Mined Tunnel Construction using Artificial Ground Freezing Technique for HATS 2A Project

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ABSTRACT

This article presents a case of using Artificial Ground Freezing (AGF) as the soil improvement method in soft ground for mined tunnel construction in Harbour Area Treatment Scheme Stage 2A (HATS 2A) project. In Hong Kong, AGF technique is not a common soil improvement approach. There are only very few local cases and experiences on this technology. There is also no local design guidance for AGF. This article summarises the design and construction considerations of the application of AGF in Hong Kong. Laboratory testing results of frozen soil strength are discussed. The consideration of strength parameter selection and design approach adopted for the mined tunnel are also presented.

1 INTRODUCTION

The Harbour Area Treatment Scheme Stage 2A (HATS 2A) comprises a 3.9m internal diameter and 28m deep Interconnection Tunnel to connect the existing and the new pumping stations. The tunnel consists of two parts, with Part A (236m long) to be built by an Earth Pressure Balanced type Tunnel Boring Machine and Part B (14m long) to be built by hand mining method. The mined tunnel will connect the new Launching Shaft to the existing Riser Shaft (Figure 1). To facilitate the mined tunnel construction, AGF was selected as the soil improvement method to form a 2m thick frozen soil ring in Alluvium and Marine Deposits as the temporary support for tunnel excavation.

![Figure 1: Tunnel Layout Plan](image)

2 APPLICATION OF ARTIFICIAL GROUND FREEZING IN HONG KONG

There were only very few local cases reporting the application of AGF in Hong Kong. The Kowloon Canton Railway Corporation Lok Ma Chau Spurline project adopted AGF for the construction of three crosspassages
(Storry et al. 2005). Horizontal ground freezing was carried out in completely decomposed volcanic and intact volcanic rock using brine solution system.

AGF was also applied in Harbour Area Treatment Scheme Stage 1 for launching of a 1.8m diameter pipe jacked tunnel in Kwun Tong (Pakianathan et al. 2002). The depth of the tunnel launching level was 22m and a frozen block was formed in marine deposits by vertical freezing pipes. Liquid nitrogen was used as the coolant.

Mass Transit Railway Corporation West Island Land adopted ground freezing for the construction of a 26m long tunnel for obstruction removal.

In HATS 2A, vertical ground freezing was also carried out to form a 2.5m thick frozen block for TBM launching break-through for Part A of the Interconnection Tunnel.

3 REVIEW OF GEOTECHNICAL DESIGN CONSIDERATIONS

The formation of the ice block is the key consideration for the AGF technology in the feasibility assessment. The factors include the soil water content and groundwater flow rate at the freezing location. In general, minimum water content of 10% is required to bond the soil particles (Harris 1995). Laboratory test results by Enokido & Kameta (1987) for river sand indicated that UCS of 6 to 7MPa could be developed with water content at only 5% at -30°C. The testing results on sand by Kuribayashi et al. (1985), as shown in Figure 2, showed a strong relationship between the soil water content and the strength of the frozen soil. For Part B of the Interconnection Tunnel, the soil to be frozen is fully submerged and the water contents of the in-situ soil are all larger than 20%.

With regard to groundwater flow rate, it is difficult to form a continuous frozen soil if the water flow velocity is larger than 1 to 2m/day for brine solution system (Anderson 2004). PRC code of practice DG/TJ08-902-2006 (SUCCC 2006) recommended that detailed investigation should be carried out when groundwater flow rate is larger than 5m/day. However, liquid nitrogen system could tolerate a much larger water flow rate. Shuster (1972) reported that ice block was able to form where groundwater flow was as high as 50m/day for liquid nitrogen system. The estimated groundwater flow at this mined tunnel site is only 0.12m/day and therefore brine solution system can be applicable.

Strength and deformation behaviours depend on the ice content, unfrozen water content, air content and original soil structure of the frozen soil. Temperature has direct effect on the strength of the frozen soil because of its influence on the amount of unfrozen water. Laboratory test from Bourbonnais and Ladanyi (1985a, 1985b) indicated that strength of frozen sand increased sharply with decreasing temperature to about -40°C but tended to level off at about -100°C. Very different trend was observed for overconsolidated clay, on which the strength exponentially increased when temperature was below -60°C. The strength of frozen clay could exceed frozen sand when the temperature was sufficiently low.
Because of the creep property of the ice, frozen soil also creeps under loading. When subject to loading, the frozen soil would keep deforming for a certain period of time. Eventually, strain of the sample may reach a constant value, if the stress level is low, or failure may occurs, if the stress level is high. In general, the long-term strength of frozen soil is approximately 40% to 60% of the instantaneous strength (Schultz & Hass 2011).

4 TESTING OF FROZEN SOIL

The package of tests carried out for frozen soil included UCS test, creep test, frost heave/thaw consolidation test, freezing temperature test, thermal conductivity test and heat capacity test. Determination of strength parameters for temporary support design relies on the results of UCS test and creep test. PRC standards MT/T 593.4 and MT/T 593.6 (Ministry of Coal Industry of PRC 1996) were adopted as the testing standards for UCS tests and creep tests respectively. As the design temperature is -15°C, the samples were tested at -10°C, -15°C and -20°C to establish the temperature and strength relationship. For UCS test, the samples were first frozen to the test temperature and then subject to increasing loading under a constant strain rate of 1%/min. It is essential to control the strain rate during the test because the strength of frozen soil is strongly influenced by the strain rate applied. In this article, the instantaneous UCS value \( q_i \) is defined as maximum stress attained or stress at 20% axial strain, whichever is obtained first during the performance of a test. The UCS test results are presented in Figure 3 below.

With regard to creep tests, the frozen soil samples were subject to a constant loading at 0.7, 0.5, 0.4 and 0.3 of the instantaneous UCS value. The termination criteria of the tests are failure of the frozen sample or 24 hours after the strain rate of the sample is less than 0.0005 h. Where the latter criterion is met, the sample is considered to be able to sustain the corresponding load for sufficient long period of time without failure. A typical strain and time relationship for frozen Marine Deposits is shown in the Figure 4.

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![Figure 3: UCS Testing Results](image)

![Figure 4: Relationship of Creep Strain Rate and Time under Creep Test](image)
5 SELECTION OF DESIGN STRENGTH PARAMETERS

In selecting the design parameters, the creep property is a key consideration. The instantaneous strength of frozen soil was considered not suitable for design. Based on some published results (Fish 1991, Sheng 1997), approximately 50% of strength reduction from instantaneous strength would happen in less than 24 hours.

It was considered that strain rate is also a critical consideration for selecting design parameters. Typical stress strain relationship for Marine Deposits was shown in the Figure 5. It can be seen from the plot that although the stress level kept increasing, the strain level significantly increased when the stress is beyond 2MPa. It was felt that the sample was effectively yielded at this stress level and as such it was adopted as the yield strength.

![Figure 5: Typical Stress Strain behaviour of Marine Deposits in UCS test](image)

To account for the creep effect, the design creep UCS, i.e. long-term UCS, was taken as half of the yield strength, which is equivalent to 40% of the instantaneous UCS. It was considered that the design strength adopted had properly addressed the concern of excessive deformation and at the same time fell within the reasonable range of strength reduction from instantaneous UCS. The design strength parameters adopted are presented in Table 1.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Lab test result</th>
<th>Adopted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Instantaneous</td>
<td>Yield</td>
</tr>
<tr>
<td></td>
<td>UCS test</td>
<td>strength</td>
</tr>
<tr>
<td>Marine Deposits</td>
<td>2.8 (no creep failure)</td>
<td>2</td>
</tr>
<tr>
<td>Alluvium</td>
<td>4.7 (no creep failure)</td>
<td>3.5</td>
</tr>
</tbody>
</table>

1. The determination of instantaneous UCS is based on a moderately conservative fit line on data at test temperature -10°C, -15°C, -20°C as shown in Figure 3. Although moderately conservative fit line was adopted, sufficient safety margin was allowed for the sample strength at -15°C.

2. All samples passed the test at stress 0.5q, but no sample pass the test at 0.7q in the creep test.

The stability of the tunnel construction was assessed using finite element method. The conventional factoring approach for tunnel design would have an overall FOS of 2.1 contributed by load factor of 1.4 and material factor on structural member of 1.5. However, this approach was considered not able to cater the situation when the stability of the tunnel relies heavily or solely on the strength of frozen ring, because there is no factor applied onto the frozen soil. As such, a partial material factor of 2.0 was applied onto the design creep UCS, together with load factor and material factor on structural member. Face support was also considered in the analysis. To cater for the uncertainties related to the contribution of face support, the design needed to tolerate a relaxation factor of 0.3 to 0.7.
6 CONSTRUCTION CONSIDERATIONS

The temporary support of the tunnel comprises a 2m thick frozen ring together with steel rib installed every 600mm. The frozen soil ring would be formed by two rows of freezing pipes surrounding the tunnel using brine solution system. The layout design of the freezing pipes was based on the contractor’s experience and PRC code of practice DG/TJ08-902-2006 (SUCCC 2006). Thermal analysis was carried out to further verify the freezing pipe layout design and estimate the active freezing period. Thermal couples would be installed to monitor the temperature of the frozen soil. The layout of the thermal couples would be carefully arranged such that the temperature gradient along the radial direction of the tunnel could be clearly revealed. The temporary support and horizontal freezing pipes layout are shown in Figure 6 below.

![Figure 6: Tunnel Temporary Support and Horizontal Freezing Pipe Layout](image)

Generally, for horizontal freezing, the gap between the end tip of the horizontal freezing pipe and the surface of the shaft at far end is the weakest location for water leakage. In the present design, two rows of the freezing pipes would be installed from opposite directions to ensure a better water cut-off at this gap. As the accuracy of the position of the freezing pipe would affect the formation of the frozen ring, the allowable out of position for each pipe is 200mm. Where this limit is exceeded, additional freezing pipe is required. The design of the freezing pipe is shown in Figure 7. Brine solution will be brought to the tip of the freezing pipe by an inner tube and made contact with the outer casing along the full length of the pipe to effect freezing.

![Figure 7: Horizontal Freezing Pipe Design](image)
The estimated active freezing duration is 60 days. During this period, the water pressure within the frozen ring will increase and therefore pressure relief holes would be installed. The pressure relief holes could also serve as a means of checking the formation of ring closure. When water pressure inside the frozen ring increases, it can be inferred that a confined frozen ring is forming.

Excavation can commence when the readings at the thermal couples indicate the temperature of the frozen ring has reached the design requirement. The advance length is 600mm and steel rib would be installed for each advance length. Steel plate would be installed between steel rib and thermal insulator would be mounted onto the steel plate. Cast in-situ permanent lining would be installed after the tunnel is excavated through.

7 CONCLUSION

The AGF design presented in this article had gone through careful consideration on the fundamental mechanical properties of frozen soil. The construction method presented incorporated some recognised good practice from overseas. It was felt that the parameters selection and factoring approach adopted are sufficiently safe, but further study is needed to assign an appropriate factor to cater the uncertainties related to AGF. As the AGF technology becomes a more readily available soil improvement method in the Hong Kong market, there is a need to develop more compatible construction practice and a unified local design approach which is both suitable for local environment and compatible with other design standards in Hong Kong.

REFERENCES


Harris, J.S. 1995. Chapter 4, In Ground Freezing in Practice: 85. Thomas Telford, The Institution of Civil Engineers.


