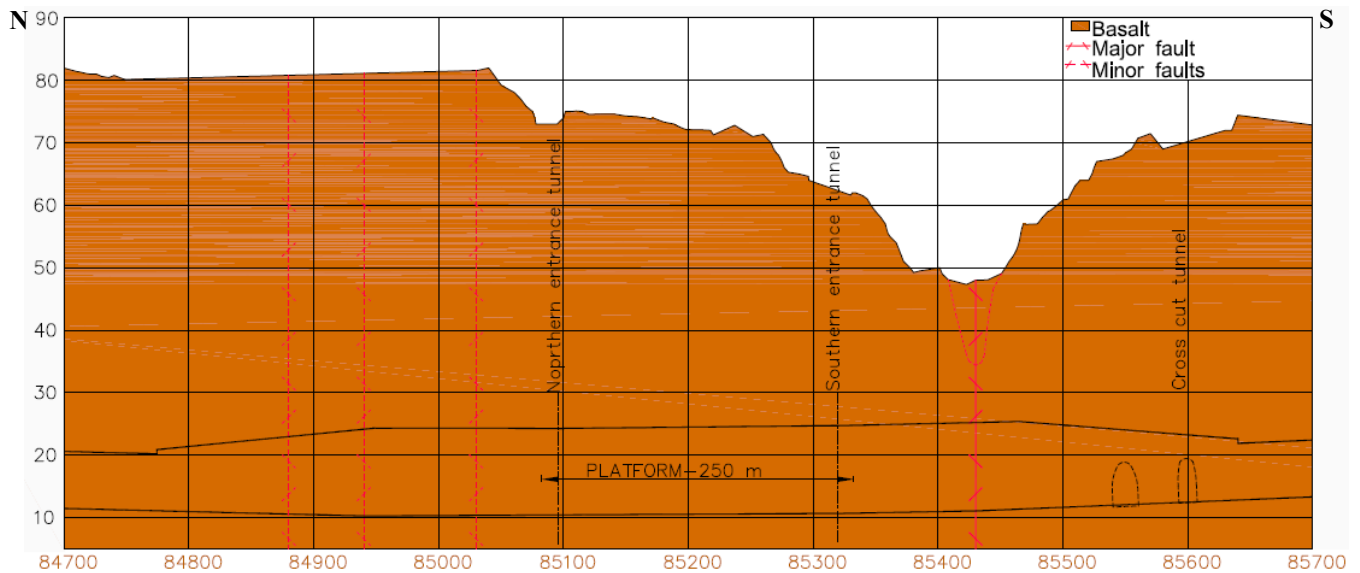


Fig. 3 Longitudinal Profile of the Railway Station Hall and Tunnels to the North and to the South (Investigations indicate a limited rock overburden of 9 m in the area where the tunnel is excavated through the major fault)

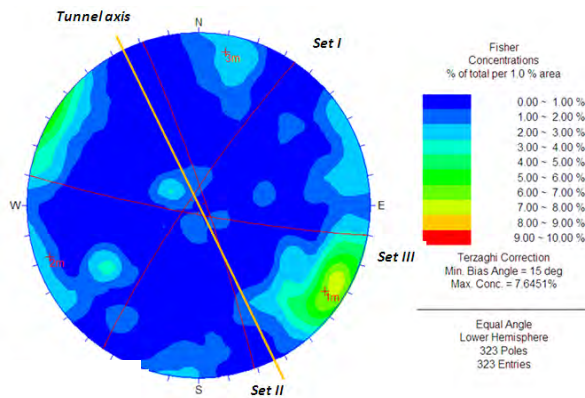


Fig. 4 Stereonet Plot Showing Three Steeply Dipping Joint Sets in the Area (The fourth joint set is a sub horizontal joint set. The yellow line indicates orientation of the tunnel)

| Set | Dip/dip-direction | KH4   |         | KH5   |         |
|-----|-------------------|-------|---------|-------|---------|
|     |                   | Freq. | Spacing | Freq. | Spacing |
| I   | 83°/304°          | 0.54  | 1.9     | 1.49  | 0.7     |
| II  | 86°/068°          | 0.55  | 1.8     | 0.61  | 1.7     |
| III | 84°/191°          | 0.27  | 3.7     | 0.22  | 4.5     |
| IV  | 17°/208°          | -     | -       | 0.57  | 1.7     |

Table 1 Average Fracture Frequency and Spacing for the Joint Sets I-IV Logged in Boreholes KH4 and KH5

- To the south of the major fault; only gravitational stresses exist.
- To the north of the major fault; remnant stresses exist:  
 $\sigma_h = \sim 1.7\text{--}3.9$  MPa, oriented parallel to the tunnel.  
 $\sigma_H = \sim 3.0$  MPa (might be as high as 11 MPa), oriented approximately perpendicular to the tunnel.  
( $\sigma_h$  and  $\sigma_H$  refer to the minor and major horizontal stress, respectively)

## 5. Main Challenges

Given the geology and the design criteria, two main challenges have been identified in this project. A combination of unfavorable oriented joint sets forming potential unstable wedges in the roof of the large spanned railway station, and excavation through a major fault located to the south of the railway station, where the span of the tunnel is 30.5 m. These two challenges will be discussed in the following, and solutions will be presented.

### 5.1 Unstable Wedges in the Crown

Use of the program “Unwedge” identified early in the design process the potential of large, unstable wedges in the crown of the railway station and the connecting tunnels to the north and south of the station. Modeling in “Phase2” was conducted to design the rock support as shown in Figure 5. The model represents the sub-horizontal bedding (with an average dip of  $17^\circ$ ) as well as the three steeply dipping joint sets (set I, set II and set III) with a spacing corresponding to the true spacing multiplied with a factor of 4. It also includes 6 and 12 m end-anchored rock bolts (spacing = 2.2 m) and 20 cm thick fibre reinforced shotcrete lining.

Numerous investigations including laboratory tests were carried out on the basaltic lavas and resulted in parameters as following:

- Uniaxial compressive strength  $\sigma_c = 102$  MPa (average value from laboratory tests).
- E-modulus for intact rock = 35 000 MPa (average value from laboratory tests).
- GSI (Geological Strength Index) = 67 (calculated from Eqn. 1).
- $m_i = 25$  (obtained from RocLab)
- Disturbance factor = 0 (corresponding to “Excellent quality controlled blasting or excavation by tunnel boring machine results in minimal disturbance to the confining rock mass surrounding a tunnel” in RocLab).
- Poisons’ ratio  $\nu = 0.25$  (the value is estimated, results from testing of the uniaxial compressive strength is 0.17).

$$GSI = 9 \log_e Q' + 44 \quad (1)$$

where  $Q' = RQD/J_n \times Jr/J_a$

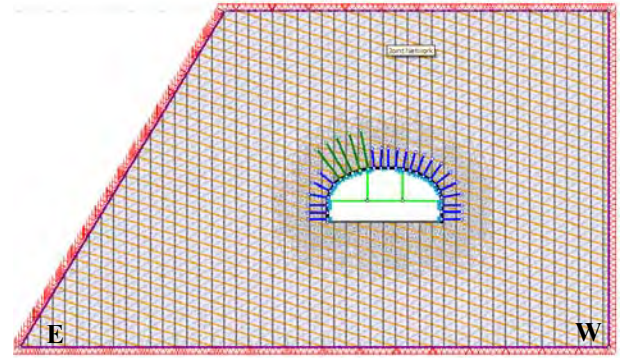


Fig. 5 Outline of the Model in Phase2.

Three stress models were run, as displayed in Table 2, according to results from the stress measurements.

|    | $\sigma_v$<br>MPa | $\sigma_h$<br>MPa | $\sigma_H$<br>MPa | Trend<br>$\sigma_h$ | Trend<br>$\sigma_H$ |
|----|-------------------|-------------------|-------------------|---------------------|---------------------|
| S1 | Gravity           | 1.7               | 3.0               | N140°               | N050°               |
| S2 | Gravity           | 1.7               | 8.0               | N140°               | N050°               |
| S3 | 7.2MPa            | 3.9               | 10.9              | N146°               | N056°               |

Table 2 Input Parameters for Stress Models S1, S2 and S3 (The trend of  $\sigma_h$  is sub-parallel to the tunnel axis, while the trend of  $\sigma_H$  is almost perpendicular to the tunnel axis)

### 5.2 Discrete Element Modeling

To further predict and understand the expected modes of failure to be encountered during the excavation, 3-dimensional discrete element models were developed using Itasca’s “3DEC”. The models are designed to assess how the rock mass will behave under specific in situ strength and stress conditions. In total nine models were evaluated under varied initial conditions. Specifically, the structural discontinuities near the station hall excavation boundary were under evaluation.

The 3DEC models run in the stress regimes reported in Table 1 all displayed similar modes of failure. The two main processes are the release of individual blocks (Fig. 7) and convergence of large zones (Fig. 8).

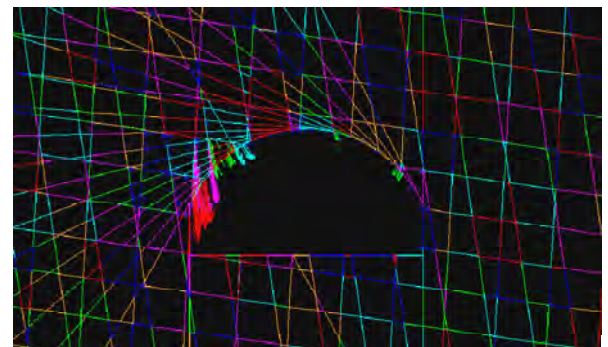


Fig. 6 Displacement Vectors Mapped to Individual Blocks (Individual block failure on left side of the model)

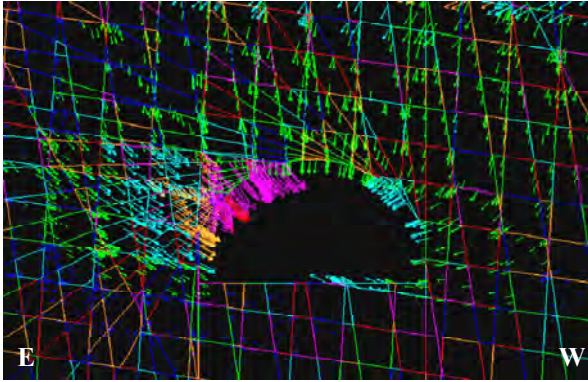


Fig. 7 Displacement Vectors Mapped to Individual Blocks (Large zone of convergence on the left side of the model)

### 5.3 Excavation Process and Support Design

The peak compressive strength of the fibre reinforced shotcrete lining is 35 MPa, and the residual compressive strength is to 5 MPa. The peak and residual tensile strengths is 5 and 0 MPa, respectively. The model was loaded according to:

- Stage 1: 30% of the total stress field was induced, simulating excavation of the tunnel before installation of rock support.
- Stage 2: The remaining 70% of the total stress field was implemented, simulating the installation of rock support.

The load split simulates that some deformation is will occur in the rock mass before the tunnel face is excavated and then supported. The load splitting is in conducted in accordance with state-of-the-art practices

as discussed by Vlachopoulos and Diederichs [2009]. The magnitude of the split factor can however be discussed.

Modeling resulted in the design of 12 m long rock bolts to be installed in the tunnel crown to the east as shown in Figure 8. Analysis of the results from stress model “S1” illustrate that the surrounding rock has failed at multiple locations and the peak strength of several joints has been exceeded. However, the load carried by the rock support is much smaller than the capacity:

- The maximum axial force in the lining is 2.3 MN, corresponding to 11.5 MPa (thickness of lining = 0.2 m), which is 1/3 of the peak compressive strength of the lining.
- The minimum axial tensile force in the lining is -0.2 MN, corresponding to -1.0 MPa, which is 1/5 of the assumed tensile strength of the lining.
- The maximum axial force in a rock bolt is 0.11 MPa, which is 1/2 the tensile strength of the weakest bolt types which will be used during construction.

The railway station is excavated in a total of five sequences (three sequences at the top bench, and two sequences in the bottom bench as indicated in Figure 9). In this design, the eastern side of the top bench will be excavated first and the long bolts will be installed at the working face immediately. The bolts act as initial support, as well as being part of the final support. Also, the strict requirement due to water-inflow to the tunnel necessitates the need for bolts which can be grouted under high pressures (up to 100 bar). However, since

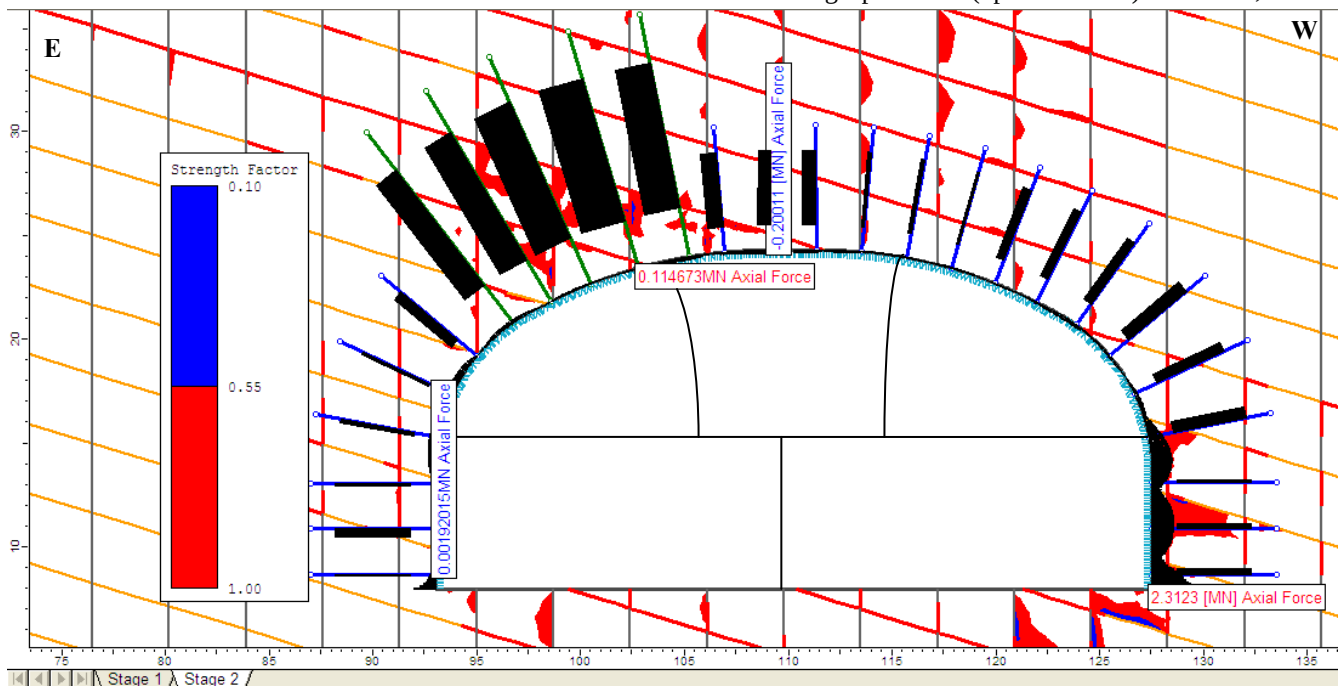


Fig. 8 Results of Stress Model “S1”: Axial Forces in the Lining (Black areas) and Rock Bolts (Black Rectangles) (Strength factor (ratio between rock strength and induced stresses at any point) less than 1 in the surrounding rock mass and yielded joints (red joints). Excavation sequences are also shown in the profile.)

the rock mass must be allowed to deform before grouting of the bolts is carried out, special designed end-anchored bolts which can be grouted at high pressures behind the tunnel face has been selected.

#### 5.4 Crossing of a Major Fault with a 30.5 m Spanned Tunnel

The fault to the south of the railway station is a major feature in the area; the fault can be traced as a lineament for several kilometers. In order to investigate this fault, two inclined boreholes were drilled through the fault (one the northern and one from the southern side of the fault). The core material revealed a central zone in the order of 15 m containing mainly crushed material. As indicated in Figure 2, the rock overburden is limited to 9 m in the area where the tunnel is being excavated through the fault. In addition, stress measurements has revealed that only gravitational stresses occur in this area, resulting in the need for careful design of the excavation procedure and rock support through this region.

In order to obtain information regarding the fault before the excavation, the tender documents have described three drillcores with a so-called “triple-tube system” which allows potential fine material such as clay to be obtained in the drillcore. Figure 10 shows the planned core drilling.

An excavation procedure including excavation in nine sequences as shown in Figure 9 is described in the tender documents.

Rock support in this area was designed based on analysis of modeling in Phase2 (not described in this paper). The support will consist of lattice girders, shotcrete and bolts. The excavation method is in principle as following:

1. The rock mass is supported with spiling bolts before section 1a is excavated. Rock support consisting of 20 cm fibre reinforced shotcrete and bolts are then to be installed. Section 1b is excavated and supported in the same way. The blast length is set at 3.0 m for these sections, but will be reduced if the rockmass conditions are poorer than anticipated. A section of the lattice girder is installed every 0.5 m and incased in shotcrete. Also, lattice girders for temporal support are installed every 0.5 m in the wall against section 5 and incased in shotcrete.
2. Section 2a and 2b are excavated and supported as described for section 1a/1b.
3. Section 3 is excavated after installation of spiling bolts. Rock support consisting of 20 cm fibre reinforced shotcrete and bolts are installed. The blast length is generally 3.0 m, but will be reduced if the rockmass conditions are poorer than expected. New sections of lattice girders are installed and

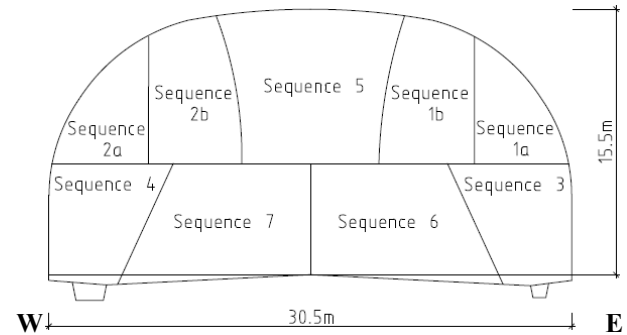


Fig. 9 Excavation Sequences through the Major Fault (Sequence 1a and 1b, as well as 2a and 2b, can be excavated in one sequence if the rock mass condition is good)

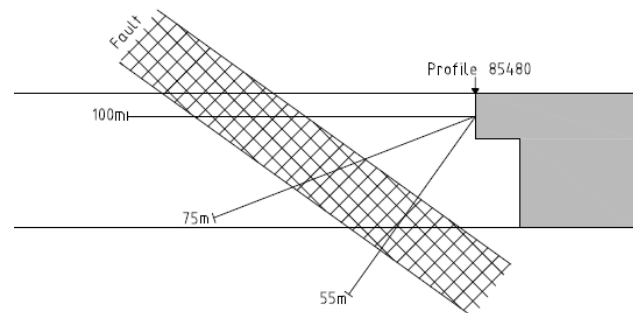


Fig. 10 Core Drilling through the Major Fault from “Sequence 1a/1b” in Fig. 9

connected to the lattice girders installed in the already excavated sections above. The lattice girders are incased in shotcrete. If the rock mass conditions are poor, a concrete base foundation for the lattice girder will be constructed. Suitable temporary support is installed in the wall next to section 6.

4. Section 4 is excavated and supported as described for section 3.
5. Section 5 is excavated after installation of spiling bolts. Rock support consisting of 20 cm fibre reinforced shotcrete and bolts are carried out. The blast length is set to 1.5 m. New sections of lattice girders are installed in the roof and connected to the lattice girders installed in the already excavated sections to the left and to the right. The excavated profile is now fully supported by the lattice girders.
6. Section 6 and section 7 is excavated.

It must be emphasized that the excavation method described above is preliminary, and that the final decision on how the excavation will be performed is made after the drilling cores through the fault have been obtained and analyzed.

#### 6. Conclusion

A large-spanned railway station with connecting tunnels to the north and the south of the station is designed; the station will be one of the largest

underground railway stations in the world when construction is completed in 2016. The span of the railway station is 34.5 m and the length is 250 m. The design of rock support and excavation methods has been challenging due to the large span in combination with geological features such as unfavorable jointing, lack of horizontal stresses and the crossing of a major fault.

One result observed in all 3DEC models is the influence of the bedding planes in the crown on overall tunnel stability. If the uppermost day lighting bedding plane is observed in the crown, the failure of large wedges of rocks is observed in the models. However, if the uppermost day lighting bedding plane is observed in the haunch the failure is generally restricted to smaller groups of individual blocks. This interaction between the tunnel crown and dominant bedding planes should be monitored closely during construction and supported appropriately.

In the railway station, the need of 12 m long rock bolts in the eastern part of the roof has been identified due to the joint sets forming potential large unstable wedges. Excavation through a major fault south to the railway station is carried out by excavation of the profile in nine sequences. Final decision regarding the excavation procedure through the fault will be based on drill coring through the fault from the head of the tunnel.

### Acknowledgements

The work reported in this article has been financed by Norwegian National Rail Administration (JBV) and the authors would like to thank them.

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# THE DESIGN OF ROCK CAVERNS FOR UNIVERSITY AND SAI YING PUN STATIONS

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## Abstract

The only existing example of a Mass Transit Railway Corporation (MTRC) station rock cavern is Tai Koo, constructed in 1985. In 2011 two new underground stations, Sai Ying Pun Station (SYP) and Hong Kong University Station (HKU) are under construction. Unlike Taikoo the West Island Line (WIL) rock caverns will be constructed beneath dense urban areas which comprise a variety of existing structures. The new stations will form a westward extension to the MTRC Island Line. This paper presents in outline the design concepts for SYP and HKU caverns and summarises the site constraints and ground conditions influencing concept selection. The analytical approach adopted for design of the temporary and permanent excavation support is summarised. Key areas including the method of rock mass classification and the methodology to determine ground loading on the cavern lining is presented.

## Keywords

Geological Models, Rock Mass Classification, Cavern Engineering, Drained Caverns, Permanent Cavern Lining, Discrete Element Model

## 1. Introduction

The West Island Line (WIL) will extend the Mass Transit Railway Corporation (MTRC) Island Line (ISL) from the existing Sheung Wan Station (SHW) to the western district of Hong Kong Island, and add approximately 3.3 km of underground route to the existing Line. Three new underground stations, Sai Ying Pun Station (SYP), Hong Kong University Station (HKU) and Kennedy Town Station (KET), will be built along the line. KET will be a cut and cover station, and SYP and HKU will be deep cavern stations. Figure 1 shows the underground view of the WIL.

Selection of a cavern option was primarily on account of the lack of available land in the densely populated areas of the western district. The alignment was conceived following evaluation of a number of alternatives taking into account the physical constraints (for example existing buildings and their foundations), the need to connect the three new stations (SYP, HKU and KET) to ISL west of SHW and the need to locate the caverns within a competent rock mass. The depth

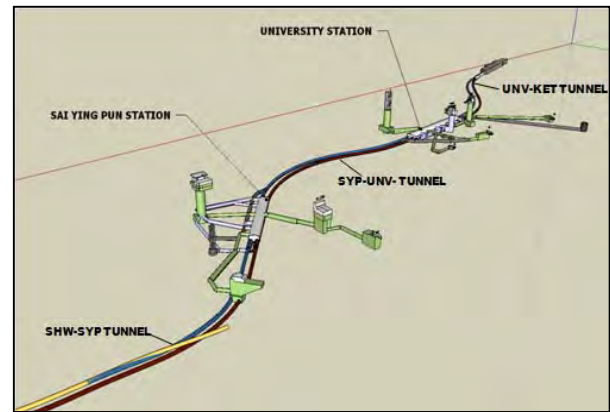


Fig. 1 Underground View of the WIL

of the proposed cavern stations vary between 50 m and 80 m below ground level.

The two WIL caverns will be the first station caverns built by MTRC for over 25 years and will be the first caverns of these spans to be built beneath Hong Kong's densely urbanised areas. There was little local precedent to guide development of the design concepts, which also had to be developed under close scrutiny from Buildings Department and the Geotechnical Engineering Office. Experience with cavern engineering from overseas was combined with knowledge of local practice and design standards to produce an approved reference design.

## 2. Site Condition and Structural Geology

The proposed SYP cavern is located beneath the colluvium-covered sloping ground at the foot of Victoria Peak, beneath the Centre Street and High street area. More than 30 buildings constructed on various types of foundation, such as friction piles, end-bearing piles and pad footings, are located above the footprint of the cavern. The station is orientated in a southwest-northeast direction (Fig. 2) with the ground level dipping steeply towards the north from over 50 mPD in the southwest to less than 30 mPD in the northeast.

The proposed HKU cavern is located in an area of sloping ground to the north of Lung Fu Shan, beneath Pokfulam Road, with a number of existing buildings around the area (Fig 3). The cavern is orientated in an east-west direction, and ground levels decrease from +80 mPD at the western end to less than +40 mPD at the eastern end of the cavern.

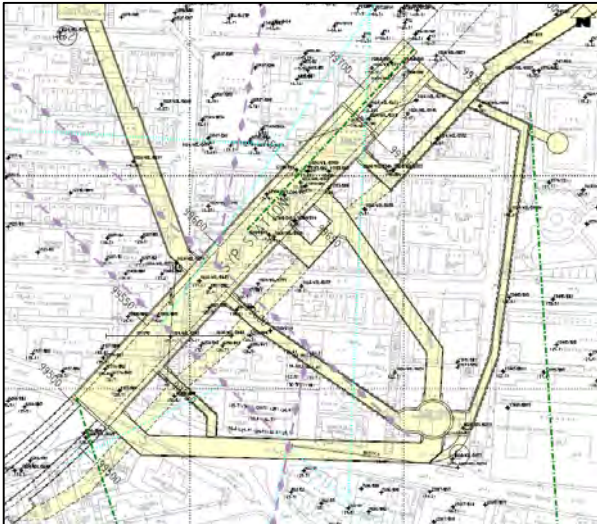


Fig. 2 General Layout Plan of SYP Station

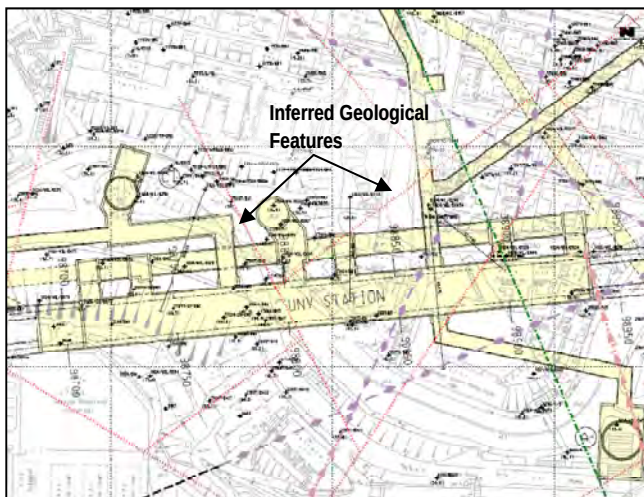


Fig. 3 General Layout Plan of HKU Station

The published geological maps of the region indicate the overall trends of faults with northeast-southwest and northwest-southeast orientation. HKU station has an east-west orientation and is therefore intermediate to each of the trends, while SYP is sub-parallel to the dominant northeast-southwest trend. No major faults are shown in the vicinity of SYP and a single fault with a northwest-southwest orientation is shown just to the south of HKU, which if extrapolated towards the northeast intersects the eastern end of the HKU cavern. A number of other lineaments, representing possible geological structures had been identified by studies of the topography and aerial photographs [MTRC, 2008a].

Both vertical and inclined boreholes were planned to investigate the rock mass conditions. The aim of the inclined holes was to provide data on the subvertical structures, including major structures and faults which could impact on the stability of the cavern excavation. However, due to access difficulties and the presence of existing buried utilities, complete coverage of the cavern areas was not possible. The evidence for most of the inferred geological features was limited.

However, one of the inferred features predicted to intersect the eastern end of the SYP cavern and platform tunnels was encountered in several boreholes. This adverse geological feature, was characterised by closely spaced structure and altered material some 0.7 m thick. The eastern end of the SYP cavern thus comprises a shallow rock cover of partially weathered rock mass intersected by an altered adverse geological feature. In order to investigate this feature further and to provide an opportunity for detailed characterisation, and also if necessary to undertake treatment, an advanced pilot tunnel, from the construction shaft at King George V [KGV] Memorial Park was recommended. The pilot tunnel will form a part of the permanent works for access. The anticipated location of the adverse feature and the proposed details of the pilot tunnel are shown in Figure 4 below.

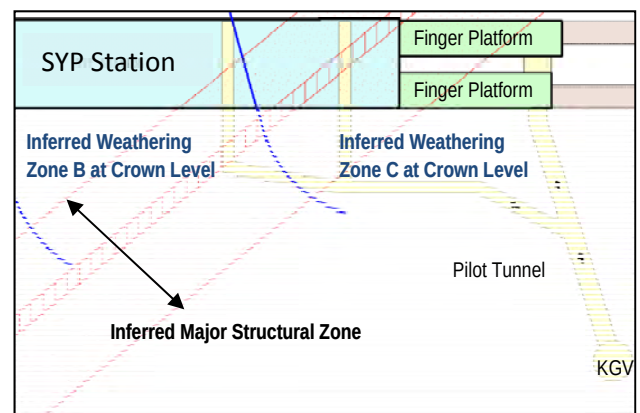


Fig. 4 Envisaged Pilot Tunnel at the Eastern End of SYP

### 3. Cavern Layout

The SYP station is 225 m long and comprises a 187 m long cavern and a 38 m long platform tunnel extending from the northeastern end. The cavern is located approximately 40 - 65 m below ground. It has an excavation span of 22.8 m and a height of 16 m, and will be built with a moderate rock cover of 50 m at the western end and a rock cover of only 10 m at the eastern end. In view of the shallow rock cover at the eastern end of the cavern construction, two-finger platform tunnels with a reduced size of approximately 9.5 m in width and 10 m in height have been proposed. The platform tunnels will be separated from one another by a 3 m wide rock pillar. A summary longitudinal geological profile of the SYP cavern is shown in Figure 5.

The HKU station comprises a 240 m long cavern with an excavation span of 22.4 m and a height of 16 m. The overall depth of the cavern is similar to that of SYP and varies between 48 m and 80 m below ground level. However, the rock cover is greater and is anticipated to be around 65 m at the western end and 25 m at the eastern end of the cavern.

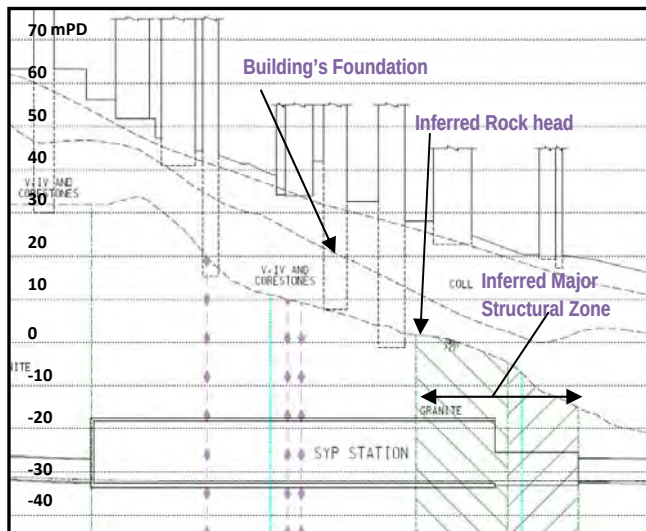


Fig. 5 Longitudinal Geological Section along SYP

The orientations of the caverns were governed by the existing SHW station and its overrun tunnels, allowable turning radius of the railway line, physical constraints of existing structures and their foundations, and the location and depth of competent rock. A further review of the geological data at the detailed design stage concluded that shifting the position of the SYP cavern 50 m towards the west would increase the rock cover significantly and hence reduce the risks associated with cavern construction. However, because of the railway operational constraints on cant and curvature, as stipulated in the MTR Design Standard Manual on railway engineering design, the station was shifted 18 m to the west.

The various rock pillars associated with each cavern were designed taking into account the anticipated quality of the rock mass, joint orientations, and the width and height of the adjacent cavern and adits. Stress concentration and distribution at each construction stage were estimated. The derived layouts have a pillar width typically a half to two-thirds of the tunnel cavern height [GEO, 1992].

#### 4. Cavern Design Concepts

##### 4.1 Excavation Sequence and Support

The cavern will be excavated using drill and blast methods with temporary support to enhance the intrinsic stability state in order to provide adequate stability reserve at all stages of construction. Temporary support will comprise rock reinforcement and shotcrete installed in the crown and sidewalls incrementally as the excavation proceeds. The supports will also be installed within a specified distance from the excavation face in order to benefit from the support provided by the excavation face and to maintain confinement. The maximum advance will be varied according to the ground conditions and proximity of adjacent openings. Rock bolts and shotcrete, which can

be installed quickly and easily, were adopted in the reference design. Preliminary excavation sequences of the caverns themselves together with those of their adjacent adits were developed in the design. Development considered the benefits of various sequences in terms of rock engineering and maintaining stability, access from side adits to the main opening and considered construction issues including typical plant size for the installation of rock bolts with reference to the height of the top heading. The typical sequence is shown in Figure 6.

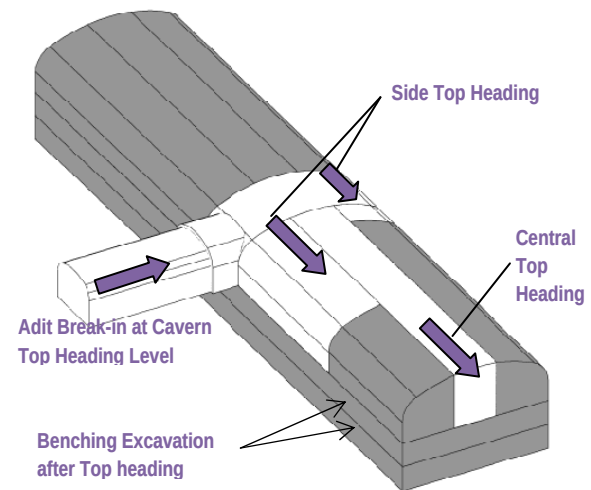


Fig. 6 Excavation Sequence

##### 4.2 Selection of Drained versus Undrained Lining

The key design consideration for both caverns, which has far reaching implications for the overall cavern design was the selection of design concept in terms of a drained versus undrained permanent lining system. In drained lining systems, a mechanism for the relief of water pressure on the lining is provided which results in an economical design in terms of the thickness and reinforcement content of the lining. By contrast, an undrained lining system requires the full water pressure to be supported by the lining, which increases the structural thickness. Straight side walls become difficult to engineer due to the quantity of reinforcement required and a curved invert and base slab are required. As a consequence excavation volume and the cost and complexity of construction are all increased.

A drained system is permanently pressure relieved and allows flow of groundwater into the cavern, which could cause a drawdown of groundwater levels in the superficial materials in the long term. This has implications for settlement and potential consequences for buildings and structures. This risk is particularly pertinent to SYP that is located in the centre of Sai Ying Pun where many old buildings are built on a variety of foundation types including settlement sensitive friction piles.

A hydrogeological assessment was undertaken to study the potential impacts of drained caverns and to examine the influence of depressurisation on the hydrogeological regime. The results suggested that a drained cavern concept would be feasible for both HKU and SYP. A drained system was therefore designed and accompanied by specifications for groundwater control during construction, including for probe holes and water inflow criteria to govern pre-excavation grouting requirements; a comprehensive instrumentation and monitoring scheme; and a grouting design at critical locations.

To reduce the risk of groundwater pressure build up on the lining with time the design includes the installation of a highly conductive geotextile behind the lining to facilitate inflow of water from the surrounding rock mass to a drainage invert of 300 mm thick rock fill. The collected water will be diverted to a sump via drainage pipes located along the cavern invert. A typical waterproofing and drainage arrangement for the cavern lining is shown in Figure 10 in sub-section 4.6.

This paper does not include details of the hydrogeological analysis which involved a comprehensive sub-regional hydrogeological study. It is hoped that a further paper to discuss the hydrogeological assessment for this project will be prepared when more information from construction becomes available.

#### 4.3 Rock mass classification

The Q-classification system is often adopted as a basis for tunnel design in Hong Kong and it is straight forward to apply in terms of design and implementation. As is well known, Q - support correlations are based on the assessment of case studies covering a wide range of rock mass conditions [BARTON,1998]. However, for wide span caverns in weathered rock, perhaps influenced by particular discrete structure or for those with a small amount of rock cover the precedent data is limited. The predictive reliability of Q based support estimation for such conditions is therefore also limited.

A project-specific rock mass classification system was adopted to provide a framework for core characterisation and for developing deterministic geological and rock mass models representative of the range of probable ground conditions for each cavern. The classification is summarised in Table 1 [MTRC, 2008b] [BANDIS, 1997].

|                    |   | Sub-Vertical Structure Type                                                 |           |      |
|--------------------|---|-----------------------------------------------------------------------------|-----------|------|
|                    |   | SV1 & SV2                                                                   | SV3 & SV4 | SV5  |
| Rock Mass Zonation | D | Not anticipated to influence stability directly due to cover of other Zones |           |      |
|                    | C | RM03                                                                        | RM05      | RM06 |
|                    | B | RM02                                                                        | RM04      |      |
|                    | A | RM01                                                                        |           |      |

Table 1 Rock Mass Classification and Zonation

A key attribute of the Hong Kong granite is the weathering state of the sub-horizontal structure which can form weak persistent structures and thus be very influential on stability of large spans [SHARP, 1989]. The rock mass zonation subdivides the cavern area into rock masses with similar attributes in terms of weathering characteristics, particularly the weathering state of the subhorizontal structure. Four zones were identified ranging from Zone A, which mainly contained Grade I and II rock with unweathered sub-horizontal joints, to Zone D, which mainly contained Grade III rock with highly to completely weathered sub-horizontal joints.

Superimposed on and cutting through the weathering zones are a variety of subvertical structure types. From a review of the published geology and the inspection of cores, five principal types can be identified ranging from SV1 comprising widely spaced joints with no infill to, SV4 and SV5 which represent discrete zones of alteration and geological faults.

This rock mass classification system was provisional at the time of the detailed design stage and will be reviewed and updated as data becomes available during the construction process.

#### 4.4 Cavern Shape

Once drained caverns had been selected as the preferred option the cavern shape could be developed. Selection of cavern geometry was driven largely by the aim of maximising the intrinsic stability of the excavation, so as to decrease temporary support requirements and reduce construction risk. Persistent weathered sub-horizontal joints have been identified as having a key influence on crown stability when present. A flatter arch profile was evaluated as a method of reducing the extent of intrinsically unstable haunch areas and minimising potential overbreak. Adoption of vertical sidewalls allows a relatively rapid excavation using vertically drilled bench blasting and simplifies formwork for casting the final lining. The combination of vertical walls and a flat arch roof offer more functional space.

#### 4.5 Design of Temporary Supports

Rock mass models developed by applying the classification system described above were used directly as a basis for Distinct Element Models. The models represented, as faithfully as possible, the distribution of structure types as actually encountered in the borehole core. The force-displacement behaviour of the discontinuities was modelled using the Barton Bandis failure criteria, applied to each discontinuity type based on their physical attributes [BANDIS et al, 2011] [BARTON & BANDIS, 1987 & 1990]. The derived constitutive model is non-linear and stress strain dependent. A typical example of Distinct Element Modelling (DEM) (using UDEC) for the cavern design is shown in Figure 7.

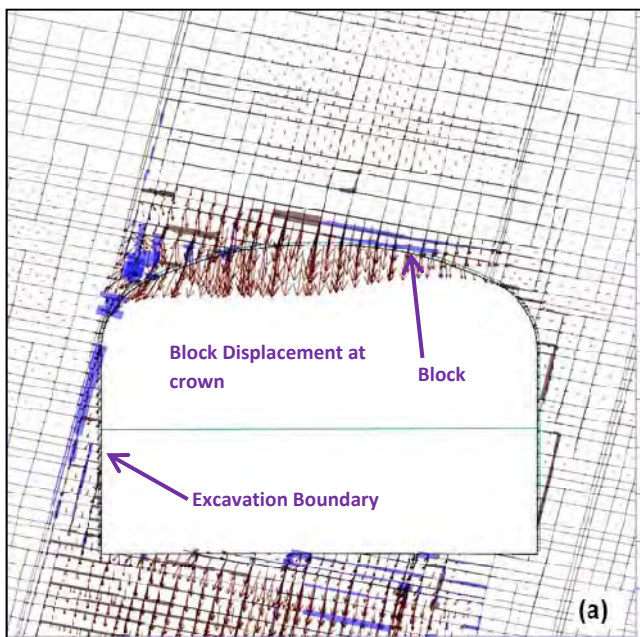


Fig. 7 UDEC Model for Cavern Design

Modelling using UDEC was undertaken to assess the intrinsic stability at each stage of the proposed excavation sequence. The influence of model attributes were studied, including the sensitivity of the rock mass response to strength of particular structure, water pressures and in situ stress. Based on the observed rock mass behaviour, the modes of failure and the extent of overbreak, rock reinforcement layouts were designed and the models re-run with the support installed incrementally with each stage of excavation. Adequate stability reserve was demonstrated at each stage of the cavern cross section development and the loads induced in the rock reinforcement used to design and specify the units. A cross-section of the UDEC model based on rock mass class RM02 is given in Figure 8, which illustrates also the adjacent adits that were included in the model.

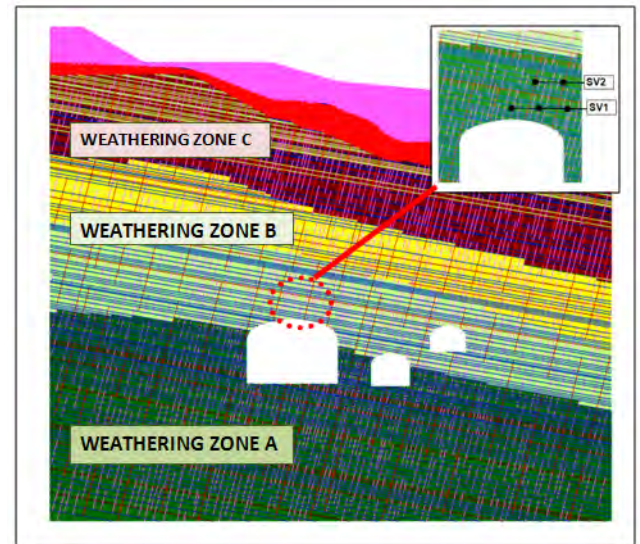


Fig. 8 UDEC Model Rock Mass Class RM02

It is recognised that the outcome of a UDEC model is entirely dependent on the validity of the geological model and related assumptions. These assumptions need to be verified on site based on detailed deterministic records of the geological structure encountered, using the project geological structure classification or similar. From Authors' experience, for the design implementation and verification process for major cavern developments it is not adequate to rely solely on records of Q values of the encountered conditions as the influence of major structure, which can dictate the behaviour of the excavation, become unduly attenuated, and their significance lost.

From Authors' experience, the management of the design implementation and verification process is one of the key risk mitigation tasks for successful cavern engineering. It should be recognised however that the required resources, methods and standards of data presentation are different to those adopted on long, smaller span, tunnel alignments in HK and elsewhere.

#### 4.6 Permanent Cavern Support - Concept

A cast in-situ locally reinforced concrete lining was the preferred permanent support option and was adopted for both caverns. The principal benefit was to enable a water proof membrane to be placed in order to achieve the high standards of water proofing required for an MTRC station [MTRC, 1997]. The lining was also designed to support the concourse base slab. A secondary benefit was installation of the overhead electrical and mechanical (E&M) system which would be more difficult with a permanent shotcrete lining. The concrete linings have been designed for long term durability with the aim of minimal maintenance costs.

The performance of the final lining in terms of structural efficiency is sensitive to the shape of the cavern, and is also influenced by the geometry of intersecting adits. Three cavern arch profiles ranging from a relatively circular profile, to an elliptical arch and a flat arch were studied to optimise lining design, and are illustrated in Figure 9. Sensitivity analyses showed that the haunch zones are prone to build-up of bending moments, particularly in the case of haunches with small radius.

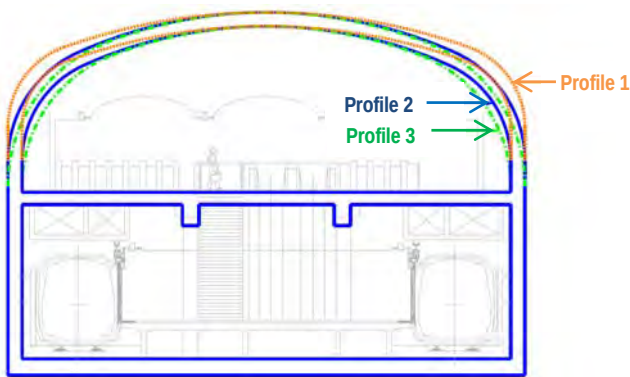


Fig. 9 Sensitivity Check on Cavern Shape

All adits that connect to the cavern will have internal finished levels coincident with that of the concourse slab. High horse-shoe adit openings were essential to provide sufficient space to house ventilation ducts and E&M equipment. As a consequence the crown of the adits will intersect the main cavern at the haunches rather than within the vertical sidewalls. Two alternatives were considered as follows:

1. Increase the height of the cavern such that the adits intersect with the straight sidewall of the cavern to form a simple interface. This reduces bending moments in both adits and the cavern lining but increases the excavation volume;
2. Maintain the height of the cavern and let the top of the adit intersect with the cavern haunches. This has the advantage of optimising the excavation volume but increases the complexity of interface and design of the reinforced concrete collar.

By examining different models, a geometry was developed which maintained the optimised external excavation profile and an internal profile with a larger radius as shown in Figure 10. This arrangement allows for an efficient final lining and improves the collar geometry.

To minimize the number of different shutters required to form the cast in-situ concrete lining the cavern crowns, SYP and HKU adopt the same profile, except for a 400 mm flat section at SYP that has been provided in the centre point to accommodate the larger span.

#### 4.7 Methodology for the Determination of Rock Loading

The rock loading for the permanent lining design was first estimated based on an empirical approach in accordance with “Updating of the Q-system for NMT” [GRIMSTADT & BARTON, 1993] and “Engineering Classification of Rock Masses for the Design of Tunnel Support” [BARTON et al,1974].

The rock loads were also derived from the two-dimensional UDEC simulations. These loads are derived from the modelled interaction of the lining with the rock mass surround and provide loads representative of the actual rock mass behaviour and the stiffness of the lining as constructed.

Cast in-situ concrete lining was adopted for the permanent support of the caverns. The linings were designed by elastic analysis using the rock loading obtained from the UDEC model until effective stress conditions. A waterproofing membrane was provided behind the final lining to meet the requirements of water tightness. The general arrangement of water pressure relief and waterproofing membrane is shown in Figure 10. The main components are as follows:

- target radial drainage holes drilled through the shotcrete lining and into the surrounding rock;
- perforated, circumferential PVC drainage pipes (or Geodrains) (at 5 m c/c along the station), strapped and shotnailed to shotcrete substrate, and connected to invert drains;
- protective and drainage geotextile held in place by shotnailed ‘roundels’; and
- waterproofing membrane with double-welded (overlap) joints.

There are nine adit connections at HKU station and seven at SYP station. The internal dimensions of the adits vary from 4 m to 8.9 m in width and 4 m to 6 m in height. Collar structures were used to support the unbalanced loads of the cavern due to connection to the adits. Three-dimensional numerical analyses were adopted to design the intersection with reinforcements allocated in accordance with the averaged strip of bending moment and shear forces. Extra reinforcement was allocated in areas where high stress concentration was anticipated. A computer model for the design of intersection is shown in Figure 11.

The fixing of concrete reinforcement at the crowns of a high and wide cavern is difficult, high working platforms are often required and increase the risk of damage to the waterproofing membrane.

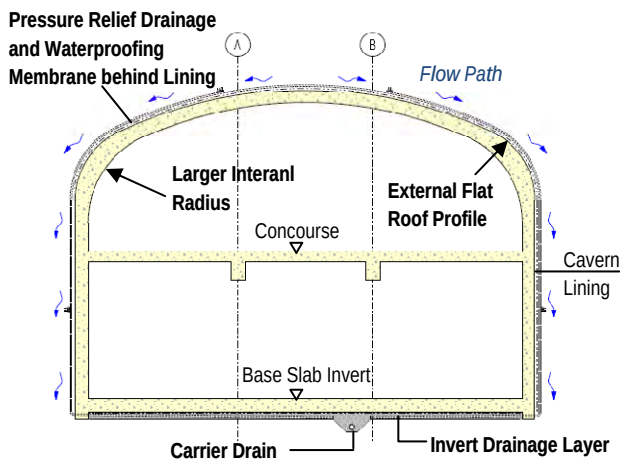


Fig. 10 General Arrangement of Lining and Water Pressure Relief System

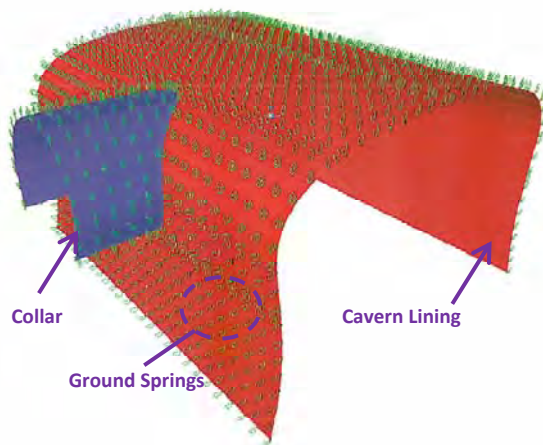


Fig. 11 3D Model for Cavern Adit Intersection Design

The option of using double corrosion protected (DCP) permanent rock bolts to reduce the bending moment of the lining were investigated to determine whether lining reinforcement at the crown could be eliminated. The bolts were introduced into the UDEC models as part of the incremental excavation and support and then were retained in the long term after the lining was formed. From the analysis undertaken it was concluded that, with the use of permanent rock reinforcement an unreinforced cavern crown would be possible in sections of the cavern away from the adit intersections. However, for the sections of the cavern with adit connections, crown reinforcement would still be required even with the use of DCP bolts. Over 60% of the cavern length is influenced by adit connections. In addition it was necessary to provide minimum reinforcement throughout the length of the cavern for fire proofing purposes and for these reasons reinforcement was adopted in the lining throughout instead of DCP bolts.

## 5. Discussion and Conclusions

HKU and SYP illustrate that, with the right topography and geological conditions, railway stations formed within rock caverns can bring significant benefits. The cavern option for WIL allows SYP and HKU to be built in a congested urban area with minimum access required from the ground surface (and thus minimal disruption to the public) and the release of more land for other uses.

Various key factors in the rock cavern design were discussed. The advantages of project-specific rock mass classification systems, which assist in development of distinct element models were outlined, and the benefits of a drained cavern structure described including the influence on cavern geometry.

The design was largely based on information from surface boreholes and published geological data. The extent of this information is consistent with the complexity of the project however uncertainties exist in the models which will have to be developed during construction to facilitate design verification and validation. These stages are a vital component of risk mitigation on major rock engineering schemes. The design will need to be reviewed based upon data obtained during construction and revised as necessary to suit the conditions encountered.

## Acknowledgements

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# EXPLICIT CHARACTERISATION AND INTERACTIVE ANALYSIS FOR ENGINEERING DESIGN OF ROCK CAVERNS

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## Abstract

*Optimum engineering designs for large span underground structures require appreciation of the rock mass responses and in particular the structure-controlled, anisotropic, stress and deformation dependent behaviour. The paper presents the principles and selected applications of comprehensive procedures for explicit geological and rock mass characterisation, design analyses and engineering assessment as the key elements for the interactive excavation - support design of caverns. Emphasis is placed on explicit characterisation as the basis for modelling of the rock mass, derivation of representative input parameters and application of discontinuum analysis techniques. The value of detailed engineering assessment of the design analyses results for developing reliable solutions is discussed and generic, practical guidelines are proposed.*

## Keywords

*Engineering geological assessment, Rock Mass Modelling, Design Analysis, Cavern Engineering*

## 1. Introduction

The design methodology for large scale underground openings has developed significantly over the past 20 years through advances in the understanding of the complex, interactive (coupled) processes involved, as well as explicit simulation of the rock mass as a structural medium.

This paper provides a review of the current practice covering the interactive analysis of the rock mass response – groundwater influence – engineered stabilisation systems. Particular emphasis is given to the analytical simulation of the rock mass in order to provide a comprehensive, reliable model of the intrinsic load bearing response and hence the degree of engineering stabilisation measures required.

The principal elements for the design of underground structures are as follows:

- (i) Engineering geological interpretation to produce zonal subdivision (domains) and subsequent rock mass characterisation of each domain
- (ii) Constitutive (numerical) model development

(iii) Engineering analyses

(iv) Engineering assessment of the analysis results

Stage (i) includes interpretative evaluation and synthesis of data relating to the types, physical state, distribution and engineering properties of the lithological materials and the structural discontinuities as well as, the in situ stress and ground water conditions. The objective is to derive representative models which are amenable to development into deformation-based methods of analysis, which form Stages (ii) and (iii).

Rock engineering practice to date has generally adopted one of two approaches for the characterisation and simulation of rock masses, for the purposes of rock engineering design:

- a discontinuum approach, in which the geological structure is explicitly represented and controls the modes of deformability and mechanisms of ultimate failure of the simulated rock mass
- a continuum approach, which involves an empirical transformation (simulation) of the rock mass from the in situ discontinuous state into an equivalent continuous mass, where the weakening and softening influence of discontinuities is allowed for implicitly

The broad range of rock mass types and the applicable methods of analysis and classification are presented in Table 1.

As indicated, each method is applicable to specific rock mass conditions and cannot therefore be used interchangeably. The equivalent-continuum approach is appropriate where the density and orientation of jointing are such, that no preferential paths of stress-strain responses are present, which perhaps represent less than 20% of rock masses. The discontinuum approach is appropriate when the geological structure controls anisotropy, deformation modes and the constitutive response of the rock mass. In terms of future cavern engineering in Hong Kong within the granitic and volcanic terrains, a discontinuum approach would be applicable to most of the potential sites.

Past design practice has often misused equivalent continuum principles for detailed design because of the attractions of simplicity and apparent convenience particularly for often unjustifiably extensive sensitivity

analyses. Unfortunately designs still relying on equivalent continuum principles (or the use of discontinuum models that bear little or no relationship with the actual geology) are all too frequently utilised.

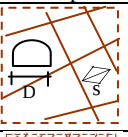
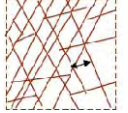
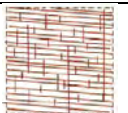
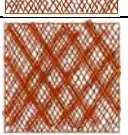
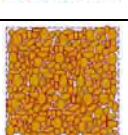
| Description of rock mass                                                           |                                                                                    | Analysis       |          | Classification                                                                                         |
|------------------------------------------------------------------------------------|------------------------------------------------------------------------------------|----------------|----------|--------------------------------------------------------------------------------------------------------|
|   | Intact massive<br>$s > 2\text{m}$<br>$D/s < 5$                                     | Continuum      | FEM, BEM | $Q > 1000$<br>$\text{RMR} > 80$<br>$\text{GSI} > 80$                                                   |
|   | Jointed<br>$s = 0.5\text{--}2.0\text{m}$<br>$D/s = 5\text{--}20$                   | Discontinuum   | DEM      | $Q = > 10 - 100$<br>$\text{RMR} = 61\text{--}80$<br>$\text{GSI} = > 40\text{--}80$<br>(applicable?)    |
|   | Bedded<br>$s = 0.1\text{--}0.5\text{m}$<br>$D/s = 20\text{--}100$                  |                |          | $Q = > 1 - 10$<br>$\text{RMR} = 41\text{--}60$<br>$\text{GSI} = > 30\text{--}40$<br>(applicable?)      |
|   | Very densely jointed<br>$s = 0.05\text{--}0.1\text{m}$<br>$D/s = 100\text{--}1000$ |                |          | $Q = 0.1\text{--}1$<br>$\text{RMR} = 21\text{--}40$<br>$\text{GSI} = > 20\text{--}30$<br>(applicable?) |
|   | Tectonically disturbed<br>$s < 0.1\text{m}$<br>$D/s > 500$                         | Equiv. Contin. | FEM, FDM | $Q < 0.1$<br>$\text{RMR} < 21$<br>$\text{GSI} < 20$                                                    |
|  | Rockfill or talus material<br>$d = 0.01\text{--}0.5\text{m}$<br>$D/d > 10000$      | Continuum      |          | $Q < 0.01$<br>$\text{RMR}, \text{GSI}$<br>not applicable                                               |

Table 1 Broad Distinction of Rock Mass Types in Relation to Classification and Method of Analysis

## 2. Principles of Underground Engineering Design

The challenges of underground engineering design in rock contrasts with other engineering designs, in terms of the extent of the uncertainties and residual risks as well as the practical limitations of geological data acquisition (and resulting parameter definition). In addition there are no Design Codes covering key issues such as acceptable limits of rock mass deformation or levels of stability reserve against failure (traditionally considered as a factor of safety).

Excavation support designs have in the past often been based on a combination of empirical and numerical techniques. Rock mass quality classification schemes, such as Q or RMR were often used, (notably in the early stages of design), to classify the rock mass into categories of quality and also to derive temporary support schemes (reinforcement and shotcrete) for the corresponding rock quality categories. The empirically predicted temporary support schemes were then incorporated into numerical models of the rock mass and their adequacy examined.

Whilst this approach appeared rational it can lead to a false appreciation of stability state, particularly if inappropriate numerical methods are adopted. It is

clear that if the rock mass is poorly represented in the design analysis models or if globalised characterisation schemes for strength and deformability parameters are adopted (pseudo-continuum approach), meaningful physical understandings of response are lost and hence the implied engineering support issues and levels are uncertain and as a result prone to undue risk.

### 2.1 Summary of Key Requirements

The basic requirements of underground engineering design can be summarised as follows:

- Use of a credible proven approach such that the required stability state can be reliably demonstrated for the range of geological conditions that could be encountered and with the adequacy of engineering measures explicitly demonstrated.
- Adequate flexibility in terms of support application to address any potential variations, as may be encountered in situ.
- Provision for construction issues, uncertainties and contingencies as to how these will be addressed during the works execution.
- Incorporation of best engineering practice, proven technology and experience.

### 2.2 Design Implementation

Due to inevitable uncertainties in geological prediction, a planned design implementation process, that includes designer involvement and appropriate expertise, should be treated as an essential component of a design methodology. The key elements of the process should include:

- Specialist evaluation of geological conditions ‘as excavated’.
- In-line performance monitoring of key indicators (deformation, stress change, groundwater etc) in order to validate design predictions.
- Design model assessment and review (confirmation) for each construction stage.

Performance monitoring is recognized as being a significant element of a comprehensive design approach. It should however only be utilised as a confirmatory process for a comprehensive predictive design, and not relied upon as a primary input for support application (as for instance, might be applied in “NATM-based” tunneling procedures).

## 3. Engineering Geological Models and Rock Mass Characterisation

The preparation of geological models and subsequent rock mass characterisation for Cavern Design should be treated using a coordinated approach and aimed at development of the following:

- Representative geological zonation
- Deterministic rock mass (constitutive) models as a basis for design analyses
- Realistic representation of the stress-deformation behaviour of both the rock materials and discontinuities
- Related in situ stress and groundwater states

On account of the overriding influence of geological structure, in the majority of cases involving development of large underground space, deterministic discontinuity representation and associated explicit characterisation is generally required as the basis for design. (This requirement may often be overlooked in favour of non-specific rock mass classification schemes).

### 3.1 Generic Engineering Classification Schemes

Generic schemes of rock mass quality rating have been devised as tools for handling the geological and geotechnical variability. Q, RMR, RSR, BGD, RMI, GSI are the acronyms of the more widely applied classification schemes.

Notwithstanding the broad contribution to the understanding of rock mass quality, the development of classification schemes has in some cases been extended beyond their useful potential. The expansion of empirical classifications into the development of pseudo-continuum rock mass design parameters, for example, raises particular concerns as to the inability to properly represent certain failure mechanisms commonly governed by geological structure (which is either attenuated or completely lost).

The above limitations, potentially set the Q system apart from other systems in that the constitutive parameters involved have been established based on case studies, where stability was assessed largely in terms of physical causes and mechanisms (usually related to geological structure). Implicitly, by representing back-analysed parameters, the Q scheme recognises to some extent the discontinuous, anisotropic nature of rock masses.

### 3.2 Explicit Engineering Characterisation for Cavern Design

Precedent practice on several major underground engineering schemes has involved the semi-deterministic definition of rock mass structure. The approach is based on the application of site specific classifications of discontinuities via inspection of outcrop, core and core photographs and reference to televiewer data. The characterisation procedure also incorporates material types present where lithological variability is important, notably in weak, bedded rock masses or variably weathered rock masses.

An example of borehole data compilation into a rock mass model cross section involving the principal material and discontinuity types is illustrated in Fig. 1.

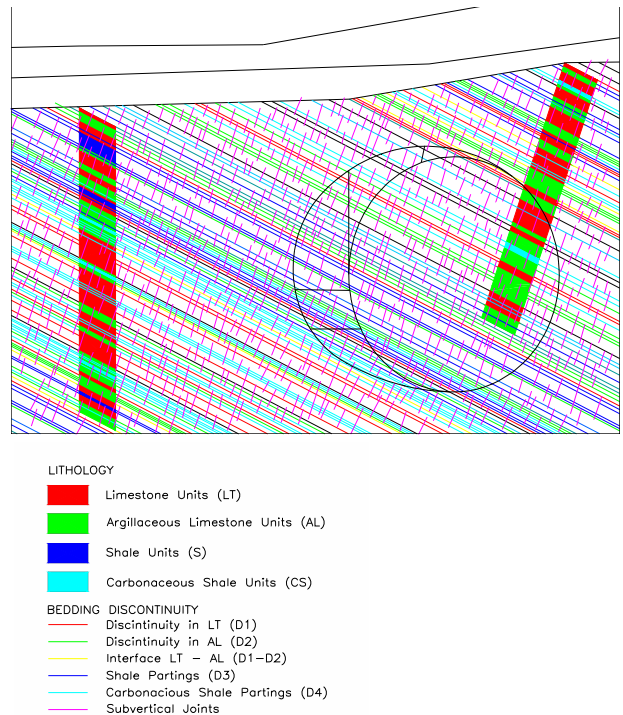


Fig. 1 Explicit Model of a Bedded Rock Mass (Dublin Port Tunnel Crossover) [GEO-DESIGN, 2004]

An example of the application of a specific classification scheme applied in design models for the Caverns of West Island Line, HK is outlined below.

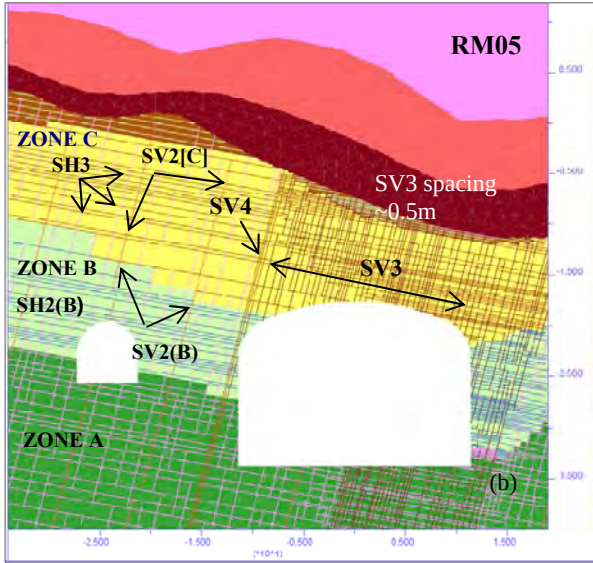
From a review of the published geology, inspection of core and outcrop, principal types of sub-vertical (SV) and sub-horizontal (SH) structure were identified, namely:

- SV1 minor joints – SV2 major joints - discrete persistent structure – SV3 joint swarms – SV4 discrete faults.
- SH 1 unweathered – SH 2 heavily stained joint sidewalls, no infill – SH 3 heavily stained walls and typically <5 mm infill – SH 4 - weathered walls and infill of HDG/CDG >10 mm <200 mm.

Each structure type was defined and characterised based on their physical characteristics such as persistence, roughness, aperture, degree of associated alteration, infill and wall strength. These characteristics provided the basis for derivation of the roughness (JRC), wall strength (JCS) and residual friction indices of the Barton-Bandis (BB) model for shear strength and stiffness modelling.

Rock mass classes were established from a matrix classification of weathering zone versus structure type and were developed on representative cross sections. As an example case, the Sai Ying Pun (SYP) station

design model in “poor” (RM05) quality rock mass class is illustrated as typical examples in Figure 2.



Note: Rock mass class RM05 characterised by presence of weathering zone C at Cavern crown, of typical RQD 80-100%,  $Q=0.5-1.0$  and predominantly SH3 & SV3 discontinuities. Fault feature SV4 present at RH haunch.

Fig. 2 Sai Ying Pun Cavern, Design Model in Poor Rock Mass Conditions [GEO-DESIGN, 2009]

### 3.3 Shear Strength and Stiffness Behaviour of Discontinuities

Important behavioural features of discontinuities include non-linearity, stress-, displacement- and scale-dependency. These aspects are allowed for in the Barton and Bandis (BB) model for predicting shear strength and stiffness parameters. The model is implemented in the version of UDEC-BB, which relates the shear strength to the normal stress using the equation:

$$\tau_{peak} = \sigma_n \tan \left[ JRC_n \log_{10} \left( \frac{JCS_n}{\sigma_n} \right) + \Phi_r \right] \quad (1)$$

The required input data for modelling discontinuity behaviour according to the BB model are:

- $JRC_0$  = Joint Roughness Coefficient at  $L_0$  scale
- $JCS_0$  = Joint Compressive Strength at  $L_0$  scale
- $JRC_n$  = Joint Roughness Coefficient at  $L_n$  scale  
( $JRC_n = JRC_0 (L_n/L_0)^{-0.02JRC_0}$ )
- $JCS_n$  = Joint Compressive Strength at  $L_n$  scale  
( $JCS_n = JCS_0 (L_n/L_0)^{-0.03JRC_0}$ )
- $\Phi_r$  = Angle of Residual Friction
- $E_0$  = Initial Mechanical Aperture at  $L_0$  scale

A particular and important requirement of discontinuity modelling is to allow for the effects of scale on the shear strength and stiffness behaviour. Barton and Bandis [1985] developed the concept of the

“effective” block size, which represents an elemental discrete unit, beyond which scale effects in the shear behaviour along the block contacts (bounding discontinuities) probably diminish to insignificant levels. The sides of the “effective” blocks define the “effective” length of joints ( $L_n$ ), which forms the basis for scale effect allowances in the shear strength and stiffness parameters. In well-developed rock mass structure the effective block size is represented by the natural block size. Otherwise, the network of fractures representing the relatively better developed sets of discontinuities, are the ones which determine the effective block size.

Examples of the range of behaviour predicted by the BB model for different types of discontinuities are given in Figure 3(a) & (b).

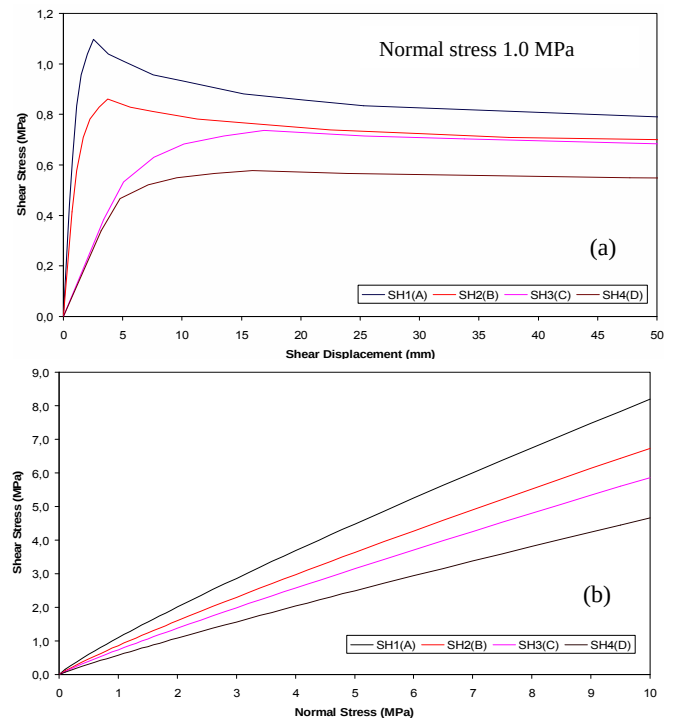


Fig. 3 B-B Model Derived Shear Stress-displacement Curves and Peak Shear Strength Envelopes for Different Types of Discontinuities

### 3.4 Groundwater Pressure

Hydrogeological studies are an integral part of the rock mass characterisation studies in order to establish the head profiles and GW pressure distribution in relation to construction, including assessment of groundwater drawdown impacts.

An example case of the distributions of groundwater pressure assumed for the SYP Cavern is illustrated in Figure 4. The inferred patterns represent partial drainage with zero pressure at 1m from the excavation periphery, gradually reverting to full hydrostatic pressure through inferred intermediate regimes according to the permeability profile.

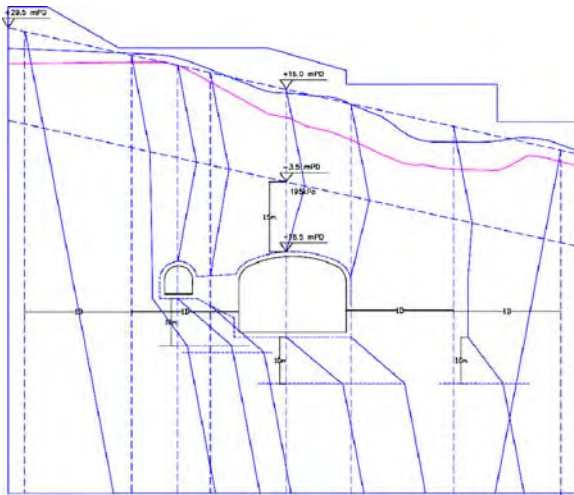


Fig.4 Distribution of Groundwater Pressure as Modelled in the SYP Cavern Design Analyses

#### 4. Design Analyses

The advances over the past few decades in explicit numerical modelling techniques have enhanced greatly the capabilities and scope of engineering simulation and analysis. Such analysis now forms a reliable basis for the design of underground structures and can be utilised to:

- Evaluate and optimise alternative design concepts.
- Address the sensitivity of design to uncertainties associated with the key input parameters (geotechnical, in-situ stress state and groundwater conditions).
- Optimize the temporary support requirements for the expected range of geological conditions.
- Identify and define critical failure mechanisms.
- Assess the final lining requirements under long-term loadings.
- Evaluate the encountered (actual) conditions during project implementation and refine the design accordingly (design implementation stage).

The “Distinct Element Method” (DEM) provides the most appropriate approach and representative simulation for analysis of discontinuous rock mass behaviour. The DEM in rock engineering is implemented by the state-of-art software UDEC and 3DEC, principally developed by Dr Peter Cundall [CUNDALL, 1971, 1980, 1987].

In the UDEC approach, the rock mass structure is explicitly represented and the response to stress changes (including external forces, groundwater pressures etc) is controlled by directly modelled interactions between the solid blocks as a result of induced translational or rotational block movements.

Water pressure in UDEC is represented as explicit water forces acting on the contacts of the discontinuities creating effective stress state in the

fractures and new total stresses in the impermeable rock blocks. Thus both the shearing reactions along the discontinuities and the mode and magnitude of block strain and motion are affected.

UDEC determines if a rock mass will fail under applied loadings based on the specified (derived) discontinuity strength – deformation criteria or otherwise calculates the incremental, progressive displacements that occur within the system. In brief, the generic modelling capabilities of UDEC include:

- Detailed simulation of the rock mass including the fundamental properties of the discontinuities
- Representation of multi-stage construction procedures (excavation - support sequence) in two or three dimensions
- Progressive, path-dependent, interactive load - deformation responses of the rock mass, groundwater state and engineered support system
- Derivation of induced loadings on rock reinforcement and shotcrete lining support systems
- Derivation of long term, final lining interactive loadings
- Effective stress states generated by the construction and long term groundwater regimes

UDEC-BB has been adopted extensively as a design analysis tool in a variety of rock engineering problems, notably for the design of tunnels and caverns including complex underground schemes.

Selected case study examples of UDEC-BB applications are given in Figures 4, 5 and 6 and discussed below.

#### Case Study 1 : Lai King Hill Road Portal, HK - KCRC West Rail - Contract DB350

The case study refers to the detailed design of large span portals, in this case with extreme skew to the face between a cut-and-cover structure and the underground alignment forming spans up to 27m under <10m rock cover.

UDEC analyses were employed to investigate the complex ground-excavation interactions both in intrinsic and supported tunnel response states. The aim was optimisation of the design in terms of excavation sequence and type/quantity of support.

Figure 5a illustrates a critical section of geometry and the rock mass weathering profile at the portal, as represented in the UDEC analyses. Figure 5b is a UDEC vector plot of the displacement responses of the excavation following staged excavation and using a design optimised sequence. A view of the Lai King Road Portal upon completion of the excavation with temporary support in place is shown in Figure 5c.

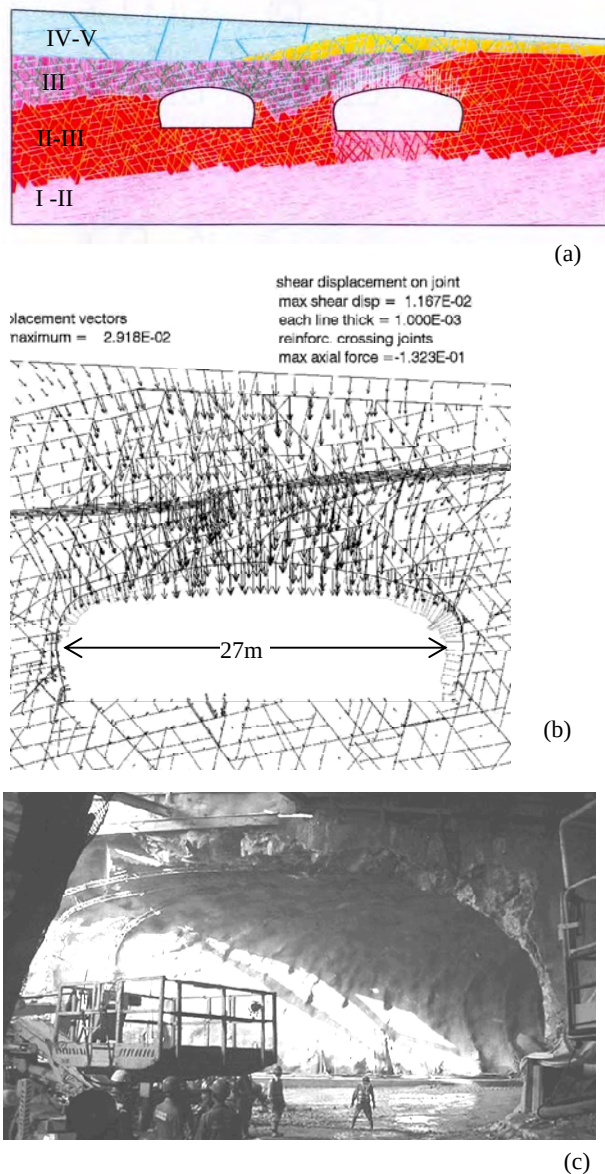


Fig. 5 Lai King Hill Portal – KCRC West Rail [BANDIS et al, 2000]

Case Study 2: West Island Line, HK - Sai Ying Pun Station Caverns - Detailed Design [GEO-DESIGN, 2010]

UDEC analyses were undertaken to cover a range of potential stability and deformation responses as follows:

- Intrinsic behaviour (unsupported) of the caverns
- Detailed stability assessment of incrementally excavated and supported cavern stages
- Response of cavern with permanent concrete lining and derivation of lining rock loadings

Figure 6 illustrates the predicted profile deformations as controlled by temporary support measures (Fig. 6b) and is compared with the response of the unsupported Sai Ying Pun Station cavern simulating mass class RM05 (Figure 6a - NB: geological model shown in Figure 2).

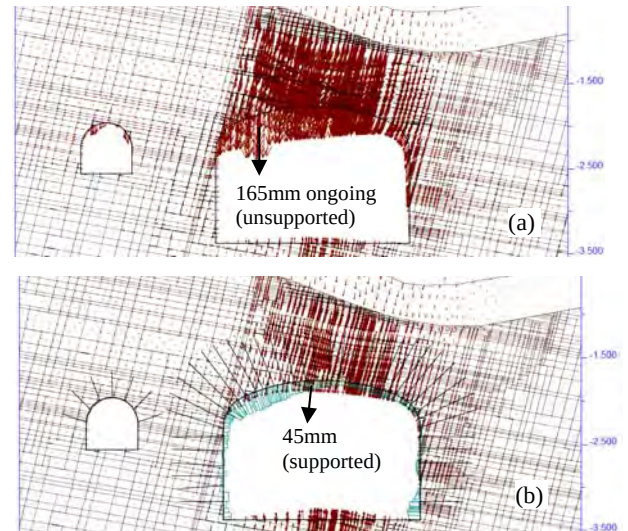


Fig. 6 Intrinsic vs. Supported Deformation Responses of Sai Ying Pun Station Cavern at Full Span

Case study 3 : Triangeln Station Cavern, Citytunneln Project, Malmö, Sweden [SHARP et al, 2008]

Triangeln Station required a ~30 m span rock cavern built beneath buildings at limited depth (ca.15m). Overburden consisted of ca 5 m glacial tills and 10 m weak, bedded, calcareous rock. UDEC-BB modelling studies indicated that a conventional excavation-support development was not applicable due to high loadings on the reinforcement units and shotcrete (Fig. 7a). Alternatives examined included a central footing - column - headbeam system with haunch replacement technology (concrete abutment and sidewall elements) and intermediate shotcrete crown arches (Fig. 7b). The option of a centrally supported cavern was eventually selected as the optimum design concept.

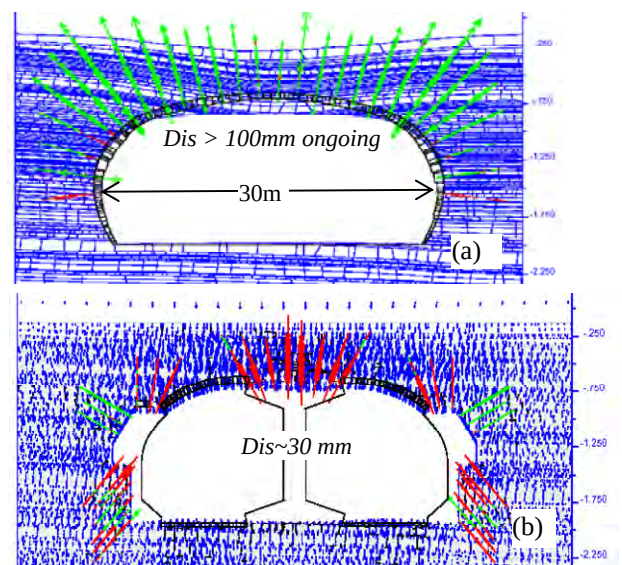


Fig. 7 UDEC Analyses of Concept Designs Without and With Central Support System

#### Case study 4: Independent Design Study on the Predicted Performance of Low Cover Large Span Excavations in Weak, Bedded Rock

The design of shallow cover highway tunnels has been extensively studied by the authors for a stratified sequence of weak sedimentary rocks with dominant sub-horizontal bedding partings with thin clayey interbeds. The purpose of the study was to investigate differing predicted response of explicit discontinuum the (bedded and structurally anisotropic UDEC model) versus equivalent continuum solutions and implicit rock mass strength concepts.

The plots in Figure 8 illustrate that both the implicit continuum and explicit discontinuum models of the rock mass predicted grossly unstable conditions for the intrinsic (unsupported) state, involving:

- (a)Implicit continuum model: Shear yielding of the material above the outer crown zones and extensive destressing in the central crown zones;
- (b)Explicit discontinuum model: Shear overstressing along the bedding partings creating physical delamination and flexure of the rock layers which eventually failed in tension above the central crown and haunch zones.

Although both models predicated poor levels of intrinsic stability, mechanisms of failure were very different in terms of implied failure extent (continuum being a much greater volume). The different deformation mechanisms gave rise to very significant differences between the corresponding ground reaction curves as shown on Figure 9. The equivalent continuum-based ground reaction curves indicated a very much more favourable loading condition compared to the discontinuum-based equivalent.

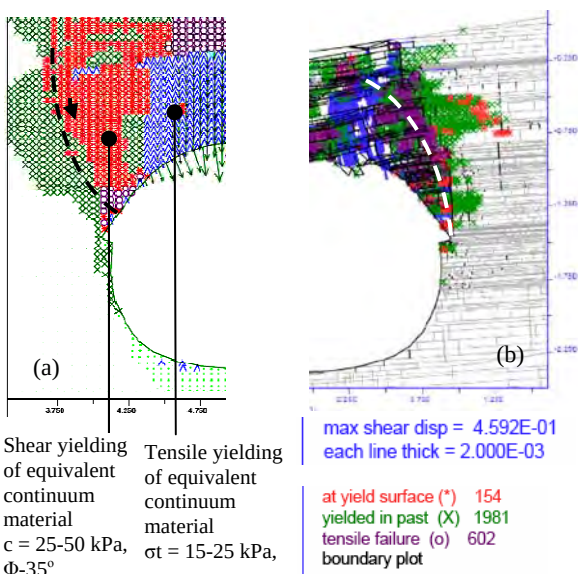


Fig. 8 Comparison of Intrinsically Unstable Behaviour of a Bedded Rock Mass Modelled as (a) Implicit Continuum, (b) Explicit Discontinuum

The discontinuum model provided a realistic simulation of the delamination and bending of the rock layers, developing initially at the tunnel periphery and then extending, with a decreasing failure zone span, upwards into the rock mass. The mechanism was a close simulation of what is commonly observed in failures of such excavations. The failure resulted in significant early loading and on-going load accumulation. Shear overstressing along the bedding discontinuities was the key mechanism for releasing these effects until failure (collapse) was observed.

The example case clearly demonstrates the significance and importance of understanding the correct physical response mechanisms appropriate for the rock mass as a basis for providing a reliable, safe and effective design solution.

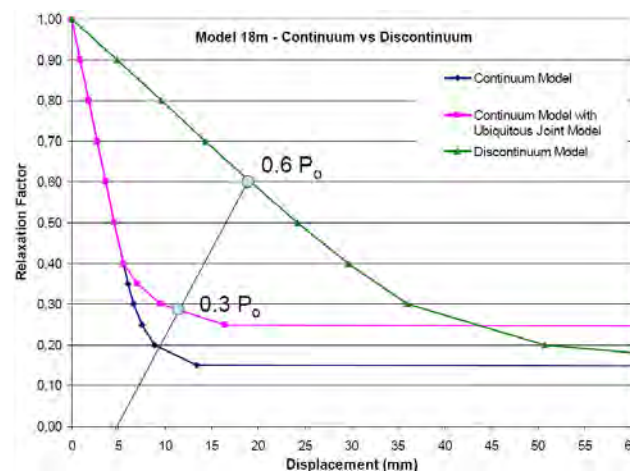


Fig. 9 Comparison between the Ground Reaction Curves Derived from the UDEC-BB, FLAC (M-C material behaviour) and FLAC-Ubiq Joint Analyses

#### 5. Engineering Evaluation – Critical Design Issues

Detailed engineering evaluation of numerical (UDEC) analyses is an extremely important stage in developing comprehensive detailed designs. Key engineering elements of the process are outlined below.

In general, underground engineering design lacks standard, Code-based procedures. In most cases, it is the Designer's perception of the problem and experience that lead to design concepts and detailing. Key issues emerging in most designs may be summarised as follows:

- Selection of appropriate and adequate temporary support, including passive vs. pre-tensioned reinforcement
- Shotcrete as a composite, integrated load bearing component
- Excavation - support sequence
- Acceptable profile deformation during excavation - support development
- Required stability reserve for large scale underground opening

- 3D effects and rock mass “relaxation”
- Permanent support (concrete linings, etc.)
- Design of intersections

A summary technical assessment and practical guidelines on key issues are given below based on monitored response from various projects.

### 5.1 Temporary Support

#### Rock Reinforcement Capacity

Correct representation of the interactive rock mass-support responses is fundamental for rock reinforcement capacity predictions with the rock mass simulation approach playing an instrumental role. Reinforcement unit modelling details (notably the type of bonding and allowable axial strain) are also significant. Typically, Safety Factors of 1.5 against tensile failure of bar for temporary units and 2.0 for permanent units are usually considered.

#### Shotcrete Capacity

The design of shotcrete support for underground excavations is an imprecise process. A shotcrete lining with a controlled (uniform thickness) parabolic profile acts mostly as a membrane (shell), predominantly being under axial compression and some flexure, due to the section shape and the deflection imposed by the rock mass response.

Deviation from the perfect parabolic lining curve on account of a non-uniform excavation profile produces parasitic bending moments ( $M_{par}$ ), resulting from the variable thickness and consequently deviations of the axial force vectors. A rigorous design approach should involve calculation of  $M_{par}$  which requires deriving eccentricities, which involves practical difficulties.

In practice, shotcrete capacity is demonstrated in terms of structural Code compliance on the basis of N-M interactive diagrams applicable to reinforced concrete design. This implies that the actual variable section thickness does not affect the distribution of axial forces (this is valid if the bulk moduli of the rock and shotcrete do not differ significantly). The issue needs to be considered further taking into account the yielding nature of the shotcrete material and to assess the technical requirements for optimum design in relation to support performance and cost.

### 5.2 Implied Degree of Rock Mass Stabilization

A methodology found useful in practice is to assess rock mass stabilisation in terms of the cavern profile deformation, overstress state and capacity reserve.

#### Cavern profile deformation

Maximum vertical deflection of the crown approximately 1/250 of the horizontal span for temporary support and 1/500 for permanent support is

a reasonable limit for cavern spans of 20 – 25 m, provided that an adequate stability reserve can be demonstrated for the anticipated range of geological conditions.

#### Discontinuity shear overstress

The degree of overstress in a discontinuous rock mass is primarily dependent upon the amount of induced shearing along the discontinuities caused by the excavation process and in response to the support measures selected. The degree of joint shear overstress can be quantified in terms of a Shear Strength Ratio (SSR), defined as  $SSR = \text{mobilized shear strength} / \text{peak shear strength}$ .

In essence, SSR represents a "Safety Factor" against joint peak shear strength exceedance. An upper limit  $SSR \leq 0.75$  should be achieved in the cavern surround forming an arch zone at small depth from the crown to accommodate the required minimum bonding length of the rock reinforcement units.

#### Material overstress

This can be examined in terms of the ‘maximum allowable to induced principal stress ratio’ values in the rock mass surround at the end of the excavation-support development. A quasi-elastic rock material state is the optimum requirement, which would correspond to maximum ratio of allowable-to-induced principal stress equal to or higher than 0.5.

### 5.3 Permanent Support (Concrete Lining)

There are no structural engineering codes specifying procedures for the design of interactive structures such as cavern concrete linings.

Rock loads are usually derived from empirical approaches based on classification schemes, ground reaction curves and numerical modelling. The criteria usually adopted for the lining structural design include the N-M interactions for a suite of factored loadings and deflections within the serviceability and limit state design framework.

Structural analysis software provides adequate tools for simulating the post-elastic behaviour of reinforced concrete, but lacks capabilities for realistic simulation of the interactions of the lining structure with the geological surround.

A rigorous method for modelling the variable stiffness of reinforced concrete linings according to EN1992-1 was proposed, taking into account stage II cracking and creep. The method was verified for its accuracy and was implemented in UDEC using the FISH programming language [PASCHALIDES & BANDIS, 2011].

The comparisons in Figure 10 demonstrate the resultant enhancement of bending moments at both the crown and sidewall sections as predicted from explicit discontinuum analyses compared to continuum analyses of a 20m span cavern in bedded rock under 150 m overburden. ( $M_{\text{DISCR\_CRK}}$  corresponds to the cracked concrete liner model in the explicit discontinuum analysis).

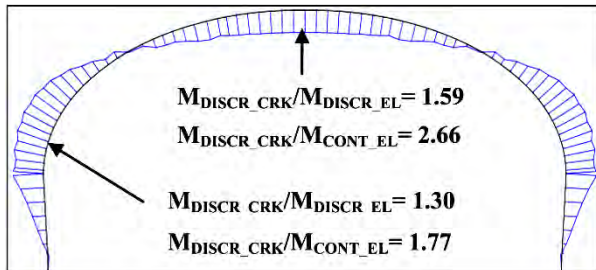


Fig. 10 Comparisons of Bending Moments in Concrete Liners Modelled as Cracked and Elastic Materials in Implicit Continuum and Discontinuum Analyses [PASCHALIDES & BANDIS, 2011]

#### 5.4 Three-dimensional effects

Related design issues include the displacements occurring ahead of the face and the implied relaxation of stresses and the excavation/support of intersections.

##### Rock Mass Relaxation

If the ground is a continuum and behaves elastically, the intrinsic elastic convergence provides a basis for some 30% initial relaxation, as demonstrated by various studies. In such cases, the corresponding reduction in ground loading may also be assumed to be ~30%.

UDEC analyses of a horizontally bedded rock mass (without and with joints normal to the cavern axis) and comparisons with equivalent continuum model and bedded / jointed models revealed considerable differences in rock mass relaxation responses, as follows: elastic continuum ~30%, bedded ~15%, bedded/jointed ~10%, as summarized in Figure 11.

##### Cavern Intersections

Intersections in underground excavations are expected to attract enhanced loadings compared to the individual counterparts due to the overlapping 2-D loads and increased effective span. In practice, the approaches to derive rock loadings are empirical and/or based on evaluation of 3D analyses. The Q-approach is based on increasing  $J_n$  by 3 in the Q equation. The resulting lower Q-value is in turn used to estimate loadings on the roof and sidewall using the  $P_{\text{roof}}$  and  $P_{\text{wall}}$  relationships.

Engineering of intersections requires careful development of the excavation-support details ensuring

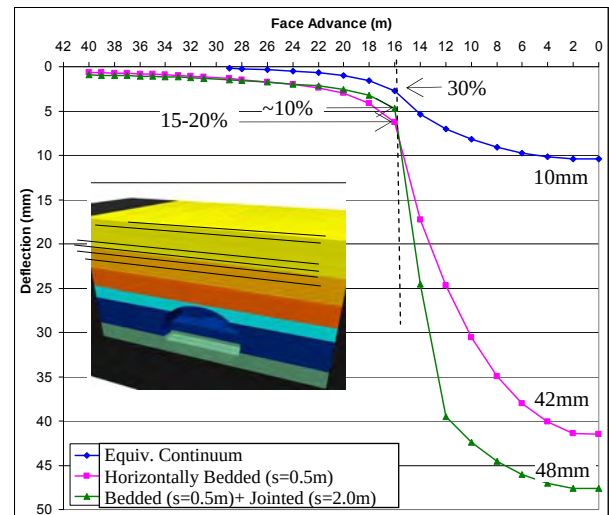


Fig.11 Comparison between 3DEC Predicted Longitudinal Displacement Profiles of a 20 m Cavern

adequate control of the deflections of the smaller opening, which will attract the highest loadings, particularly until development of its complete section.

The design principle for the development of intersections can be based on the utilization of rock reinforcement, reinforced shotcrete and the rock surround to form a composite support system comprising two shells, “linked” in order to achieve structurally continuous response. Structural arch members such as an embedded shotcrete rib may be utilized to achieve a robust link between the shells. An example design case of an intersection under very limited rock cover is illustrated in Figure 12.

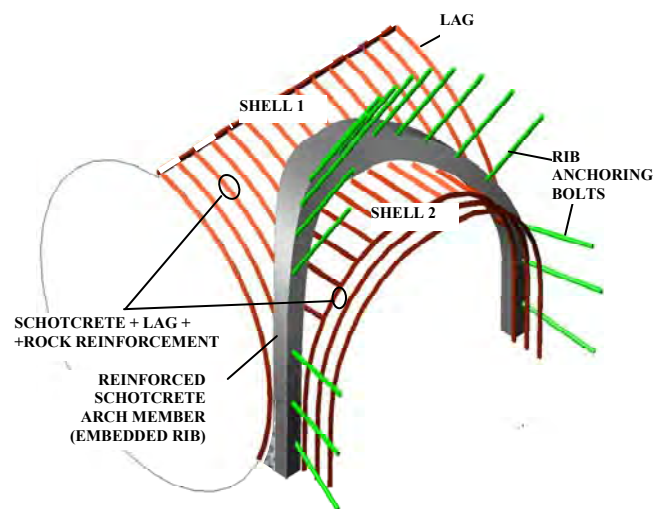


Fig. 12 Support Concept for Intersection Development (Dublin Port Tunnel Crossover) [GEO-DESIGN, 2004]

#### 6. Conclusions

The use of comprehensive engineering design for rock caverns has been made possible through the development of explicit modelling techniques for rock

masses using the Distinct Element Method (UDEC). The method is capable of simulating the complex, interactive behaviour of a structure (discontinuity) controlled medium, and interactive loading and groundwater effects. Through appropriate simulation of engineered reinforced and supported measures (rock anchors, reinforcement, linings etc) it is possible to quantify and dimension the various stabilising options.

A key issue that needs to be addressed through the provision of appropriate engineering expertise is the degree of safety (safety reserve against failure) that exists for a given design. Because of the complex interactive and path-dependent response of the staged cavern development and associated introduction of engineered stabilisation, it is necessary to develop and present a clear understanding of the design prior to construction using comprehensive, explicit methods. Such methods also form the basis for defining the associated monitoring (validation) rationale.

In summary, the benefit of the deterministic approach is the definition and understanding of the interactive behaviour of the controlling structure. This is essential if appropriate and realistic failure mechanisms leading to a clear understanding of the engineering response of the rock are to be established. By comparison the use of other empirically based pseudo-continuum representations results in obscure transformations of physical properties into analytical models and a resultant loss of understanding of the critical interactive response of the discontinuity system.

When considering the interaction of groundwater pressure on rock mass response (strength, stability etc.) the need for an explicit simulation of the discontinuity system is further enhanced.

Particular challenges facing Hong Kong cavern developments involve the excavation of caverns beneath existing high rise structures and the increasingly complex architecture of three dimensional space, to produce more acceptable underground layouts and environments.

### Acknowledgements

The authors would like to acknowledge the immense contribution of Peter Cundall for his innovative invention of the UDEC analysis codes. They would also like to pay special tribute to the Greek analytical team of Geo-Design who have solved many complex application issues and as a result have made UDEC a powerful core component of applied rock engineering. In particular they would like to pay tribute to the outstanding contributions of Dr Christos Schinas, who sadly passed away earlier this year.

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# GEOGUIDE 4 AND THE DEVELOPMENT IN CAVERN ENGINEERING OVER THE PAST 20 YEARS

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## Abstract

*This paper reviews the contents of Geoguide 4 - Guide to Cavern Engineering [GEO, 1992] identifying the areas in which the state of the art of tunnelling has progressed since the Geoguide was published back in 1992, providing updated information.*

*In 2010, Ove Arup & Partners Hong Kong Ltd (Arup) was commissioned by the Geotechnical Engineering Office (GEO) to undertake the study on the Enhanced Use of Underground Space in Hong Kong – Feasibility Study under Agreement No. CE 66/2009 (GE). with support from Norconsult AS, Urbis Ltd and other specialist sub-consultants. Part of the study involved a Review of Geoguide 4 which forms the basis of this paper [ARUP, 2011].*

*Developments in terms of site selection and ground investigation techniques are also included as well as the design and construction of facilities within caverns during the past 20 years. The various types of analysis and design of the caverns will be discussed in the context of numerical modelling and advances in rock engineering.*

*Advances in construction technology and materials technology that have improved the safety and constructability of underground space will be discussed and the paper will review the latest technologies and trends in the tunnelling equipment market including developments in tunnelling equipment and methods (drilling jumbos, shotcrete robots and pre-grouting equipment, computer control and automatic logging of performance).*

## Keywords

*Underground, Geoguide 4, Hong Kong, Cavern Engineering*

## 1. Introduction

The existing edition of Geoguide 4 was published in 1992. The Geoguide presents a recommended standard of good practice for the civil engineering aspects of rock cavern applications in Hong Kong.

The guide is aiming at engineers engaged in the design and supervision of caverns as well as non-specialist professionals, including planners and developers. The

Geoguide concentrates on the geotechnical aspects of cavern engineering and aims to produce cost-effective and safe integrated designs.

In 2010, Ove Arup & Partners Hong Kong Ltd. (Arup) was commissioned by the Geotechnical Engineering Office (GEO) to undertake the study on the Enhanced Use of Underground Space in Hong Kong – Feasibility Study under Agreement No. CE 66/2009 (GE) with support from Norconsult AS, Urbis Ltd and other specialist sub-consultants.

Part of the study involved a review of Geoguide 4 which forms the basis of this paper.

## 2. The Background to Geoguide 4

As stated in the Foreword to the Geoguide, the contents are derived from Scandinavian practice but with suitable modifications to make it applicable to conditions in Hong Kong.

The Scandinavian countries of Norway, Sweden and Finland represent typical hard rock provinces, as does also Hong Kong. The Scandinavian countries all have long traditions of underground excavations in the form of sub-surface mining. During the late part of the 19<sup>th</sup> century the technology and knowhow was adopted to the development of transport tunnels (rail, road and hydropower tunnels). The utilization of excavated hard rock caverns for military purposes, shelter, storage, energy production and recreational activities started during the 2<sup>nd</sup> World War and has since then developed rapidly as to size, functions and number of such facilities.

In hard rocks, the stability of a tunnel or cavern is mainly governed by the dimensions and shape of the cavity, the rock jointing, the rock stress situation and the influence, if any, of faults and weakness zones within the rock mass.

The basic rock support is usually a combination of rock bolting and fibre reinforced shotcrete. Under more adverse stability conditions, this may be supplemented by spiling bolts (forepoling) and reinforced shotcrete ribs.

In cases where water ingress into the cavity represents a problem, systematic pre-grouting of the rock prior to excavation may be performed.

The design of hard rock tunnels and caverns and their rock support is basically based on empirical experience. Site investigations in combination with geological logging during the excavation works forms the basis for the detailed design of the rock support.

Rock mass quality classification systems have been developed that based on the dimensions of the cavity and geological parameters can give guidance as to the appropriate rock support required. These systems are based on empirical experience, and consequently should be updated to reflect developments in support technology and the ever increasing experience platform.

### 3. Is Geoguide 4 Still Relevant?

Almost 20 years after its first publication, the guidelines for site investigation strategy and rock cavern design given in the Geoguide are still valid. It represents an approach based on a combination of sound engineering judgement and empirically based design guidelines that has proven its usefulness for hard rock cavern projects in Scandinavia as well as in other parts of the world, including Hong Kong.

Consequently, it has been decided not to rewrite the Geoguide. Some Technical Guidance Notes covering areas needing update have been prepared recently by GEO to supplement the Geoguide. With this supplementary update, the Geoguide presents general information and guidelines that are still relevant in planning, design and construction of cavern schemes in Hong Kong.

### 4. Development in Site Selection and Ground Investigation Techniques

The principles for site investigations have not changed. A summary of investigations commonly required for the various stages of the project is described as follows:

- Planning stage: Desk studies and site reconnaissance including study of aerial photos
- Preliminary design: Geophysical investigations, boreholes, permeability testing
- Detailed design: New boreholes, directional coring may be considered, packer tests
- Construction stage: geological and engineering geological mapping using a rock classification system, probing ahead may be required

The methods used have developed, but refraction seismic survey combined with diamond core drilling and sampling of rock cores are considered to give the best information of ground conditions during the design phase of a project.

The most prominent development and new techniques are briefly described below.

### 4.1 Geophysical Methods

Down-hole geophysical methods that have been developed may be relevant for cavern investigations in Hong Kong. The new techniques are described in GEO Technical Guidance Note No. 3 [GEO, 2004]. TGN 3 provides technical guidelines on the use of down-hole geophysical investigation techniques in the identification of weak layers in the ground, which could also affect slope stability.

The down-hole methods may be useful supplements to diamond core drilling where cores from existing boreholes and wells do not exist or when difficulties with retrieving cores arise:

- The Acoustic Televiewer (ATV) log provides a very-high resolution, sonic image of the drillhole wall with millimetre resolution, and can be presented as a flat, “unwrapped” diagram or as a 3-D “pseudo-core” which reveal fractures, bedding planes and orientation of those features. This log is useful where orientations of failure surfaces, fractures and bedding are critical for design, and can also be used to evaluate compressional-wave velocity, drillhole deviation and eccentricity. Attitudes of features may also be presented on stereonet plots to aid slope and foundation design.
- The Optical Televiewer (OTV) generates a continuous, 360° orientated image of the drillhole wall, using an optical imaging system allowing for both accurate drillhole deviation data to be recorded as well as accurate and precise image orientation in the same log.

### 4.2 Directional Drilling

The accuracy of a drillhole may be increased by the use of directional drilling. Equipment and technology permitting control of the direction of long drillholes has been developed during the past 20 years that makes it possible to drill in a forced straight line as well as controlling the drillhole in one or more curves, hitting the target with very high precision. The technology generally consist of a forced steering mechanism of the drill string combined with a logging function that maps the exact position at any point along the drillhole.

With this method it is also possible to perform Horizontal Directional Coring (HDC). HDC has been carried out in various projects (Po Shan Drainage Tunnels and Tsuen Wan Drainage Tunnel projects) in Hong Kong to drill along the alignment to verify the ground conditions and potential inflows during construction.

### 5. Cavern Design

The rock conditions for large span caverns in Hong Kong are generally good. The granites and volcanic

rocks can have quite high compressive strength. Thus, the rock can be used as a structural material that only needs to be strengthened by rock bolting and shotcrete for long term stability.

Geotechnical design of caverns is usually based on precedents, i.e. empirical methods. Recent cavern schemes are proven examples of this design approach:

- Stanley sewage treatment plant in Hong Kong Island from 1994, dimensions: 15 m span x 16 m high
- Island West Refuse Transfer station at Mount Davis from 1997, dimensions: 27 m span x 11 m high (main cavern)
- HK University Water Service Reservoir in Mid-levels from 2009, dimensions: 17 m span x 17 m high

In Norway, a cavern with span 61 m and height 25 m was constructed for the Winter Olympics in 1994 at Gjøvik (Fig. 1). The cavern houses an ice hockey rink, but is also used for other sports events, concerts and exhibitions. The rock quality at Gjøvik is good. Similar and even better conditions may be found within the strong granites and volcanic rocks of Hong Kong and thus, caverns with similar dimensions may be constructed here.

Another recent example is the underground ammunition storage completed in Singapore in 2007. The use of rock cavern freed up nearly 300 hectares of surface land for other more valuable uses (Fig. 2).



Fig. 1 Gjøvik Mountain Hall



Fig. 2 Singapore Ammunition Storage

## 5.1 Numerical Models

Numerical Models such as finite element as well as finite difference models have been increasingly used to allow prediction of the movement and impacts on the surrounding environment. Finite element models such as PLAXIS (Fig. 3) and FLAC have been used to model continuum related rock mass problems. These problems have been typically analysed when the rock is heavily jointed and appears to have soil like characteristics relative to the size of the excavation, additionally analysis is also performed when there is low ground cover. In a similar manner where the rock is very massive then continuum models tend to be more appropriate as the intact rock properties tend to dominate the response of the ground to the excavation. Where the rock mass is blocky, discontinuum models are used in the design of underground space such, as UDEC or 3DEC. It should be noted the 2D UDEC can be quite conservative when considering the overall nature of the rock.

If there is sufficient joint data for the underground scheme then an assessment of the likely rock block or wedges can be assessed for different geometries within the underground space. Computer programmes such as UNWEDGE are frequently used to design specific rock block support and to identify the preferred length of support based upon known or suspected joint set geometries.

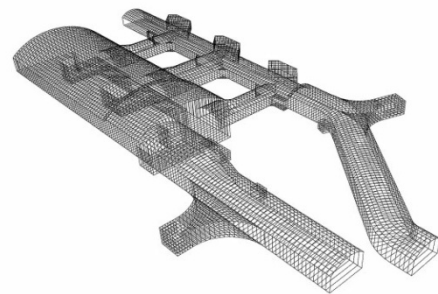


Fig. 3 Island West Transfer Station Computer Model

## 5.2 Evaluation of Rock Mass Quality and Rock Support

Preliminary design of rock support may be made on the basis of rock classification using the most commonly used rock classification systems such as the, Q or RMR systems. Other systems may be used such as the GSI system (Geological Strength Index) or the Rmi system (Rock Mass Index) [PALMSTROM, 1996] & [PALMSTROM & SINGH, 2001].

The Q-system was developed by the Norwegian Geotechnical Institute in the early 1970s [BARTON et al, 1974] with a major update released in 1993 [GRIMSTAD et al, 1993]. The most important element of the update was the introduction of Reinforced Ribs of Shotcrete (RRS) for rock support in poor rock. In earlier days, rock support in poor and very poor rock

was carried out with concrete linings. RRS may replace cast in situ concrete lining as rock support at the excavation face, see figure 4.

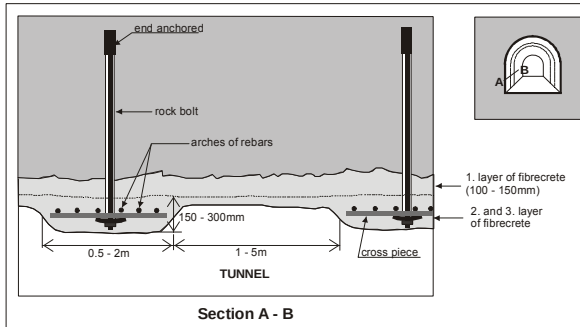


Fig. 4 Example of Bolted Ribs of Reinforced Shotcrete

## 6. Advances in Construction and Materials Technology

Since the publication of Geoguide 4 there have been significant advances in equipment and materials for underground construction. The following information details important items that the authors consider should be highlighted to the reader.

### 6.1 Underground Equipment

#### (1) Computerised Drill Jumbo

Drill rigs are designed to work fully automatically with up to 4 booms. Each boom has a drill machine which is capable of drilling up to 3 m per minute. The largest current available models can drill close to 200 m<sup>2</sup> faces from a single set up and have a maximum drilling height of 13 m. The computer controlled drilling performance, automatically performs alignment for the next round and surveying of the previous excavated profile.

#### (2) Mucking Out

Mucking out of underground spaces is performed by wheel loader and trucks. Diesel/electric loading equipment is applied to improve the environment at the tunnel face. A further improvement on air quality and speed of excavation was seen on the West Rail Tai Lam Tunnel (KCRC Contract DB350) when a mobile crusher was installed underground close to the face and connected to a conveying system. This reduced the number of trucks significantly and as a result improved the air quality.

#### (3) Ground Support

Ground support is performed by highly automated and computerized robots for sprayed concrete. The sprayed concrete robot can apply wet sprayed concrete up to a capacity of 25 m<sup>3</sup>/h. Rockbolting can be performed either by a jumbo drilling rig, or dedicated rockbolting machine. The rockbolting rig is especially good for large cavern complexes and allows drilling and installation of the rockbolts with high capacities.

#### (4) Hydraulic Splitter

Hydraulic rock splitting is an old technique normally by handheld equipment. But recently in Hong Kong rock splitting equipment was successfully used on a larger scale for the excavation of a cavern. A large hydraulic splitter mounted onto a jumbo was used allowed excavation to be extended to a 24 hour operation due to minimal impact from the underground night time working. [MACKAY et al, 2009]

### 6.2 Advances in Materials Technology

Advances in materials in conjunction with application equipment has allowed for safer and speedy installation of ground improvement and support including permanent linings. This section provides information on the latest materials technologies that the authors consider should be disseminated to the reader.

#### (1) Steel fibre-reinforced Sprayed Concrete

Steel fibre-reinforced concrete (FRC) has been used for over 30 years and at the end of the 1990s synthetic fibres were being introduced including micro and macro-synthetic fibres which are manufactured from polypropylene & polyolefin. Polypropylene fibres can be used to improve the performance of concrete in fires and the polyolefin for structural performance, similar to the use of steel fibres. The bending capacity of the shotcrete layer can be improved by the use of these fibres.

Other ways to improve the bending capacity is to use shotcrete arches with rebar reinforcement, or lattice girder. The advantage of using lattice girders is, beside a high bending capacity, the quick installation and the even arc shape. But it should be noted that lattice girders can be regarded as non-deformable, and therefore are not suitable in rock with high stress where deformations must be expected.

#### (2) Alkali-free Accelerator

Compared to traditional sprayed concrete accelerators, new alkali free accelerators offer a safer method of acceleration with final strength increase; by forming a homogeneous and compact concrete matrix without internal tension and micro-cracking. Traditional accelerators produce a loss of 15-50% in potential compressive strength. The various types of alkali-free accelerators on the market can provide very high early strength, low rebound and may be sprayed to 300mm thickness in one operation. The accelerators also produce low corrosiveness and reduced permeability of the shotcrete.

#### (3) Grouting

Grouting and sealing works should preferably be performed ahead of the excavation face ('pre-grouting'). [TATTERSALL et al, 2009]. Coarse-grained and expanding cement grouts are used to stem wide open joints. Pure cement grouts are used to treat rock with

Lugeon values down to 0.5, with fine cements (microcements) employed for the lower range giving better penetration into the rock mass.

Superplasticizers and microsilica are used in cement grouts to improve their properties such as penetrability and bleed reduction. Polyurethane-based foam may be considered as an alternative material to stem large inflows of water, particularly where low grout pressures have to be used. This grout type expands on contact with water. Grouting with silicates or epoxy grouts is appropriate where good watertightness is needed, and is commonly applied after first stage sealing with cement grouts.

Rock mass grouting takes place using computerized units that can deliver grout to several grout holes simultaneously. Grouting pressures are normally dependent on the hydrostatic head, and pumps can reach up to 80-100 Bar pressure. Modern grouting rigs may be equipped with computer logging systems of the injection process.

#### (4) Explosives

Explosives can come in various forms, high-energy explosives in the form of cast boosters for initiating bulk emulsion and cartridge emulsion for production blasting. Low cost bulk explosives based on Ammonium Nitrate prill or site-sensitised emulsion is used for main production blasting. Detonating Cords used in the contour holes to aid smooth wall blasting due to the high velocity of Detonation (VOD) and decoupled charge.

Detonators are placed in 3 main categories; electric detonators are initiated by an electric current and are only used for initiating blasts in Hong Kong. Non-electrical detonators are initiated by shock tube. These detonators are the most widely used in Hong Kong at present and are safer than electric detonators as they are not affected by stray current. Electronic detonators are the latest detonators to be used in production in Hong Kong. They have a highly accurate timing system that enables a wide division of the blast round, which allows a better prediction of blast induced vibration and are not affected by stray current.

The following list of documents is considered essential references when explosives will be considered for excavation:

- CAP 295 - Dangerous Goods Ordinance [HKSARG, 1997].
- Project Administration Handbook Chapters 4 & 7 [HKSARG, 2010].
- APP-72 Control of Blasting [BD, 2009].
- Mines Division Guidance Note on Vibration Monitoring [GEO, 2009].
- ETWB TC(W) 15/2005. Geotechnical Control for Tunnel Works [ETWB, 2005].

- General Specification for Civil Engineering Works (2006 edition) [HKSARG, 2006].
- GEO Technical Guidance Note No. 28 (New Control Framework for Soil Slopes Subjected to Blasting Vibration) [GEO, 2010].
- A Study of the effects of Blasting Vibration on Green Concrete, GEO Report No. 102 [KWAN & LEE, 2000].
- Prediction of Blast Vibration [LI & NG, 1992].
- Tunnel Construction Guidelines 02 – Underground Rock Blasting [AGS-HK, 2010].
- List of Explosives Approved for Use in Hong Kong [CEDD, 2011].

#### (5) Rock Bolts

Rock Support is provided in various forms and since the publishing of Geoguide 4 friction anchor bolts e.g. swellex have proved to be the most popular temporary support for rock excavations. There are also new developments in rock bolts, for example the CT-Bolt which is typically a mechanical bolt, installed as a temporary rock support and later grouted to become a permanent rockbolt. It also has a polyethylene sleeve which seals the bolt against corrosion.

#### (6) Cable Bolts

Cablebolt is an alternative method for support in large span caverns. The cable consisting of a number of steel wires wound into a strand and grouted into a drillhole which allows the transfer of loads from the cable bolt into the rock mass. Cablebolts are installed to provide reinforcement and support to walls, roof and floor of underground openings. It is a flexible form of bolting as it allows the installation of long bolts in confined spaces.

#### (7) Reinforced Ribs of Shotcrete

Reinforced Ribs of Shotcrete (RRS) are installed in poor rock which is highly overloaded. In such cases, shotcrete arches with rebar reinforcement, or lattice girder can be used (Fig. 5).



Fig. 5 Ribs of Shotcrete Reinforced with Lattice Girders in a Large Underground Hydropower Cavern

Ribs of reinforcement bars are comprised of steel reinforcement which is placed around the tunnel profile and fixed in place with radial bolts which are fully grouted. The reinforcement is usually covered by 5 cm

thick shotcrete. Convergence measurements may be established to record future deformations.

#### (8) Lattice Arches

Arches of Lattice Girders consists of rolled rebar girders. In its most simple form, three rebars are assembled in an oblong lattice with a triangular cross section. When the different elements of the girders have been assembled, they are entirely embedded in shotcrete. The advantage of using lattice girders is, beside a high bending capacity, the quick installation and the even arc shape.

#### (9) Waterproofing

Caverns should preferably be located in good rock where the natural strength of the rockmass creates the permanent opening. Waterproofing in these circumstances will consist of pregrouting to reduce the water leakage to an acceptable level for the operation of the cavern and for the environment (prevention of unacceptable ground water draw down).

The remaining leakage or water drip may be handled in several ways. Where the use the cavern requires dry conditions, the water may be drained down to a drainage system in the invert using an “umbrella” type of lining. The “umbrella” may be provided in several ways:

- Drip screens of corrugated plates (steel or other material).
- Drip screens or total enclosure of ceiling and walls with plastic membranes (e.g. Giertsen Tunnel sealing)

For caverns not exposed to frost, new advances with cementitious polymer-based, spray-applied membrane that bonds to the inner and outer lining preventing water migration behind the waterproofing membrane may be used [VERANI, 2009].

Cuspated sheets of HDPE material may be placed as drainage in the walls and crown of the cavern and covered by shotcrete. The cuspates can be formed in various sizes to cater for different flow rates.

#### (10) Permanent Lining

Permanent Lining has traditionally been concrete cast in-situ behind wooden or steel formwork. This is still the case on many projects with additives to concrete allowing better pumping characteristics and faster setting times giving better production rates. Alternatively permanent shotcrete lining have become more common where irregular or non-standard profiles are required.

### 7. Conclusion Remarks

From the information provided the authors have given a brief insight into some of the more important

developments across the process of cavern engineering since the publication of the Geoguide 4 in 1992. It should be noted that some of these innovative ideas have already been adopted successfully across the various disciplines of the cavern engineering industry.

Even with the new developments it should, however, be noted that the main site investigation strategies, design principles and the construction methods described in the Geoguide 4 is still highly relevant.

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# SUBTERRANEAN STATIONS – SAFETY & COMFORT

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## Abstract

Today the word “Underground” tends to have negative connotations, but this was not always the case. Early mankind relied on caves for shelter and protection, and the ancient desert city of Petra in Jordan was intricately carved into solid rock. The modern Canadian cities of Montreal and Toronto boast extensive “underground cities” which provide a comfortable connection to surrounding offices, commercial space and transportation in spite of the harsh winters. Japan also has extensive underground shopping streets associated with metro railway stations. Underground railways, such as the London Underground which dates from the 19th Century, provide unobstructed high volume movement of passengers and address issues of urban congestion and limited land availability. Hong Kong already features a sizeable metro rail network and five new lines are currently under design or construction which will ultimately provide rail connectivity to all 18 districts within Hong Kong. Much of this new railway construction needs to be underground due to urban congestion and topography. In light of current fire safety standards and increasing public expectations for comfort, some special design features have been adopted for the new underground stations in Hong Kong and these are introduced in this paper. As cities become more congested, and in the face of climate change and pollution, perhaps it is time to review our perceptions of underground spaces and recognise their value and potential.

## Keywords

Railway, Station, Underground, Safety, Comfort

## 1. Introduction to Underground Space

When the term “underground” is mentioned today it can evoke negative connotations as evidenced by some of the on-line Oxford Dictionary definitions:

- “beneath the surface of the ground”;
- “in or into secrecy or hiding, especially as a result of carrying out subversive political activities”;
- “relating to or denoting the secret activities of people working to subvert an established order”;
- “a group or movement seeking to explore alternative forms of lifestyle or artistic expression”; and of course not to forget,
- “an underground railway, especially the one in London”.

In prehistoric times, survival was entirely dependent on the basic necessities for life: air, water, food and shelter, in order of priority. When considering shelter, a naturally occurring underground cave (Fig. 1) could provide protection from the elements (rain, wind, heat, cold) and safety from predators, and as such would have been a treasured amenity.



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Fig.1 Prehistoric Cave Dwelling

In later historic times, underground construction was still sometimes adopted. For example, the famous ancient Jordanian city of Petra (Fig. 2), a UNESCO World Heritage Site, dates from around the 6<sup>th</sup> Century B.C. and was intricately carved into sandstone gorges. This construction not only provided a concealed and secure location but also addressed extreme desert heat during the day and cold at night, and facilitated water collection. Other more modest examples can be found in Cappadocia, Turkey.



© Bernard Gagnon / Wikipedia

Fig.2 Treasury Building at Petra, Jordan

In modern times, examples of large scale “Underground Cities” are Montreal’s RÉSO (Fig. 3)

and Toronto's PATH (Fig. 4), both developed in response to Canada's harsh winter climate. RÉSO was started in 1962 and is now the largest underground city network in the world, stretching 20 km and covering 4 million square metres. RÉSO's network connects to 10 metro stations, 1,200 offices, 2,000 stores, 40 movie theatres, 7 major hotels, 40 banks, 3 exhibition halls and much more. Orientation and wayfinding is enhanced by different aesthetic treatment in different areas of the complex [A VIEW ON CITIES].



[simplyairlines.com](http://simplyairlines.com)

Fig. 3 RÉSO, Montreal



[justinlagace.com](http://justinlagace.com)

Fig. 4 PATH, Toronto

Toronto's PATH is over 100 years old, but grew significantly in the 1970's and 1980's and is now the world's largest underground shopping complex with 27 km of shopping concourses, over 350,000 m<sup>2</sup> of retail space with 1,200 shops and services employing in excess of 5,000 workers. PATH connects to 20 parking garages, 5 subway stations, 6 major hotels and much more, and has over 125 ground level access points [A VIEW ON CITIES].

Japan has also developed extensive underground shopping streets often associated with subway stations which generate high patronage flows.

Given the increasing demand for accommodation, the potential for use of underground space, and perceptions about that space, need to be reconsidered. For example, in the case of an enclosed auditorium or cinema, differences in location (above-ground or underground) should be barely perceptible to occupants. Modern technology in both construction and building services ensure that a safe and comfortable environment can be created underground for a wide variety of uses.

This paper focuses on the safety and comfort of underground railway stations, highlights some special features of the new generation of underground stations under design and construction in Hong Kong during the "teen years" of the 21<sup>st</sup> Century, and endeavours to shed some new light on the potential for wider use of underground space in Hong Kong.

## 2. Subterranean Stations

### 2.1 International Examples

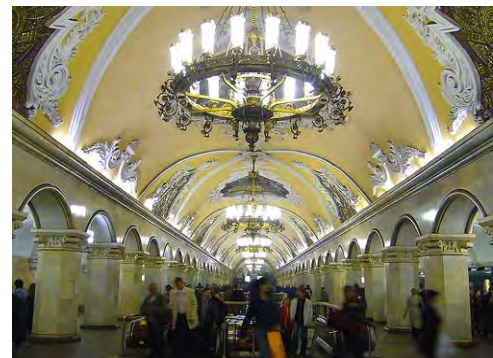
Underground railways and stations are not a new phenomenon. The London Underground (Fig. 5) dates from the latter half of the 19<sup>th</sup> Century, while the Paris Métropolitain and the Moscow Metro date from 1900 and 1935 respectively.



[964rap.com](http://964rap.com)

Fig. 5 London Underground

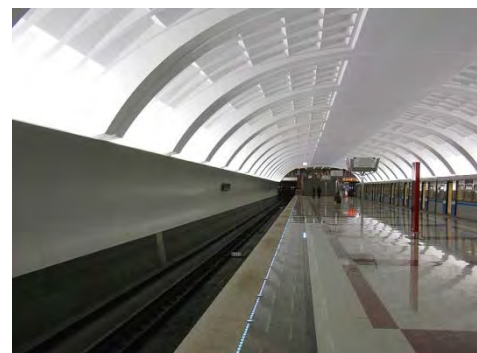
The palatial decoration in some of the early Moscow Metro stations is a testament to the importance placed on these facilities at that time and the efforts made to create a high quality and appropriate internal environment, even under a Communist regime (Fig. 6).



[yeowatsup / flickr.com](http://yeowatsup/flickr.com)

Fig. 6 Komsomolskaya Metro Station (1935), Moscow

Modern stations in Moscow are more austere but are still distinctive (Fig. 7).



[Eternal Triangle / wikipedia](http://EternalTriangle/wikipedia)

Fig. 7 Mitino Metro Station (2008), Moscow

Some unique underground “art” stations can be seen in Sweden (Fig. 8).



Fig. 8 T-Centralen Station, Stockholm

Of course such individual station interiors require a particular maintenance commitment, while the alternative adoption of line-wide or system-wide consistency can provide greater efficiency for procurement and long-term maintenance.

## 2.2 Hong Kong MTR System - Existing

The MTR metro railway network in Hong Kong was first opened in 1979 and has progressively expanded over the years, most recently with the addition of the former KCRC network in 2007, and currently boasts more than 80 railway stations, a significant number of which are built underground, plus 69 light rail stops.

The majority of the underground MTR stations are built using the cut-and-cover technique, however in some special instances cavern construction is adopted such as in Tai Koo Station (24 m span, 16 m height and 250 m length, Fig. 9), part of the Island Line opened in 1985 [ROBERTS et al, 1996] [SWALES et al, 2010].

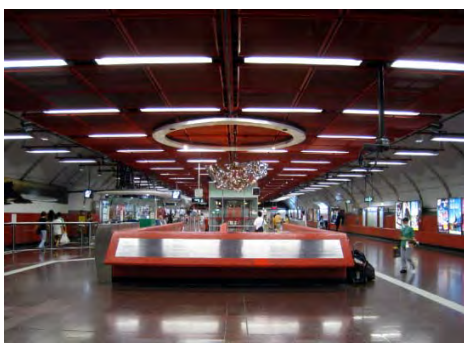
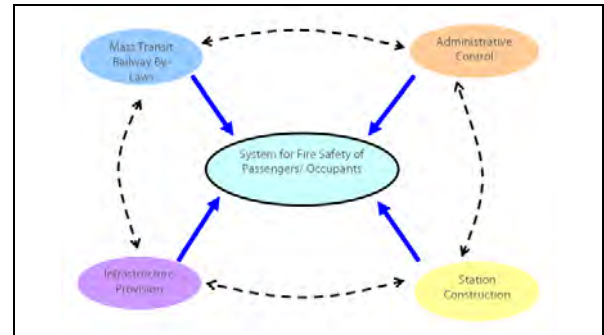


Fig. 9 Concourse of Tai Koo Station, Hong Kong

Underground railway/ stations are significantly more expensive than those built above-ground and are usually only adopted where ground level constraints or the topography and alignment gradients/ levels dictate.

Fundamental considerations for underground station safety and operational integrity include: waterproofing, flood protection, minimising combustibles, fire

prevention, fire suppression, ventilation, smoke control, dual power supplies, emergency lighting, surveillance, public address systems, means of escape (primarily by escalator), means of access, clear and unambiguous signage, all of which are supported by stringent by-laws and 24 hour management (Fig. 10).



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Fig. 10 MTR Fire Safety Management System

With regard to comfort and convenience, fundamental considerations include: accessibility for mobility impaired; circulation capacity and efficiency; type of circulation elements (stair, lift, escalator, moving walkway); travel time from street to platform; air quality and temperature; lighting level and lighting quality; acoustics; materials and finishes; colour, design perceptions (e.g. of distance); and strategic provision of artwork.

For an underground station the ground level presence is minimal, mainly entrances (which most people want) and ventilation buildings (which most people loathe), while the vast majority of the complex is hidden below ground, perhaps similar to an iceberg (Fig. 11).



© Ralph A. Clevenger

Fig. 11 The “Tip of the Iceberg”

Therefore there is very limited scope for design of external elements, and the treatment of internal public spaces becomes a prime consideration, although severely constrained by fire safety and operational considerations in the choice of materials and the complexity of design. A mass transit station must facilitate the smooth flow of a high volume of people, which is quite different from a shopping mall, for example, which may be designed to “trap” customers for more business while of course still providing adequate means of escape in the event of an emergency.

The extensive use of one bold colour to identify an individual station, especially useful when viewed from a crowded train, is a feature of many MTR stations in Hong Kong, originating from the earliest lines. Tai Koo Station (Fig. 9), for example, is primarily red in colour. A more subdued and “architectural” palette of colours was adopted on the Airport Express Line stations opened in 1998, but these were intended to cater to a more business class clientele and are the exception rather than the rule. Recent metro station designs still maintain bold colouring.

### 2.3 Hong Kong MTR System - Ongoing Expansion

Five new lines are now under design and/or construction in Hong Kong, including West Island Line (WIL) - an extension of Island Line, South Island Line- East (SIL-E), Kwun Tong Line Extension (KTE), and Shatin to Central Link (SCL) - which will ultimately combine with other existing lines to form “East-West” and “North-South” corridors (Fig. 12). Upon completion these new lines will not only provide mass transit connectivity to all 18 districts in Hong Kong, but will also provide enhanced connectivity and interchange with other lines, resulting in more convenient MTR journeys for an even greater number of Hong Kong residents. Additionally, the Express Rail Link (XRL) with a large terminus in West Kowloon will provide high speed rail connectivity to the expanding network within Mainland China.

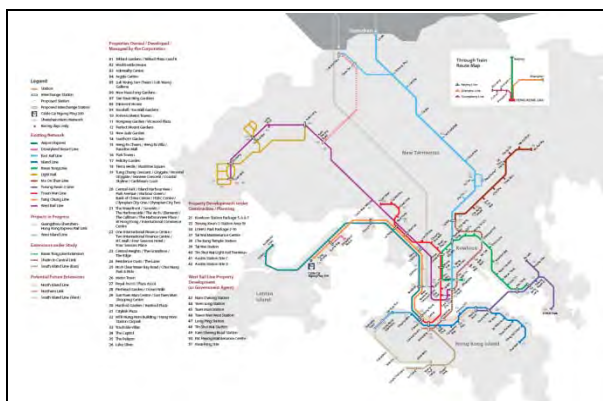


Fig. 12 The Growing MTR Network

Construction of a railway in a dense urban environment is extremely challenging, resulting in not only a significant amount of underground construction, but also difficulties in siting even the relatively small entrance buildings needed without impacting unduly on private lots.

For example, the West Island Line (WIL), which some 30 years after it was first considered will provide an extension to the Island Line, not only must be built underground but also the alignment must be threaded between the piles of an increasing number of large developments.



Fig.13 Part of the WIL Alignment

### 2.4 Hong Kong MTR – New Generation Underground Station Case Studies

The following brief case studies, with particular emphasis on West Island Line which is now under construction, highlight some special features of the new generation stations which set a new standard in Hong Kong underground railway design.

#### (1) West Island Line (WIL) – Sai Ying Pun Station (SYP) and HKU Station (HKU)

SYP and HKU are both cavern stations which must also address the steep topography of Hong Kong Island and have a wide enough reach of entrances to provide adequate catchment of passengers. For these stations the cost of entrances and connecting adits is very high, roughly equivalent to the station cavern cost.

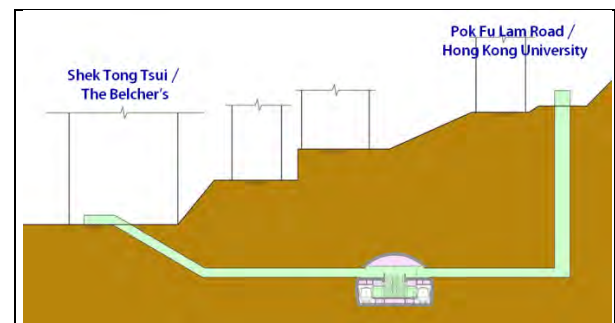


Fig.14 Simplified Section through HKU Station

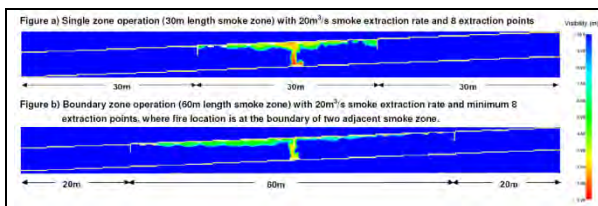


Fig.15 Sai Ying Pun Station and Entrances

Special features of these stations include long adits, lift-only entrances together with lift-assisted

evacuation, station caverns with high space efficiency, and a generous provision of community art in recognition of WIL as a “Community Railway”.

Safety within long adits, in one case up to 350 m long, is addressed by sprinkler suppression, multiple smoke zones of maximum 30 m in length (due to gradients) using a single overhead smoke extraction duct and local damper control, plus variable exit signage which changes depending on the location of an incident. Figure 16 shows typical computational fluid dynamic (CFD) model for fire in a long adit. Figure 17 presents an example of the use of variable exit signage.



© MTR Corporation

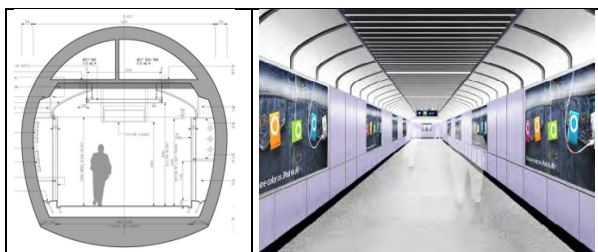
Fig. 16 Typical CFD Output for Long Adit



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Fig. 17 Variable Exit Signage at Long Adit

The travel distance within the WIL long adits is longer than usual and a moving walkway is provided where tidal patronage is high enough to warrant such an installation. Additional Help Points are provided for communication with station staff. The perception of distance is also addressed by a transverse ceiling pattern, rather than longitudinal, while strategically located artwork, together with advertising panels whose appearance regularly changes, add interest and variety (Fig. 18).



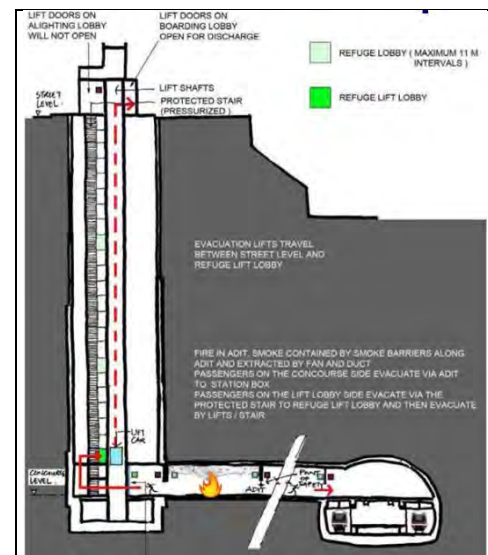
© MTR Corporation

Fig. 18 Long Adit Design Concept

Where a significant vertical rise is required between the station and an entrance, say in excess of 35m, the use of escalators starts to become impractical. Provided that entrance patronage is not too high, say less than 2000 people per hour per direction (pphpd), the adoption of high capacity lifts becomes attractive, not only to

shorten travel time and improve the passenger experience but also to reduce running costs. The WIL project has pioneered the use of lift-only MTR entrances and true walk-through lifts with separate boarding and alighting lobbies.

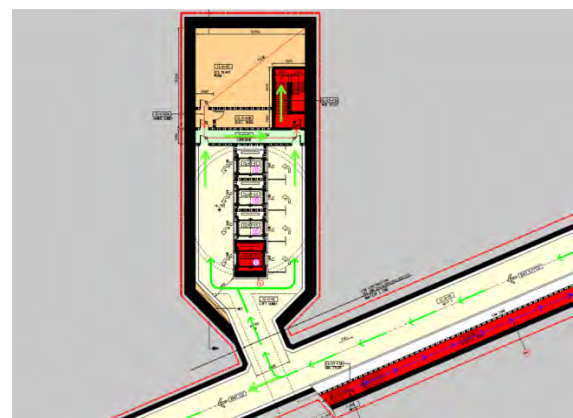
Such lift-only entrances are discounted in evacuation calculations, however in order to address an adit fire scenario, lift assisted evacuation has been developed in consultation with Electrical Mechanical Services Department (EMSD), Fire Services Department (FSD) and Buildings Department (BD). In the event of an incident in an adit leading to a lift-only entrance, evacuees are directed to a pressurised refuge lift lobby one level above the normal lift lobby and from there are safely conveyed to the ground level by the same high capacity lifts running in emergency mode (Fig. 19) [YEUNG et al, 2009].



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Fig. 19 Lift-assisted Evacuation, “One Level Up”

HKU Entrance C1 is one of the most complex lift-only entrances, combining lift-assisted evacuation and dedicated access for emergency personnel over a rise of close to 70 m (Fig. 20 & 21).



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Fig. 20 HKU Entrance C1 Lower Lobby

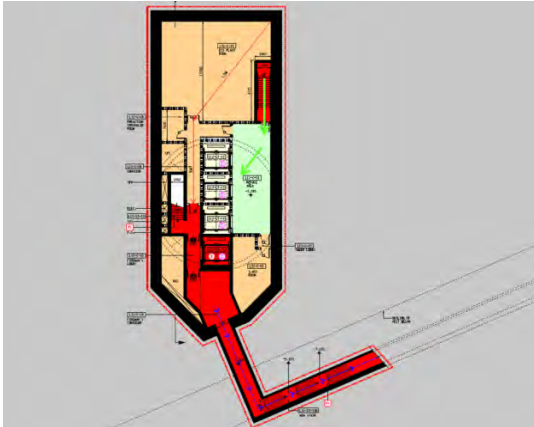


Fig. 21 HKU Entrance C1 Upper Lobby

The use of space within the caverns of these WIL stations is necessarily highly efficient as can be seen in the layouts and section for SYP Station (Fig. 22 to 25).

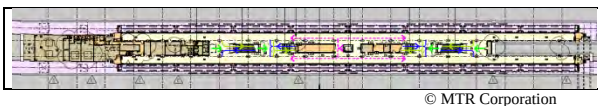


Fig. 22 SYP Platform Level



Fig. 23 SYP Concourse Level

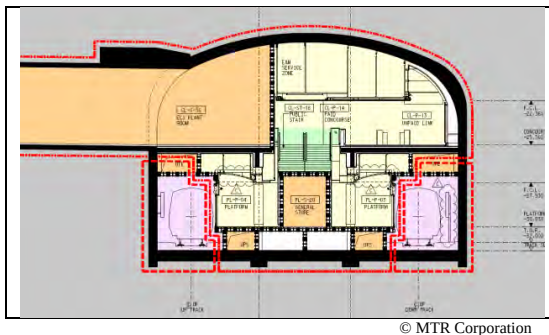


Fig. 24 SYP Station Section



Fig. 25 SYP Station Concourse Environment

With a block of plant room space intruding into the public concourse and with much of the ceiling void devoted to E&M services, the challenge has been to provide a pleasant station environment and to avoid feelings of claustrophobia.

The provision of Artwork will also help enhance the station environment and the local community is involved in its development (Fig. 26).



Fig. 26 SYP Concourse + Artwork (indicative only)

Many of the same design principles developed on these two WIL stations have been directly applied to Lei Tung Station on the SIL-E project, which although a smaller station (for 3-car rather than 8-car trains) still faces similar topographical challenges and space constraints.

## (2) South Island Line - East (SIL-E) and Shatin to Central Link (SCL) – Admiralty Station (ADM) Expansion

Upon the completion of both the SIL(E) project and the fourth harbour crossing under the SCL project, ADM will become Hong Kong's most complex interchange station serving 4 lines and with 8 platforms (Fig. 27 & 28). Due to site constraints the new portion of ADM will be built deep underground and has also required careful planning of interchange flows which will far exceed street level patronage.

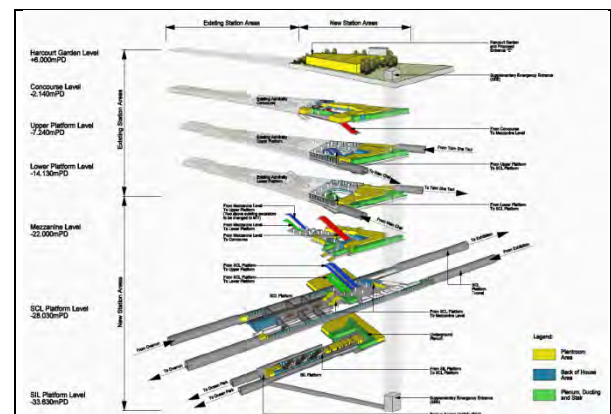


Fig. 27 ADM Expansion – Isometric View

Due to the scale and complexity of the station, and the significant number of fire scenarios identified during fire engineering studies, the variable exit signage and other safety-critical elements in an emergency are expected to be handled by a sophisticated decision support system in order for station staff to better manage an incident.



Fig. 28 ADM Section - New (L) and Existing (R)

The tall atrium containing multiple circulation elements will allow natural light to flood deep into the new station area (Fig. 29), however this concentration of circulation has necessitated various fire safety enhancements. In the event of a fire in the sprinkler-protected atrium, smoke will be detected by beam detectors and full height smoke curtains will be activated to separate the atrium from adjacent areas, except for the escalator openings which form make-up air paths. Static smoke control will be enhanced by the provision of multiple booster fans at the top of the atrium to boost smoke removal.



Fig. 29 ADM Atrium and Interchange Circulation

The SCL concourse area between the two SCL platforms provides connectivity to the SIL(E) platforms below, and to the atrium which leads upwards to other platforms and the ground level. In this case it has been spatially possible to better express the cavern structure in the station architecture and enhance the feeling of space.



Fig. 30 ADM SCL Concourse Cavern

### (3) Express Rail Link (XRL) – West Kowloon Terminus (WKT)

Upon completion the West Kowloon Terminus (WKT) will be by far the largest station in Hong Kong and will provide a strategic connection to the high speed rail network in Mainland China. Provision has been made for 15 platforms located at Basement 4 level - 9 platforms approximately 450 m long to accommodate 16-car long-haul trains, plus 6 platforms approximately 220 m long to accommodate 8-car shuttle trains. Departure level and Arrival level are located at Basement 3 and Basement 2 respectively.

Much effort has therefore been put into the design to enhance the arrival and departure experience and to enhance the overall quality of internal space. Central to this is a very large atrium which provides visual connectivity to other levels and also to the outside environment (Fig. 31).

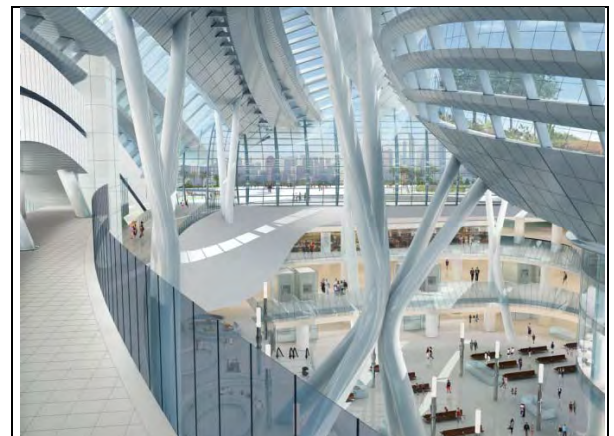


Fig. 31 WKT Atrium

Fire safety provisions for all of the underground areas have been extensively discussed with FSD and enhancements provided include multiple points for means of access, special consideration of the static smoke control for the atrium, and full coverage by fire suppression including the adoption of long throw sprinklers where required.

WKT closely interfaces with the planned West Kowloon Cultural District and has also been subject to challenging planning requirements such as 30 percent of green space within the site. Therefore it has been necessary to “hide the junk and show the art”, resulting in most of the plant rooms, including large arrays of power transformers, being located deep underground.

The resulting landscaped park surrounding the WKT entrance building and extending over the roof will be a valued amenity for the Hong Kong public and has only been made possible by the extensive use of underground space (Fig. 32).



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Fig. 32 WKT Entrance Building and Landscaping

### 3. Conclusion

Compared with overseas examples the use of underground space for public facilities in Hong Kong could still be considered to be in its youth. Nevertheless, the potential and capability exist for further underground development, and this may only be constrained by cost and the willingness, or otherwise, of statutory authorities to reconsider the nature and hazards of underground space given the technology available today.

The use of underground space for railway stations in Hong Kong has evolved over the decades as above-ground space becomes more congested and as expectations by the public and statutory authorities continue to rise.

MTR Corporation, of necessity, is doing its part to “push the boundaries” by not only proposing enhanced designs for underground space, both in terms of safety and comfort, but also by introducing new technology to Hong Kong such as lift-assisted evacuation which has potential applications both below ground and above.

It is to be hoped that Hong Kong will see increasing and more diverse use of underground space, strongly supported by all in the construction industry.

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# FIRE SAFETY OF CAVERN SPACES

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## Abstract

*Underground spaces such as basement, underground MTR stations, road / rail tunnels, utility tunnels, etc are popular in Hong Kong. A few cavern developments have been purpose-built in the past 30 years. In recent years, the Hong Kong Government encourages cavern development to make a better use of underground spaces. A consultancy study on the Enhanced Use of Underground Spaces in Hong Kong has been commissioned to investigate the issues of the underground planning, strategies and practice related to traffic, accessibility, development compatibility and development intensity. However, fire and life safety have not been fully addressed. This paper intends to address the fire safety requirements for cavern development and draws attention to the fire safety issues when further studies are conducted in this topic.*

## Keywords

Cavern, Fire Safety, Fire Engineering Approach

## 1. Introduction

Underground spaces developed as part of a building or for underground transportation systems or for utilities are very popular worldwide, such as basement, underground MTR stations, road / rail tunnels, utility tunnels, etc. In general, the basements and tunnels are located right underneath the ground surface - subsurface.

The use of subsurface space is quite common in Hong Kong in relation to the MTR and associated underground retail complexes. But the purpose-built cavern developments in Hong Kong are limited. The history of cavern development in Hong Kong can be categorised into two generations. The first generation were caverns built out of necessity for infrastructural and transportation purposes. These included the valve chamber of the Western District Aqueduct and the Taikoo MTR Stations both built in the 1980s. The second generation developments generally refer to the caverns built in response to government promoted planning initiatives geared at the exploitation of underground space at specific locations to reduce aboveground land take, e.g. the Stanley Sewage Treatment Plant and Island West Refuse Transfer station.

In order to encourage cavern development, Hong Kong statutory planning practice is moving toward allowing caverns for industrial usage underneath a different

planning zone, and Hong Kong Government has appointed a consultant to undertake the study on the Enhanced Use of Underground Spaces in Hong Kong – Feasibility Study under Agreement No CE 66/2009 (GE). With these efforts, the issues of the underground planning, strategies and practice related to traffic, accessibility, development compatibility and development intensity will likely be resolved.

However, fire and life safety will be a major technical issue, which have not been fully addressed. In related to the usage, the solutions for cavern development can be very costly. If the regulation requirements are not properly interpreted, construction of caverns will become impossible. This paper intends to address the fire safety requirements for cavern development and draws attention to the fire safety issues when further studies are conducted in this topic.

## 2. Codes Requirements

### 2.1 Codes for Buildings

In Hong Kong, the Buildings Ordinance regulates the planning, design and construction of buildings, including caverns. In 1983, the Buildings Ordinance was amended to make building permit mandatory for the development of oil storage caverns. In 1990, the Buildings Ordinance was again amended to extend its control for all cavern developments. It is therefore necessary to obtain building approval prior to the development of any cavern project.

In addition to the Buildings Ordinance, the Building (Planning) Regulations (B(P)R) also directly/indirectly address the development of cavern. For instance, under B(P)R 30, the provision of lighting and ventilation is mandatory for habitable rooms and offices. Also, in accordance with B(P)R, all buildings must be provided with adequate means of escape so that occupants can be discharged safely to an open space, and reasonable access must be given to fire fighters to attend an incidence. This makes housing residential, office developments and places of public entertainment within caverns technically difficult, further diminishing the opportunities for cavern development.

Fire safety for buildings in Hong Kong is designed to the following codes of practice in order to meet Building Ordinance, Building (Planning) Regulations and Building (Construction) Regulations:

1. Code of Practice for the Provision of Means of Escape in Case of Fire (MoE) [BD, 1996a]

2. Code of Practice for Fire Resisting Construction (FRC) [BD, 1996b]
3. Code of Practice for the Provision of Means of Access for Firefighting and Rescue Purposes (MoA) [BD, 2004]
4. Code of Practice for Minimum Fire Service Installations and Equipment and Inspection, Testing and Maintenance of Installations and Equipment (FSI) [FSD, 1998]

These codes are prescriptive in nature and written to address the fire safety design of typical buildings. Requirements for basements are also given in the codes to address structures directly below a building (Note: these codes are not directly applicable to underground rail/metro stations, which are under Railway Ordinance).

Since the codes of practice mentioned above are written for above ground buildings, some clauses within the codes cannot be easily met by large underground structure/cavern. Some require a large investment in construction cost and may not be cost effective. A number of examples are given below:

- Travel distance is limited to 18m for single direction escape. A cavern deeper than 18m must be provided with alternative exit route (MoE Code).
- When there is more than one escape direction, the travel distance is limited to 30m. In other words, an escape staircase has to be provided every 60m at most (MoE Code).
- All elements of construction below ground should have 4 hour FRP (FRC Code).
- Every basement should be provided with smoke outlets at certain intervals (in addition to smoke control system), e.g. every 30m and/or 3500m<sup>3</sup>. This may lead to a large number of shafts for large underground site.
- All buildings should have at least 25% of facade served by emergency vehicle access (EVA). For places of public entertainment, this is increased to 50% and the EVA width up to 12m in order to fulfil thoroughfare requirements (MoA and MoE Codes).
- Fireman's lift is required for basement to cover 60m length. In other words, a fireman's lift has to be provided every 120m at most. A fireman lift is required to be reached within 18m from the EVA (MoA Code).
- For basements over 3 storeys, smoke extraction is required and each basement compartment should be served by separate extraction systems (i.e. fan, ductwork system and independent shaft, FSI Code).
- For places of public entertainment (stadium, theatre, cinema etc), Part III of MoE Code require them to abut upon and have frontages to two or more thoroughfares for means of escape.

## 2.2 Design Guidelines for Caverns

Underground structure is recognised to possess different level of fire risk to life safety. Some of the unique hazards to underground structures are outlined below:

- Occupants have to escape upwards. For deep underground structure, staircase may not be appropriate for all occupants, considering those with mobility difficulty.
- Hot smoke entering staircases or vertical shaft will rise upwards and cause hazard for occupants escaping in the same direction.
- Fire fighters enter at ground level may travel against rising hot smoke to gain access into the compartment of fire origin.
- Smoke trapped within underground structure can only be vented through mechanical extraction.
- Heat accumulation is more severe in an underground space with limited ventilation.

In accessing the feasibility of introducing a wider use of underground caverns, the Hong Kong Government commissioned a study on the potential use of space underground (SPUN) in 1988. Subsequent to the SPUN Studies, a document titled 'Guide to Fire Safety Design for Caverns' was jointly published by the Building Authority and the Fire Services Department in 1994 [BA/FSD, 1994] to provide planning and design guidance for cavern development for use by public utilities e.g. refuse transfer station, water service reservoirs and sewage treatment works, etc. With the above categories the guidelines presume that the caverns will accommodate a low population i.e. facility workers, of which are familiar with the underground cavern and are not required to work and sleep in the cavern. There should be an effective management scheme and a suitable contingency plan provided to the workforce. The guidelines also assume the fire load is generally localised, relatively low and closely controlled. In essence, this document cannot be readily applied to typical habitable underground space.

Besides, some requirements stipulated within the document are relatively restrictive. For example, travel distance for single direction is limited to 18m (similar to MoE Code). Provision of smoke extraction may also be required subject to Fire Services Department (FSD). Provision of Emergency Vehicular Access is also required with setting-down point. A full list of fire services installations are given with the actual provision to be discussed and agreed with FSD.

Note that alternative approach is allowed within Section 5.6 of the Guide to Fire Safety Design for Caverns. In other words, performance based design using fire engineering approach is acceptable.

### 2.3 Areas of Improvement

Fire engineering approach is recognised as an alternative approach to prescriptive codes for fire safety design in Hong Kong. When fire engineering approach is adopted, the solution is considered on a case-by-case basis. This may be appropriate for unique underground space project. However, if there are a large number of underground projects, it may be more efficient to develop a generic guidance specifically for underground space. Alternative strategies may be developed to suit underground spaces with the following considerations:

- Occupants may be considered “temporary safe” when entering adjacent fire compartment(s) that are adequately protected from the compartment of fire origin.
- In long tunnel design where twin-tunnel is constructed, occupants are allowed to escape into adjacent tunnel as temporary safe place. Such concept may be adopted for large underground space using “streets” concept.
- For deep underground space, lift evacuation may have to be incorporated for a large proportion of the occupants. “Refuge area” may need to be considered.
- Enhanced fire protection systems such as smoke extraction and automatic sprinklers shall be provided. Fire suppression system is highly recommended for high-hazard rooms.
- Fire fighting strategies shall be developed with the fire services department. Special fire fighting and rescue technique may have to be developed to address large underground space.
- Control of material combustibility or surface retardant in space with larger crowd should be considered. Limit of fire load density may also be considered as part of the mitigation measures.

It is imperative to protect life safety in the event of fire. However, over-design resulting in significant investment (cost and space) should also be avoided. A carefully engineered strategy and solutions will aim to achieve a balance between level of life safety and cost effectiveness.

## 3. Fire Engineering Approach

### 3.1 Initial Agreement on Performance-based Design Issues

It is essential to agree with the approval authorities at the beginning of the project on the issues that a performance-based design is required to address. Commonly, the performance-based approach is adopted to address large fire compartments, long travel distances, large population, large smoke zone areas, smoke control system, fire separation, and structural fire protection issues.

Refer to the Guide to Fire Safety Design for Caverns, the major issues that need to be addressed for designing a cavern via fire engineering approach, including: Place of Safety, Place of Safe Passage, and Setting-down Point.

- “Place of Safety” means the open air.
- “Place of Safe Passage” means escape stairs, corridors or another fire compartment adequately protected from fire and smoke, and comprises the elements of the exit or route to be followed to reach a place of safety.
- “Setting-down Point” means an area in between the access tunnel and the underground cavern compartment, which provides turning facility for fire services appliances, ample spaces to set up controls and serves as a frontier during the course of fire fighting.

In an underground cavern, the distance from any location to the nearest exit leading to a place of safe passage should not exceed 18 m for single direction and 72 m for two or more choices of direction. This distance may be extended if it can be demonstrated that fire loads are low and escape routes are extremely simple. Exit time for the individuals in a fire compartment is normally limited to 2.5 minutes. In areas with a low population, the exit time will be primarily related to the distance required to travel. In assessing exit times on this basis, a travel speed of 1 m/s would be appropriate.

A place of safe passage for a cavern is normally access tunnels. The tunnels could be a few hundred metres long leading directly to a place of safety.

### 3.2 Performance-based Analysis

The initial agreement with the authorities gives the direction of the detailed performance-based design and analysis. The timeline analysis technique is well accepted. Analysis of fire safety is primarily concerned with time: time of ignition, time at detection, time for evacuation and time to reach untenable conditions (Fig. 1). Therefore, rather than examine fire safety in terms of travel distance and fire compartment areas or volume, the level of fire safety is assessed in terms of the time taken for the occupants to evacuate compared with the growth of the fire and time to reach untenable conditions. Therefore, life safety of occupants will be evaluated using a quantitative ‘timeline’ assessment. This primarily involves determining the time to untenable conditions (ASET) within the fire compartment and compares these with the time required for evacuation (RSET). The requirement for occupant safety is:

$$\text{RSET} + \text{Safety Margin} < \text{ASET}$$

Accordingly, determination of the development of the fire, release of smoke, temperature of the smoke layer, time to reach untenable conditions and time for evacuation form part of the assessment.

Safety of occupants in a cavern for a particular fire scenario can be achieved if it is demonstrated that the time taken to evacuate the occupants to a place of safe passage is less than the time taken for the fire to develop untenable conditions. This relationship incorporates an appropriate safety margin to accommodate for uncertainties in the assumptions and calculations.

In addition to the life safety of the occupants, fire fighters should also be given adequate protection by providing adequate access facilities. Given the nature of the underground space, the use of Setting-down Point can be considered as an alternative to traditional access provision. The engineering analyses should be conducted with full understanding of the fire fighting operation and technique. A unique fire fighting and rescue operation procedure may be developed to address large cavern site with public access.

### 3.3 Statutory Approval

Fire and life safety design for underground space represent a unique challenge. When a performance based approach is adopted, it is important that early consultation with the approving authorities, i.e. both Buildings Department and Fire Services Department, to agree principles and acceptance criteria. This is because the solution may require a number of iterations before final agreement can be achieved. A statutory approval framework that allows early and quick consultation will enable designers and engineers to make amendments in case fundamental provisions are identified through consultation.

### 3.4 Ventilation Consideration

As the underground space cannot obtain the fresh air naturally through the entry/exits, therefore mechanical ventilation shall be provided under normal operation within the space to provide an acceptable indoor environment. Sufficient plant space shall be provided for housing of ventilation equipment for fresh air supply and exhaust. In addition, ventilation shaft shall be extended from the plant room to the ground level for intake/discharge purpose. The ventilation requirements for different space types will be different. These requirements are expressed as an outdoor airflow rate per occupant or per unit floor area, or often both if the space is used as public. If the space is used as plant room, the minimum ventilation requirement usually expresses as air change rate per hour. If the ventilation rate is very large to maintain a good comfort for those spaces, some degree of cooling shall be considered to avoid large amount of untreated fresh air introducing into the underground occupied space which will degrade indoor comfort.

Beside the consideration of the normal operation, pressurization to the adit of the underground space and smoke control system within the underground space will also be required under emergency operation. The smoke control rate and pressurization rate can refer to the relevant fire services code.

## 4. Case Study

### 4.1 Case 1: Salt Water Reservoir in Cavern

A salt water reservoir adjacent to an important development has to be relocated to an enclosed cavern, and the only access path is the passage which joins the two new reservoirs. The length of the passage together with the reservoirs is up to 100 m, forming two large pocket spaces at each side. The fire safety design for

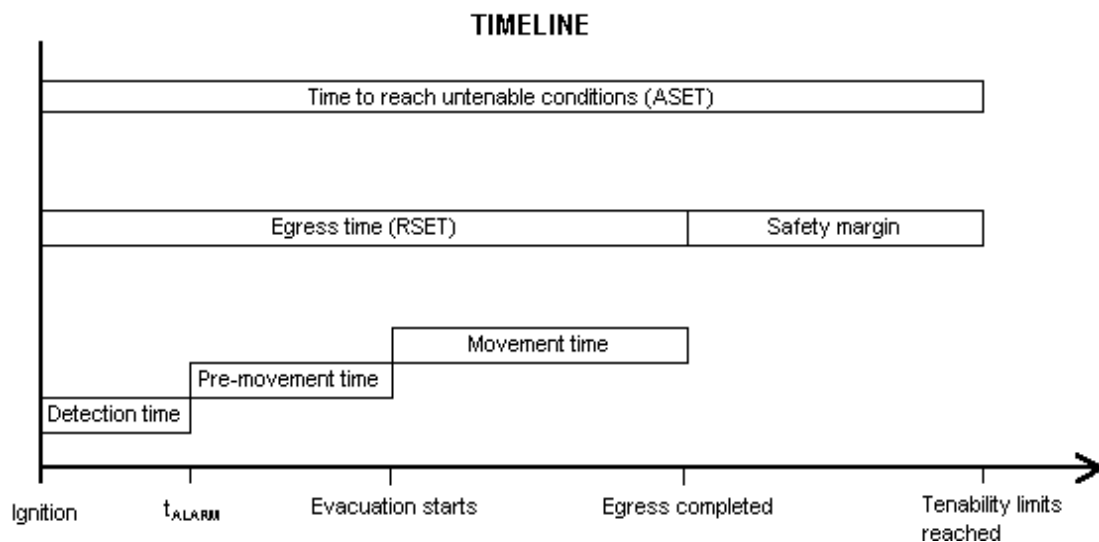


Fig. 1 Quantitative 'Timeline' Assessment

these two pocket spaces has been raised as an issue in particular the dead end escape travel distance can be up to approximate 100m, which is significantly longer than 18m requirement by the Guide to Fire Safety Design for Caverns.

The site is a utility facility that will be strictly accessed by staff only to carry out maintenance works. Two salt water tanks and a passage are connected to the exterior for cleaning and maintenance use. The passage is the path, which joins the two reservoirs together, and forms a dead end passage on each side. The overview of the reservoir is shown in Figure 2.

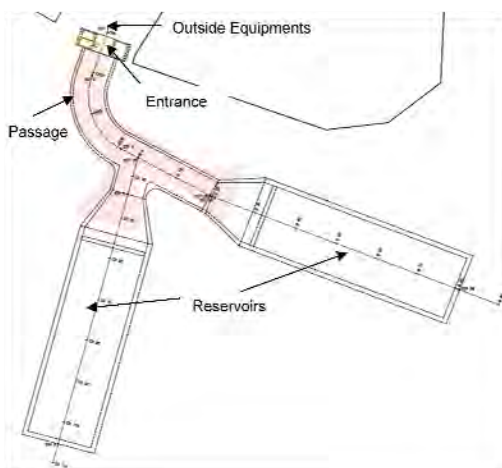


Fig. 2 Overview of Saltwater Reservoirs

The fire safety provisions of the facility have been designed and operated by referring to the Code of Practice on Safety and Health at Work in Confined Spaces [LD, 2000] published by the Hong Kong Labour Department. It is an enclosed space but not a confined spaces defined under the Factories and Industrial Undertakings Regulation [HKSARG, 2000]. At the same time, a qualitative analysis based on a fire engineering approach has been carried out to investigate the design of the reservoir.

The passage and the reservoirs are built with reinforced concrete walls. There are polythene sheets and a drainage layer typically underneath the ground. The proposed volume for each reservoir is 6,000 m<sup>3</sup>. A steel platform is built on top of the saltwater reservoir. The platform is 10 m above the ground level and the dimension of the platform is 46.8 m x 13 m x 0.8 m. A catwalk is also constructed on top of the platform. The maximum ceiling height is 4m from the platform, and the catwalk is 1.6 m wide.

The following fixed installations are recommended:

- Automatic alarm system
- CCTV
- Emergency lighting
- Exit and directional signs
- Manual fire alarm system (manual call points)

- Fire control centre (at the entrance portal)
- Fire detection system
- Gas detection system
- Portable fire extinguishers
- Ventilation control system
- Communications system

In addition to the fire protection provisions above, a management and operational plan has also developed specifically for the facility.

Since there is only one way in and out, it is necessary to control the occupant load for each operation. A minimum of 2 persons are recommended. The reason for a two persons team is that one can carry out the duty while the other one wait at the junction and serve as a watcher. The team can communicate through radio if an accident happens. There is no public access and the door is locked under normal situations.

If a maintenance vehicle is to enter the space, it will always enter before any other workers that enter by foot. This is to reduce the likelihood of accident and prevent a vehicle fire occurring near the entrance in the very unlikely event and affecting the escape route for occupants already inside the space.

Hot works such as welding and cutting in the space is only undertaken where there is no other alternative. If welding and cutting work in the space is unavoidable, the following safety precautions must be strictly followed:

- The supervisors and the workers should made reference to relevant ordinances and codes of practice.
- Appropriate pre-entry screening procedures for safe entry of the space must be performed.
- Supervisors must complete and post a welding permit before allowing any welding or cutting to be carried out.
- Sufficient ventilation must be provided.
- Work atmosphere must be constantly monitored by appropriate gas detection.
- No ignition source should be introduced into the space until testing confirms that no flammable or explosive atmosphere exists. Continuous monitoring of oxygen concentration and flammable levels should also be maintained
- Under no circumstance is welding gas cylinders permitted inside the space. The gas should be turned off at the cylinder valve when not in use and remove unattended torches and hoses from the space.
- If it is possible, the welding transformer and regulator should be placed outside the enclosed space.

- Never, under any conditions, should compressed oxygen be used to provide “fresh air” to someone working in the enclosed space.

A fire safety management plan has also been developed with the inputs of relevant stakeholders such as designers, users, maintenance team etc. Important aspects such as staff training plan, fire action plan, maintenance plan and housekeeping are included in the management plan.

### Staff Training Plan

The supervisors and the workers of the maintenance should be well-trained according to the training requirements stipulated in the Code of Practice for Safety and Health at Work in Confined Space. In addition, the staff training plan should clearly define responsibilities of each party (e.g. supervisors, standby persons and workers). The content of the staff training plan covers the following topics:

- Work activities, duties and responsibilities of maintenance staff;
- Where appropriate, the use of fire fighting equipments – portable fire extinguishers;
- Action to be taken in case of fire.

### Fire Action Plan

Fire action plan will be executed upon occurrence of fire incidence. All members of the maintenance team should be fully aware of their duties if a fire situation arises. The fire action plan takes into account the possible scenarios of fire. The scenarios are:

- Spot of outbreak of fire by the standby person;
- Spot of outbreak of fire by the workers;
- Fire occurs at the outside portal;
- Fire occurs inside the unmanned/enclosed space.

### Maintenance Plan

It is critical that the fire protection systems function and work as required because they form the first line of defence in any fire safety strategy. Any damage should be reported immediately and effective measures should be implemented to compensate for the damage until they are repaired. A detailed maintenance plan is developed and forms a part of the Fire Safety Management Plan.

### Housekeeping

Good housekeeping and sensible fire precautions will reduce the possibility of a fire occurring. The maintenance team should ensure that accumulation of trash will not occur. Any trash shall be removed immediately after each access. Smoking is strictly prohibited.

### Risk Assessment

If major work requiring access for staff into the enclosed space, a risk assessment shall be conducted

by a competent person to identify the fire hazards likely to be present on site if major work is to be conducted. One aspect of the risk assessment is the possibility of fire in the space. Therefore, any fire risk will be assessed and given consideration for appropriate protection if necessary.

### **4.2 Case 2: Rock Cavern Stadium**

The World Largest cavern hall for public use is situated in Gjøvik Norway. The cavern hall was used for hosting the Winter Olympic Games in 1994 [MELAND, 1993].

After the Olympic Games, the hall has been used for multi-purpose arena for ice hockey, figure skating, curling, track & field, and swimming as well as for conferences, exhibitions and fairs with a capacity of 5,800 sports-spectators. The hall was built inside the Hovdetoppen mountain right in the centre of Gjøvik. The rock is gneiss, perhaps as old as 800 to 1100 million years old.

### Scale of the Cavern

The dimensions of the cavern and usable area arrangement are listed as follows:

Height : 25 m  
 Length: 91 m  
 Width: 61 m  
 Seating Capacity: 5,800  
 Net Area: 10,010 m<sup>2</sup>  
 Gross Area: 14,910 m<sup>2</sup>  
 Stadium: 1,965 m<sup>2</sup>  
 Public Areas: 2,224 m<sup>2</sup>  
 Media Centre: 2,110 m<sup>2</sup>  
 Participant Facilities: 263 m<sup>2</sup>  
 Admin. & VIP Centre: 577 m<sup>2</sup>  
 Operations/Maintenance: 515 m<sup>2</sup>  
 Technical & Electrical: 638 m<sup>2</sup>  
 Additional Facilities & Storage: 1,100 m<sup>2</sup>

The cavern hall is 9 stories high. In addition to the game hall, a number of special areas and rooms for the athletes, the media and VIP are constructed. The layout is illustrated in Figure 3.

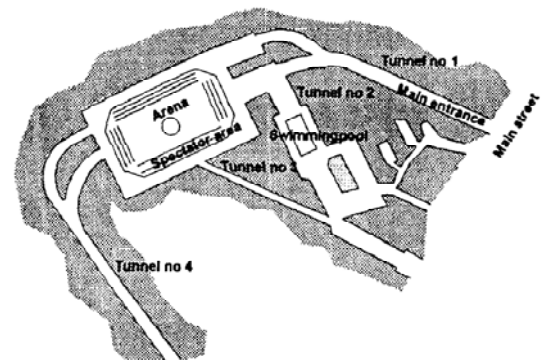


Fig. 3 Gjøvik Cavern Hall Layout

### Fire Safety

Fire safety strategy has been carefully considered for construction, smoke control, and evacuation of 5,800 spectators and staff members. All construction materials, finishing, and furniture are specified as non-combustible materials, fire-retardant materials and limited flammability materials. Ventilation for the spectator area is designed with a ventilation-by-displacement system as an alternative to ventilation by complete mixing.

Smoke control systems are provided in the cavern hall, “safe area”, and the access / evacuation tunnels. Fire and smoke separations between the cavern hall, the safe area and the tunnels ensure that a fire in the cavern hall or in relevant service rooms will not affect the safe area and the tunnels.

To protect the spectators and the staff members in case of fire, “safe area” has been constructed surrounding the spectator stands. The safe area provides a minimum of 0.6 m<sup>2</sup> per person for emergency use. The safe area is fire separated from the cavern hall. In this area, no combustible material and / or hazardous material are allowed.

Should a fire break out in the spectator stands, the spectators will evacuate from the cavern hall to the safe area and wait for further instruction. If full evacuation from the cavern is necessary, all spectators and staff members are instructed to evacuate to the open space via the tunnels. The tunnels are over 100m long from the cavern hall to outside.

### **4.3 Case 3: MTR Underground Stations**

Many of the MTR stations in Hong Kong are underground. The West Island Line stations can be as deep as 80m below surface. A line wide fire safety strategy is usually developed to address the means of escape, fire fighting access, fire resisting construction and appropriate fire services installation, including smoke control system. Fire safety management also forms part of the strategy to ensure that the safety measures are properly implemented and maintained during occupation of the stations.

The objective is to minimize risk of fire occurrence through a combination of active and passive fire prevention measures. It is recognized that ignition sources within a station is limited and the fire loads are relatively low compared to other occupancy type [ITA, 1998].

In principle all public areas, including the adits connecting the station and remote entrances, are provided with adequate smoke control to maintain smoke layer at least 2.5 m above occupied level. Escape routes are provided such that all occupants can evacuate from an incident zone or platform within a

fixed period of time, e.g. 4.5 minutes in the modern lines strategy. The maximum number of occupants in a station is dictated by the full train load, peak hour occupancy at platforms and headways of train. Occupants are expected to use the escalators as part of the means of escape. This is recognized internationally as an appropriate means of escape within train stations. In order to minimize fire risk to occupants, public areas are always fire separated from non-public areas such as back of house and MEP plant rooms. The main underground structure is required to achieve 4 hour fire resistance period as indicated in the code of practice of fire resisting construction in Hong Kong.

It is important that fire-fighting personnel can get access to the fire affected area at the earliest possible for fire-fighting and rescue operations. Fire fighting access for underground stations represent a challenge especially at urban area due to limited space above ground for access of fire fighting vehicles. Some stations are remote or deep down from the entrances at ground level. In such case, protected route is provided from the fire fighting stair and lift so that the unprotected distance is limited to certain distance, e.g. 60m as prescribed in the code of practice for fire fighting access.

### **5. Conclusion Remarks**

Underground spaces in subsurface are popular in Hong Kong, in particularly the MTR and associated underground retail complexes. However, purpose-built cavern development is rare.

Hong Kong Government has engaged a consultant to undertake a study on the Enhanced Use of Underground Spaces in Hong Kong. The study aims to investigate and resolve the issues of the underground planning, strategies and practice related to traffic, accessibility, development compatibility and development intensity.

Fire and life safety is a major technical issue. The existing codes are prescriptive in nature and written to address fire safety design of typical buildings. The clauses cannot be easily met by large underground structure/cavern. The Guide to Fire Safety Design for Caverns published by Building Authority and Fire Services Department in 1994 is a starting point for fire safety design for caverns. Fire engineering approach will be an appropriate alternative to address the fire safety issues of cavern development.

Three cases demonstrated the variety usage of the cavern spaces. Fire engineering approach has been adopted to address the issues of fire and life safety. Up to date, fire safety design for cavern development has not been fully addresses. It is essential to carry out further studies on the issues.

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# DEEP SHAFTS USING DIAPHRAGM WALL TECHNIQUES IN HONG KONG

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## Abstract

Numerous projects have been under construction in Hong Kong over the past few years, which have necessitated the construction of deep shafts using diaphragm wall techniques. Whether for temporary purpose, such as access of tunnelling equipment, or for permanent usage, such as riser shafts for drainage or sewage transfer schemes, deep excavations must be constructed as part of tunnelling or cavern engineering works. A short review of the different techniques available for construction of deep shafts is presented, and the specific aspects of shafts constructed using diaphragm wall techniques are developed in terms of design and construction, with a particular emphasis on the construction challenges of very deep shafts with small diameter. Results obtained on 4 shafts, up to 90 meters depth, constructed by the Author in Hong Kong in 2009 and 2010 are presented

## Keywords

Diaphragm Wall, Circular, Deep Shaft

## 1. Introduction

With the ever growing population density in Hong Kong and in other major cities in the world, there is an increased necessity to create and maximize usable space for people and vehicles, as well as to provide or expand public transportation systems.

Part of the solution is to create more and ever deeper underground space. As this must be done in already very dense and congested environments, minimizing the impact on existing structures is primordial. Various techniques can be used for creating earth and water retaining structures. However, for deep excavations, diaphragm wall techniques, up to 1.5 m in thickness are most suitable as they offer:

- large stiffness, thereby minimizing ground movements in surrounding areas. As we will see later, this is even more so in the case of circular layouts (e.g. shafts) where hoop stiffness plays a major role and reduces wall deflections to extremely small values, generally a few millimeters only, even at very large excavation depths.
- good to excellent verticality control, thereby ensuring continuous water cut-off when compared to pile-based retaining structures which often necessitate additional grout

treatment behind the wall and/or a soft/hard pile layout which incurs increased excavation wastage due to overlaps.

- a wall type which can be considered as a permanent structure where applicable, thereby avoiding the construction of a second wall and maximizing space within the final structure while reducing construction time.
- a type of retaining structure which can also be used as load-bearing and contribute to the foundation of the superstructure.

As a result, diaphragm walls are increasingly being used for:

- temporary or permanent walls for deep basement developments (e.g. car parks, shopping malls)
- deep foundations for skyscrapers, combined with large diameter circular shafts for basements (such as ICC Tower in Hong Kong or Nakheel Tall Tower in Dubai)
- cut & cover tunneling
- subway stations and TBM launching / receiving shafts
- temporary shafts for tunneling and/or cavern construction (for access, exit, ventilation, etc.)
- permanent shafts for drainage / sewage tunnels (riser shafts, etc.)

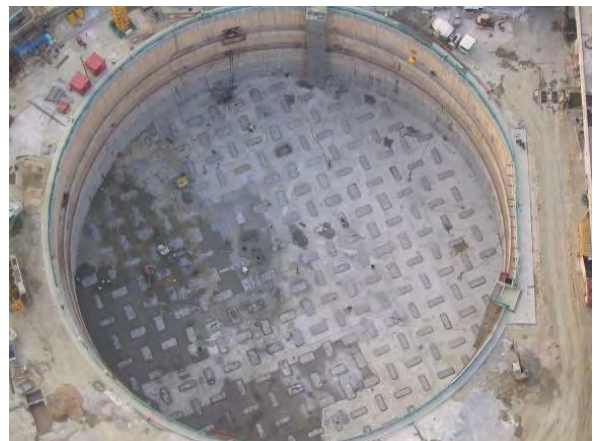


Fig.1 76 m diameter basement shaft & foundations, ICC Tower, Hong Kong

In the following, we will concentrate on the design and construction aspects of deep circular shafts using diaphragm wall techniques.

## 2. Diaphragm Wall Shaft Design Considerations

### 2.1 General

One essential feature of diaphragm wall circular shafts, as opposed to other type of circular, pile-based shafts, is that they provide a horizontal continuity which enables transfer of hoop stresses in concrete between adjacent panels. As such, the primary mode of resisting the earth and water pressures is in horizontal hoop compression as opposed to vertical bending between strutting elements. This is obviously a much more efficient way of making use of the available concrete thickness, since vertical bending moments are minimal. In fact, those small vertical bending moments are only a by-product of vertical deflections caused by varying horizontal hoop strains with depth. In fact, circular hoop stiffness acts as a continuous strutting from top to bottom of the wall. This circular stiffness of the wall is calculated as follows:

$$K = \frac{E \cdot T}{R^2} \quad (1)$$

Where: K = circular stiffness (resisting pressure per unit wall deflection)

E = Young's modulus of the wall's concrete

T = usable wall thickness<sup>(i)</sup>

R = Radius of shaft<sup>(ii)</sup>

<sup>(i)</sup> After appropriate deductions for vertical tolerances

<sup>(ii)</sup> From shaft centre to wall's axis

This working mode in hoop compression is what enables the design of very deep shafts, when other types of conventional retaining structures would be limited to 30-35 m in depth.

The hoop stress can be easily calculated as

$$\sigma = \frac{PR}{T} \quad (2)$$

Where: P = Total resulting pressure at depth  
R and T as in (1)

As an example, for a 13 m diameter, 1.5 m thick shaft assumed to maintain a 0.9 m overlap at its toe, with full bulk excavation down to its toe at 90 m depth, such as was recently constructed in Hong Kong, the hoop stress at its toe would be in the order of 8.75 MPa, which is within the capacity of typical 45 MPa (cube strength) tremie concrete.

In the case of shafts reaching rockhead, it may happen that the rockhead level is uneven over the footprint of the shaft. In this case, additional excavation of rock to equalize the panels' toe to the deepest level is a very costly and time consuming solution. It is usually preferable to start casting ring beam(s) or a continuous lining when excavation proceeds near the highest rock

level, so as to ensure the continuity of hoop stress transfer where the diaphragm wall itself no longer forms a continuous horizontal ring.

### 2.2 Diameter

A very wide range of diameters can be used, from about 7 m internal diameter for small access shafts to 76 m (ICC, Hong Kong) or even 130 m (Tall Tower, Dubai) for basement construction.

The smallest diameters are governed by the size of excavation equipment used, for which the standard length is 2.8 m and the standard thicknesses are 0.8, 1.0, 1.2 and 1.5 m. So the smallest practical diameter of about 7 m imposes a panel layout which, depending on the chosen excavation tools and type of joint, can range from an octagon to a dodecagon. This, however, is not an issue as hoop stresses can develop even in non-perfectly circular shapes.

Whilst there is no theoretical limitation to maximum shaft diameter, due consideration should be given in the design to the possibility of unbalanced loading as it is more likely to occur, due to differential earth and/or water pressure and/or asymmetric excavation in large shafts. The adverse effects of such unbalance obviously increase with increasing diameter. The parasitic horizontal bending moments and shear created in the "ring" by asymmetric loading can be estimated using standard structural "beam on spring" software or other appropriate geotechnical software and, if they exceed the moment / shear capacity of the diaphragm wall at the joints, this can be dealt with, first of all using a capping beam, then if necessary using ring beams built as bulk excavation proceeds. Although there is generally no horizontal structural connection between diaphragm wall panels, it must be noted that there is still a moment and shear transfer capacity due to the hoop compression over the thickness of the joint.

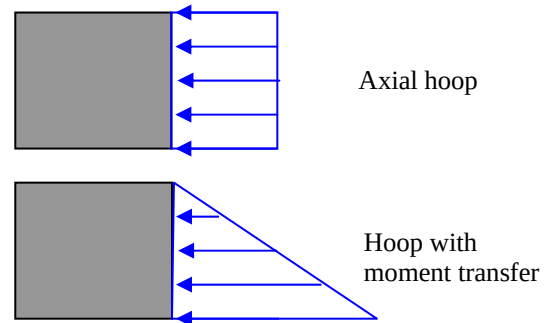


Fig. 2 Moment Transfer with Hoop Eccentricity

### 2.3 Panel Layout

Workable panel layouts for diaphragm wall circular shafts will depend mainly on the chosen excavation method and tools, as well as type of joints between

panels. Such methods, tools and type of joints will, in turn, depend on the geological conditions, the depth of diaphragm wall excavation and the depth of bulk excavation. The smaller the shaft diameter and the deeper, the more challenging it will be to come up with a workable layout. There is no universal recipe, and it is advised to consult with specialist contractors on this issue.

A few general rules, however, are as follows:

- For relatively shallow walls, either cutter type of equipment or clamshell may be used although cutter equipment is preferable in terms of verticality control. In both cases, either milled joints or stop-ends may be used.
- For wall depths beyond 40 or 45 m, it is advisable to use cutter equipment with milled joints over part or all of the wall depth. This is because this type of equipment, when skillfully operated, offers much better verticality control than clamshells. Good verticality control is essential for deep shafts with large hoop stresses, in order to ensure proper overlap of panels at depth. This 40 - 45 m depth range also corresponds to practical limits for installation of “stop-end” type of joints.
- For small diameter shafts, the angle between adjacent panels becomes larger (Fig. 4). In the case of milled joints, this means that significant quantities of hard concrete from two adjacent, already cast panels will need to be milled during excavation of the closing panel. As the amount of concrete to be milled is large on the inner wall face and minimal on the outer wall face, there is a tendency to deviate from the vertical and this must constantly be corrected while excavation proceeds. Unless special measures are taken, as discussed in more details later in this paper, this can become quite an issue especially in terms of construction time and cost.



Fig. 3 Verticality Control using Cutter Excavation

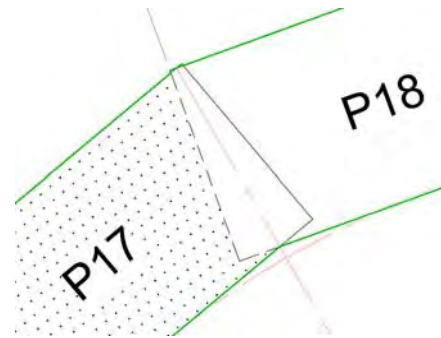


Fig. 4 Joint Angle

## 2.4 Trench Stability

For convenience, trench stability calculations for circular shafts using diaphragm wall techniques are usually carried out assuming a straight trench, in the same manner as for linear walls. For panels using single, 2.8 m long bites, this assumption is geometrically true. However, for panels with multiple bites, it is an approximation. It must be noted that for the outer wall face, this approximation is conservative because of the reduced mass of potentially unstable soil. However, the situation on the inner face is opposite. For large diameter shafts, the difference is insignificant as the curvature is quite flat. Typical assumptions for surcharge loadings, such as 15 or 20 kPa up to the edge of the trench are rarely encountered in practice, and this conservativeness somehow compensates for the “straight trench” approximation. For small diameters, however, it is recommended to avoid placing any significant surcharge within the footprint of potentially unstable wedge on the inner side of the shaft.

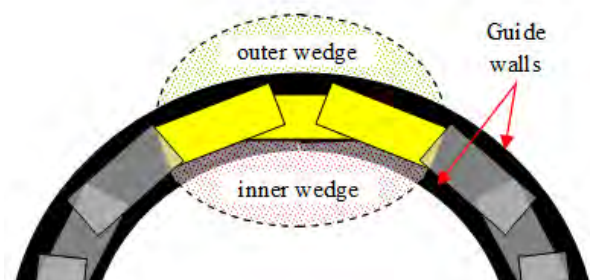


Fig. 5 Trench Stability, Effect of Curvature

## 2.5 Reinforcement

Since circular shafts work primarily in horizontal hoop, the vertical reinforcement can be particularly light. The design hoop stress is generally calculated using the remaining wall thickness after due deductions for vertical tolerances (Fig. 6). In this approach, the hoop stress is assumed to develop within a true circular ring and therefore acts axially. The thickness of the wall is generally designed such that the resulting hoop stress falls within the capacity of plain concrete. Then the section can be assimilated to the case of a plain stocky wall, and the reinforcement cage can be minimized.

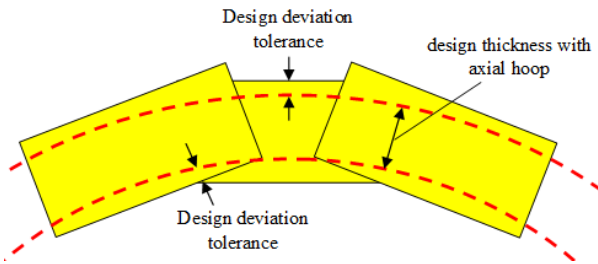


Fig. 6 Assumption for Hoop Stress Calculations

When necessary, for example in the case of unbalanced loadings on the wall, more detailed analyses can be made to estimate parasitic horizontal bending moments in the ring and see whether they can be accommodated without additional ring beams.

A typical reinforcement cage arrangement for milled joint type is shown in Figure 7 below, where P1 is a primary panel. P2 and P18 are closing panels.

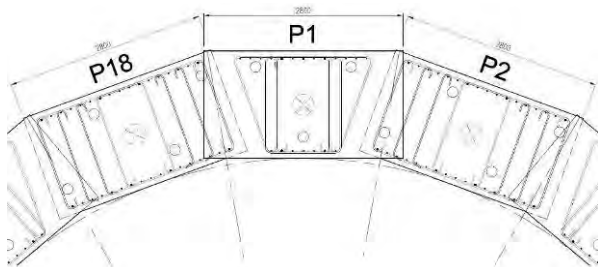


Fig. 7 Typical Rebar Cage Arrangement

### 3. Excavation Tools and Equipments

Excavation tools and equipments shall be carefully selected to suit various ground conditions at site. Generally, we can categorize excavation equipments into two main types, (1) cable grab also called clamshell and (2) hydraulic cutter.

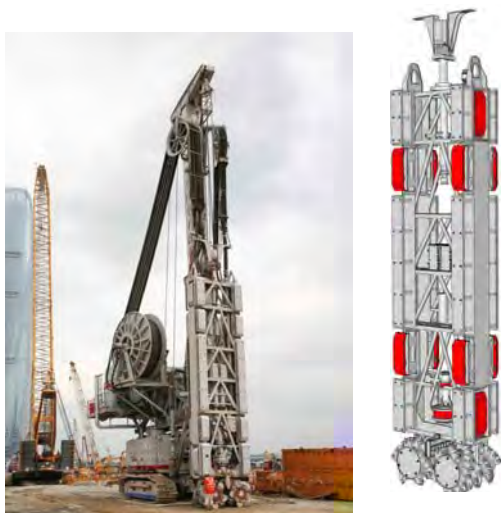


Fig. 8 Hydraulic Cutter (BAUER model BC40 shown)

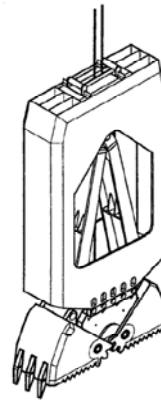


Fig. 9 Normal Grab with Crawler Crane

Normal grab is suitable for excavation in shallow shafts. It is quick to mobilize and fit within congested site area. It is commonly used for small to medium size projects. There are different sizes of grabs ranging from 600 mm to 1500 mm available in the industry to suit various wall thicknesses. The standard width of the grab is 2800 mm.

The hydraulic cutter gives better trench quality and faster excavation progress, particularly for deep shafts. High efficiency bentonite circulation system is essential when using cutter. With the aids of sensors and computerized controls, the cutter offers real time monitoring on excavation in progress. Corresponding to the depths and orientations, the steering plates in cutter frame can be automatically or manually activated to ensure excavation is in line with the targeted alignment. The cutter frame is mounted with four high speed rotating wheels. A wide selection of cutting bits can be mounted on the wheels to overcome different soil strata.

For easy maneuvering on site, the cutter frame is carried by a base machine. The base machine is equipped with a hydraulic power pack for operating the cutter frame. Automatic coilers are mounted at the top of base machine for coiling and un-coiling of the mud hoses and hydraulic hoses.

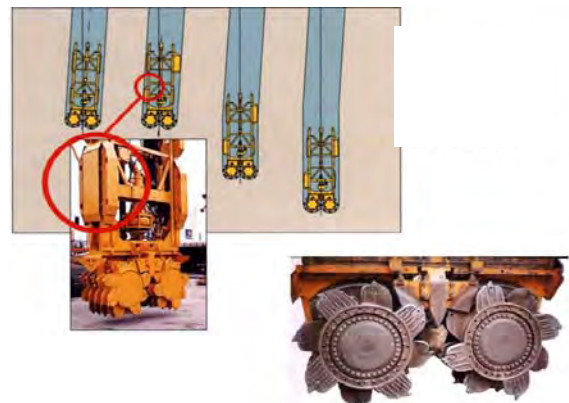


Fig. 10 Cutter Frame with Steering Plates

#### 4. Method of Diaphragm Wall Construction

Sequence of works can be briefly described as follows:

- i. Construct guide wall
- ii. Excavate diaphragm wall panels
- iii. Fill & top up trench with bentonite slurry during excavation
- iv. Excavate to the predetermined toe level
- v. Substitute fresh bentonite into panel or desanding the bentonite to acceptable criteria
- vi. Check verticality with chisel or koden machine (if cutter is used, refer to cutter operation report from cutter machine)
- vii. Install stop end (except in case of concrete milled joint)
- viii. Install prefabricated reinforcement cages
- ix. Install concrete hopper & tremie pipe
- x. Pour concrete by tremie method
- xi. Interface core to check concrete and rock interface if applicable, normally carried out at 14 days after concrete casting
- xii. Toe grouting and rock fissure grout as per design

Sufficient bentonite storage is needed before commencement of excavation. The bentonite is critical for maintaining the trench stability. If in any case, there is sudden leakage of bentonite, spare bentonite can be used to maintain a minimum bentonite head in the trench. The bentonite is to be tested at intervals and controlled to required standard. Admixtures can be added to the bentonite as necessary. The bentonite circulation system shall be properly maintained to ensure continuous supply and smooth excavation progress. New type of bentonite pool is shown in Figure 12 for an example.



Fig. 11 Steel Circular Bentonite Pool, 400 m<sup>3</sup> Capacity

Rock excavation is a time consuming process in diaphragm wall construction. When encountering rock, chiseling method will normally be necessary.

Toe grouting or contact grout will be carried out after completion of all diaphragm wall panels. It is normally a bentonite / cement mix injected through the reservation tubes to the bottom of wall filling up the voids, if any, between the toe and founded rock layer.

This forms a grout seal at the perimeter of the diaphragm wall as water cut-off.

Depending on ground conditions, rock fissure grout can be carried out on the rock layer below the diaphragm wall to further seal the fine cracks. The rock fissure grout may consist of micro-fine cement.

In the final stage, pumping test shall be used to verify the water-tightness performance of the diaphragm walls. Pump wells and observation wells associated with stand pipes shall be installed. Water will be drawn out from the shaft and will be measured to see if there is any abnormal ingress of water. The water level on adjacent ground will be monitored to see if there is any significant drawdown of water level outside the diaphragm wall. Pumping test is recommended as it will safe-guard the operatives who will carry out the bulk excavation.

#### 5. Particulars of Deep Shafts

The excavation of small diameter deep shaft requires highest techniques and expertise.

Verticality control is extremely critical in order to meet with the design requirement. On deep shafts, verticality requirement is normally in the range of 1:350 to 1:400. Deviation will occur when cutting through different ground conditions. If excessive deviation is found, the excavated panel shall be backfilled with low strength or lean-mix concrete. In this case, the panel is to be re-excavated, which in turn imposes an impact on the progress.

Water-tightness is an important subject on diaphragm wall construction. To make the shaft functioning as it is designed to be, all panels are to be constructed in the correct alignments in both vertical and horizontal directions within tolerances. Failure to comply may result in forming a “window” through the wall. This may cause a severe water leakage and possibly structural failure due to insufficient overlapping thickness of the diaphragm wall.

Hydraulic cutter is recommended to excavate deep shafts. Then, concrete milled joints are to be formed between panels to ensure watertightness. For shafts with depths up to about 40m, traditional stop-ends are suitable. See Fig. 12 below for reference.

To achieve high quality on verticality and to form a water tight concrete milled joint, a new technology named as ‘Sacrificial Void Formers’ (SVF) has been developed by the Author. The improved method of milling primary panel edges has been adopted recently in a 90m deep circular shaft constructed in Hong Kong. The method greatly reduces the amount of structural concrete (high strength) which must be milled when the closing panels are excavated.

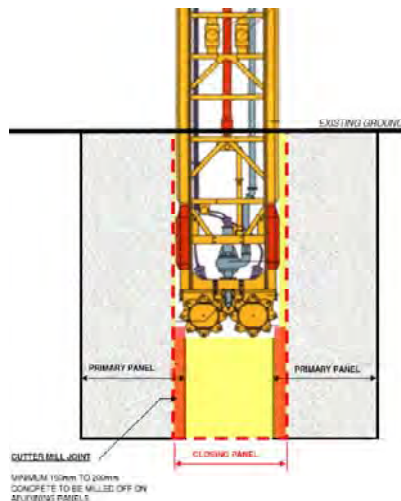


Fig. 12 Cutter Milled Joint Arrangement

The SVF method involves casting a diaphragm wall comprising two primary panels joined by a closing panel. In a first step, a casting volume is excavated for each primary section, then the primary sections are cast in the primary casting volumes. A sacrificial displacement element (SVF) is positioned down each side of the primary sections, in a position where the milling of the secondary panel will take place. A secondary casting volume is then excavated for casting the secondary section, and the material of the SVF is removed. At the same time, a joining surface profile is milled on to the edges of the primary sections so that, when the secondary panel is cast, a sound, clean joint will be formed between the adjacent panels.

The rebar cages accompanied with the void formers, are lowered into the diaphragm wall trench by crane, and the structural concrete is then poured with the primary panels.

The SVF assembly can comprise a rebar cage for providing reinforcement for the primary section when the primary section is cast, with the sacrificial displacement element(s) being secured to the rebar cage. Many materials can be suitable for the SVFs, provided that it is softer, or more brittle, or more friable, or less dense than the concrete used to cast the primary sections.

The prefabricated sacrificial displacement elements can be built up in modules. By assembling the sacrificial displacement elements in modular fashion, the assembly process can be simplified and sped up.

This new method is particularly useful where the adjacent primary and secondary panels are at an angle to each other. Angled sections such as these are often used in the construction of curved diaphragm for containment tanks, for example, or deep circular shafts.



Fig. 13 Installation of Rebar Cage with SVF



Fig. 14 A 13 m Dia. Shaft under Excavation at 82 m Depth (with the application of SVF, diaphragm walls are in perfect alignment)

## 6. Results for 4 shafts recently constructed

### 6.1 General

The case of 4 small diameter, deep shafts built in Hong Kong between 2009 & 2010 are summarized hereafter, in terms of construction time, type of equipment used and joint type as well as verticality achieved.

Geometrical data of these 4 shafts are given in Table 1 below.

|                           | Shaft          |                             |                              |                   |
|---------------------------|----------------|-----------------------------|------------------------------|-------------------|
|                           | 1              | 2                           | 3                            | 4                 |
| Int. dia.(m)              | 12             | 12.3                        | 9.55                         | 13.75             |
| Thickness (m)             | 1.5            | 1.5                         | 1.2                          | 1.2               |
| Wall depth (m)            | 56 - 58        | 85.2 – 89.8                 | 38.2 – 44.0                  | 33.7              |
| Bulk excavation depth (m) | 48.7           | 96.2 m (far below wall toe) | 111.6 m (far below wall toe) | 20.5              |
| Stratum / thickness (m)   | Fill 20        | Fill 19 – 22                | Fill 10                      | Fill 11           |
|                           | Alluvium 6 - 8 | Marine Clay 4 – 9           | Marine Clay 4                | Marine Clay 4 - 5 |
|                           | CDG 30         | Alluvium 0-4                | Alluvium 17                  | Alluvium 18       |
|                           |                | CDG 35 – 50                 | CDG 7 - 13                   |                   |
|                           |                | Grade II Boulder 1-4        | Grade I/II Boulder 0.3 - 3   |                   |
|                           |                | Corestones 5 – 18           |                              |                   |
|                           |                |                             |                              |                   |

Table 1 Geometry and Geology for 4 Shafts Studied

## 6.2 Construction Time

Construction time can of course vary significantly depending on ground conditions and chosen equipment. In particular, excavation of rock (grade III or II, boulders and corestones) is extremely time consuming, regardless of excavation equipment. On small diameter shafts such as those presented herein, space is limited and therefore only one rig can operate. If, as it is usually the case, time and cost are of essence, it is important to minimise the amount of rock to be excavated. In all cases presented, grout curtains were built underneath the diaphragm wall to minimise water inflows during bulk excavation whilst eliminating the need to excavate down to “perfect rock”. Such curtains were built either using Tube-a-Manchettes (cases 1 & 4) or rock fissure grouting (cases 2 & 3).

The diaphragm wall and curtain grouting construction durations for each case are shown in Table 2.

|                                    | Shaft                                |                                |                                 |             |
|------------------------------------|--------------------------------------|--------------------------------|---------------------------------|-------------|
|                                    | 1                                    | 2                              | 3                               | 4           |
| Excavated area (m <sup>2</sup> )   | 2455                                 | 3801                           | 1353                            | 1581        |
| Excavated volume (m <sup>3</sup> ) | 3683                                 | 5701                           | 1623                            | 1897        |
| Nos. of panels                     | 6                                    | 18                             | 6                               | 8           |
| Main type of equipment             | cutter                               | cutter                         | clamshell                       | clamshell   |
| type of joint                      | stop-end                             | milled                         | stop-end                        | stop-end    |
| Guide Walls (months)               | 1                                    | 0.5                            | 0.75                            | 0.5         |
| RockFill pre-treatment (months)    | N/A                                  | 1                              | N/A                             | N/A         |
| <b>Excavation</b>                  | <b>1.5</b>                           | <b>6.5</b>                     | <b>4.25</b>                     | <b>1.75</b> |
| Curtain grouting (months)          | 0.75                                 | 2                              | 1.5                             | 1.75        |
| Overall excl. pump test (months)   | 3                                    | 10                             | 6.5                             | 4           |
| Comments                           | virtually no rock except 1 panel toe | large boulder, many corestones | sloping grade I/ II rock at toe | No rock     |

Table 2 Construction Time for 4 Shafts Studied

## 6.3 Achieved Verticality

Verticality achieved are detailed in Tables 3 to 6.

## 7. Conclusion

Referring to the results and the experiences we gained from the above shafts, we observe the following:

- The use of cutter will generally give better progress on excavation compared that to ordinary grab method. However, the grab method will have an advantage if space is limited.
- On deep shafts which are over 50m, the verticality requirements have proved to be achievable. Even on the shaft which is over 90m deep, the excavation by cutter method has proved capable of achieving high verticality standard.

| Panel          | Axis deviation @ toe from Kodan tests (m) | Difference between adjacent panels (m) | Toe depth (m) | Angular deviation (1 in) |
|----------------|-------------------------------------------|----------------------------------------|---------------|--------------------------|
| 1L             | 0.05                                      | 0.05                                   | 55.817        | 1116                     |
| 1M             | 0                                         | 0.05                                   | 55.817        | -                        |
| 1R             | 0                                         | 0                                      | 55.817        | -                        |
| 2L             | 0                                         | 0                                      | 55.727        | -                        |
| 2M             | 0.1                                       | 0.1                                    | 55.727        | 557                      |
| 2R             | 0.1                                       | 0                                      | 55.727        | 557                      |
| 3L             | 0.05                                      | 0.05                                   | 55.767        | 1115                     |
| 3M             | 0.1                                       | 0.05                                   | 55.767        | 558                      |
| 3R             | 0.14                                      | 0.04                                   | 55.767        | 398                      |
| 4L             | 0.1                                       | 0.04                                   | 58.75         | 588                      |
| 4M             | 0.12                                      | 0.02                                   | 58.75         | 490                      |
| 4R             | 0.13                                      | 0.01                                   | 58.75         | 452                      |
| 5L             | 0.1                                       | 0.03                                   | 58.747        | 587                      |
| 5M             | 0.15                                      | 0.05                                   | 58.747        | 392                      |
| 5R             | 0.05                                      | 0.1                                    | 58.747        | 1175                     |
| 6L             | 0.05                                      | 0                                      | 58.777        | 1176                     |
| 6M             | 0                                         | 0.05                                   | 58.777        | -                        |
| 6R             | 0                                         | 0                                      | 58.777        | -                        |
| <b>Min</b>     | <b>0</b>                                  | <b>0</b>                               | <b>55.727</b> | <b>392</b>               |
| <b>Max</b>     | <b>0.15</b>                               | <b>0.1</b>                             | <b>58.777</b> | <b>1176</b>              |
| <b>Average</b> | <b>0.07</b>                               | <b>0.04</b>                            | <b>57.26</b>  | <b>&lt;705</b>           |

Table 3 Verticality Achieved by Cutter (Shaft 1)

| Panel          | Axis deviation at toe level from Kodan tests (m) | Difference between adjacent panels (m) | Toe depth (m) | Angular Deviation (1 in) |
|----------------|--------------------------------------------------|----------------------------------------|---------------|--------------------------|
| 1              | 0.1                                              | 0.1                                    | 88.4          | 884                      |
| 2              | 0                                                | 0.1                                    | 88.17         | -                        |
| 3              | -0.05                                            | 0.05                                   | 87.5          | 1750                     |
| 4              | 0.1                                              | 0.15                                   | 87.34         | 873                      |
| 5              | 0                                                | 0.1                                    | 87.95         | -                        |
| 6              | -0.05                                            | 0.05                                   | 87.5          | 1750                     |
| 7              | 0.1                                              | 0.15                                   | 89.34         | 893                      |
| 8              | 0.15                                             | 0.05                                   | 88.39         | 589                      |
| 9              | 0.1                                              | 0.05                                   | 85.21         | 852                      |
| 10             | -0.1                                             | 0.2                                    | 88.06         | 881                      |
| 11             | 0                                                | 0.1                                    | 86.56         | -                        |
| 12             | 0                                                | 0                                      | 89.8          | -                        |
| 13             | 0.1                                              | 0.1                                    | 89.71         | 897                      |
| 14             | 0.1                                              | 0                                      | 88.76         | 888                      |
| 15             | 0                                                | 0.1                                    | 88.43         | -                        |
| 16             | 0.05                                             | 0.05                                   | 88.56         | 1771                     |
| 17             | 0.05                                             | 0                                      | 88.85         | 1777                     |
| 18             | 0                                                | 0.05                                   | 88.9          | -                        |
| <b>Min</b>     | <b>-0.1</b>                                      | <b>0</b>                               | <b>85.21</b>  | <b>589</b>               |
| <b>Max</b>     | <b>0.15</b>                                      | <b>0.2</b>                             | <b>89.8</b>   | <b>1777</b>              |
| <b>Average</b> | <b>0.04</b>                                      | <b>0.08</b>                            | <b>88.19</b>  | <b>&lt;1150</b>          |

Table 4 Verticality Achieved by Cutter (Shaft 2)

- The application of new SVF method significantly reduces the risk of deviations during excavation.
- Constructing a small diameter diaphragm wall shaft, less than 60 m deep and without rock excavation, can normally be completed in 2 months, see results from Shaft 1 and Shaft 4. If excessive rock excavation is encountered, the time to complete is dramatically longer, see result from shaft 3, therefore excessive rock excavation shall be avoided as much as practicable.
- Watertightness performances are assured if diaphragm wall construction method is used. Noted that the Shaft 2 is 90m deep, the water pressure is extremely high and the concrete milled joint is functioning well.
- With the aid of new SVF technique lately developed, we are confident that future shafts may be constructed even deeper than 90 m.

| Panel          | Axis deviation @ toe from Koden tests (m) | Difference between adjacent panels (m) | Toe depth (m) | Angular Deviation (1 in) |
|----------------|-------------------------------------------|----------------------------------------|---------------|--------------------------|
| 1              | -0.05                                     | 0.05                                   | 38.23         | 765                      |
| 2L             | 0                                         | 0.05                                   | 38.34         | -                        |
| 2M             | 0                                         | 0                                      | 38.38         | -                        |
| 2R             | 0                                         | 0                                      | 38.49         | -                        |
| 3L             | -0.1                                      | 0.1                                    | 41.01         | 410                      |
| 3M             | -0.15                                     | 0.05                                   | 40.97         | 273                      |
| 3R             | -0.15                                     | 0                                      | 44.03         | 294                      |
| 4L             | -0.1                                      | 0.05                                   | 44.1          | 441                      |
| 4M             | -0.05                                     | 0.05                                   | 44.03         | 881                      |
| 4R             | -0.05                                     | 0                                      | 44.01         | 880                      |
| 5L             | 0                                         | 0.05                                   | 42.14         | -                        |
| 5M             | 0                                         | 0                                      | 39.84         | -                        |
| 5R             | 0                                         | 0                                      | 39.87         | -                        |
| 6L             | 0                                         | 0                                      | 38.34         | -                        |
| 6M             | 0                                         | 0                                      | 38.34         | -                        |
| 6R             | 0                                         | 0                                      | 38.3          | -                        |
| <b>Min</b>     | <b>-0.15</b>                              | <b>0</b>                               | <b>38.23</b>  | <b>273</b>               |
| <b>Max</b>     | <b>0</b>                                  | <b>0.1</b>                             | <b>44.1</b>   | <b>881</b>               |
| <b>Average</b> | <b>-0.04</b>                              | <b>0.03</b>                            | <b>40.53</b>  | <b>&lt;563</b>           |

Table 5 Verticality Achieved by Clamshell (Shaft 3)

| Panel          | Axis deviation @ toe from Koden tests (m) | Difference between adjacent panels (m) | Toe depth (m) | Angular Deviation (1 in) |
|----------------|-------------------------------------------|----------------------------------------|---------------|--------------------------|
| 1              | 0                                         | 0.05                                   | 33.91         | -                        |
| 2L             | 0.05                                      | 0.05                                   | 33.78         | 676                      |
| 2M             | 0                                         | 0.05                                   | 33.68         | -                        |
| 2R             | 0                                         | 0                                      | 33.9          | -                        |
| 3L             | 0                                         | 0                                      | 33.83         | -                        |
| 3M             | 0.15                                      | 0.15                                   | 33.9          | 226                      |
| 3R             | 0.1                                       | 0.05                                   | 33.85         | 339                      |
| 4L             | 0                                         | 0.1                                    | 33.67         | -                        |
| 4M             | 0.05                                      | 0.05                                   | 33.85         | 677                      |
| 4R             | 0                                         | 0.05                                   | 33.66         | -                        |
| 5L             | 0                                         | 0                                      | 33.73         | -                        |
| 5M             | 0.05                                      | 0.05                                   | 33.85         | 677                      |
| 5R             | 0                                         | 0.05                                   | 33.65         | -                        |
| 6L             | 0                                         | 0                                      | 33.79         | -                        |
| 6M             | 0                                         | 0                                      | 33.93         | -                        |
| 6R             | 0                                         | 0                                      | 33.89         | -                        |
| 7L             | 0                                         | 0                                      | 33.88         | -                        |
| 7M             | 0                                         | 0                                      | 33.73         | -                        |
| 7R             | 0                                         | 0                                      | 33.67         | -                        |
| 8L             | 0                                         | 0                                      | 33.95         | -                        |
| 8M             | 0                                         | 0                                      | 33.75         | -                        |
| 8R             | -0.05                                     | 0.05                                   | 33.85         | 677                      |
| <b>Min</b>     | <b>-0.05</b>                              | <b>0</b>                               | <b>33.65</b>  | <b>226</b>               |
| <b>Max</b>     | <b>0.15</b>                               | <b>0.15</b>                            | <b>33.95</b>  | <b>677</b>               |
| <b>Average</b> | <b>0.02</b>                               | <b>0.03</b>                            | <b>33.80</b>  | <b>&lt;545</b>           |

Table 6 Verticality Achieved by Clamshell (Shaft 4)

# WATERTIGHT PERMANENT SHOTCRETE LININGS IN UNDERGROUND CONSTRUCTION

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## Abstract

*Only one method has traditionally been available to construct watertight tunnel linings. This method depends on installing a polymer sheet membrane between the initial temporary support and the permanent Cast-In-Place (CIP) concrete lining. The method is slow, expensive and unreliable, presenting difficult technical problems when leakage repair is required. Fully bonded spray-on membrane, sandwiched between layers of shotcrete, offers a favourable alternative. Especially when the permanent support consists of high quality shotcrete, substantial project savings can be achieved. As an example, in a drill and blast project, the overall concrete materials consumption can be reduced by more than 50%. Another important advantage is that if there are damages in the installed membrane, causing spot leakages, these are very quick and easy to repair by point grouting. The reason is that the membrane is fully bonded on both sides, thus preventing water migration along the membrane on either side. The system has been successfully implemented all over the world for more than 10 years. Some selected examples are presented in the paper.*

## Keywords

*Watertight lining, Spray-on membrane, Sandwich structure, Permanent lining, Shotcrete, Cost saving.*

## 1. Introduction

Sprayed concrete, or shotcrete, as is also an accepted term, has become an integral part of initial support for many traditional, non-mechanized (i.e. drill and blast) tunnel excavations. However, even with the substantial increase of shotcrete use over the last decades, there still exists great potential to save both time and money by using shotcrete for both initial and final lining, especially because it can now be made watertight.

## 2. Watertight Tunnel Linings

Waterproofing of underground structures has long been a problem that has challenged even the most seasoned designers and contractors. Shotcrete alone has been presented as a possible solution and also tried a few times, but with very limited success. Even though shotcrete is successfully used in the construction of swimming pools, it falls short of providing a viable

watertight system for use in tunnel linings. There are primarily two reasons for this:

1. Ground water ingress through the rock substrate underlying the shotcrete layer.
2. Shotcrete and any other concrete, even with high cement content, need more than 7 days of undisturbed hydration to create non-continuous capillary pores.

It is reasonable to expect that water flow and migration will find its way through the shotcrete in a much shorter time than seven days and therefore, the channels created will become permanent water conduits. Many projects have focused on preventing shrinkage and other cracking phenomena in the shotcrete to make it waterproof, but that alone will not change the requirement of undisturbed hydration to avoid the otherwise unavoidable leaks.

Furthermore, shotcrete has for many years been viewed as a low-quality concrete product with insufficient means of quality control during placement. This has been an additional argument against the use of shotcrete for permanent linings in tunnels. Shotcrete for waterproofing never became popular for reasons given above.

## 3. Traditional Solution

Traditionally, waterproof tunnel linings are composed of an initial temporary shotcrete layer, followed by a protective sheet of geo-textile, the polymer sheet membrane and a Cast-In-Place (CIP) concrete final lining. Typically this is executed as an “umbrella” covering the tunnel walls and roof and leaving the invert without membrane coverage. This can also be termed a drained solution, since the invert is left open to ground water ingress.

Traditional sheet membranes are watertight, strong, elastic and durable; however, as experience has often shown, they do not guarantee a truly waterproof lining. One problem in particular is that of undetected leakage spots in the membrane (e.g. punch damage during subsequent reinforcement and concreting works) that are hidden behind the CIP concrete. Water is then free to migrate along both sides of the sheet membrane to the outside of the CIP lining, eventually making its way through construction joints, shrinkage cracks, areas of honeycomb or other local defects. Repair of this type of water ingress is difficult due to the problem

of determining the exact location of the leak. It is mostly the case that the source of the leak is far from where the water visually appears on the inside of the lining.

A majority of drill and blast tunnels executed using the traditional way of lining and waterproofing end up being seriously over-supported, a fact that has so far received very little attention. Because of this, a substantially reduced overall thickness of concrete materials would be fully sufficient for safe permanent rock support. Of course, this was not an option when there was no alternative lining method, but that has changed.

#### 4. Shotcrete and Sprayed Membrane Basics

The perception of shotcrete as a low-quality material whose application is difficult to control has been influenced greatly by the way it is typically used. For decades, specifications have called for an initial, temporary (and low-quality) shotcrete support that is not to be considered part of the final, permanent lining. This way of specification and lack of priority and focus on quality, in turn often leads to the supply of less-than-desirable shotcrete. Significant developments in both shotcrete mix design technology and the equipment side have provided the means to place uniform, high-quality shotcrete linings in a fully controlled manner. The key to success is no longer limited by technology. It is now a matter of proper decision-making and specification as well as proper training of operators and control of execution.

To ensure a top-quality lining the following are recommended: Wet-mix shotcrete applied by hydraulic manipulator; use of fiber reinforcement; alkali-free accelerator and accurate accelerator dosing system and water/cementitious-ratio less than 0.42 [MELBYE et al, 2006].

A spray-on waterproofing membrane must also be watertight, elastic and durable and it must bond well to concrete. Membrane today offer bond strength of more than 1.0 MPa on both sides when it has been sprayed onto a shotcrete substrate and shotcrete has been applied against the membrane, creating a monolithic “sandwich” structure. This type of application prevents any water migration along the shotcrete/membrane interface on either side and represents a major advantage if a weak spot in the membrane should occur. The repair of a drip or damp spot can then be completed easily through simple point injection with a very small amount of grout.

One potential limitation of spray-on membranes is that the system needs adequate curing time from spray-applied paste to fully-cured membrane. This is frequently seen as a major obstacle and counter-intuitive to the ground water control problem (i.e. the

substrate must have no active water before the membrane can be placed). To address this concern, simple procedures have been developed in which local drains are installed prior to applying the membrane. Where the substrate shotcrete is humid or wet, it can be sprayed first with a quick-set version of the membrane. Where water penetrates this first layer, a short 10 mm diameter hole can be drilled and a drainpipe is fixed with quick-set mortar. When over spraying with the normal membrane, water is no more interfering with membrane curing. Once the complete waterproofing structure is in place, these temporary drains can be sealed by grout injection.

#### 5. Permanent Shotcrete Lining

The possible use of shotcrete as final and permanent tunnel lining still sparks much debate and opinions are frequently against this solution. However, where permanent lining shotcrete has been successfully used design has been targeting high-quality concrete that will be durable over the design life of the project. As is well known, many of the properties of a finished concrete structure can be indirectly addressed through requiring a high enough compressive strength value. In the case of a permanent shotcrete lining, a minimum of 35 MPa at 28 days should be considered as a reasonable starting point.

The most important benchmarks to ensure durability and uniform quality for any shotcrete mix design are:

- Minimum binder content of 400 kg/m<sup>3</sup>
- Water : cementitious ratio less than 0.42
- Slump (without fibers in the mix) at time of spraying to be between 150 and 200 mm
- The above combination is possible only with the proper use of chemical admixtures selected based upon project-specific requirements (e.g. required open time of the concrete mix)
- Use of so-called “sulfate resistant cements” is not recommended. If sulfates are expected to be a problem, further reduction of water cement ratio has a much stronger positive effect, without the problem of poor reactivity with shotcrete accelerators caused by the low C3A cements. Some micro silica would also be beneficial
- Alkali-free accelerators are highly recommended
- Maximum aggregate particle size of 10 mm or less to ensure minimal rebound (less than 10%) and to create a surface roughness suitable for membrane application

High shotcrete spraying jet velocity is desirable to ensure good compaction (i.e. sufficient air pressure and good nozzle design).

## 6. Properties of Spray-On Membrane

A currently available membrane technology comes in powder form and is applied using dry-mix shotcrete equipment with water added at the nozzle. The membrane is typically applied to a concrete substrate (sprayed or cast) at a thickness of 3.0 - 4.0 mm. It cures from a sticky paste to an elastic, waterproof membrane in a matter of hours. The main technical properties are summarized in Table 1.

| Form                             | Powder                           |
|----------------------------------|----------------------------------|
| Colour                           | Light brown                      |
| Bulk density at 20°C             | 590 g/l $\pm$ 100 g/l            |
| Application thickness            | 3 to 10 mm                       |
| Application temperature          | 5°C to 40°C                      |
| Failure stress (20°C @ 28 days)  | 3.5 MPa                          |
| Failure strain (20°C @ 28 days)  | > 100%                           |
| Bond strength to concrete (28 d) | 1.2 $\pm$ 0.2 MPa                |
| Shore hardness                   | 80 $\pm$ 5                       |
| Flammability                     | Self-extinguishing (DIN 4102-B2) |

Table 1 Membrane Properties

### 6.1 Use of Steel Fiber Reinforcement

Practical experience and tests have demonstrated that steel fibers may be used in both the substrate concrete and in the shotcrete covering the membrane without any damage to the membrane.

### 6.2 Membrane Durability

The durability of the membrane is oftentimes viewed as a concern when this type of waterproofing technology is under consideration. While this attitude is common when dealing with any new technology the basic chemistry of the waterproofing membrane has a nearly 40-year history. Experience with both polymer-modified mortars and concrete is widespread and well-documented. Currently, hundreds of tons of these specialty mortars are being used each year leading to the conclusion that there is little reason to expect any durability problems with the described spray-applied waterproofing membrane in concrete environment.

The chemical stability of the spray-applied waterproofing membrane has also been brought into question. Investigations have been conducted using a number of different chemicals and exposure times. The results of such tests have shown that the membrane is susceptible to compromise only when exposed for long periods of time to strong organic solvents and concentrated acids.

## 7. Comparison of Waterproofing Process Steps

Table 2 presents an overview of waterproofing process carried out using the traditional lining approach as that

uses approach of shotcrete lining with spray-on membrane.

| Traditional Lining with Sheet Membrane                                                        | Shotcrete Lining with Spray-on Membrane                                                                                    |
|-----------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------|
| Initial shotcrete support, typically 50 - 250 mm                                              | Initial shotcrete support, typically 50 - 250 mm                                                                           |
| Sprayed leveling shotcrete (D&B method) to fill in undulations in the substrate               | Where active water flow, drill drain holes and install short pipes using quick-set mortar                                  |
| Affix geo-textile fleece to the substrate to prevent sheet membrane puncture                  | Locally spray fast setting membrane where no need for drainage points, but still risk of drips                             |
| Install sheet membrane; weld and control joints. Special measures needed for any penetrations | Spray final membrane layer. Rate of application: 50 m <sup>2</sup> /h manually and 200 m <sup>2</sup> /h using manipulator |
| Place formwork and cast final concrete lining. Typical <u>theoretical</u> thickness 300 mm    | Spray final shotcrete lining, typically another 50 to 250 mm and grout the drainage points                                 |

Table 2 Waterproofing Process Overview

For projects where the required thickness of the shotcrete lining (combined with systematic rock bolting when needed) is less than 300 mm, the use of a spray-applied membrane offers significant advantages.

In the case of a drill and blast excavation with traditional lining and waterproofing, the following design approach is typical:

- The designer must allow space for the final lining concrete, (say 300 mm) and must add necessary space for the maximum thickness of shotcrete temporary support (say 250 mm). These allowances will define the location of the theoretical blasting line (no protruding rock inside of this line)
- Unavoidable look-out angle for blast holes and overbreak from blasting will typically create an actual rock contour on average 300 - 500 mm outside of the above theoretical line and sometimes more
- The total volume of concrete materials is therefore frequently 1.0 m<sup>3</sup>/m<sup>2</sup> of wall and roof in the tunnel (shotcrete, leveling shotcrete and CIP concrete)

There are many advantages when choosing the shotcrete and spray-applied membrane solution:

- Total consumption of concrete materials can be reduced by 50% to 70% depending upon the project situation
- No need for expensive formwork

- Work execution rate throughout the tunnel is substantially faster leading to lower overall construction time
- Easy adaptation to complicated and changing geometries and if multi-stage face excavation becomes necessary
- Elimination of problems caused by protruding rock bolts or other penetrations through the membrane
- Integrity of the final waterproofing system can be verified by simple visual inspection with weak- spot repair often accomplished in minutes per location

## 8. Project examples

### 8.1 Californian Wine Caves

For several years the Californian wine cave construction industry has been using spray-applied waterproofing membrane against ground water ingress during and after the construction of the caves. Wine caves are generally small in cross section (3m diameter) with horseshoe or circular shape. A common challenge is the oftentimes complex excavation geometries. An initial support layer of mesh and shotcrete is typically covered with spray-applied membrane and followed by a final layer of shotcrete. The overall shotcrete thickness is typically about 100 mm.



Fig. 1 Wine Cave Portal Near Land Owner's Residence

Many of these caves have luxurious hospitality and event facilities under ground that can easily compete with any top class hotel. Water drips on the dinner table is of course unacceptable.

The success of the system is well illustrated by the fact that full roof and wall coverage by spray-on membrane in the wine storage caverns caused them to become too dry. The air humidity dropped so low that the evaporation loss from stored wine increased. The simple solution was to leave the lower part of the cave walls without membrane. Water then migrated through the shotcrete and evaporated to keep air humidity at the right level.



Fig. 2 Wine Cave Entrance after Application of Spray-on Waterproofing Membrane

### 8.2 Nordøy Subsea Road Tunnel

The Nordøy Subsea Road Tunnel reaches a maximum depth of 150 m below sea level, with a minimum rock cover of 40 m and was constructed using pre-excavation grouting [LAMHAUGE et al, 2007]. To prevent the formation of ice and the development of potholes caused by water dripping locally onto the roadway, about 40,000 m<sup>2</sup> of the tunnel roof was waterproofed with the spray-applied membrane system.

The application was done by computer controlled hydraulic manipulator, which ensured a very uniform membrane thickness of 4 mm. See Figure 3.



Fig. 3 Spraying of Intersection with Computer Controlled Equipment – Nordøy Tunnel

Overall output (including drainage points, local spraying of fast-setting membrane, injection, final inspection and weak-spot repair) was over 1,600 m<sup>2</sup>/week. The spray-applied membrane is sandwiched between two 50 mm layers of shotcrete and the completed 100 mm sandwich structure represents the final permanent lining.

### 8.3 Chekka Tunnel

The Chekka Tunnel in Lebanon was constructed in 1971 and has a Cast-In-Place (CIP) concrete lining of about 300 mm thickness. Seasonal rains caused

leakage through construction joints and cracks in the concrete and spray-applied membrane was chosen to solve the problem.

The 3 mm layer of membrane was spray-applied by computerized robot (Fig. 4), covering more than 150 m<sup>2</sup>/hour. Shotcrete with synthetic fibers was sprayed for protection at a thickness of 40 mm, followed by 5 - 10 mm of gunite coating as a cosmetic finish.



Fig. 4 Computer Controlled Spraying – Chekka Tunnel

#### 8.4 Other Projects

During the last more than 10 years, the spray-on membrane waterproofing has been used for a number of different project types and around the globe. A small selection is shown in Table 3.

| Project                                     | m <sup>2</sup> |
|---------------------------------------------|----------------|
| Zapata and Lo Prado road tunnels, Chile     | 20000          |
| Chatswood/Epping rail, 4 stations, Sydney   | 40000          |
| Metro M2 Lausanne, Switzerland              | 5000           |
| Pinalito Power Station, Dominican Republic  | 18000          |
| Road Tunnel de Bracons, Spain               | 43000          |
| Road Tunnel Clem Jones, Brisbane, Australia | 70000          |
| Hindhead Highway Tunnel (A3), UK            | 70000          |

Table 3 Some Other Projects

#### 9. Conclusions

Waterproofing of tunnel linings has until recently been completely dominated by the use of polymer sheet membrane installed between initial shotcrete support and final lining CIP concrete.

By combining the improvements made in wet mix shotcrete technology and the now one decade old spray applied membrane, entirely new solutions are possible. By utilizing high quality shotcrete, the initial support can be made part of the final permanent support and the waterproofing membrane can be sandwiched between the two construction stages. The total product comprises a watertight permanent final lining for the underground opening.

Drill and blast excavation by its very nature creates overbreaks and in most cases, the total thickness of concrete materials with the traditional solution is far beyond what is structurally needed for safe permanent rock support. In such cases the shotcrete solution can save 50% or more of the concrete materials otherwise used. In underground openings with complicated geometry (e.g. metro stations, tunnel intersections and variable size cross-sections), the practical flexibility of final lining shotcrete with embedded spray-applied membrane will offer the most advantage.

In recent years, a series of successfully executed projects using the integrated spray-applied membrane system has confirmed its viability in a variety of applications all around the world, including hydroelectric power facilities, road tunnels and metro projects. When the fully-integrated, spray-applied waterproofing membrane system is suitable for the project, the reduced construction time will generate significant project cost savings far beyond and in addition to what is saved on the actual lining system itself.

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# UNDERGROUND EXPLOSIVES MAGAZINE COMPLEX FOR WEST ISLAND LINE

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## Abstract

*The MTRCL's proposal to extend the existing Island Line westward to Kennedy Town was first conceived as far back as the early 1980s. The technical difficulties associated with constructing an underground railway in such a heavily developed area were always going to be considerable, but the logistics associated with the Project have also proven to be just as challenging. Not least of these challenges is the need to provide explosives to all three major civil construction contracts to facilitate blasting of up to thirty four work faces per day. For this reason a decision was taken early on in the planning stage to construct an explosives magazine on Hong Kong Island to supply the needs of the Project. An extensive study was carried out by MTRCL, their Consultants and the Hong Kong Government to find a suitable location for the magazine, which had to be as close as possible to the work sites to allow for efficient usage, but remote enough from the heavily populated areas of Western District to allay public concerns relating to safety and security.*

*The recommendation from this study was to construct an underground Explosives Magazine Complex below Mount Davis with dedicated explosive storage chambers for each of the three main West Island Line Contractors. Construction of the facility commenced in July 2009, was completed by August 2010 and commissioned in late October 2010.*

*This paper will provide details of the planning process, design constraints and the construction works for this underground explosives magazine complex.*

*Major areas addressed will include:*

- *Selection of the site on the western coastline of Hong Kong Island*
- *Design specifications related to underground explosives storage facilities*
- *Portal entrance design and construction issues*
- *Construction of underground facilities*
- *Fire and safety design and installation works*

## Keywords

West Island Line, Underground, Explosive Storage, Planning, Design, Construction, Fire Safety

## 1. Introduction

The MTRCL's West Island Line (WIL) was initially conceived in the late 1970's and early 1980's as a future extension to the Island Line that serves the northern coastline of Hong Kong Island. The envisaged alignment has been through many iterations due to geological constraints, enhanced environmental awareness and the changing demographics within the Western Districts of Hong Kong Island over the past 30 years. The final alignment adopted was proposed in the West Island Line/South Island Line Feasibility Study produced by MTRCL in March 2004 which was subsequently developed through to the final detailed designs for the West Island Line and approved for construction by the HK Government in the summer of 2009.

The West Island Line extension comprises just over 3.3 km of running tunnels between the existing Sheung Wan Station and the end of the new WIL over-run and refuge siding tunnel beneath Mount Davis. There will be three new underground stations, Sai Ying Pun (SYP), Hong Kong University (HKU) and Kennedy Town (KET) with an extensive network of underground adits leading to fourteen Station Entrances dispersed across the entire Western District to serve the population. Of the three underground Stations, two, SYP and HKU will be constructed within rock caverns while the third will be a cut and cover station. The majority of the running tunnels between the stations, the over-run tunnel and the majority of the entrance adits will also be constructed within rock and it is intended that primarily drill and blast methods will be used for their construction. The explosives to be used for construction will be stored in an underground explosives magazine facility being constructed for the exclusive use of the West Island Line Contractors. This paper will provide details of the planning process, design constraints and the construction works undertaken to complete the Underground Magazine complex in time for the major blasting activities for WIL commencing in the autumn of 2010.

## 2. Background to Selection of Facility

### 2.1 Why Did the West Island Line Need an Explosives Magazine?

The West Island Line is constructed entirely underground and in a heavily built-up urban area. The majority of the works are constructed by drill and blast construction. The volume of rock being excavated is in excess of 500,000 m<sup>3</sup> comprising of both granites and volcanic tuff.

The volume of rock to be removed does not in itself necessitate an explosives magazine to be provided for the WIL construction. More significant was the daily explosives deliveries demands placed upon the construction works due to the complex layout of the underground stations and hence the number of multiple workfronts operating at the same time. As can be seen in Figure 1, the future WIL Stations at Sai Ying Pun (SYP) and Hong Kong University (HKU) incorporate a labyrinth of construction, ventilation and passenger adits, in addition to two large underground station caverns, nine deep shafts with significant portions of rock excavation and twin running tunnels between the three underground stations, the existing Sheung Wan Station and the over run/refuge siding tunnel beyond Kennedy Town Station.

From a very early stage in the design process it was recognised that the key to successfully implementing the construction works within the shortest possible period of time would be the ability to simultaneously open up as many underground workfaces as possible, in order to minimise linearity within the programme. This would require maximising the number of delivery points for explosives across the Project and the frequency of deliveries to these points.

During the planning stage of the Project nine locations for explosives deliveries were identified to maximise blasting production and minimise the construction programme (this was subsequently reduced to five locations during the detailed design stage). With two deliveries per day needed at each of these locations to serve up to seventeen blasting faces, and at delivery times to suit mucking out schedules, it became apparent that a guaranteed supply of explosives would be essential.

The Civil Engineering and Development Department (CEDD) Mines Division control all explosives storage

and movements within Hong Kong. From their main explosives storage facility at Kau Shat Wan on Lantau Island, explosives are brought to Hong Kong Island once per day for delivery to the various work sites where drill and blast activities are being undertaken. From discussions with Mines Division it was clear that WIL demands of up to nine delivery points requiring twice daily explosives deliveries could not be entertained, therefore MTRCL would have to construct its own explosives storage magazine.

## 2.2 Why an Underground Explosives Magazine?

Having concluded that an explosives magazine was essential for the success of the WIL Project, attention turned to its size and location. Preliminary design [MCAL, 2007] estimates indicated that demand for explosives would vary greatly, but that at peak demand when all three main Contractors were carrying out blasting activities along the full length of the Project, a daily requirement for explosives approaching 2000 kg would not be unrealistic. This demand would vary greatly and would inevitably be further refined depending on the final construction programme, selection of explosives (use of bulk emulsions could have a significant impact) and the individual blast face designs. The magazine would need to be sized to accommodate a number of variables in the supply of explosives and detonators.

As mentioned above, the Mines Division's primary explosives storage facility is located on Lantau Island. Deliveries by boat can be affected by extreme weather and adverse sea conditions, during the Typhoon Season when in particular a T3 Strong Wind Signal or higher stops all deliveries [GEO, 2009]. This is a frequent occurrence within Hong Kong, with the signal often staying in place for a duration of two to three days. It was therefore considered prudent that the WIL explosives magazine be designed for a 5 day storage capacity to ensure that the progress of the works would not be affected by lack of explosives (this was subsequently reduced to 2 days, following discussions with Mines Division, due to security concerns) [GNJV, 2010].

The 4000 kg of explosives required for 2 days of

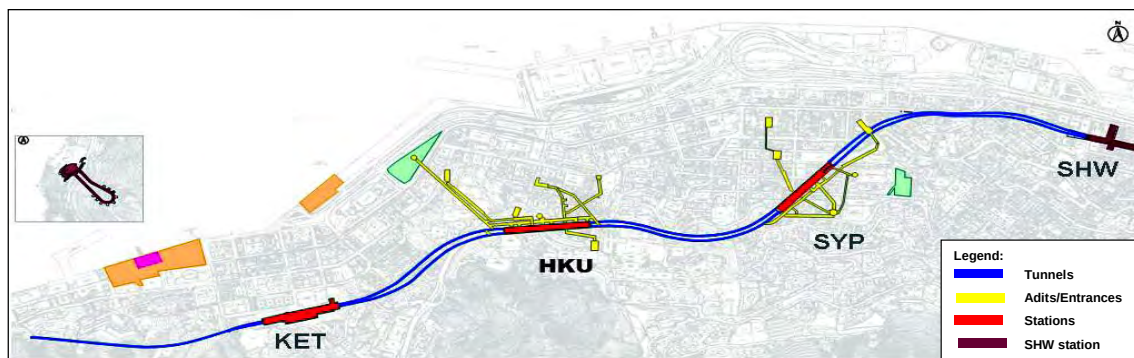


Fig. 1 Alignment of WIL

blasting activities is a very large quantity of explosives. It was apparent that such a large storage facility would be difficult to accommodate within the heavily populated areas close to the WIL works sites. Initially, it was assumed that either a conventional above ground facility or underground storage incorporated into the proposed running tunnel construction sites would be required. A desktop study was undertaken to find a suitable location that could satisfy the known constraints.

Hong Kong Government regulations dictated that for security reasons, each Contractor on WIL would require a separate Mode A Explosive Store within any future magazine complex [HKSARG, 1997c]. Detonators would need to be stored separately from explosives, and again each Contractor would require their own detonator store. Separation distances between above ground stores and various groups of sensitive receivers were dependent on design criteria set out in the, "UK Manufacture and Storage of Explosives Regulations 2005", as required by Mines Division guidelines for Mode A Explosives Stores, and on the net explosive quantity (NEQ) of explosives held in each store. Blast zones around above ground explosives storage facilities are dictated by the NEQ of explosives stored within the footprint of the facility. Within these zones strict criteria were imposed by the HK Government on numbers of residences permitted to be affected in the event of an accidental detonation [HKSARG, 1997a, b, c, d]. Existing features such as slopes, retaining walls, water retaining structures, electrical and communication facilities all had to be considered when selecting a suitable above ground location. Many of these would have to be upgraded or relocated if they fell within the blast exclusion zone.

Figure 2 shows the six most promising locations that were identified during the desktop study for further detailed review, while Figure 3 indicates the blast exclusion zones developed for the potential above ground explosive magazine sites. The location identified at Green Island was quickly discounted as it did not provide any security of delivery during a T3 Typhoon event. The two underground facilities proposed were studied extensively and, technically, did satisfy all of the safety requirements for storage of explosives. However, they were rejected as their proximity to residential areas of Kennedy Town presented significant security and political issues with respect to acceptance by the local community. The remaining locations were visited to evaluate their suitability.

It was during this further review that the idea of putting the magazine below ground at a location remote from the proposed works was first considered. During a visit to the above ground sites south of Victoria Road, the design team came across an old World War 2 military tunnel.

On further investigation it was established that this tunnel had in fact been used as an ammunitions and explosives magazine. Reuse of the facility was briefly considered but discounted due to its generally poor condition and close proximity to an existing Government Building directly above it, however the possibility of constructing a similar facility nearby, at one of the above ground sites identified in a previous study seemed feasible. The advantages of such a facility were:

- The facility would be close to the work sites
- It could be made as large or as small as required by simply lengthening the tunnel, adding more flexibility in design to suit the Contractors storage requirements
- It would be more secure than an above ground facility
- It would not require a large above ground footprint, and thereby reduce environmental impacts
- It would not require the large exclusion zone for residential buildings required for the blast zone of an above ground facility
- It was far less likely to attract opposition from the public, district councils, local residents or government departments.
- Being located deep underground it would be safer in the unlikely event of an accident.

The major drawback was that it would be considerably more expensive and would take longer to construct. However, as the benefits clearly outweighed this drawback, all efforts were channelled into developing a detailed design for an underground facility south of Victoria Road and gaining the necessary government approvals and community acceptance for the scheme.

### 3. Design of Facility

#### 3.1 Government & Stakeholder Engagement (Buy-In)

Use of explosives in an urban environment is a very emotive issue. The general public, and many government departments and other stakeholders were understandably very nervous about allowing what they perceived to be a potentially dangerous facility to be built within their backyard. They had to be convinced not only of the need for such a facility, and that it would be safe, but also that it would only be temporary and used only for the construction works on WIL.

From the outset, MTRCL engaged the local community and kept them fully informed on the proposals for the explosives magazine. District Council (DC) representatives were briefed on the various locations studied including the proposed underground site beneath Mount Davis. All of the alternatives were presented to concerned stakeholders by MTRCL throughout Western District. Briefings were given to government departments involved in the WIL approval

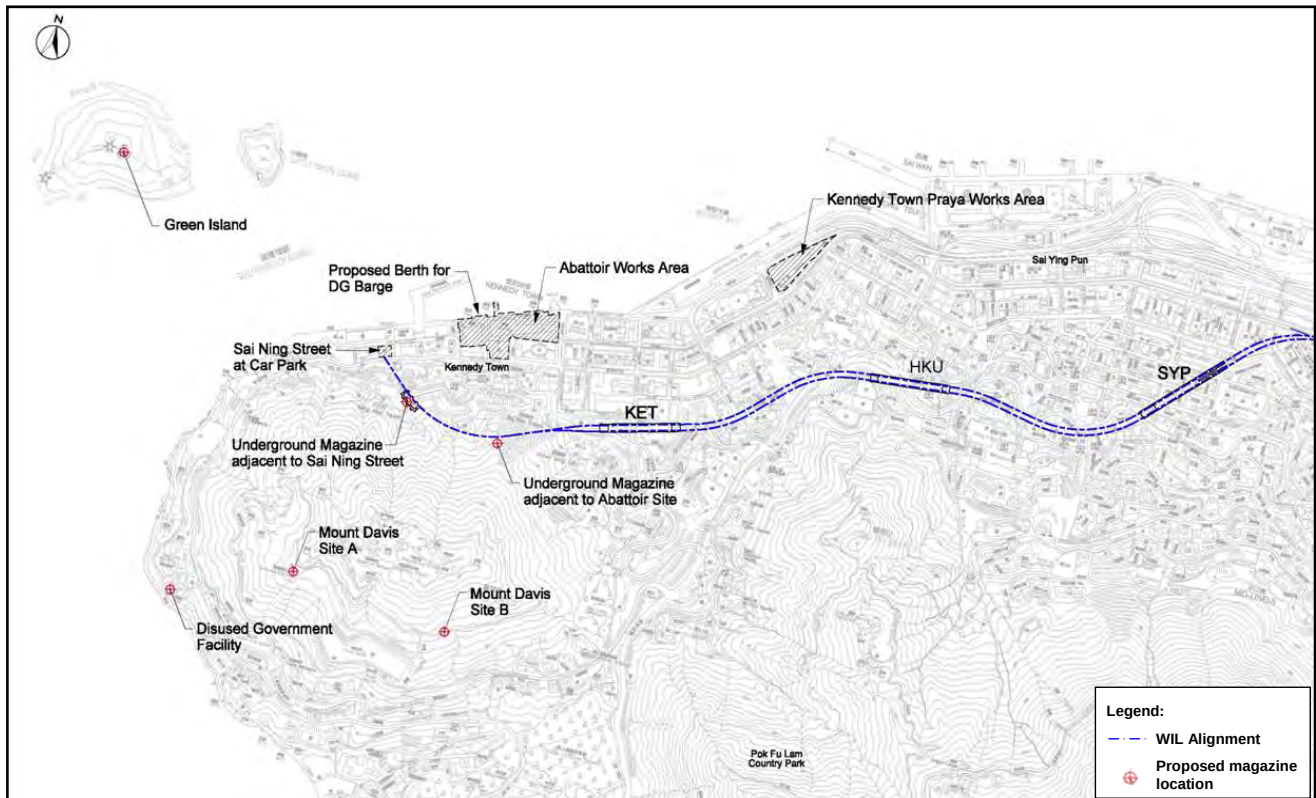


Fig. 2 Possible Magazine Sites

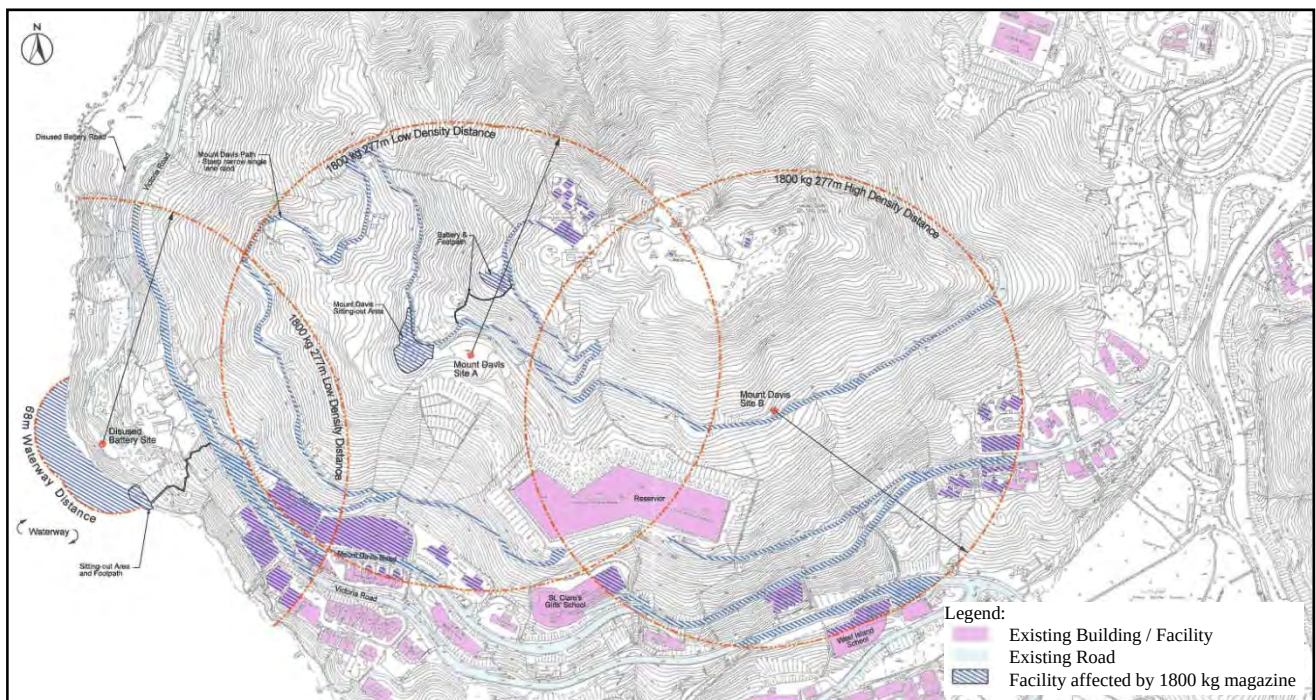


Fig. 3 Blast Zones Around Above Ground Magazine Sites

process. One-on-one presentations were held with the Hong Kong Police Force (HKPF), Fire Services Department (FSD), Environmental Protection Department (EPD), Lands Department (LD), Planning Department (PD), Civil Engineering and Development Department (CEDD), CEDD (Mines Division), CEDD

(Geotechnical Engineering Office) and the Transport and Housing Bureau (THB), who had to be confident that the facility was both safe and viable. In all of these discussions the Railway Development Office (RDO) played an active role in driving the proposals through and ensuring that the various concerns of the

government department were met. From these discussions the general layout of the facility was developed [MTRC, 2008d].

By far the most difficult and important obstacle to overcome, was ensuring that the requirements of the EPD and the Mines Division, with respect to the impact on the environment and of the safety of the public within the vicinity of the Magazine facility respectively, could be achieved.

A comprehensive Environmental Impact Assessment (EIA) was undertaken around the portal area of the magazine to investigate any potential impacts on flora and fauna in the area. While this did not identify any rare species or specimens that required special protective measures, it did identify several trees which were of significant value. To protect these trees the design was revised several times with the tunnel portals and support facilities being relocated to avoid them.

Of great significance in the design of the facility was the Quantitative Risk Assessment (QRA) carried out to assess the safety of overnight storage of explosives in the Magazine [MTRC, 2008a, b]. This was insisted upon by EPD as part of a wider obligation to carry out a QRA on the use, transportation and overnight storage of all explosives for the WIL Project. The study, which was carried out over an 18 month period, was based on a risk/probability/consequence analysis of the handling and storage of explosives within the magazine complex. It covered risk to life from any blast occurring either within the storage chambers or when being transported within the facility. It included potential impacts on boating offshore from the facility, vehicles and pedestrians on Victoria Road above the facility, and residents living in the buildings along Victoria Road. It has to be stressed that the risks that were identified were minuscule, as illustrated in the FN Curve shown in Figure 4. FN Curves show the predicted cumulative frequency of accidents (F) versus the predicted number of lives (N) that would be lost as a result of such an accident. The curve produced as part of the QRA indicated that the likely frequency of an incident occurring at the magazine facility within any one year period that resulted in loss of life was in the order of less than  $1 \times 10^{-7}$  i.e. a 1 in 10 million chance. However, this level of confidence was essential in persuading not just EPD but also the public at large that an underground magazine complex would be safe. The study also clearly illustrated that risk to life associated with the underground cavern design was far less than for an above ground facility.

An undertaking was given to the Government and the DC that the facility would not be used by anyone other than WIL as an explosives magazine, and that the facility would be backfilled and the site returned to its original condition prior to handing it back to

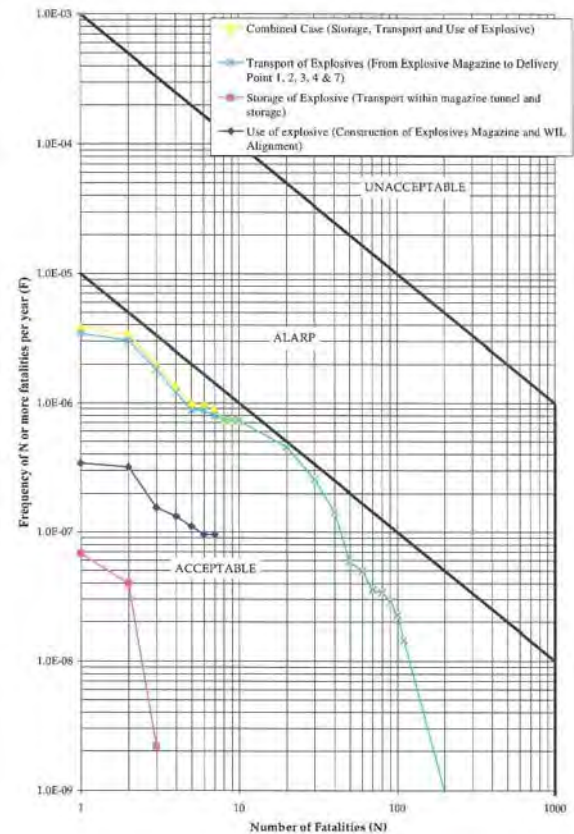


Fig. 4 FN Curve for Accidental Explosion at the Magazine

Government on completion of the WIL Project. Subsequently, both Government and the DC have been reconsidering whether or not the facility can be put to the beneficial use of the community rather than filling it back in, and it is hoped that this can be progressed further by Government before WIL is completed.

### 3.2 Design of the Underground Explosive Magazine

#### (1) Site Geology and Ground Conditions

The preliminary design for the underground magazine was undertaken by the preliminary design consultant [MCAL, 2007] in parallel with the stakeholder engagement process before being taken through to detailed design by another consultant [MTRC, 2008c], see figures 5 & 6. This comprised 320 m of 5.5 m diameter horseshoe shaped access tunnel constructed primarily within rock, but with approximately 80m of soft ground tunnel construction at both the north and south tunnel portals. Explosives storage chambers linked to the access tunnel, associated slopeworks/retaining walls, earthworks, utilities diversions and all the usual support works that accompany tunnel portal construction were also included in the final design. Available GI was sparse as the dense vegetation and difficulties of accessing the steep slopes (up to 35 degree angle of repose) above the tunnel alignment had prevented detailed investigation during feasibility and preliminary design stages (at detailed design stage this was obtained by

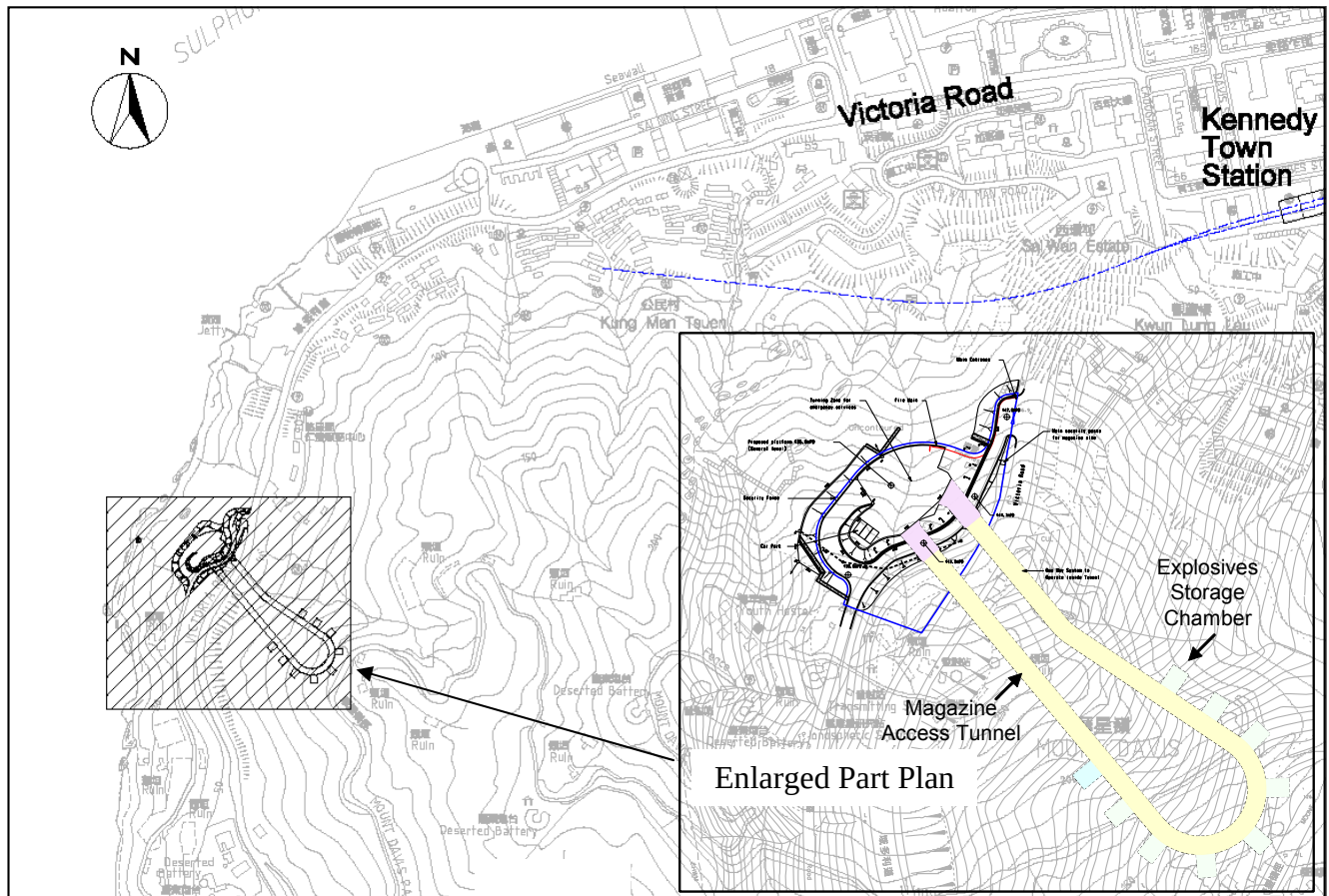


Fig. 5 Underground Magazine Site Location South of Victoria Road



Fig. 6 Oblique Aerial View of Underground Magazine Site South of Victoria Road

using helicopters to lift in Ground Investigation (GI) drilling rigs onto the hillsides). One horizontal borehole was drilled within the profile of the eastern tunnel drive.

From available geotechnical data it was established that the predominant rock geology consisted of volcanic tuff of the Mount Davis Formation which is dominated by coarse ash crystal tuff with intercalated tuffaceous siltstone and sandstone. Overlying the rock was predicted to be layers of colluvium or residual soil of thickness varying from 1.5 m to 3.5 m which in turn overlay a layer of completely to highly decomposed

tuff. The Rock Quality Designation RQD, a measure of the degree of jointing or fracture in a rock mass) of the bedrock below was predicted to be between 70 to 100, with ground water levels assumed to be approximately 1m above bedrock level. A perched water table was also likely. Figure 7 indicates the Geological Map of the Site.



Fig. 7 Geological Map of the Site

At the portal location a small boulder field was located along the alignment of a previous watercourse that had been diverted by culverts in recent years beneath Victoria Road. The platform area in front of the tunnel portals was formally used as a squatter settlement and consisted of a series of man-made terraces constructed using uncontrolled fill materials retained by a mixture of both earth berms and crudely built concrete and masonry retaining structures. Several small dwellings were also present as were suspended concrete slabs with large voids beneath them. The underlying ground beneath these terraces dipped steeply to the coastline and had similar characteristics to the rock geology described above. All of the uncontrolled fill materials within the site and the associated retaining walls and structures had to be removed to form a platform area in front of the future explosives magazine facility.

## (2) Design Parameters and Specifications

To store the explosives and detonators, nine chambers 5 m wide x 8.5 m deep x 4.5 m high were designed offset from the main access tunnel. As well as being designed for overburden and hydrostatic loads, these chambers also had to be designed to withstand an uncontrolled explosion of the explosives stored within them, this was the main criteria which drove the design of the overall facility.

Designing an underground explosives magazine is not a common occurrence. There was very little guidance or detailed specification documentation available to guide the preliminary design consultant in producing a safe and robust design. The existing facility used by Mines Division on Kau Shat Wan was designed as a “one off” by a Specialist Consultant working for the then British Administration in Hong Kong prior to 1997. There was no available detailed specification for the facility, so although the design produced was undoubtedly very robust, there was no way for design consultant to use it as a template for their design specification. Instead, a search was conducted worldwide in an attempt to obtain appropriate design specifications for any underground magazines in existence. Specifications were obtained from Norway, Australia, UK, USA and France. Of these, the most appropriate were considered to be those obtained from the US Department of Defense and NATO. In both instances, these specifications provided detailed and practical information on design specifications for accidental blasts and how to calculate the impact and influence zones of such events.

Uncontrolled blasts of the magnitude possible within the facility were THE defining design criteria for the facility. From the US Department of Defense (DoD) 6055.09-STD “DoD Ammunition And Explosives Safety Standards” [DOD, 2008] design specifications were adopted to ensure that such a blast would not have a destabilising effect on the existing slopes above the underground facility and to ensure that there was

adequate rock cover to prevent a collapse within the explosives storage chamber, which would result in surface impacts. The slopes above the magazine facility are mainly natural slopes of varying angles of repose, varying depths of overburden and contain numerous boulders. The accepted approach for the design of a rock tunnel located beneath such topographic constraints would most probably be dictated by the vibration impacts imposed during drill and blast construction. However, these were relatively small in comparison to the potential impact on the slopes due to an uncontrolled blast in a chamber fully stocked with up to 300 kg of explosives.

The detailed design of the chambers is not the subject of this paper, therefore we will not go into detail here on this aspect of the facility. However, it is important to appreciate that the uncontrolled explosion of a large quantity of explosives stored within a chamber is quite different to that of a delay-interval controlled initiation of explosives within a charged face of a rock tunnel. The release of energy from such an explosion envelopes the void between the point of explosion and the chamber walls with a massive force, and this is then transmitted through the rock mass itself. In addition, the instantaneous air over pressure (AOP) is massive and large enough so as to render the design of effective blast doors as redundant. It is in fact preferable not to have blast doors at all, but rather to allow this AOP to escape out of the chamber and into the access tunnel.

Again, the predominant design specification guidance came from DOD6055.09-STD. The impact of such an explosion can cause sympathetic detonation of explosives within adjacent chambers, not to mention collapse of rock pillars between them or the roof of the chamber. Analyses have shown that the required separation distances of chambers can be calculated based on rock properties and explosives quantities. Previous studies have established that sympathetic detonation of explosives in adjacent chambers could only be initiated by impact from rock spalling caused by shock wave transmission through the rock mass, rather than any direct propagation by blast pressure waves or thermal effects. Separation distances required to prevent this effect in sound rock are derived from the DOD6055.09-STD (Eq. C5.2-4) reproduced below:

$$D_{cd} = 0.99 Q^{1/3} \quad (1)$$

$D_{cd}$  = Required Separation Distance (m)  
 $Q$  = Charge mass (kg)

Based on Eqn. (1) a safe separation distance of 6.63m was derived. For safety purposes a minimum separation of 11 m was adopted between chambers.

With respect to the potential impact on the stability of the roof of the chamber in the event of an uncontrolled blast, DOD6055.09-STD defines the critical chamber

roof thickness to prevent collapse in terms of the equation:

$$C_c = 1.0 Q^{1/3} \quad (2)$$

$C_c$  = Critical depth above the chamber (m)  
 $Q$  = Charge mass (kg)

Based on Eqn. (2) the safe ground cover depth above a chamber containing 300 kg NEQ was estimated to be 6.7 m. It should be noted that the rockhead cover at the explosives storage chambers was significantly more than this value with the design controlling event being where the access tunnels passed beneath Victoria road at a depth of just over 7 m to the road surface.

To deal with the shotgun effect of the AOP rushing down the access tunnel and out of the portals, blast barriers were placed at a clear distance of 6 m from the portal face with absorbent rubber matting and an angled profile to direct any shock wave upwards and out of harms way.

#### 4. Construction Phase

##### 4.1 Risks

It was recognized at Tender award that not only did the construction programme present a significant challenge, but that program and project delivery risks, if they eventuated, would place additional pressure on the program, and more crucially, onto the WIL project. The registered risks that transpired throughout the course of the works included:

- heavily vegetated and steep terrain above Victoria Road resulting in greater than anticipated extent of boulder surveys, affecting blasting commencement and;
- un-surveyed trees, resulting in re-designing site formation and;
- un-identified location and condition of existing structures, with poor existing masonry walls and walls with significant cavities resulting in re-designing site formation support and backfilling and;
- changes to/additional FSD requirements for CCTV supply. This latter risk did not have a direct program impact, but required additional resource.

Several unidentified and unexpected risks arose during construction, and when these eventuated, presented significant challenges, as follows:

- early afternoon explosives delivery times placing constraints on mucking out cycles and;
- a significant boulder field at and above the eastern portal resulting in an extended pipe-pile installation period, TAM (tube-a-manchette) grouting period, excavation period, tie-back drilling and grouting, and soft ground grouting volumes and;

- meeting FSD requirements for ventilation airflow, resulting from variations to design-stage overbreak, lining and support assumptions and;
- meeting FSD requirements for fire hydrant water supply pressures, with original assumptions based on WSD flow, but not pressure commitments.

This latter risk presented one of the most significant challenges on the Project and was overcome by rapid provision of a separate emergency water pipe with a take-off point located 500m from the site.

Grouting volumes in the first round of 12 m long, 89mm dia. double layer canopy tubes was significant, at almost 12,000 litres, increasing the pre-excavation works period prior to portal excavation.

One risk that did not transpire was the pre-tender identification of a possible 20m long weak-ground zone in the northern tunnel identified at pre-tender stage by the horizontal site-investigation drillhole mentioned above.

##### 4.2 Construction Challenges

###### (1) Portal Entrance - Design and Construction

The steep terrain, heavy vegetation and alignment and condition of existing structures presented difficulties in site survey throughout the course of the Works. As discussed in the section on risks above, constraints in pipe pile installation works forming the main entrance structures, grouting of tie-backs and soil nails, bulk excavation and temporary support on steep terrain, were overcome by careful selection of small but manoeuvrable construction plant, and recognition of the need for flexibility in sizing of support members.

With access into site off the busy Victoria Road, the main artery to Cyberport to the Western side of Hong Kong Island, traffic management was crucial to ensure continued material supply and muck-out, without undue traffic disruption.

Retaining wall design modifications due to topography, tree locations of trees not designated for felling and exposed ground conditions was required for at least 4 of the 5 retaining walls comprising portal-area platform support.

A key element in procuring community support, and indeed avoiding environmental concerns was placing high value on tree-preservation. This manifested itself in minimal felling, tree-transplantation and significant site formation retaining wall and slope design changes to avoid further tree-felling. Despite the large rubber-tyre-covered blast walls, the resultant facility remains surrounded by trees, and being below the road, is unobtrusive to all but the most inquisitive passers-by.

## (2) Construction of Underground Facilities

The tunnel blasting phase of the Project, once underway, was relatively straightforward, with risk mitigations put in place for both soft and hard rock drives.

Soft ground tunnelling (total 2,300 m<sup>3</sup>) comprised conventional 12m long canopy-tube support, initially in two layers and overlapping by 4m and perimeter grouting (Fig. 8). The face was advanced using mechanical excavation, by heading and bench methods, across Victoria Rd, until sufficient rock cover permitted safe blasting. Maximum settlements of 20mm remained within limits for an existing retaining wall located between the two tunnels.

In the hard rock section (total 8,700 m<sup>3</sup>), blasting was conducted once per day, on up to 3 headings, using primarily cartridge explosives and NONEL (non-electric) detonators delivered directly to site by the Mines Division. Small quantities of bulk emulsion product were used in the later stage of tunnelling.

Due largely to depth below rockhead, the overall rock quality was generally very good, with a total of only 321 total rock bolts required over the entire main tunnel length, less than a bolt per linear meter of tunnel excavated.

For blasting works:

- peak particle velocities (PPV) values for the 112 blasts were all below predictions, and none exceeded 10 mm/s. All AOP values were below 120 dB. No blasting complaints were received.
- most faces required 70 blastholes per round, and average “pulls” were 3~4 m, with the greater values in the west tunnel.
- Powder factors were grouped in two ranges; 1.5 for the majority of the tunnels, with higher values of 2.5 closer to the portals.
- 35,000 man-hours/month were worked at the peak, with zero reportable accidents over the entire project.

The U-shaped horizontal tunnel profile required a tight radius of curvature at the end of the tunnel deepest under Mount Davis, which presented challenges in maintaining sufficiently long rounds without resulting in excessive overbreak due to drilling for a curved drive, with straight drill rods.

As the facility design life is short (just 5 years) regulatory bodies permitted use of shotcrete-only support in the rock tunnel sections as both the primary support and secondary support, adopting NGI Q-system support requirements. While having significant program advantages, this impacted as-built profiles, challenging size and smoothness assumptions for ventilation flow requirements.

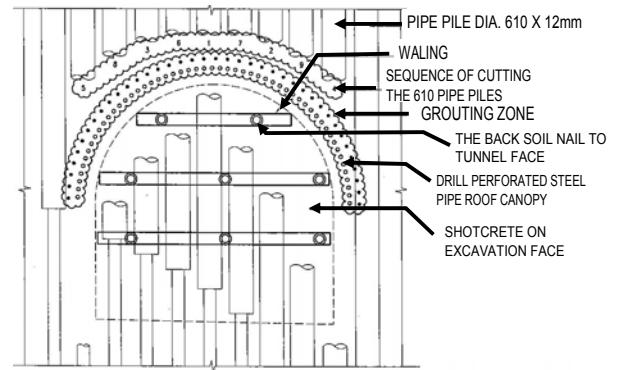


Fig. 8 Soft Ground Tunnel Support at Portals

Issues that placed pressure on the program included:

- explosives delivery times due to distances from Kau Shat Wan
- difficulties in establishing boulder survey extent
- un-surveyed trees
- un-identified existing structures.

To meet the programme, a number of delay recovery measures were proposed, and implemented. These included:

- reducing the two original “u-shaped” pipe-pile portal entrance walls to one by introducing a soil-nailed slope as part of the works. This significantly reduced piling time, and added only a short section of soft ground tunnel, with overall programme advantages.
- adopting a spray-applied membrane for the upper roof sections of the soft ground sections of the tunnel and in the explosives niches. This permitted straight-sided forms to be used to simplify and speed up this section of works, while simultaneously allowing traffic access for mucking out.

Early liaison with the Transport Department, Police and FSD was also planned in advance to ensure construction phase safety, traffic and security provisions were agreed and in place immediately prior to commissioning.

Fitting out of the explosives niches was controlled to ensure as-constructed security, storage and safety provisions were acceptable by Mines, FSD and the HKPF. Separate Mode A store applications was made for the group of niches assigned to each of the 3 WIL Contractors.

Pre-tender magazine operation manuals, combined water spray and sprinkler systems with controls at each niche, robust 3-key gates all assisted in ensuring the approval process went smoothly following completion of the physical elements of the facility.

### 4.3 The Importance of Community Acceptance during and after Construction

With the community liaison preparation work laid out during the design and pre-tender periods, the majority of the groundwork for community acceptance was well underway and following on from this groundwork involved implementing provisions identified to do so. Liaison works continued throughout the construction phase, resulting in enquiries numbering only 3, with no complaints or queries related to:

- trees or
- magazine re-use or
- spoil disposal routes

Experienced MTRCL senior inspector input for blasting works ensured that residents of a nearby old peoples' home were more concerned about pre-blasting sirens that reminded them of World War 2, rather than concerns over vibration. The MTRCL's site-signage programme throughout WIL ensured all passers-by were informed of the purpose of the site, ensuring complete transparency.

Post-construction, a number of queries were received which pertained to post-WIL Magazine usage, for which there was (and remains) a keen interest. Current requirements are to backfill the facility and reinstate to original condition, however District Council, Government Departments, and private interest groups have all expressed interests ranging from wine storage facilities to car parks to crematoriums.

### 5. Discussion and Conclusions

The WIL Explosives Magazine facility is a practical example of how underground space planning can be incorporated into large civil infrastructure projects to minimise the use of valuable ground-level sites within the very congested environment of Hong Kong. The planning of the Project required many Government Departments and external stakeholders to work in a collaborative and proactive manner to overcome the many technical, environmental, security, safety, land utilization and political issues that surrounded what could have been a very emotive Project involving the storage of large quantities of explosives close to a densely populated urban environment.

The use of design specifications not normally adopted in Hong Kong and the commissioning of a Quantitative Risk Assessment on use and storage of explosives for the WIL, which was far more detailed and comprehensive than any previous studies carried out in Hong Kong, were key to ensuring that the underground explosives magazine facility could be constructed and operated safely and to the satisfaction of all stakeholders including the general public.

From concept to completion the Project took almost four years, which given that the construction phase itself only lasted for fourteen months, would appear to be excessive. However, it was important to ensure that for such a sensitive Project all issues and options were fully explored and that the final product was as safe and as secure as possible. The benefit to the WIL Project blasting duration is estimated to be a time saving well in excess of one year which translates into a similar time saving for the Project as a whole, allowing the WIL to open in the summer of 2014.

Looking forward, the discussion will now shift to the long term use of the facility. The access tunnels and storage chambers were designed without permanent concrete linings and for a design life of just 5 years. In reality the facility will be structurally secure for well in excess of the design life, however, if it is to be used beyond that period a review of the current design will be required and a permanent lining may be needed to satisfy any future use design criteria. Several Government Departments have already expressed an interest in the long term use of the facility for purposes other than that of an Explosives Magazine. However, time is short. MTRCL will commence backfilling of the magazine in 2013 in preparation for handing it back to the Hong Kong Government in 2014. The same spirit of collaboration and proactivity will be required if the facility is to be retained for the benefit of community in Hong Kong.

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# CONSTRUCTION AND OPERATION OF REFUSE TRANSFER STATION IN MAN-MADE CAVERN ISLAND WEST TRANSFER STATION

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## Abstract

*In early 1980's, the scarcity of land with sky-high market values in Hong Kong had lead to the Government in undertaking Studies on Potential Uses of Underground space (SPUN), which followed by further feasibility studies in exploring the strategies on the usage of underground space for infrastructure. In 1991, the Government identified a suitable location to build the Island West Transfer Station (IWTS) in a cavern complex inside the mountain of Mount Davis at Kennedy Town, with waterfront available for marine transportation.*

*With innovative and cost effective approach which substantially reduced the volume of excavation and the subsequent construction and operation costs, a saving of capital cost of HK\$600M as compared with the tender budget was achieved. The construction of IWTS commenced in 1995 and it was commissioned in May 1997.*

*During the design and construction phases, close liaisons with governmental departments were undertaken with designs complied with the statutory, contractual and engineering requirements, which were more demanding than normal aboveground building construction, especially on fire safety, ventilation, means of escape, environmental controls, and geotechnical engineering matters. With 40,000 m<sup>3</sup> of rock excavated by drilling and blasting, a man-made cavern was formed with a floor area of 3,500 m<sup>2</sup> available for construction of IWTS.*

*The IWTS is capable of handling 1,000 tons of municipal waste collected daily from the Central, Western and Southern Districts of Hong Kong Island, which will be compacted, containerised and transferred to strategic landfill, the West New Territories (WENT) Landfill for final disposal.*

*With its opening since 1997, IWTS has served the community not just as a refuse transfer station, setting an exceptional example in the usage, engineering and construction of underground/cavern complex, but also acts as environment educational centre for the public and other stakeholders for the appreciation and awareness of waste management.*

## Keywords

*Underground Space, Cavern Complex, Waste Disposal Plan, Refuse Transfer Station, Strategic Landfill*

## 1. Introduction

As an integral parts of the Waste Disposal Plan set out by the Environmental Protection Department (EPD), a refuse transfer in western side of Hong Kong Island was required to serve the community in the Central and Western Districts. However, the scarcity of land with sky-high market values at that time and the requirement of a waterfront site for marine reception at the Western district have imposed tremendous difficulties in identifying a suitable location for this refuse transfer station. In the late 1980's the Government undertook Studies [ARUP, 1989 & 1991] on Potential Uses of Underground space (SPUN) managed by the Geotechnical Engineering Office, and followed by further study for two specific cavern projects (CAPRO). It was identified that potential use for underground treatment and disposal was feasible and the Government decided in 1991 to adopt the innovative concepts to build the Island West Transfer Station in an underground cavern under Mount Davis.

SITA Waste Services Limited was commissioned by the Environmental Protection Department (EPD) in 1995 under a contract to Design, Build and Operate the Island West Transfer Station, the Hong Kong's first underground waste transfer station inside a man-made cavern [SITA, 1995].

## 2. Design

### 2.1 Design Philosophies and Principles

With the experiences in design, build and operate three waste transfer stations in Hong Kong, namely Kowloon Bay (KBTS), Island East (IETS) and Sha Tin (STTS) Transfer Station, the authors committed on the development of knowledge and sharing other experience of evolving transfer station technology in their international operations, that optimal solutions were explored and developed in the design of the Island West waste transfer system.

Enhanced compactors design with refuse container for the compaction and transferred of waste were selected at IWTS, while the marine transportation and landfill

discharge systems were similar to those at IETS. The design of IWTS incorporated the essential environmental and operational requirements of any effective transfer station. The design philosophy was centred upon site activities or processes likely to cause environmental impacts within the cavern development, while maximising the use of the available external areas.

### 2.2 Detailed Environmental Impact Assessment

An Initial Environmental Impact Assessment (IEIA) was prepared as part of the Island West Transfer Station (IWTS) by EPD's consultants, and identified key environmental issues arising from the project and provided an initial assessment and evaluation of each of the key environmental impacts for making recommendations that, would influence the outline design of the facility.

A Detailed Environmental Impact Assessment (DEIA) for the Facility was prepared and took into account of all aspects of the Design, the Work and the Operation of the Facility and addressed the issues identified in the IEIA.

#### (1) General Approach

The general approach for completing the DEIA was to address the key issues identified in the IEIA during the tender design phase. The necessary iterative process between design and environmental performance requirements were undertaken at an early stage thus avoiding unnecessary delays during the detailed design.

#### (2) Existing Environment

A review of the existing environment was undertaken covering new developments in the area and an update on the status of proposed adjacent infrastructure projects.

#### (3) Construction Phase - Construction Activities

All construction activities including the programme of works and construction methodology were reviewed. Any significant changes to the initial design proposals were described and assessed for their potential environmental impact.

The main construction related impacts were noise and vibrations related to excavation, blasting, haul road traffic and access ramp construction and dust arising from the same activities. Site runoffs were being controlled through implementation of erosion control measures and installation of effective silt/grease traps. Marine impacts were minimal and the only marine works carried out was dredging of 200 to 300 m<sup>3</sup> of silt and boulders for marine access to the berthing area.

#### (4) Operation Phase

Impacts during the operation of the facility under normal, abnormal, transient and emergency conditions

were assessed and include the following:

- Noise
- Dust and Odour
- Effluent and Drainage Disposal
- Marine Transport
- Visual Impacts
- Socio-economic Impact

As part of the DEIA, an assessment was undertaken of the negative effects perceived by people likely to be affected by the project. Public consultation was undertaken with a public relations programme designed to provide project information while at the same time listen to the concerns of the public. As a result of consultations, recommendations were made for project refinements where appropriate.

### 2.3 Transfer Station Design

The station layout was designed to permit the transfer of waste from the Waste Collection Vehicles (WCVs) to the containers quickly, safely and with the minimum environmental impact. The following is an overview of the general station function and layout.

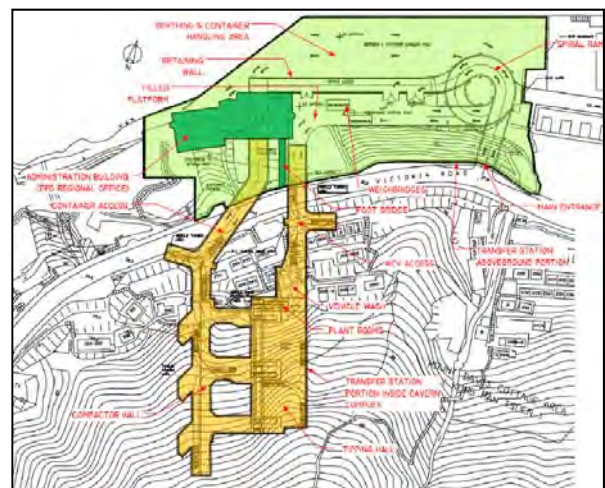


Fig. 1 General Site Layout

The layout plan of the station as shown (Fig. 1) permits the most effective use of the site and ensures safe, efficient and environmentally acceptable transfer of maximum quantities of refuse from WCVs to containers.

Vehicle wash facilities are provided in the egress lane of the WCV access. The waste transfer process starts from the weighbridge facilities which are located on the fill platform extend to the cavern complex. The facility is capable of accommodating peak throughput capacities without causing vehicles to queue.

The refuse transfer process then continues with the WCVs entering the tipping hall, discharge the waste into a receiving hopper of the live floor from which it

is passed into a compactor and subsequently compacted into containers. Containers are then shuttled to the berthing area for onward deliver to the landfill by a purposely built container vessel equipped with an on board crane. The system comprising a receiving hopper equipped with a "live floor", which will continuously propel waste to the different compactor charge boxes in either forward or backward direction.

To achieve the design capacity of 1,000 tonnes of refuse per day, careful engineering design is required to provide sufficient length of tipping edge next to the live floor whilst minimizing the demand on foot print to save the expensive volume created from excavation.

The cavern layout to accommodate this transfer operation is on two levels: an upper level for the tipping hall and a lower level for compacting and storing containers. WCVs enter and leave the first floor tipping hall through a single opening at the north-east corner of the cavern via the access road. In the compactor hall, container trailers are manoeuvred by specialist tractors which transport the waste loaded container out from the compactor hall via a single access in the north-west corner of the complex.

At the berthing area, container exchange occurs using either the crane on board the vessel, or a Container Handling Unit (CHU).

Vehicle wash facility is provided in the egress lane of the WCV access to maintain the high level of hygiene conditions before leaving the station. The facility is capable of accommodating peak throughput capacities without causing vehicles to queue.

Offices and personnel facilities for the regional office of the EPD and the facility operator are provided on different levels of the Administration Building adjacent to the container access portal and plant rooms, which also accommodate a vehicle maintenance facility and dangerous goods stores. External container storage areas have been incorporated within the berthing area.

The containers will be transported between the transfer station and the WENT landfill by a specially designed container vessel with an on-board gantry crane to enable self loading and unloading.

#### (1) Traffic Movement and Management

Site roads and access design effectively minimised the area requirements for traffic movement and management. The compact design enables all the required functions of the transfer station to be accommodated without interference with other public works to be undertaken in the vicinity of the site and along the Victoria Road. Junction and interface arrangements are kept simple and secure by separating the WCV traffic streams. Access for container vehicles

to the berthing area is kept independent of other waste streams.

#### (2) Site Access and Spiral Ramp

Access from Victoria Road was provided at a level of 27.8 mPD and in order to avoid the length and visual impact of elevated structures, a spiral ramp arrangement to Highways Department standards and approved by the Advisory Committee on the Appearance of Bridge and Associated Structures (ACABAS) was provided. The spiral ramp provides access to the weighbridge area, cavern complex, administration building and berthing area of the Transfer Station (Fig. 2).



Fig. 2 Aerial View of IWTs and Administration Building

#### (3) Weighbridges

Two weighbridges were provided to weigh inbound and outbound WCVs, with electronic platform scale for weighing the gross and tare weight of a vehicle, which will be recorded by computer system and for calculation of net weight of waste tipped in IWTs. The weighbridges were constructed on elevated platform with the rock fill excavated from the cavern with a retaining wall was built.

#### (4) Tipping Hall/WCV Access

WCV access to the tipping hall comprises a 9m wide passage with 60 m in length approximately (Fig. 3). It incorporates a pedestrian walkway, a Fire Services/Emergency Vehicle turnaround point and a Fire Services Control Room. With the aid of a queuing analysis it was demonstrated that no queue will form under normal circumstances. After discharging the waste into the hopper, WCVs leave the tipping hall and proceed through the vehicle wash located at eastern end of the access and exit the cavern complex.

#### (5) Tipping Hall

At the tipping hall entrance a indicator board activated by a programmable traffic controller, will advise the WCV driver the location for the discharge of waste. Approaching that location, the WCV driver will be guided by a green traffic light and will reverse until

coming to rest against a wheel stop, and discharge waste into a hopper. Having discharged the WCV load, the driver could then remount the vehicle and exit the tipping hall.

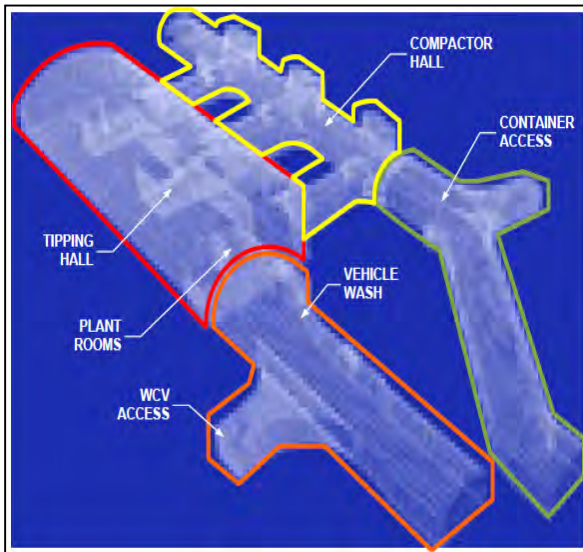


Fig. 3 3D View of Cavern Structures

#### (6) Compactor Hall

The compactor hall at the lower level (5 mPD) accommodates four compactors and a waste water treatment plant (Fig. 4). The waste collected on the live floors on Tipping Hall fell into two stage compactor units under gravity, and compacted into a closed compaction chamber of compactor, prior being ejected into 20' container. Special-made low profile trailers with the refuse container were manoeuvred by specialist tractors with a series of adequately proportioned chambers housing the compactors.

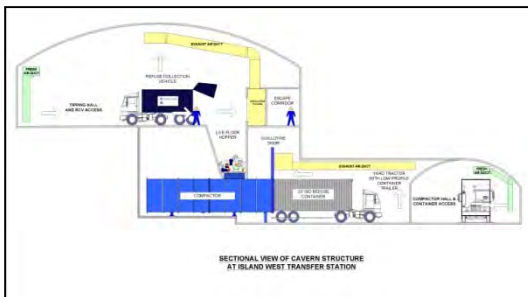


Fig. 4 Sectional View of Cavern Structure

#### (7) Container Access

From the compactor hall at the lower level (5 mPD), the containers are moved to and from the berthing area using specialist tractors and low profile container trailers. The container access is fed directly out of the cavern complex onto the Berthing Area at level 4.5 mPD, thus effectively separating the two major traffic streams and minimising travelling distances for the shuttle operation.

#### (8) Waste Water Treatment Plant

The Waste Water Treatment Plant (WWTP) which

consisted of a series of waste water and sludge holding tanks and accommodated the waste water treatment equipment with associated ancillary equipment and device, was located inside the cavern. Due to the high loading induced by the tanks, they have been located so as to be founded on rock and to ensure that as many of the retaining walls as possible are formed against in situ rock faces. This resulted in the most economical structure for the WWTP.

#### (9) Amenity, Ancillary and Plant Rooms

Amenity, ancillary and plant rooms were located at the portal structure and the administration building for the accommodation of amenity facilities and infrastructures to support the operation of IWTS and the visitors.

#### (10) Administration building

The Administration Building accommodates the office area for the regional headquarter of the Environmental Protection Department and the Contractor, including all the E&M plant, Workshops, Amenity and a Roof Garden.

#### (11) Environmental Control Systems

Environmental Control System was essential component of the whole system and was integrated into the design, with was fully integrated with the waste water treatment plant, the air handling and treatment equipment further minimising space utilisation in the cavern complex.

The air handling and wastewater treatment systems were integrated that a significant source of nutrients was present in the wastewater treatment facility. These nutrients were used as the primary reactant in a process combining wastewater treatment and odour/dust control.

#### (12) Electrical and Mechanical System Design

The Electrical and Mechanical services installation described hereafter define the type and extent of the building services work for the transfer station and generally includes the following:

##### Electrical

- Low voltage distribution for normal power, essential and emergency power supply
- Emergency diesel generator
- Normal lighting, emergency lighting and exit signs
- Earthing system
- Earthing protection system
- Fire services power supply and alarm system
- Telephone system
- CCTV monitoring system
- Public address system
- Intercom system
- Traffic light system

- UPS system

#### Fire Services

- Street fire hydrant installation
- Fire hydrant/hose reels installation
- Fire detection and alarm detection
- Manual fire alarm system
- Portable fire extinguishers
- Sprinkler system
- Total gas flooding system
- Fire and Smoke Dampers
- Fire shutters and compartmentation
- Gas detection system
- Radio –telecommunication system
- Fixed telecommunication system
- Audio and visual alarm system
- Directional signage

#### Mechanical

- Mechanical ventilation with environmental (odour and dust) control system
- Smoke extraction and make up air system
- Staircase pressurisation system
- Air conditioning system

#### Plumbing

- Potable water supply system
- Flushing water supply system
- Industrial water supply system
- Ground water reuse system

#### Drainage

- Rainwater and surface drainage system
- Soil and waste water drainage system
- Industrial waste water drainage system

### (13) Fire Protection and Means of Escape

Typical underground cavern construction with dead-end structure implies additional risks on the means of escape (Fig. 5). Thus intensive discussions with Fire Services Department (FSD) and the fire engineering consultant were undertaken. With the provision of protected escape corridor and escape staircases inside the cavern complex connected with the Tipping Hall and Compactor Hall compartment, it allows safe with alternate protected escape routes for personnel to evacuate from the underground premise during emergency or fire condition. It was also well aware that smoke hazard imposed higher risk than fire, especially to the personnel working inside underground cavern complex. Sophisticated smoke extraction and makeup air system, staircase pressurisation, fire and smoke dampers, gas detection system, fire roller shutters, ventilation and air conditioning control (VAC) system, and audio & visual alarm system were incorporated in the fire protection system at IWTS.

Emergency lightings, directional signage and exit signs were powered by maintained type battery and were connected with the essential power supply from the

emergency generator. They were provided throughout the underground and aboveground portion of the facility, with additional low-level emergency lightings being installed along main escape routes at WCV and Container Access. Low-level lightings enhances the identification and visibility of escape routes during the outbreak of fire with heavy smoke condition, which tends to accumulate at top level of the access where illumination from ceiling mounted emergency lightings will be blocked.

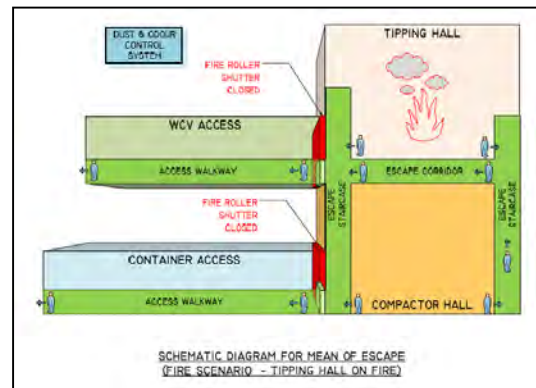


Fig. 5 Schematic Diagram for Means of Escape

### 3. Construction

The design and construction of IWTS commenced in May 1995 and the operation commenced in May 1997 and the construction operations was undertaken with the least disturbance to the environment. The design was significantly more compact than the conceptual design to the extent that construction activities were confined to only half that of the allotted area. Thus the potential area be affected by construction activities was significantly reduced. Furthermore, the duration of construction operations was also shortened.

The main construction related impacts were expected to be noise and vibrations related to excavation, blasting, haul road traffic and access ramp construction and dust arising from the same activities. Site runoff was controlled through implementation of erosion control measures and installation of effective silt/grease traps to ensure water discharge to the stormwater drainage system complies with statutory requirements. During construction there were minimal impact on the local road network since all spoil were disposed of by barge directly from the site. Marine impacts were minimal since the only marine works carried out were dredging of 200 to 300 m<sup>3</sup> of silt and boulders for marine access to the berthing area.

The underground tunnelling and cavern construction work implied a higher degree of risk to the health and safety of workers than normal construction. In addition to the activities associated with normal construction works, additional hazards due to underground environment were addressed on visibility, dust and fumes, noxious gases and radon, inundation and

flooding, handling of explosives and falls of ground (Fig. 6).



Fig. 6 Record Photo of Cavern Excavation

In addition to the health and safety of personnel working on site, the disturbances of the site operation on neighbouring public and residential areas (Fig. 7) were also considered, in addition to compliance with statutory requirements, notices and advise were given to local residents with regards the site operation in order to increase public awareness.

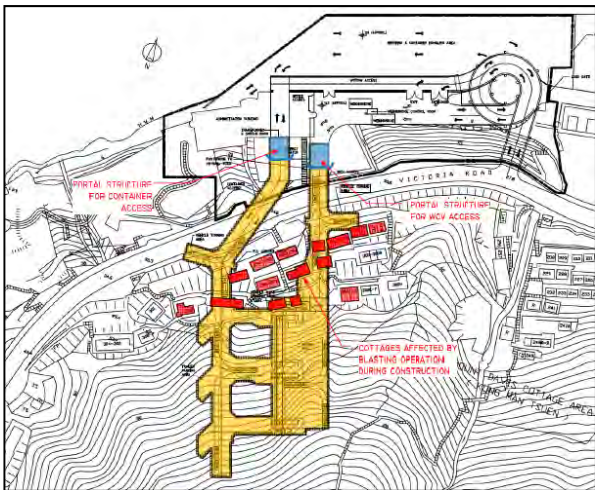


Fig. 7 Location of Neighbouring Residential Area

With the implementation of the controls and mitigation measures outlined in the DEIA, environmental impacts in respect of dust, noise, road and marine traffic were kept to within statutory or acceptable limits.

It was considered that the fundamental environmental mitigation measure was encapsulated in the design itself. In contrast to the conceptual design, only half the allotted area was used for the development, the adjacent location of the tipping hall and compactor hall tunnels eliminated the need for an elevated access road along the coastline, and the volume of rock to be excavated was about 60% of that for the conceptual design. Furthermore, the short distance between the cavern tunnels and the berthing resulted in lower overall noise and fugitive dust emissions from haul

road traffic. Along with the shorter period required for construction, the overall result was less environmental impact than that envisaged for the conceptual design. Even though no additional assessment of construction impacts was required, it was necessary to review the cumulative construction impacts of the various adjacent infrastructure projects in the light of any further information available at the time of the detailed design.

#### 4. Operation

The design capacity of IWTS is able to transfer 1,000 tonnes of municipal each day collected from the Western, Central and Southern District of Hong Kong Island, with operation hours from 7:30 am to 11:30 pm. All the wastes delivered and tipped into waste reception equipment inside the cavern structures, are compacted into purposed made 20' containers and transferred to West New Territories (WENT) Landfill for final disposal, via marine transportation by two purpose built container vessels with onboard gantry crane.

Other ancillary and supporting operation activities are also undertaken, such as all the building services including fire services, cleansing, vehicle wash, wastewater treatment, ventilation and dust & odour removal system, road sweeping, landscaping, and the associated maintenance of the plants and equipment.

Environmental monitoring is undertaken, such as air, water, noise, dust, odour, etc, to ensure compliance with the contract requirements. Otherwise deduction will be made to the payment for the operation.

As it was addressed in the SPUN study that radon, a radioactive gas which will be derived naturally from granite and volcanic rocks, will impose occupational and health hazard to the personnel working if they inhaled excess amount of radon gas inside the underground cavern structures during the construction and operation processes.

With adequate air-change rate with a well designed ventilation system, routine in-situ radon measurement in construction and operation stages are able to achieve the requirement on "Excellent Class" of 150 Bq/m<sup>3</sup> for recommended Indoor Air Quality Objectives in Hong Kong.

#### 5. Conclusions

With the single Design-Built-Operate Contract approach for IWTS, the operator was given maximum flexibility to provide innovative designs, new engineering and technology, and cost effective approaches that substantial cost saving was achieved without comprising tight schedule of completing the design and construction of IWTS in two years, and full

compliances with the stringent engineering, environmental, safety and performance requirements, at a cost of just half the original budget and cut down more than half the excavation volume as proposed in the conceptual scheme.

With the successful commissioning in 1997, IWTS set another milestone on the use of underground land in Hong Kong for infrastructure facilities. It serves not just as a reference for planning of underground land and cavern engineering, but it also provides a platform for the promotion and education of environmental awareness and protection to both local public and overseas stakeholders. More than a thousand visitors visited IWTS each year.

### Acknowledgements

The authors wish to thank Environmental Protection Department of the HKSAR for its kind permission to publish this paper.

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# CAVERN STORAGE OF EXPLOSIVES AT THE GOVERNMENT DEPOT, KAU SHAT WAN, LANTAU

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## Abstract

*The Hong Kong SAR Government's principal Explosives Depot is at Kau Shat Wan, Lantau. A major part of this facility is underground, within rock caverns designed for the storage of explosives. This paper describes key aspects of the planning, design, construction and operation of the Kau Shat Wan Depot. The use of the facility is described from the operator's perspective.*

## Keywords

*Planning, Explosives, Depot, Operation, Caverns, Design*

## 1. Introduction

The Kau Shat Wan Depot (KSW), which came into operation in June 1997, is used to store commercial explosives, including fireworks. The facility was constructed because the Government's previous Explosives Depot, at a shared military site on Stonecutters Island, had to be relocated before the site was handed over to the People's Liberation Army for their sole use in July 1997. The Depot has a notional storage capacity of 500 tonnes net explosives quantity (NEQ). In practice, the volume of explosives stored in their original packaging rather than the NEQ limits the actual capacity.

Developing a permanent explosives depot is a multifaceted process, particularly in a densely populated and administratively complex territory like Hong Kong. The social, political and environmental issues faced in the mid 1990's for the development of the Kau Shat Wan Depot were considerably less onerous than that which would likely be encountered today, but the technical issues, including the geotechnical design and construction as well as the hazard assessment process for developing such a storage facility, remain pertinent. Furthermore, the operational considerations from the user's perspective provide useful insights, which should be considered at the planning and design stages in future cavern development.

## 2. Explosives Control in Hong Kong

Commercial explosives, in particular blasting explosives, are very stringently controlled in Hong

Kong. The Commissioner of Mines through Mines Division of the Geotechnical Engineering Office of the Civil Engineering and Development Department (CEDD) is responsible for regulating the manufacture, storage, transport and use of explosives under Cap 295 – Dangerous Goods Ordinance (DGO 295) of the Laws of the Hong Kong Special Administration Region. In addition, any person intending to import explosives must first apply to the Commissioner of Mines for authorisation for their use in Hong Kong.

### 2.1 Government Explosives Depots

In Hong Kong, all blasting explosives are required (under the DGO 295) to be stored in a Government Explosives Depot or in a Mode A Store (site magazine) licensed by Mines Division for a specific blasting project. Mines Division currently manages two Government Explosives Depots, the Kau Shat Wan Government Explosives Depot (KSW Depot) and the Kowloon Government Explosives Depot.

The KSW Depot is located on the southeastern coast of Lantau Island near Ping Chau and Hei Ling Chau (Fig. 1).



Fig. 1 The Kau Shat Wan Explosives Depot; the tunnel portals to the caverns are on the left

The site, which occupies about eight hectares, comprises the wharf area, the administration area, the storage area and the demolition ground. The depot has 16 buildings for different purposes in addition to 10 underground or cavern magazines and 8 above ground earth covered magazines in the storage area.

### 3. Planning and Hazard Assessment Considerations

Safety requirements dictate the site selection process for an explosives magazine. For large facilities, in particular, the site has to be isolated and, to ensure the longevity of the facility, the selection process must ensure that the site is unlikely to be impinged by future development. Moreover, marine access is a local requirement for import of explosives (mainly by sea) and delivery, which has also traditionally been done by sea, to a suitable marine unloading point for onward delivery by road. In Hong Kong, these requirements severely limit the choice of sites to a few remote areas of the New Territories and the outlying islands.

The site selection process for the new Government Explosives Depot commenced in the late 1980s. The development of an underground magazine was not a prerequisite, but, as a study on the potential use of underground space was also being undertaken at this time [ARUP, 1990], the possibility of housing the facility underground would have been recognised. In fact, in 1991, the Hong Kong Government adopted a policy of encouraging development in rock caverns, where such development was appropriate [MALONE, 1996; CHAN et al, 2006]. In 1988, initial feasibility studies were undertaken in relation to a site at Chi Ma Wan on Lantau Island [MOTT, 1992; SHIU, 1993]. The studies concluded that the site would be technically suitable for the new depot, but it was subsequently considered unfavourable from the planning point of view; it was envisaged that there would be strong public resistance to the development, as it was located partly in a country park. A search for an appropriate site elsewhere in Hong Kong established that Kau Shat Wan on Lantau Island was the most suitable site for the new depot. The main advantages of the Kau Shat Wan site were considered to be:

- (a) it was relatively inaccessible with land access only by a narrow footpath.
- (b) the population in the general vicinity of the site was only about 160 people, primarily at Man Kok Village about 700 m to the south and the Trappist Monastery 800 m to the north.

Four layout options were taken forward for consideration amongst nearly twenty possible layout options for Kau Shat Wan. The selected options [MOTT, 1992; SHIU, 1993] comprised two above ground options, an underground option with limited reclamation and a hybrid option. They were all planned to cater for storage of up to 500 tonnes of explosives for commercial use and a possible further 500 tonnes for military use. The layouts were evaluated using the quantity-distance requirements established by the Explosives Storage and Transport Committee [ESTC, 1979], UK Ministry of Defence, and the Risk

Guidelines in the Government's Planning Standards and Guidelines [HKPSG, 1990].

The ESTC quantity-distance requirements for each layout were also used to indicate the extent of a Safety Zone (SZ) surrounding the proposed depot. In accordance with the HKPSG, no inhabited buildings or congregation of people would be allowed within the SZ. The underground layout resulted in a significantly smaller SZ than the above ground and hybrid layouts, which effectively meant that less land surrounding this layout needed to be sterilised as a safety buffer when compared to other options.

Under the HKPSG, a Government Explosives Depot is classified as a Potentially Hazardous Installation (PHI) and, as such, a Hazard Assessment (HA) is required. The HA is a technical study on the level of off-site individual and societal risks associated with the design and operation of a particular PHI. The initial HA for Kau Shat Wan produced calculations of individual risk and societal risk for the four layout options. These risks were assessed against the Hong Kong Risk Guidelines (RG), as given in the HKPSG, and the options ranked in terms of the Potential Loss of Life (PLL) per year. Both above ground options were close to or above the individual risk criteria ( $10^{-5}$ /year); the below ground and hybrid options satisfied the individual risk criteria and all options satisfied the societal risk criteria. In terms of minimising PLL, the underground option was better than the hybrid option. Associated studies showed that the underground option could also minimise the environmental impact and involve lower capital and operational costs [SHIU, 1993; MALONE, 1996].

Before construction of the KSW Depot, a number of changes were made to the design, including eliminating the storage of military explosives and reducing the total capacity to 500 tonnes NEQ (other changes are discussed in TBV [1995]). The final layout of the KSW Depot followed the main elements of the original underground layout. The initial HA as well as the subsequent reviews produced recommendations relating to the management procedures for the depot in order to minimise operational risks. These procedures cover the capacity of the magazines, the types of explosives that can be stored, and the road and vessel transport arrangements, and form the basis of the Depot Management Manual.

### 4. Engineering Geology of the Site

The KSW underground system was excavated into the hillside close to the original shoreline, before the reclamation was undertaken for the surface facilities. The rock exposed along the tunnel alignment is located in an area dominated by fine- to medium-grained granite intruded by feldsparphyric rhyolite dykes, which is lithologically similar to the southern part of

Tsing Yi Island. The Kau Shat Wan area is considered to be part of the northeast trending dyke swarm stretching from north-eastern Lantau to Tsing Yi.

Approximately equal proportions of granite and feldsparphyric rhyolite (dykes) were exposed during the tunnel construction, as well as a few thin sub-vertical basalt dykes. The contacts between all the rock types are essentially sharp, sub-vertical and trending northeast. The rocks are generally slightly decomposed to fresh with some moderately to highly decomposed zones along a few localised northeast and northwest trending faults or shears. Most of the basalt dykes exposed at the ground surface are sheared and highly decomposed. Preconstruction site assessment [FRANKS, 1993] of the jointed bedrock indicated up to 3 predominantly sub-vertical joint sets ( $80^\circ$  dip/ $040^\circ$  dip-direction,  $70^\circ/317^\circ$  and  $70^\circ/180^\circ$ ) together with one or two localised low angle sets ( $20^\circ/055^\circ$  and  $29^\circ/133^\circ$ ). This is generally consistent with the regional trend. However, not all of these joint sets are present throughout the site. The sub vertical joints were found to be typically widely spaced (generally 1 m to 2 m, although locally were closely spaced at 0.2 m or less) whilst the low angle joints were typically very widely spaced (generally  $>2$  m). The superficial deposits and completely decomposed in-situ bedrock found at the tunnel portals were thin and generally less than 10 m thick, although the preconstruction site investigation drill holes had found a wide variation in depth of the weathered mantle ranging from 30 m to less than 10 m. At the location of the caverns, the regional groundwater was indicated from the preconstruction ground investigation drill holes to be up to 30 m below the ground surface. Preconstruction site assessment [FRANKS, 1993; SHIU, 1993] indicated slight artesian conditions within a fault zone in one drill hole at the approximate elevation of the caverns; however, this was not found during excavation of the complex.

## 5. Depot Construction

Preconstruction assessment of the likely construction costs for the tunnels and caverns were in the range of HK\$80M to 153M, depending on the layout [SHIU, 1993]. The Depot was constructed under two separate contracts.

The site formation work for the first contract [LEIGHTON, 1995 & 1996] commenced in late 1994 and was substantially completed in 1996. The contract sum was HK\$214M and the construction cost of the main access tunnel adits and caverns, including a rectangular concrete magazine structure in each of the caverns, was approximately HK\$80M. In addition to the underground excavation, this contract included the reclamation, site formation and drainage works, and construction of the surface magazines.

Work under the second contract (awarded to Aoki Corporation) commenced in early 1996 and was completed in mid-1997. The contract sum was HK\$190M and included construction of the buildings, roads and drains, the electrical and mechanical installations and soft landscaping work.

### 5.1 Underground System (Tunnel and Caverns)

The KSW underground system comprises a 1.15 km long U-shaped main access tunnel with 10 adits each approximately 33 m long, connecting the main access tunnel to the 10 storage caverns (Fig. 2). The tunnels and adits are typically 5 m high with a 1.5 m high roof arch and span of 6.5 m. The caverns are typically 20 m long, 6.8 m high with 2 m roof arch and a span of 13 m. The storage magazine inside each cavern is a rectangular reinforced concrete box-structure approximate 17 m long 7 m wide and 3.5 m high. Each cavern magazine has a maximum storage capacity of 50 tonnes NEQ.

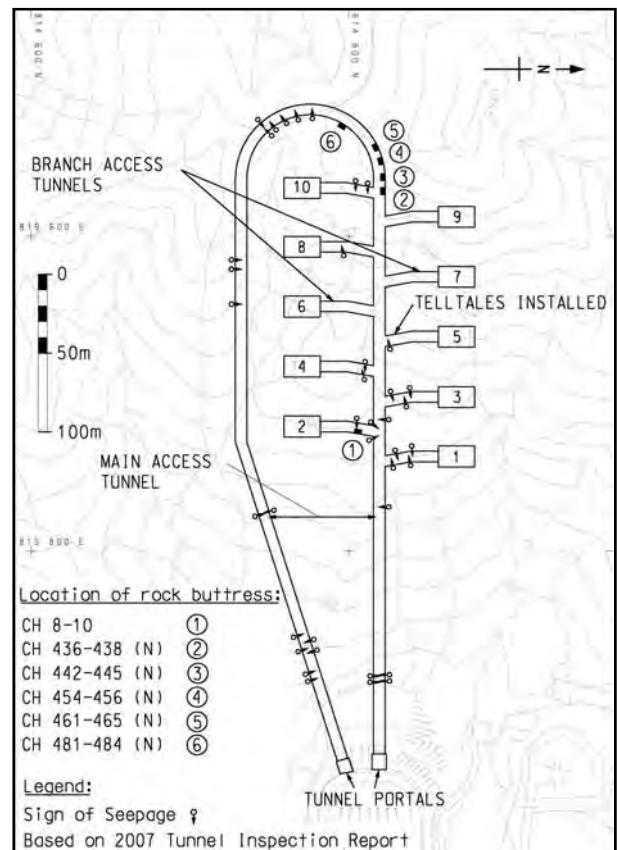


Fig. 2 Layout of Underground System at Kau Shat Wan Government Explosives Depot

The initial 100 m sections of the main access tunnel from both the northern and southern portal are concrete lined. These concrete lined tunnel sections were excavated in soft materials composed of completely to highly decomposed bedrock or superficial soil. The remaining part of the underground system was excavated by drill and blast method in jointed

moderately decomposed to fresh granitic bedrock. The support for the unlined tunnels and caverns was designed by using the NGI-Q System of rock mass classification. Preconstruction assessment of the likely Q values indicated a range of about 7 to 20 for slightly decomposed to fresh bedrock at the proposed cavern depth. This was confirmed during construction. Most of the permanent rock support was by pre-stressed pattern rock bolts and plain or fibre reinforced shotcrete, in combination, or by pre-stressed pattern rock bolts only. The preconstruction assessment indicated that the preferred cavern axis orientation was NW-SE, based on the strike of the three predominant sub-vertical joint sets. The constructed orientation of much of the tunnel axis follows an E-W alignment (except for a short length forming the U-shaped alignment at the midpoint of the tunnels), with the cavern axes N-S.

The underground system was designed such that a rock pillar, not less than twice the span of the adjacent caverns, separates each cavern. The caverns were also arranged not to be directly opposite each other. This layout utilises the rock separating the caverns as a barrier to prevent the spread of an accidental explosion to explosives in adjacent caverns through sympathetic detonation.

## 5.2 Instrumentation Monitoring

The structural stability of the tunnels and caverns was monitored during construction by ground survey using a system of 5-pin convergence arrays installed at 50 m intervals along the main access tunnel and in one of the caverns. One of the caverns was also monitored using a set of five multiple-point borehole extensometers. The monitoring data did not indicate significant movement and monitoring was stopped before the underground explosives storage system came into operation in 1997.

## 6. Post Construction Maintenance

### 6.1 Inspections for Structural Stability

Professional staff from GEO undertake routine inspections to identify the need for any structural maintenance work within the underground storage system. The inspections were initially carried out half yearly for the first three years. Thereafter, inspections have been carried out annually apart from areas between the cavern walls and the reinforced concrete storage structures, which, due to access difficulties, are only inspected every 5 years. No major signs of instability have been identified since the construction of the underground system.

### 6.2 Structural Maintenance

The follow structural maintenance works have been undertaken since commissioning of the depot:

- (a) Between 1997 to 1999, approximately 10 cubic metres of loose rock blocks were manually scaled from the tunnel and cavern rock faces.
- (b) Between 2003 and 2007, remedial works were carried out on rock bolts and assemblies, which had become corroded in the humid underground environment.
- (c) In 2007, several areas of overhanging blocks were supported by the installation of six buttresses. Five were close to the midway point along the tunnel alignment (Fig. 2 & 3) and one was along the branch access tunnel to Cavern Number 2.



Fig. 3 Rock Buttresses Installed in the Tunnel

In 2003, vertical cracks were discovered in the wall of the branch tunnel to Cavern No. 5 (Fig. 2). These cracks have been monitored using telltales, but no further movement has been recorded and no maintenance works have been required.

No major maintenance works have been needed within the KSW underground system since 2007. The cost of maintaining the structural stability of the entire system has been less than HK\$0.5M over the 14 years of operation, with the depot staff carrying out the minor works. The cost could have been lower still, had it not been for the remoteness of the site and restricted working hours, which add significantly to the cost of engaging contractors.

### 6.3 Non-structural Maintenance

For the first few years after construction, the underground system was quite wet even in the dry season. The rate of groundwater seepage varied from an estimated rate of over 10 litres/minutes along the faults and weathered basalt dykes to dripping along some joints and areas of dampness on some rock faces. Due to this seepage, the underground system was generally quite humid especially during the wet summer months. However, the tunnel has been quite dry for the last few years except in a few faulted or weathered areas.

Despite the low structural maintenance costs, the E&M installation maintenance costs of the underground system, especially the electrical and electronic systems, have been high relative to similar above ground installations in the depot. The direct dripping of groundwater onto the E&M equipment and the humid environment caused by the groundwater seepage along the unlined rock excavation faces have led to the following maintenance requirements:

- (a) Two years after the depot commenced operation, over half of the optical smoke detectors were malfunctioning. Replacement detectors installed in 1999 suffered similar problems. Waterproof smoke detectors (IP 56 rating) were tried successfully in 2005 and solved the problem.
- (b) After four years of operation, the lighting system in the caverns was found to be seriously corroded and malfunctioning due to the ingress of groundwater. As a result, the lighting system, cables and emergency exit signs had to be replaced with an explosion-proof and water-resistant system. Similarly, about half of the original explosion-proof telephone system developed faults due to water ingress and had to be replaced by another explosion-proof and water-resistant type.
- (c) After seven years of operation, the pressurised air pipelines for the pneumatic cavern magazine doors and the pressurised water pipelines for fire fighting were found to be corroded. Waterproofing works were carried out to protect these pipelines from the effects of directly dripping water in areas where groundwater seepage was significant. The works comprised installation of stainless steel drip trays (Fig. 4) directly beneath points of seepage (observed in the summer months) and stainless steel sheets to cover large damp areas where the dripping points could not be confirmed precisely.
- (d) The groundwater seepage within the underground system is lime-rich (due to seepage through shotcrete) and the lime, which accumulates by



Fig. 4 Waterproofing Works (Drip Trays) Installed to Protect Pipelines in the Tunnel

precipitation (Fig. 5), eventually blocks the subsurface drains. The drains are required to be cleared at about 5-year intervals. However, a few sections are now so strongly blocked that they cannot be cleared and these will be converted from subsurface drains to open drains in the near future. Each of the caverns has a heavy blast proof door supported on rollers in a channel. The rollers for Cavern Number 5 and later Cavern Number 3 became jammed after the roller channel had been submerged due to the ineffective surface water drainage and subsequent precipitation of lime from groundwater onto the rollers' bearings.

The total non-structural maintenance cost for the underground facility at KSW has been approximately HK\$3.5M over 14 years.

## 7. The Underground Environment

The temperature of the underground system is relatively constant at about 23 to 28°C year round. The working areas comprising the access tunnels and the caverns are well illuminated. The main tunnel is ventilated by a forced ventilation system comprising five pairs of jet fans and, consequently, the environment is slightly noisier than that above ground. These fans provide fresh air to the depot staff and air quality testing confirms that the ventilated underground environment has similar air quality to the above ground environment. The dull visual appearance of the tunnel environment has been improved by painting the rock bolt heads in bright colours (Fig. 4).

The main drawback of the underground system at the KSW Depot is the relatively wet and humid working environment, especially in the summer months, resulting from groundwater seepage. Remedial works including the installation of drip trays and diversion of dripping groundwater from the crown of the tunnel to the sidewalls have been necessary and these works have improved the working environment significantly.



Fig. 5 Removal of Lime Accumulation from the Subsurface Drains in the Tunnel

Another minor inconvenience is that the magazine caverns need to be ventilated for 5 minutes to extract the adjacent granite sidewalls) before the depot staff are permitted to enter.

## 8. Conclusions

CEDD has pioneered the planning, design, construction and operation of rock caverns for storage purposes in Hong Kong. The KSW Depot was the first underground explosives storage facility in Hong Kong. The experience gained from planning, designing and constructing the facility provides key lessons for future underground development. Mines Division's experience in operating and maintaining the underground explosives storage facility for the past 14 years has confirmed that it provides a suitable, safe and pleasant working environment.

Given the severe shortage of land for development in Hong Kong, the availability of suitable sites for a PHI, such as a Government Explosives Depot, are extremely limited. The feasibility studies undertaken for the KSW Depot have shown that less land needs to be sterilised as a safety buffer surrounding an underground facility when compared to a similar above-ground storage facility. Also, as much of the facility is effectively shielded from the community, it is less intrusive as a 'bad neighbour' and probably attracts fewer complaints than an above ground facility.

The rock quality at KSW is only average compared with the common granitic rocks found in Hong Kong. Despite this, the caverns and tunnels have only suffered a few minor structural stability issues. With hindsight, these issues could have been avoided if they had been identified and preventive works carried out during the construction stage, but the cost of the post-construction remedial works has been relatively low.

The main inconvenience encountered at KSW has been the high groundwater inflow into the underground system especially during the wet summer months. This has caused significant damage to the E&M installations as well as the drainage system due to the damp environment, direct dripping of groundwater and the precipitation of lime from the groundwater seeping through the shotcrete lined tunnel. Over the 14 years of operation the total maintenance costs incurred, directly or indirectly, by the groundwater inflow into the underground system, amounted to about HK\$3.5M. These costs could have been mitigated had they been foreseen and if more attention had been paid to groundwater inflow at the design and construction stages. The problems could have been addressed by using appropriately waterproofed components for E&M installations, such as smoke detectors, fluorescent lamps and electrical wiring. Alternatively, either a fully grouted tunnel lining system or drainage membrane layer behind a cast in-situ concrete tunnel

lining to divert the groundwater inflow into the toe drains could have been installed. This would have maintained the tunnel in a dry condition year round. Over the long term, better control of groundwater inflow would have reduced significantly the ongoing maintenance costs and may have justified the higher initial construction costs.

The remoteness of the KSW site, although ideal from a safety and security point of view, poses problems in arranging resources to carry out maintenance work. It also poses constraint on the operation of the facility. These result in somewhat higher maintenance and operation costs than would normally be the case for a site with road access.

## Acknowledgements

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development of The Government of the Hong Kong Special Administrative Region.

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# SEWAGE TREATMENT WORKS IN CAVERNS - HONG KONG EXPERIENCE

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## Abstract

*In 1989, the need of a new sewage treatment works was identified under the Stanley Sewerage and Sewage Disposal Scheme. To maintain the scenic area around Stanley, cavern option was adopted for its construction. Stanley Sewage Treatment Works, being the first secondary (biological) plant inside caverns, was constructed in November 1990 and commissioned in February 1995. It was housed inside three large caverns, each about 120 m long, 15 m wide and 17 m high together with over 450 m of road access and ventilation tunnels and shafts. The design capacity is 11,600 m<sup>3</sup>/day and is now serving a population of over 27,000 in the catchment area.*

*After operating the plant for more than 15 years, it has been symbolized as a very successful small to medium size secondary sewage treatment plant inside caverns with well proven experience both in construction and operation aspects. Some design principles and user experiences are described in this paper.*

## Keywords

Cavern, Sewage Treatment Works, Extended aeration

## 1. Introduction

The Vision of Drainage Services Department is “To provide world-class wastewater and stormwater drainage services enabling the sustainable development of Hong Kong”. Efforts have been put to operate and maintain our facilities in a sustainable manner with respect to energy conservation, use of renewable energy, resources optimization and greening. The concept of building Stanley Sewage Treatment Works (STW) inside the carven also demonstrates our effort in saving one of the scarce resources in Hong Kong, the usable land.

There are 70 treatment plants and 230 pumping stations in Hong Kong for sewage collection and treatment. However, constructing a sewage treatment facility inside caverns was not common in Hong Kong as demand for land uses with potential for cavern development was not imminent in the past few decades. Major STWs were used to be sited remotely from the urban development area. In 1989, the need of a new STW was identified under the Stanley Sewerage and Sewage Disposal Scheme. To maintain the scenic area around Stanley, cavern option was studied and finally

adopted for its construction. The STW was constructed in November 1990 and commissioned in February 1995.

## 2. General Description of Stanley STW

### 2.1 Design Principle

The STW serves a population of over 27,000 in Stanley, Ma Hang, Tai Tam, Chung Hom Kok and Red Hill areas (Fig. 1) which currently produces 8,800m<sup>3</sup> of sewage per day. As the treated effluent is discharged in the Southern Water Control Zone, the requirements for the effluent quality is stringent and can only be achieved by secondary (biological) treatment process. The effluent standards are shown in Table 1.

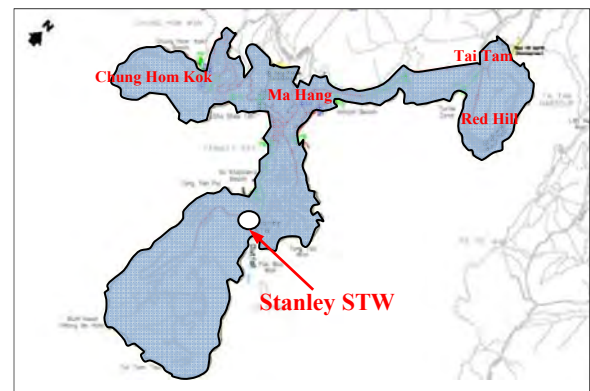


Fig. 1 Catchment of the Stanley STW

| Parameters                      | Effluent limits      |
|---------------------------------|----------------------|
| 5-day Biochemical Oxygen Demand | ≤20 mg/L             |
| Total Suspended Solids          | ≤30 mg/L             |
| Ammonia – Nitrogen              | ≤5 mg/L              |
| Nitrate + Nitrite - Nitrogen    | ≤8 mg/L              |
| E Coli                          | ≤15,000 count/100 mL |

Table 1 Effluent Discharge Standards

The treatment process generally involves 7 stages namely (1) screenings removal, (2) de-gritting, (3) denitrification, (4) carbon removal and nitrification, (5) sedimentation, (6) disinfection and (7) sludge treatment and dewatering. Pollutants are removed physically in stages 1 & 2. Screened sewage will enter the aeration tank for secondary treatment. In aeration tanks, compressed air is fed continuously to provide oxygen essential to sustain the growth of micro-organisms (activated sludge), which will assimilate pollutants in the sewage. Treated sewage and activated sludge are

separated in the final sedimentation tanks. A controlled portion of the activated sludge is fed back to the aeration tank to maintain adequate micro-organism population for biological treatment. The remaining portion (surplus activated sludge) is pumped to a separate facility for dewatering. After final sedimentation, the effluent is disinfected by sodium hypochlorite solution before discharged to the water southeast of the Stanley Peninsula via a 0.6 m diameter, 2.3 km long submarine outfall. The surplus activated sludge and collected scum are mixed with polymer and dewatered by membrane filter press to achieve a minimum dryness of 30%. The dewatered sludge is then transported by using covered containers for disposal at landfill (Fig. 2).

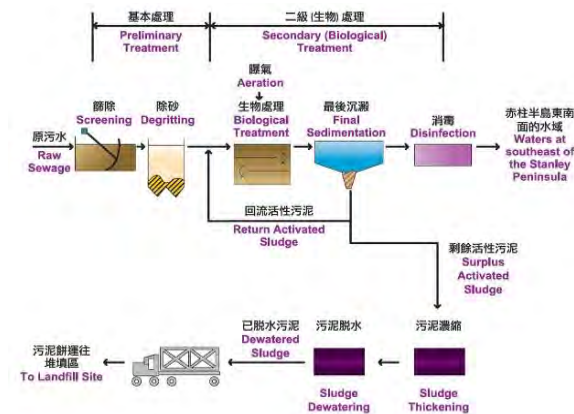


Fig. 2 Schematic of the Sewage Treatment Process

### 3. Special Design Feature

#### 3.1 Layout of the Stanley STW

With a view to maintaining the natural habitat and scenic spots at the southeast of Hong Kong Island, Stanley STW was built inside three large caverns, each about 120m long, 15m wide and 17m high. In addition, a tunnel-like structure with total length around 450m was constructed for vehicular access, ventilation shaft and effluent discharge (Fig. 3). This compact design can optimize the construction cost as well as the land requirement.

The design and layout of the STW has taken into account of the geometry of long caverns. Furthermore, due consideration was made in the design stage in minimizing footprint required thus civil construction cost.

Stanley STW is a secondary treatment plant adopting the Modified Ludzack-Ettinger (MLE) process, where extended aeration is adopted for the removal of ammonia and organic matters (Fig. 4). The extended aeration increases the retention time and oxygen supply so that secondary sludge is digested before pumped to the dewatering facilities. This would reduce the sludge treatment and odour treatment facilities, hence footprint and excavation works. Different from

traditional secondary sewage treatment plants outdoor which contain digesters for sludge treatment and gas tank for biogas collection, Stanley STW applies solely filter press for sludge dewatering, striking a balance between sewage treatment and good use of land resources.

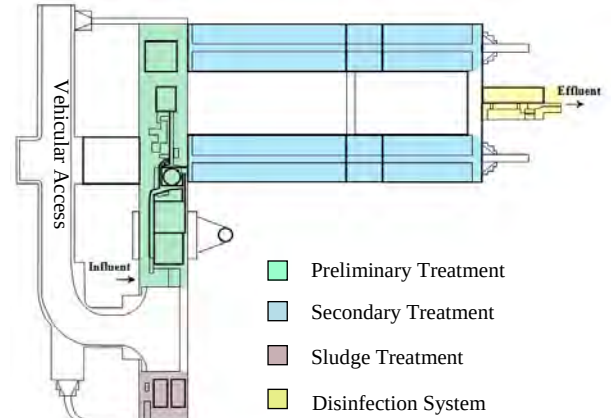


Fig. 3 Layout of the Caverns and STW

Stanley STW is a secondary treatment plant adopting the MLE process, where extended aeration is adopted for the removal of ammonia and organic matters. The extended aeration increases the retention time and oxygen supply so that secondary sludge is digested before pumped to the dewatering facilities. This would reduce the sludge treatment and odour treatment facilities, hence footprint and excavation works. Different from traditional secondary sewage treatment plants outdoor which contain digesters for sludge treatment and gas tank for biogas collection, Stanley STW applies solely filter press for sludge dewatering, striking a balance between sewage treatment and good use of land resources.

In addition, lamella settlers are applied in the final sedimentation so as to increase the surface area for settling of suspended solids in limited space (Fig. 5). Lastly, the disinfection tank for effluent is designed with zigzag flow, where dosing of sodium hypochlorite solution applies graded multipoint technique. This can meet the requirement in de-chlorination without the construction of additional chemical dosing channel (Fig. 6).



Fig. 4 Overview of the Aeration Tank



Fig. 5 Lamella Settlers in Final Sedimentation Tank



Fig. 6 Overview of the Disinfection System

### 3.2 Ventilation and Gas Detection System

The fundamental of the ventilation design in Stanley STW is to provide adequate air changes for the cavern space, not only for operators' need but also for the heat dissipation of various electrical and mechanical installations, particularly the air blowers for aeration. Most importantly, flammable gas like methane which would be generated in the treatment process has to be diluted before reaching its lower explosive limit.

The ventilation requirement is fulfilled by a system consisting of two nos. of tunnel intake fans, eight nos. of exhaust air axial fans and nine nos. of intake air axial fans (Fig. 7 and 8). All fans are operated round the clock so as to achieve 6 air changes per hour in total under normal circumstance. The exhaust air is discharged through a 65 m height ventilation shaft to the atmosphere. When the gas detection system detects the level of toxic gas inside caverns being over the threshold limits, the ventilation fans will run automatically at higher speed to attain 12 air changes per hour. The high speed operation of the fans will remain until the level of toxic gas returns to normal. In

addition, the ventilation system in Stanley STW is linked to the fire control panel, where all fresh air fans would be shut down when there is a fire incident.



Fig. 7 Fresh Air Inlets



Fig. 8 Exhaust Air Louver

The gas detection system, which is interacted with the ventilation fans, consists of methane sensors, hydrogen sulphide sensors, carbon dioxide sensors and carbon monoxide sensors.

Thirty nos. of methane sensors are mounted at ceiling level, locating in the grit removal area, aeration and final sedimentation area, polymer dosing area and sludge loading area. Ten nos. of hydrogen sulphide sensors are mounted at skirting level at the inlet works, coarse / fine screens, grit removal and hypochlorite storage and dosing area. Twenty nos. of carbon dioxide sensors are mounted at skirting level, for inlet works, coarse/fine screens, grid removal, aeration and final sedimentation area. Lastly, fifteen nos. of carbon monoxide sensors are mounted at human nose level in the car parking area.

### 3.3 Smoke Detection System

Eleven nos. of aspirating smoke detectors, which draw air through air sampling pipes, are adopted to detect

any smoke in the caverns. The 20mm diameter UPVC air sampling tubes are mounted at the ceiling of the caverns in the form of a network which is efficient and effective for smoke detection in cavern structure (see Figure 9 and the red arrow). The aspirating smoke detectors are highly sensitive devices, which are capable of giving early alarm to operating staff in the caverns whenever excessive smoke particles are detected. All smoke detectors are linked to the main fire control panel in the hydrant pump room and repeater panel in the control room.



Fig. 9 Typical Example of Smoke Detection System

### 3.4 Extended Use of Overhead Crane

Different from the sewage treatment facilities built in the outdoor areas, the head room inside caverns is limited for setting up further mounting structure. In view of the above, overhead cranes are installed along the caverns, covering the inlet works and treatment areas (Fig. 10). This feature is crucial for the maintenance staff to conduct preventive and corrective maintenance of plant equipment.



Fig. 10 Use of Overhead Crane in Service Area

### 3.5 Fire Prevention and Fighting System

Fire hazard is undoubtedly a prime concern in caverns. To address this issue, fire prevention and fighting

systems are installed. As the entrance portal is the only vehicular road entering the STW and for manoeuvring of emergency vehicles, the portal entrance and road access tunnel are protected by sprinkler system (Fig. 11) and the fire pumps are located near the access route. Adequate fire hydrants are located at the portal entrance, along road access tunnel and at the treatment and services areas for fire fighting purpose.



Fig. 11 Sprinkler System at Entrance Portal

To protect the operators and visitors inside the STW, emergency fire exits are located at both ends of the caverns leading to the open space near the Stanley Prison. As required by the Fire Services Department (FSD), the number of visitors is limited to 20.

## 4. User Experiences

### 4.1 Operator Training and Emergency Preparedness

To enhance the emergency preparedness of operator and staff, contingency plan is prepared and kept in the Stanley STW. The plan serves to provide operators with guidelines and reference for action taken under different emergency or abnormal conditions, which may relate to toxic gas, fire, chemical spill, power failure and sewage overflow. The contingency plan is updated as and when required aiming at safeguarding the public health, protecting the health and safety of the operational staff, preventing the loss or damage of the sewage treatment properties, and maintaining continuous sewage treatment process.

Drills are scheduled and conducted annually. Different scenarios are planned and team representatives would take the lead to go through the drill with all participants. Joint drills with FSD such as emergency rescue at skip loading area are also conducted regularly by resident staff and FSD rescue team. All drill records are documented.

### 4.2 Stability of the Sewage Treatment Process

As the entire sewage treatment process is situated inside the caverns, the external factors such as

temperature, ultra-violet light and heavy rainstorm will not affect the performance of the treatment process.

During winter season, the water temperature is generally low. This condition may promote the growth of filamentous bacteria which produces extracellular polymer substance that causes foaming. Foaming will affect the performance of the sewage treatment process, and cause clogging of pipework and odour problem. As the water temperature inside caverns is relatively stable and stays warm enough during winter, foaming is less prominent in Stanley STW.

Other factors such as ultraviolet and heavy rainstorm may affect the performance of the microorganisms and the hydraulic capacity of a STW in open space. Once again this can be avoided in the Stanley STW.

#### **4.3 Operators' Feedback**

Operators generally feel positive in work in caverns as they do not need to work under the scorching sunlight and heavy rain. Landscaping and odour nuisance are of lower priority in their work. However, some concern about the monotonous working environment, i.e. without the change in the daylight.

Despite the higher humidity inside caverns, the equipment breakdown patterns are similar to other above ground treatment plants. It is probably due to the slightly cooler air inside the caverns and also equipment is shielded away from the direct sunlight. Occasional water dripping and minor rubble fall were observed in the STW at its early stage of operation. Remedial works have been carried out and the situation is well improved.

There are concerns on the space constraints in caverns when replacing major and large equipment. As the inflow is increasing, the need to have more spare capacity for maintenance and overhaul of the equipment also increases. The upgrading of the STW is very difficult as the footprint is limited by the size of the caverns. Subsequent excavation work is considered to be expensive. Notwithstanding this, a recent trial of a hybrid sewage treatment process in Stanley STW proves promising and the process capacity of the aeration tank has been increased without extra footprint.

#### **4.4 Periodic Geotechnical Survey**

Besides maintenance of plant equipment, structural maintenance is one of the regular practices in Stanley STW. Independent surveyor is employed to conduct tape extensometer monitoring for caverns annually. Twelve dedicated locations are selected for periodic inspection to detect and monitor changes in the clearance between two reference points on walls (Fig. 12).



Fig. 12 Conduction of Geotechnical Survey

### **5. Conclusion**

Stanley STW symbolizes successful experience of building and operating sewage treatment infrastructures in cavern space. Hong Kong is scarce in land resources, enhancing more use of caverns will support Hong Kong's pursuit of sustainable development.



# THE FIRST SERVICE RESERVOIR IN A CAVERN

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## Abstract

*The University of Hong Kong (HKU) proposed the development of a Centennial Campus at a site to the west of the Main Campus to celebrate its centenary in 2011 and to prepare for the implementation of the four-year undergraduate degree curriculum in 2012. As the site west of the Main Campus was occupied by fresh water and salt water service reservoirs and associated waterworks facilities managed by the Water Supplies Department (WSD) of HKSAR, it required the re-provisioning of these waterworks facilities and associated infrastructure works prior to development of the campus. This paper discusses the setting of the site, the development of the adopted scheme, contract procurement and the inclusion of the user's requirements in the construction contract.*

## Keywords

Service Reservoir, Caverns, Design

## 1. Introduction

The University of Hong Kong (HKU) has been developing its new campus in response to the new academic structure for senior secondary and higher education known as the "3+3+4" reform. To achieve this goal, HKU must expand and acquire space to cater for the additional undergraduate intake. The intake of the first cohort of the new curriculum undergraduates will take place in 2012 and the new campus must be completed in time to meet this deadline.

Given that HKU is situated in the hilly and developed area of Central & Western District where availability of suitable land for developing its new campus is scarce, the site selection stage of the project had already posed a significant challenge for the University. Following a feasibility study, HKU identified that the area to the west of its Main Campus was the most suitable location for its expansion. The proposed area was zoned as 'Government, Institution or Community (GIC)' where 'Education Institution' is always permitted.

As the site was occupied by the then existing fresh water and salt water service reservoirs and associated waterworks facilities managed by the Water Supplies Department (WSD), it required the re-provisioning of these facilities and associated infrastructure prior to the development of facilities for the Centennial Campus.

An Agreement was finally reached between HKU and WSD whereby WSD would surrender a major portion of the site for the existing service reservoirs to the Government on the condition that HKU would re-provide the existing waterworks facilities in the existing site all at his own cost and that the existing water supply systems would be maintained during the re-provisioning works. HKU would seek separate agreement with the Government to grant the necessary land to HKU to extend its campus. Figure 1 shows the location of the proposed HKU Centennial Campus.



Fig. 1 Proposed Centennial Campus of the University of Hong Kong (Source: HKU Centennial Campus website <http://www.hku.hk/cecampus/>)

The following sections describe the site setting, the development of the adopted scheme, the contract procurement and the inclusion of the user's requirements in the construction contract.

## 2. Original Schemes

During the Preliminary Project Feasibility Study (PPFS) stage between 2003 and 2005, various options for the re-provisioning of waterworks facilities and the subsequent development of campus facilities were considered. The options included the construction of a decked structure covering the existing or re-provided waterworks facilities with the future campus development above the decked structure, the re-provision of the service reservoirs on the existing slopes along Pokfulam Road, and an open-cut site formation option within the Lung Fu Shan hillside to provide land for the construction of replacement fresh water service reservoirs.

Amongst the abovementioned options, the open-cut site formation option was initially selected as the preferred option. This would have comprised the construction of replacement fresh water service reservoirs on the slopes at the southern portion of the site adjacent to the existing salt water service reservoirs. This option would have involved massive excavation, amounting to about 280,000 m<sup>3</sup> in volume, and felling of about 6,000 trees to provide space for the construction of the service reservoirs. The cut slope would have been mainly in rock and about 60m high. This scheme would not only be environmentally unfriendly, it would also likely face strong objection from the nearby residents due to the possible nuisance caused during construction and the disturbance to the natural vegetation at the slopes. The layout plan and section of the open-cut scheme are shown in Figure 2.

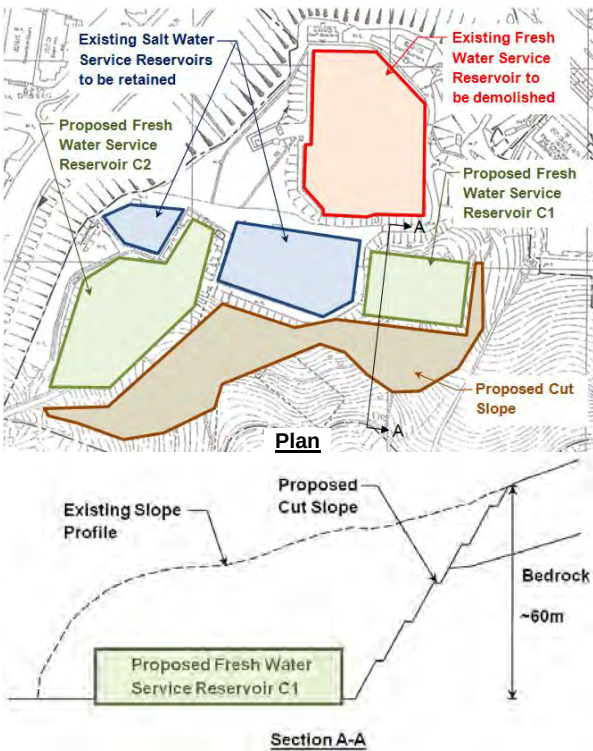


Fig. 2 Plan and Section showing the Original Scheme for the Re-provision of Waterworks facilities

### 3. Alternative Scheme

Following the investigation phase a design and construction consultancy assignment for the infrastructure was awarded to another consultant in December 2005 and a thorough review of the proposed scheme was carried out. Prompted by WSD's initiative of building salt water service reservoirs inside caverns, Geotechnical Engineering Office (GEO) and Civil Engineering and Development Department (CEDD)'s support for the utilization of underground space for housing public facilities, and the increasing awareness and expectations of the community and HKU regarding the incorporation of the concept of sustainability in

new developments, the proposal of re-provisioning the salt water service reservoirs inside caverns to be formed inside Lung Fu Shan hillside came up. This proposal is considered as an innovative, sustainable, and environmentally friendly solution. By housing the proposed salt water service reservoirs in caverns, the amount of excavation in soil and rock could be reduced to about one seventh of that in the original scheme. Moreover, substantial areas of woodland including about 6,000 m<sup>2</sup> of secondary woodland at the southern portion of the site could be saved.

### 4. Overall Project Programme

The placing of a salt water service reservoir in a cavern is unprecedented in Hong Kong. Immediately after the inception of the concept, the project team took action to turn the concept into reality. Discussions commenced in Jan 2006 with stakeholders, including the Employer, HKU (on the financial, procurement, land and programme aspects), the Operator and Owner, WSD (on the feasibility and operation of housing service reservoirs in a cavern), and the Checking Authority, GEO (on the feasibility of constructing the caverns at the proposed location). Other authorities/government departments, including District Lands Office (DLO), Fire Services Department (FSD), Agriculture, Fisheries and Conservation Department (AFCD), Drainage Services Department (DSD), and Environmental Protection Department (EPD), were also involved in the process of the design development.

The completion of the new campus for a student intake in 2012 meant that HKU would have less than 7 years to bring the Centennial Campus from concept to fruition. The development programme was extremely tight and key dates for milestone activities for housing the reservoir in the cavern were agreed by the project team as follows:

| <u>Milestone Activities</u> | <u>Key Dates</u> |
|-----------------------------|------------------|
| Inception of concept        | Jan 2006         |
| Confirmation on feasibility | Aug/Sep 2006     |
| Tendering                   | Sep/Oct 2006     |
| Commencement                | April 2007       |
| Completion                  | Sep 2009         |

### 5. Procurement Approach (Design and Build Contract)

In view of the extremely tight project programme and in anticipation of the complexity of the innovative cavern scheme for the re-provisioning works, the Employer decided at the outset to adopt a Design and Build procurement approach instead of the conventional approach. By doing so, some advanced construction works could be commenced in parallel with the development of the design, thereby shortening the overall project duration.

## 6. Finalization of the Scheme for the Re-provisioning of the Waterworks

Following the award of the Design and Build Contract, an early and effective amalgamation of a partnering team comprising HKU, WSD, the Engineering Consultant and the Contractor (after awarding the Contract) was achieved to tackle and resolve technical difficulties through their collective knowledge and expertise. This was essential to ensure that the project would be completed on time and within budget.

Through the collaboration and the dedication of the efforts of all team members, the final scheme for the re-provisioning of the waterworks was confirmed as shown in Figure 3 in a relatively short time.



Fig. 3 A General Layout Plan Showing the Final Scheme for the Re-provision of Waterworks Facilities

The scope of works involved the following major elements:

- (i) Two fresh water service reservoirs with a total capacity of 26,500 m<sup>3</sup>;
- (ii) Two salt water service reservoirs with a total capacity of 12,000 m<sup>3</sup>, to be located in two rock caverns, and an access tunnel;
- (iii) Construction of a pipe gallery housing future WSD utilities;
- (iv) Other associated works, such as a vehicular access, slope upgrading, water main diversions and protection of historic buildings.

The major requirements from stakeholders with respect to construction of waterworks facilities in the cavern, and how these requirements were met to achieve feasibility of the scheme are outlined below.

### 7. Employer's (HKU) Requirements

This innovative scheme is an environmentally friendly proposal which did not alter the existing landform and saved thousands of trees. HKU welcomed this scheme as it was in line with the guiding principles for the Campus development. The main concerns from HKU were whether the project could be completed on time

and within budget and that future operation and maintenance activities were not jeopardised

#### 7.1 Budget

According to the estimate prepared by the project quantity surveyor, the revised project cost was comparable or less than the originally accepted scheme and could therefore be readily endorsed by HKU senior management.

#### 7.2 Programme

Objections from relevant Government Departments and the public about the large number of trees that would have to be removed under the original open-cut scheme would likely cause significant delay of the project. With the innovative cavern scheme, it should be well received by the public and Government Departments, thereby increasing the chance of completing the works on time. Further, the footprint of the proposed facilities is carefully designed to ensure that it will not encroach into the nearby Country Park areas in order not to prolong the process of land acquisition. Even though this requirement imposed major constraints on the layout and design of the caverns, the probability of achieving the project programme is increased.

#### 7.3 Operation and Maintenance

To facilitate future operation and maintenance, it is necessary to have a clear land and maintenance demarcation between HKU and WSD. Also, the campus operation must not be adversely affected by possible breakdowns of waterworks facilities, such as watermain burst in the cavern.

The design of the new facilities was carefully considered to eliminate interfaces between HKU and WSD. For example, the pipe gallery was designed to allow WSD staff to inspect the watermain and carry out maintenance works inside the pipe gallery without disturbance to HKU.

## 8. Owner and Operator's (WSD) Requirements

Only the aspects associated with housing waterworks facilities in cavern are addressed. As there is limited experience of operating waterworks facilities in caverns, it was important to prepare operational procedures and agree these requirements with the operator at the outset. These operational procedures had major bearings on the layout of the reservoirs and the E&M systems as described below.

### 8.1 Ventilation

WSD operators had to gain access to the caverns on a regular basis to carry out routine inspection and

maintenance works under a safe environment. In order to avoid operators having to enter into the cavern with special gear, it was necessary to provide a sufficient natural or/and mechanical ventilation system. Natural ventilation had been considered but was finally ruled out due to architectural and environmental constraints. A mechanical ventilation was finally adopted.

Agreement on the air exchange and air quality requirements had to be established as this would have major cost and time implications for the system installation. The ventilation system adopted for the cavern is a simple “pull system”. Two fans have been installed at the top of the tunnel portal house, extracting air from the cavern. Fresh air is then forced into the tunnel due to the differential air pressure.

A concrete air vent was constructed underneath the entire access road to the front wall of the salt water service reservoirs. A vertical concrete shaft is then connected to the air vent underneath the pedestrian walkway above the salt water service reservoirs. Exhaust air louvers are provided at both sides of the air vent walls allowing air to be extracted from the cavern. A schematic plan of the ventilation system is shown in Figure 4.

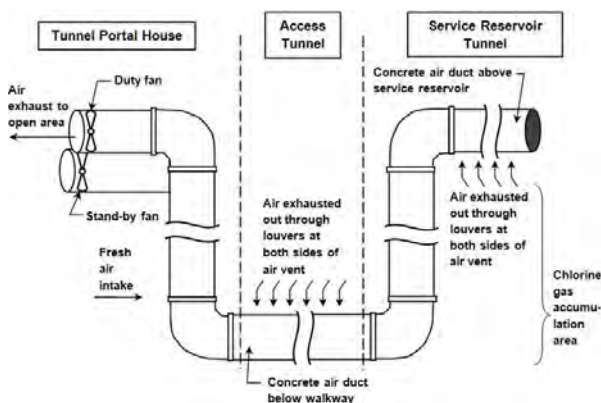


Fig. 4 Schematic Plan of Ventilation System

## 8.2 Construction Materials inside the Cavern

As the cavern would be a fairly corrosive environment due to the presence of salt water, concrete air ducts were adopted as against metals to minimize future maintenance costs. In addition, in order to eliminate fire hazard within the cavern, all cables employed were low combustible material to limit the fire risks.

## 8.3 Alarm Systems

In order to safeguard operators working within the cavern, a sophisticated alarm system is installed within the cavern. The alarm system basically consists of a series of gas detectors, such as chlorine and oxygen detectors, and smoke detectors, which are integrated with the ventilation system. If the prescribed values of any detectors are exceeded, it will trigger the operation

of the ventilation system, which could be operated remotely and manually at the cavern. A CCTV system was also installed to enhance the scrutinization of the operation of the caverns.

All metering or valves should be housed outside the cavern as far as possible and similarly the switchgear, Fire Services (FS), telemetry, Supervisory Control and Data Acquisition (SCADA), ventilation equipment, generator, and flow meter rooms were all to be located outside the cavern for easy access.

## 8.4 Layout of the Salt Water Service Reservoir in Caverns

The fact that the footprint of the caverns should not encroach into the Country Park areas became a major constraint in the layout design of the salt water service reservoirs. In the end, a twin tunnels under Lung Fu Shan was selected to be an optimum solution. Not only it avoids encroaching into the Country Park, it serves the operation requirement that the service reservoir in one tunnel could suspend in service for maintenance purpose while the other could maintain a continuous water supply.

In addition, the layout was carefully planned to avoid affecting the Grade III historic building situated in the middle of the site. Even though the layout was compromised to a certain extent, it fulfilled HKU/WSD mission of achieving an environmentally friendly design. The layout strikes a balance between future operation and maintenance of the waterworks facilities, effective land use, and the ground conditions of the tunnel site.

## 8.5 Salt Water Service Reservoir Structure

The two salt water service reservoirs were housed within the large span rock caverns. In order to fulfill the requirement of a watertight structure as per the operation requirement, the cavern permanent linings were designed as water retaining structures. Waterproofing layers were also laid on the external face to avoid ingress of water into the salt water service reservoirs. The service reservoir structures are monolithic reinforced concrete structures to reduce the length of joints as far as possible. With all these mitigation measures implemented, it was anticipated that leakage of ground water into the cavern would be insignificant and any possible contamination of the water could be avoided.

The arch roof above the service reservoir structure comprises a steel fibre reinforced shotcrete lining with a plain shotcrete smoothing layer. This shotcrete arch roof was constructed at the time of the tunnel header excavation and rested on an elephant foot in sound rock before full completion of the tunnel permanent lining. The permanent lining was designed for the

drained case incorporating a groundwater relief system. A typical section of the salt water service reservoir cavern is shown in Figure 5.

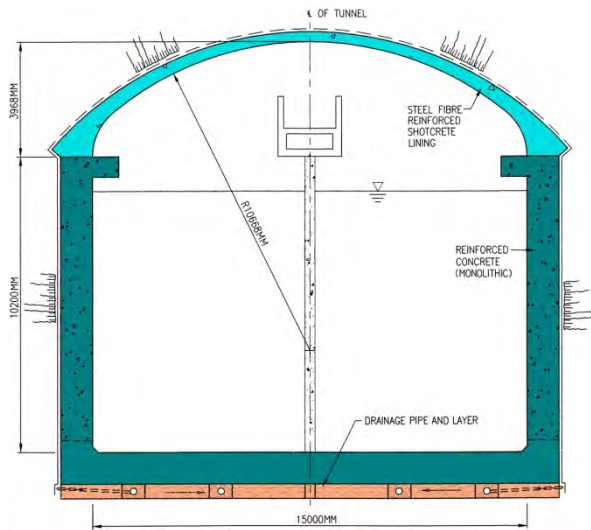


Fig. 5 Salt Water Service Reservoir Layout

### 8.6 Pipe Gallery

The pipe gallery is used to house the incoming and outgoing pipes for both proposed fresh water and salt water service reservoirs. Lifting facilities and lighting are provided inside the gallery to facilitate routine inspection and maintenance by WSD in the future. As the inspection and maintenance works would be carried out inside the pipe gallery, it would avoid the interference of the normal activities of the HKU Campus.

The pipe gallery also acts as a drainage channel capable of draining away a large quantity of water in the rare case of a burst pipe in the tunnel/cavern or pipe gallery. A typical section through the pipe gallery is shown in Figure 6.

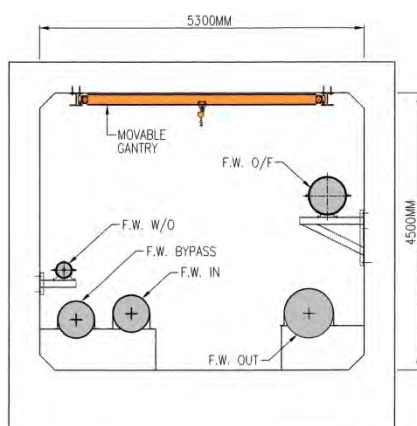


Fig. 6 Typical Section and Pipeline Arrangement in the Pipe Gallery

In order to facilitate the future inspection/maintenance of the watermain within the pipe gallery, the pressurised watermain were placed at the lowest level while those without pressure were placed at higher levels.

### 8.7 Fire Safety

As there was only one entrance/exit for the cavern, one of WSD's concerns was whether sufficient fire safety/escape measures were provided for the future operation. In this connection, a comprehensive Fire Engineer Report was prepared by the consultant to address the issue and was subsequently accepted by FSD. The report covered a qualitative fire engineering assessment of the cavern and highlighted a series of recommendations for the fire safety provisions and the future operation of the cavern.

The qualitative assessment included a review of the fire hazard of each component of the cavern, the nature of the usage of the cavern, the fire protection system to be provided and the management system to be implemented during operation. The report concluded that the fire risk for the cavern was low given that the cavern was basically constructed with concrete with low fire load, the cavern was used for water storage without any dangerous materials and sufficient protection systems like ventilation, gas detectors, lighting, alarm systems, etc were provided.

As regards the future operation, a series of control measures was recommended during inspection and maintenance works within the cavern. Other than the operation of the basic protection systems like ventilation and gas detectors, these measures included the introduction of a pre-entry screening procedure, regular maintenance and housekeeping and the carrying out of a risk assessment for fire hazards if major/extensive works would be undertaken.

## 9. GEO Requirements

GEO publications Technical Guidance Notes TGN 24 and 25, which provide guidelines on site investigation and risk management associated with cavern design and construction.

### 9.1 Additional Ground Investigations

During the inception stage, additional ground investigation works were promptly designed and tendered according to TGN 24 requirements. Horizontal drillholes through the full length of the caverns at crown and invert levels, and vertical drillholes at critical locations, were required to provide the ground conditions at the proposed cavern locations to confirm the feasibility during design, construction and final stages.

## 10. Access Tunnel

Starting from the cavern portal, an access tunnel with an overall span of 8.6 m and height of 8 m was excavated into Lung Fu Shan at a level of about +85 mPD. This access tunnel is about 40 m in length running parallel to the existing open cut slope in a southeast direction. The access tunnel runs via an enlargement transition zone of 10 m in length towards one of the caverns which had a 17.6 m span and is 50 m in length. Another tunnel branches off from the access tunnel at about 28 m from the portal and drives towards the southwest with the same dimensions of the enlargement transition zone and cavern. At the enlargement transition zone, the cavern/tunnel progressively increases in height from 8 m to 17 m and in span from 9.6 m to 17.6 m.

## 11. Permanent Works for the Access Tunnel and Enlargement Transitions

To suit the ground conditions and to avoid excessive groundwater drawdown due to the tunnel works, the first 15 m of the access tunnel were designed as an undrained tunnel, with the remaining portion designed as a drained tunnel. The finished internal width and height of the access tunnel after construction of the lining are both 7.2 m. The principles of the design and construction of the enlargement transitions are similar to those for the drained access tunnel.

The undrained portion of the tunnel was designed to withstand full hydrostatic pressure and was surrounded by waterproofing membrane. The permanent lining was constructed using traditional reinforced concrete methods excluding the upper circular arch roof. The arch roof for the access tunnel was constructed using steel fibre reinforced shotcrete with plain shotcrete as the smoothing layer after completion of the lower portion of the permanent lining.

The drained portion of the cavern was designed with a groundwater relief system. Drain holes of 50mm diameter and 1m length were drilled and proprietary drainage strips were installed to facilitate groundwater drainage. A drainage composite was also installed with the drainage strip to further enhance drainage performance. The groundwater drained into a perforated drainage pipe surrounded by a drainage layer below the tunnel invert. Other than the incorporation of the groundwater drainage system, the construction of the permanent lining for the drained tunnel was similar to the undrained tunnel.

Ventilation ducts are located at a low level to facilitate the extraction of any possible chlorine gas (which is heavier than air) that might be released from the salt water. The mains are purposely laid on the two sides of the access roads to facilitate future inspection and

maintenance. A typical arrangement of the access tunnel is shown in Figure 7.

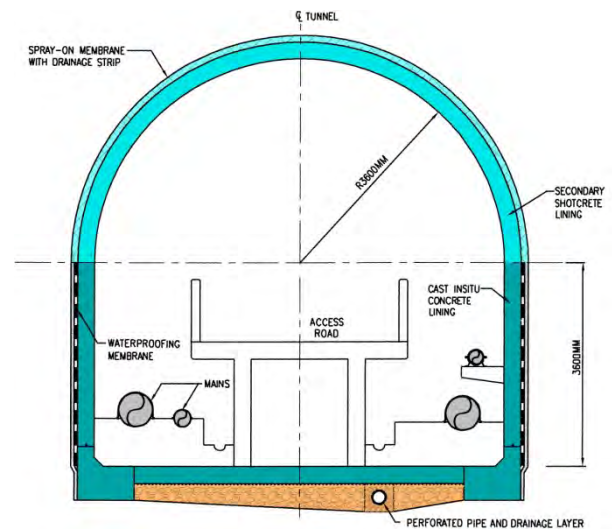


Fig. 7 Access Tunnel Layout

## 12. Conclusion

The planning, design and construction of the Re-provision of waterworks facilities was a vivid demonstration of successful integration of a partnering team comprising HKU, WSD, Government Departments, the Engineering Consultant and the Contractor. With the innovative design for housing salt water service reservoirs inside caverns, not only it achieves an environmentally friendly project, it was also completed on time and within budget to allow the timely commencement of construction of the new HKU campus.

## Acknowledgements

The authors wish to express their sincere thanks to the University of Hong Kong for their kind permission to publish this paper.

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# THE UNIVERSITY OF HONG KONG SALT WATER STORAGE CAVERNS – AN OVERVIEW OF DESIGN AND CONSTRUCTABILITY CONSIDERATIONS

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## Abstract

*To build the Centennial Campus of the University of Hong Kong, two caverns were formed to accommodate salt water service reservoirs. The caverns were located on Lung Fu Shan hillside where complex geology, comprising bedded tuffs, geological faults and irregular rock head posed challenges to the excavation works. The construction programme of the caverns was challenging, as excavation was to use mechanical methods in order to minimise disturbance to the surroundings such as nearby residents, the university operation and the ecology and stability of the natural hillside.*

*To construct caverns safely required a high level of engagement between the designers and frontline production teams. An example on this project was the production team and designers jointly developing a solution using “elephant foot” to key the permanent cavern crown lining in the top heading allowing the emission of temporary vertical posts that were originally designed to support the crown lining. This increased working space for workers and reduced the risk of accidental machine impacts.*

*To complete the caverns on time, it was necessary to plan carefully the construction methodology and carried out detailed risk assessment identifying potential hazard and mitigation measures. An effective control system was set up for monitoring of the instrumentation, personnel working in the caverns, and other temporary works.*

## Keywords

Rock Cavern, Constructability, Safety by Design, Tunnel Intersection, Løset's Formula, Localised Weak Zone

## 1. Introduction

The University of Hong Kong (HKU) is developing a Centennial Campus at a site west of the Main Campus. The site was previously occupied by fresh water and salt water service reservoirs that were managed by the Water Supplies Department (WSD), Government of the HKSAR. The associated waterworks facilities, such as Western Senior Staff Quarters, Western Staff Quarters, and Elliot Treatment Works, built around the

1920s were graded historic buildings and are to be preserved.

Prior to construction of the Centennial Campus, it was necessary to re-provision the water service reservoirs. Two new salt water service reservoirs were relocated in rock caverns beneath the adjacent Lung Fu Shan hillside. Chung et al [2008] presented the background and the development process of the re-provision scheme using caverns and its benefits over the original open cut proposal.

The layout of the two Salt Water Service Reservoirs (SWSR) caverns beneath Lung Fu Shan is shown in Figure 1. The construction of the caverns comprised an 8.4 m access tunnel approximately 62 m long connecting to two transition tunnels with increasing span from 8.4 to 17.5 m over 10 m length. These lead to two reservoir caverns 17.5 m wide and 50m long. Typical sections of the access tunnel and the SWSR caverns are shown in Figure 2.

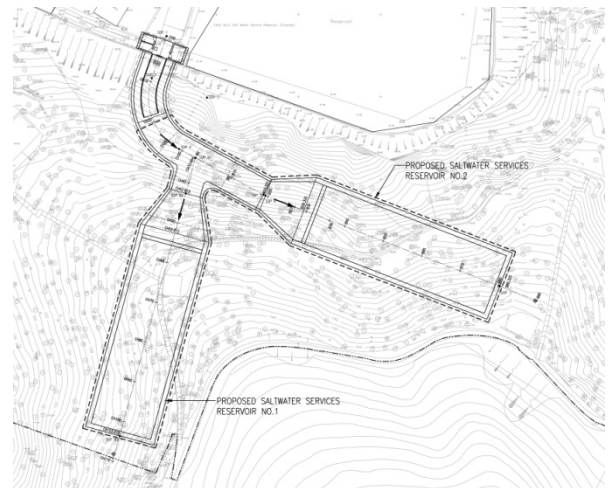


Fig. 1 Layout of SWSR Caverns

The cavern permanent roof lining thickness was 400 – 500 mm, and the wall lining was 1100 mm. At the crown of the cavern drainage layer comprised of 500 mm drainage strip was provided at a spacing of 4 m. Drainage geotextile layer and waterproofing membrane were installed along the cavern wall directing groundwater seepage to the drainage layer at the cavern invert.

This paper describes the temporary support for the SWSR caverns, including the challenging ground

conditions and construction constraints. Other topics highlighted include: careful planning of the excavation sequences; risk assessment and works control that were implemented in forming the caverns.

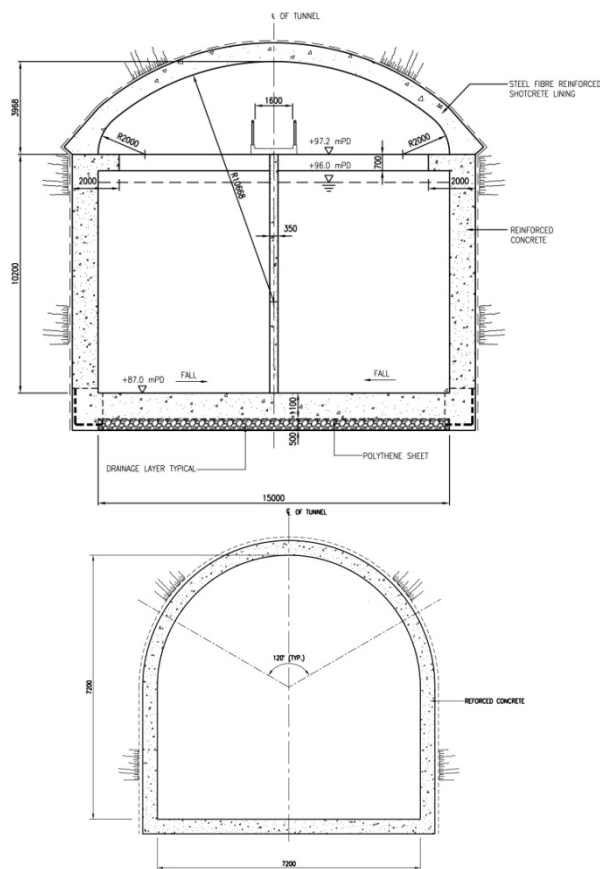


Fig. 2 Typical Sections of SWSR Cavern and the Access Tunnel

## 2. Ground Conditions

The caverns are located within the Mount Davis Formation of the Repulse Bay Volcanic Group, comprising generally coarse ash tuff with siltstone and sandstone inclusions. The major geological structures in the area include the Sandy Bay Fault and a major syncline with an axis perpendicular to the fault. In addition the Kowloon Granite pluton has intruded the area. Further details of the geological features of the site are described by [MACKAY, 2009], which include

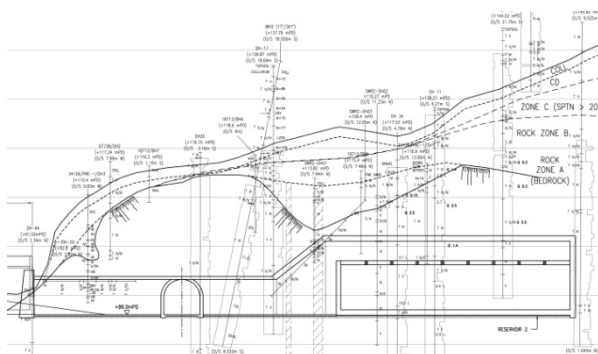


Fig. 3 Longitudinal Geological Section

findings from the ground investigation consisting horizontal, inclined and vertical drill-holes with in-situ packer tests and rock laboratory testing. Figure 3 and 4 shows a longitudinal and a cross section of the caverns.

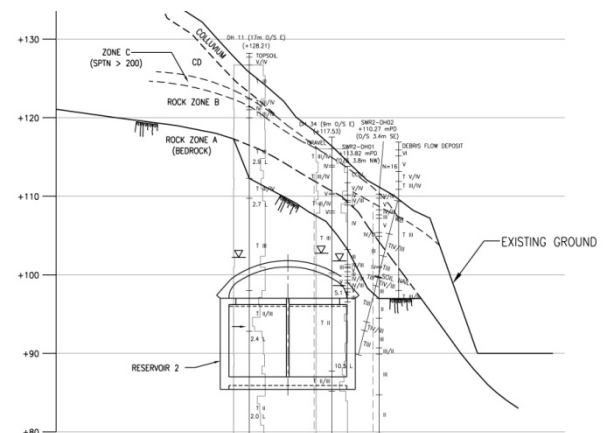


Fig. 4 Cross Geological Section

## 3. Constructions and Safety

TOH [2010] highlighted the importance of safety by design to improve the safety performance of deep excavation. Safety in construction is often presumed to mostly concern the wearing of the Personal Protective Equipment by construction workers. People make mistakes and it is necessary to have several layers of defence to ensure that these mistakes do not translate into accidents. [ASKEW, 2011] presented a robust safety management system that has several layers of protection with an accident only occurring when all of these layers are breached (Fig. 5). The first layer of protection is the design that begins at the engineer's "drawing board."

It is expected that the excavation progress of the SWSR and temporary support design would be reviewed as part of an interactive design process during the excavation. Therefore close engagement between designers and frontline production team was paramount.

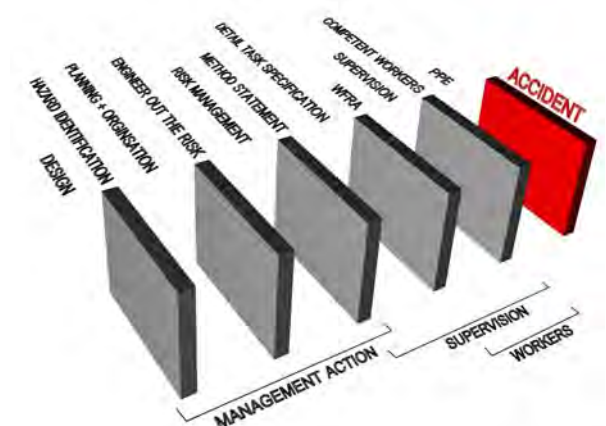


Fig. 5 Layers of Protection

### 3.1 Construction Methodology

To allow early construction of the SWSR tunnel portal, the existing slope immediately above and adjacent to the portal location was upgraded using soil nails typically at 2 m grids. Pre-support and support frames were designed for areas with limited rock cover extending to the first 30 m from the tunnel portal.

Excavation of the access tunnel and the SWSR caverns was by mechanical method to prevent any adverse effect on the stability of the natural hill slopes above the caverns, and to minimise disturbance to the surrounding community. Hydraulic breakers mounted on a 30 ton backhoe were used. Drill and rock-splitting techniques using a drilling jumbo were adopted when the rock UCS exceeded 200 MPa. The excavation sequence was heading and bench (Fig. 6).

In the access tunnel the average excavation rate was 30 m<sup>3</sup> per day. The daily works from 7 am to 7 pm comprised excavation, pre-support installation, support frames erection and shotcrete. A 24-hr works permit was granted for construction inside the caverns. The average excavation rate in the heading stage was 130 – 160 m<sup>3</sup> per day. At the benching stage a higher average rate of 200 m<sup>3</sup> per day was achieved.

Ground verifications using probing and continuous mapping of the exposed tunnel face were carried out. Probes included horizontal probing 10m ahead of the excavation face. These were inclined in the tunnel crown generally at 30° from horizontal and up to a projected target of at least half of the tunnel span above the crown. Geological mapping was carried out after each round of excavation to determine the Q-value of the rock mass and design the temporary support.

The correlation of rock strength and the rate of penetration of the probe were first established. Figure 7 shows the correlation chart using a percussive pressure of 180 Bar and drill bit diameter of 64 mm.

To control groundwater inflow, grouting was specified if the rate of inflow exceeded 10 litres per minute from an individual source. Groundwater inflow into the

tunnel was then measured from the advanced probes and from a designated point at the tunnel excavation. During the works groundwater inflow was less than the limit specified and no grouting was carried out.

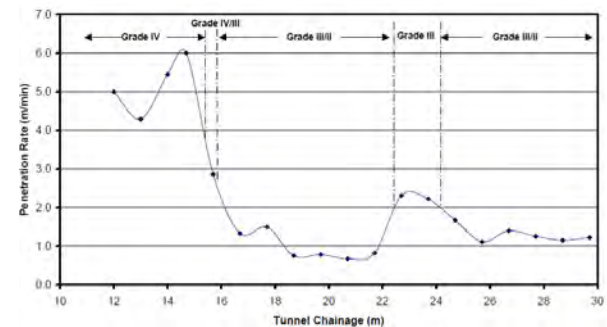


Fig. 7 Correlation of Rock Strength and Penetration Rate

### 3.2 Effective Control Management

To verify the behaviour of the tunnel and caverns and control the effects on the surrounding slopes and structures, a system of instrumentation monitoring was set up. Convergence arrays were installed typically at 10 and 15 m intervals depending on the support category and rock mass class with 5 array-points for the access tunnel and 7 array-points for the SWSR caverns. The monitoring frequency was set to give early warning of potential hazards at critical stages of the excavation such as daily monitoring when excavation was within 15 m from the arrays and weekly monitoring thereafter until the completion of the permanent lining. The convergence measured was less than 5mm, which was within the trigger level of 0.1% of design span.

To further enhance safety in the caverns construction, an engineering management system was established as part of the safety policy of the contractor to achieve zero harm in construction. The system enabled all elements of the temporary works such as tunnel support, steel arches, design changes or working platform to be predefined, design delivered and certified, built, endorsed for loading and removal and regularly checked and inspected.

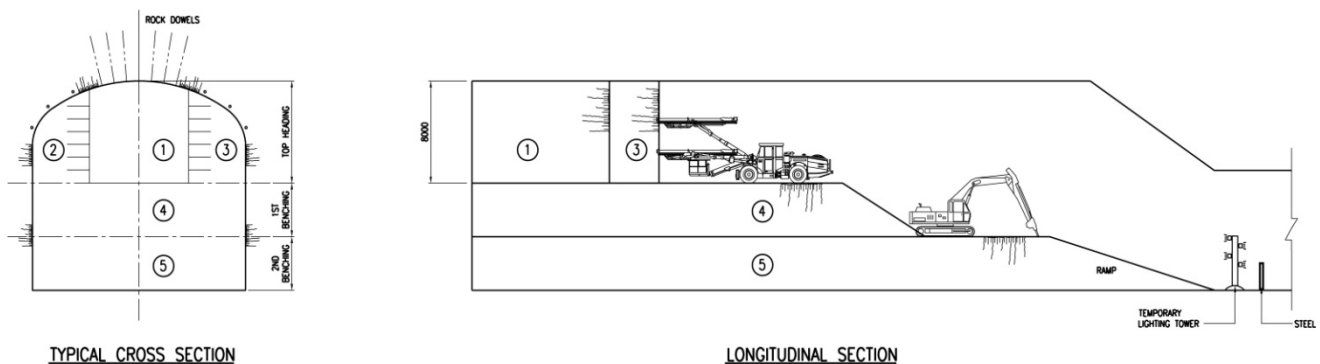


Fig. 6 SWSR Cavern Construction Sequence

In addition, a risk assessment management system was implemented to identify construction hazards and design risks, assessing the consequence of the hazards, setting up mitigation measures to reduce the impact of the hazards if they occurred, and identifying competent person to manage the risks during construction. Risk assessment review meetings were conducted regularly including participants from project managers, site engineer, site agents, supervisors, frontline production teams and designers of the temporary works.

In the event of unexpected hazards occurring during cavern construction it is important to identify personnel in the tunnel. Traditionally tally system is used for this purpose with registration and distributing tally tags. In the SWSR caverns, an electronic tally system was set up using hand-recognition and electronic display.

#### 4. Temporary Support Design

The design of the temporary support was based on the Norwegian Geotechnical Institute's (NGI) Rock Mass Quality or 'Q-System' [BARTON, 2002]. The minimum support requirement consisting of systematic dowels, shotcrete and steel fibre reinforcement as determined by the rock mass quality.

##### 4.1 Rock Cover Conditions

The geology in the caverns comprised irregular rock head that would reduce the rock head cover above the cavern crown. Three cases of rock cover were considered in the design of the temporary support. A generalisation of the temporary support applied for each rock cover cases is presented in Figure 10. In the Limited Rock Cover (LRC) cases pre-support comprising either 60 mm steel pipe or 32 mm reinforcement bar were installed to form a canopy prior to excavation, and steel support frames were erected to provide internal support where necessary as excavation

proceeded.

Probing was carried out in advance to verify the rock cover, and it allowed early planning for the support works specifically for the condition where steel support frames or lattice girders were required.

##### 4.2 Weak Zones

Numerous persistent localised weak zones, identified from the ground investigation and verified by probing, were expected to intersect the tunnel. The standard Q-value was supplemented with Løset's formula [LØSET, 1990] to estimate a mean Q-value for an enhance support spanning across the weak zone (Fig. 9). In the design, rigorous soil-interaction finite element modeling was carried out to verify the support design (Fig. 10). Details of this innovative design approach have been presented by [MACKAY et al, 2008].

During the excavation of the cavern aligned parallel to the existing hill slope, weak zones of partial weathering were identified from the probing. Additional ground investigation was carried on the slope to determine the extent (Fig. 4). Apart from the temporary support, the stability of the existing slope required careful consideration. Numerical finite element models were set up to assess the stability of both the slope due to the cavern excavation (Fig. 10) and the temporary support. Temporary support based on the Løset's formula was modeled. The maximum deformation at the weak zone was predicted to be 7 mm at the completion of the top heading. The measured displacement from the convergence points was about 3 mm.

This design approach for the localised weak zone was successfully implemented and found to improve constructability and safety compared to the steel ribs that would otherwise be required.

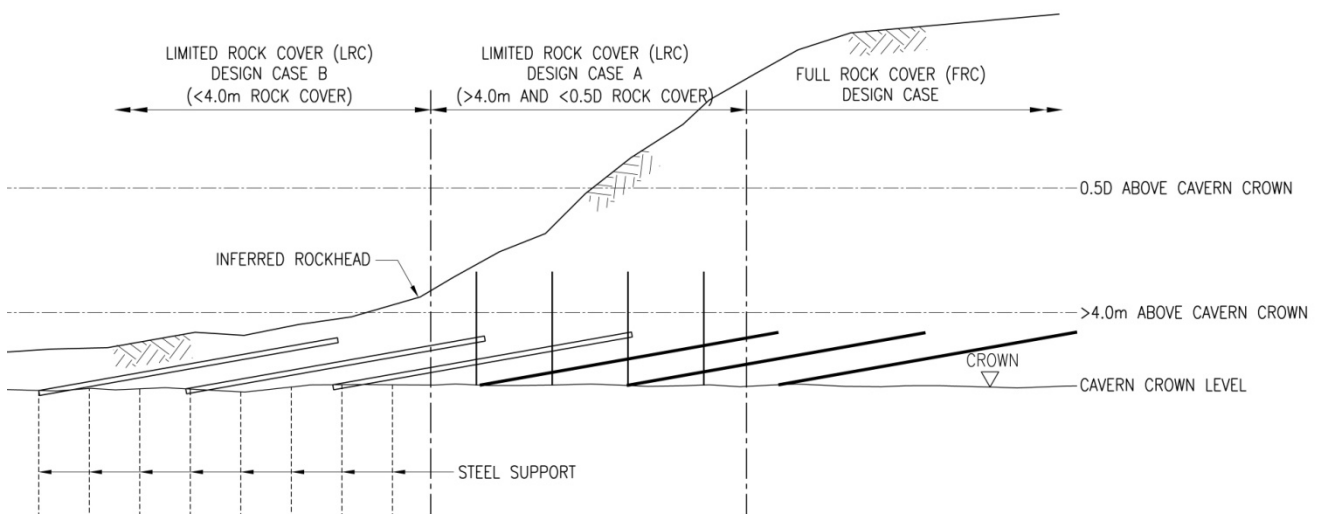


Fig. 8 Support for Different Rock Cover Cases

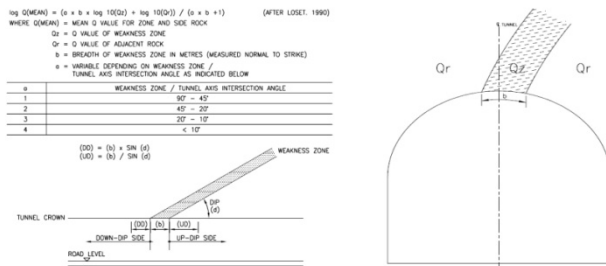


Fig. 9 Support for Localised Weak Zone (Løset)

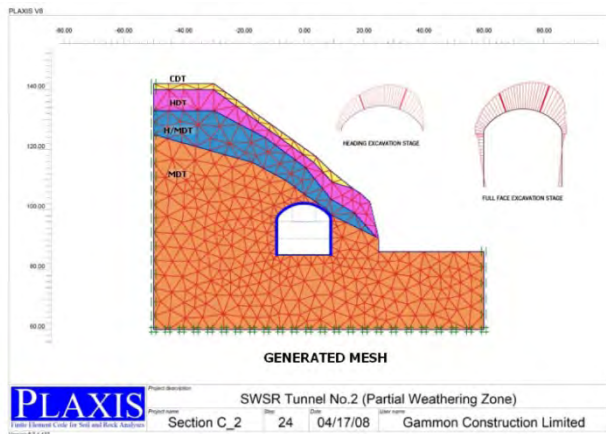


Fig. 10 Numerical Finite Element Model

### 4.3 Intersection Zone

The Q-value estimated from the initial GI showed a high degree of variability. At the intersection where the second cavern branched from the access tunnel, it was necessary to plan the excavation sequence early and to carry out rigorous assessments for the support design and to check the “pillar” stability. The site-specific correlation (refer to Figure 11) between the rock point load test (PLT) strength and its Unconfined Compressive Strength (UCS) was established from the initial ground investigation. A rock strength of 20 MPa was used to assess the stress condition. Finite element modeling was undertaken to verify the result. In the design of the support at the intersection, the Stress Reduction Factor (SRF) was taken as 2.5 and a  $J_n$  (joint set number) x 3 factor was applied to determine the Q-value.

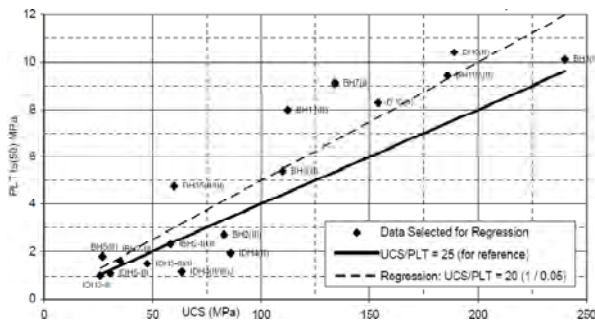


Fig. 11 UCS and PLT Correlation

During the excavation work, advance probing was carried out to assess the rock quality and rock samples at the intersection area (prior to the excavation of the second cavern) were taken to determine the PLT, and hence, verify the design assumptions of the rock stress condition.

### 4.4 Elephant Foot Support

In the initial design it was planned to cast the permanent crown lining with the temporary support at the top heading stage of the SWSR caverns to benefit construction safety and programme. The permanent crown lining would then be supported at the sides on temporary steel posts erected in stages during bench excavation. Subsequently the permanent wall lining would be installed in phases while removing the steel post.

An “elephant foot” support was used where the permanent crown lining was keyed into the sidewall. This eliminated the steel posts (refer to Figure 12). This was particularly beneficial since the installation and removal of the steel posts are both costly, time consuming and hazardous.

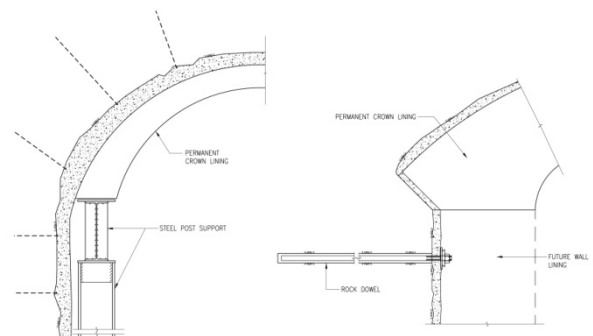


Fig. 12 Vertical Steel Post vs. Elephant Leg Support

## 5. Conclusion

Complex geology would pose hazards to cavern excavation works. The experience from the SWSR project demonstrated that the engagement between temporary support designers and frontline production team was vital in completing the caverns safely.

With the geological details from the ground investigation, comprehensive design of the temporary supports was carried out for various conditions such as different rock cover cases and variation of rock quality at the intersection zone. Site verification consisting of advance probing, monitoring of groundwater inflow and instrumentation was imperative. Weak zones were identified hence rigorous analysis combined with the application of the Løset's formula was needed to confirm the support design. Output, from close working relationship between designers and the front

line production team, including “elephant foot” support detail made the project easier to build safely.

Furthermore, the paper outlines the importance of effective implementation of control procedures spanning instrumentation, risk management and temporary works control for cavern construction.

### Acknowledgement

This paper is published with the permission of the University of Hong Kong, Water Supplies Department, the Government of the HKSAR and Gammon Construction Limited.

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