Western Geotechnical Research Centre

Geotechnical Research Centre
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The University of Western Ontario
Research Report GEOT-10-05

Evaluation Of Swelling Properties Of Shales For The Design Of Underground Structures

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Evaluation of Swelling Properties of Shales for the Design of Underground Structures

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1. INTRODUCTION

For the design and construction of underground structures in swelling rocks, it is important to understand the time-dependent deformation behaviour and develop the stress-strain-time relationships for the analysis of rock-structure-time interaction problem. Over the past 30 years, extensive researches were carried out to investigate the causes and mechanism of swelling behaviour of shales of different geological formations at various sites in Ontario and the adjoining United States [1,2,3,4,5]. These researches have led to establishment of a methodology for the design of underground structures in these regions. The methodology comprises a systematic process based on laboratory testing , analysis and design. The methodology has been used for design in many projects [e.g. 6,7,8, and 9]. The experience gained in these projects are discussed in this paper.

The results of long term swell tests of different shales and shaly limestones in various geological formations are critically reviewed and summarized. It was found that the behaviour of these rocks generally follow the swelling mechanism proposed by Lee and Lo (1993). The amount and rate of swelling are governed by the calcite content, which reflects the cementation bonding, and salt concentration gradient in the pore water towards the ambient fluid. This concept is extended to clarify the relationships between osmotic pressure, bonding and swelling behaviour, and to interpret some previously unexplained experimental observations. Finally, a more systematic and complete protocol of measurements of swell properties of shales is outlined for practical applications to the design of underground structures in these formations.

2. BRIEF SUMMARY OF TEST METHODS AND DESIGN METHODOLOGY

Three methods of laboratory testing for time-dependent deformation of rocks were developed by Lo et al. (1978) [1]; free swell and semi-confined swell tests to measure a swelling potential, and a null swell test to measure a swelling suppression pressure. These methods of testing have been extensively used for design over the past 30 years. Table 1 summarizes some of the shallow and deep shale formations investigated in various projects, ranging from excavations for tall structures to over 600 m depth for the radioactive waste depository.

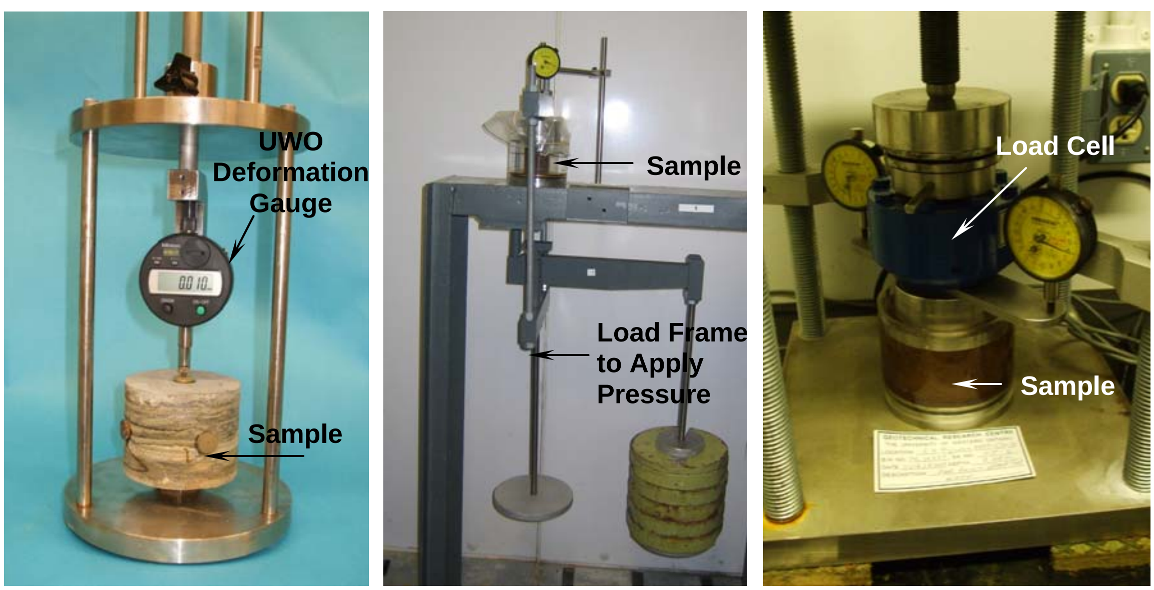
In free swell tests, freshly trimmed rock specimens are permitted to deform unrestricted in all directions. A typical specimen for a free swell test is shown on Fig. 1a. Their orthogonal dimensional changes of the specimen preserved under constant temperature and 100% humidity are measured with time. The “UWO deformation gauge” shown on Fig. 1a is used to measure the dimensions of two horizontal (X and Y) and a vertical (axial) (Z) directions for 100 days.

In semi-confined swell tests, the strain changes of the rock sample in one direction are monitored by the dial gauge reading. A constant pressure is applied to the rock sample in the direction of measurement while deformations in perpendicular directions remained unrestricted. A typical setup for a semi-confined swell test is shown on Fig. 1b. A more complete version of semi-confined swell tests have also been developed [3] in which the swell strains in all three directions were measured.

Table 1. Summary of free swell test results of shale formations from different projects

Shale Formation	Project or Location	Year of Investigation	Depth (m)	Calcite Content (%)	Swelling Potential (%/ log cycle)	
					Vertical	Horizontal
	Bruce Site DGR	2008	483.5-492.0	26.1-68.6	0-0.6	0-0.3
	SABNGS no.3	1984	79.4-121.6	2.1-7.8	-	0.27-0.33
Queenston	SABNGS no.3	1985-1987	95.6-114.3	3.5-8.5	0.37-0.54	0.22-0.34
	Hamilton	1987	11.6-25.4	13.1-27.8	0.02-0.26	0-0.05
	Fletcher's Creek Tunnel	2005	10.8-12.5	5.6-16	0.1-0.18	0.04-0.08
	Geo Terre TG07-079	2008	21-32	2.5-9.2	0.6-1.1	0.1-0.42
	Bruce Site DGR	2008	561.2-578.7	1.3-4.0	1.4-1.5	0.1-0.7
	Scotia Plaza	1984	11.5-27.9	1.8-20.5	-	0.08-0.16
Georgian Bay	Skydome (Roger Centre)	1984	11.3-34.5	2.3-5.1	0.23-0.62	0.08-0.26
	Lakeview Deephole	1986	14-140	1.8-7.6	0.52-1.4	0.1-0.34
	John St. Tunnel	1987	13.5-17.9	1.5-2.9	0.2-0.32	0-0.07
	Island Tunnel	2001	62-98	2.2-3.1	-	0.06-0.22
Blue Mountain	Bruce Site DGR	2008	632.3-642.5	1.6-9.2	0.9-1.0	0.1-0.3
	Lakeview Deephole	1986	177	3.6-18.5	0.9-1.05	0.15-0.16
Shale	Ohio	2009	34-63	<1	0.5-2.5	0.25-0.3
Shale	Kentucky	2008	13-90	1.5-14.2	4.7	0.8-1.7
Porcupine Hill	The Bow, Calgary	2008	18	<1	-	0.24-0.53

Note: Suppression pressures are also measured in some of the formations.



(a) (b) (c)

Figure 1. Swell tests set-ups: (a) free swell test, (b) semi-confine swell test and (c) null swell test

Test data from free swell and semi-confined swell tests are analysed by plotting strain vs. logarithm (to the base of 10) of elapsed time. The slope of the curve between 10 and 100 days is represented by a straight line and is termed the "swelling potential" (e.g. Fig. 2). The swelling potential from free swell tests gives an indication of tendency of the rock to expand upon stress relief. The swelling potential, thus determined, serves as a simple parameter for a preliminary evaluation of the degree of importance of the rock squeeze problem. The results of semi-confined tests characterize the effect of stress on swelling and provide parameters for design analyses [e.g. 10 and 11].

In null swell tests, the critical pressure required to completely suppress swelling in a horizontal or vertical direction is measured. A typical setup is shown in Fig. 1c. The measured stresses are plotted against the time to determine the swelling suppression pressure. The procedure and method of interpretation for the null swell tests have been discussed in Lo (1989) [7] and Lo and Lee (1990) [3].

3. FUNDAMENTAL CONCEPT-EXTENDED

Shales, mudstones and shaly limestones underlie a large area of Southern Ontario and the adjoining United States into which deep excavations and underground structures are constructed. Both laboratory tests and case histories have shown that rocks of these formations expand with time under certain conditions. Detail mineralogical investigations have shown that these rocks contain abundance of clay minerals, but do not contain any significant amount of swelling minerals, or anhydrate and pyrite. The swelling characteristics of the rocks in some of these formations have been studied in detail by Lee and Lo (1993) [4] who postulated that the swelling is a result of the dilution of pore water salt concentration, causing expansion of spaces between clay particles. The process of osmosis and diffusion are the mechanisms of dilution of pore water salt concentration, when the rock is surrounded by an ambient fluid of lower salt concentration. In practice, therefore, the conditions required for swelling are (i) the accessibility to water, and (ii) an outward salt concentration gradient from pore fluid of the rock to the ambient fluid. The relief of in-situ rock stress during excavation serves an initiating mechanism.

The concept discussed in the preceding paragraph serves as the underlying principle of the testing and design methodology described in Section 2. In the past 15 years, more data has been accumulated in different rock formations. It has been found that to interpret the diverse behaviour of swelling, it is necessary to expand the Lee and Lo concept to cover a wider spectrum of swelling behaviour.

The extended concept of swelling may be described as follows:

> In shales and mudstones with a higher salt concentration in the pore fluid than the ambient fluid, the outward gradient generates an osmotic pressure which acts as the driving force for swelling. Swelling is also controlled by the interparticle bonding. If the bond strength exceeds the osmotic pressure, swelling will not occur, and vice-verse. The bond strength is reflected in the calcite content which forms cementation bond between rock particles.

This extended concept provides a linkage between the swelling parameters measured in various methods described in Section 2 and an interpretation of some previously unexplained experimental observations. It should also be noted that in some rock formations, other compounds such as iron oxides and aluminum oxides may act as cementing agents instead of calcite.

4. EXPERIMENTAL ILLUSTRATION OF THE EXTENDED CONCEPT

In this section, test results on several shale and mudstone formations are used to illustrate the interaction of salt concentration gradient and bond strength on the swelling behaviour.

4.1 Effect of Salt Concentration Gradient

An extensive investigation on the swelling behaviour of Queenston shale, a dark grey mudstone from the Queenston Formation, has been performed in connection with the development of hydroelectric power in the Niagara area [4]. The results of series of free swell tests under different salt concentration between the pore fluid and the ambient fluid are shown in Fig. 2. The calcite contents are not much different, suggesting approximately about the same bond strength. It can be seen that the vertical swelling increases with salt concentration gradient.

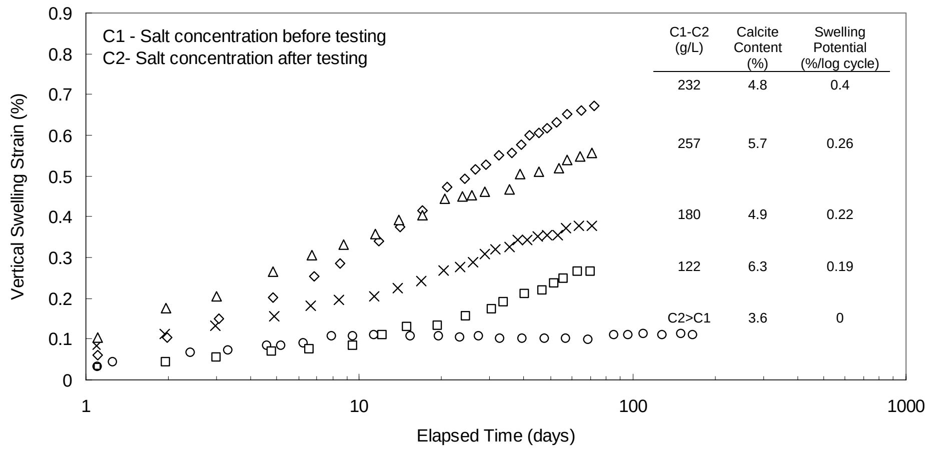


Figure 2. Results of free swell tests under different salt concentration between the pore fluid and the ambient fluid (Queenston shale, Ontario; adopted from Lee and Lo, 1993 [4])

By performing a series of free swell, water content and salinity tests on adjacent samples at different times, the effect of salt concentration gradient on the progress of swelling with time is illustrated in Fig. 3. It can be seen that as the concentration gradient decreases, the rate of swelling also moderates. Ultimately, swelling stops as the concentration gradient decreases to the extent that the osmotic pressure is less than the bond strength.

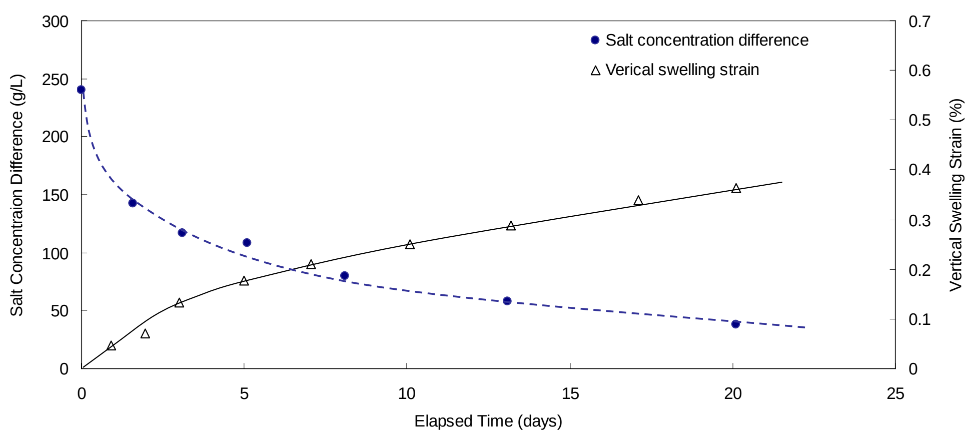


Figure 3. Change in salt concentration and vertical swelling strain with time (Queenston shale, Ontario; after Lee and Lo, 1993 [4])

In most free swell tests, some swelling still occur at the end of test of 100 days. However, when the force of osmotic pressure and bond strength counterbalance within the time of testing swelling may tend to cease. Figure 4 shows the horizontal swelling of a shale, which swelled at a horizontal swelling potential of 1.4% per log cycle of time until 40 days. The rate of swelling then decreases abruptly and almost ceased. This behaviour indicates that at 40 days the osmotic pressure generated by the concentration difference at that stage of testing was balanced by the cementing bonds such that swelling became insignificant.

4.2 The Relationship of Osmotic Pressure and Suppression Pressures

Two methods of measuring suppression pressure has been described in Section 2. In the null swell test, the applied pressure is increased continuously to keep the swelling from occurring at any time. Thus, the suppression pressure balances the osmotic pressure generated by the salt concentration gradient at that instant. In the semi-confined swell tests, a series of specimens is allowed to swell under different pressures, and the swelling potentials measured are plotted against the applied pressure to obtain the suppression pressure. The two methods give very comparable results, while the null swell test yields result at a shorter time. Examples of results of test on a shale from Kentucky is shown in Fig. 5.

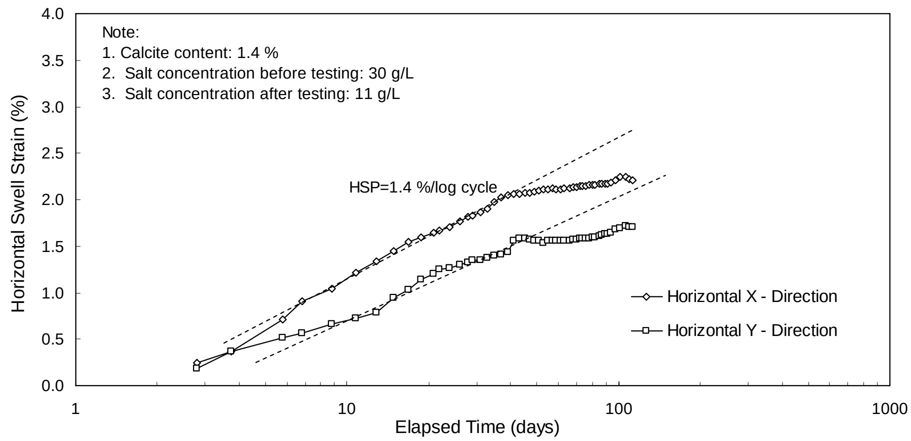


Figure 4. Results of free swell tests (Shale, Kentucky)

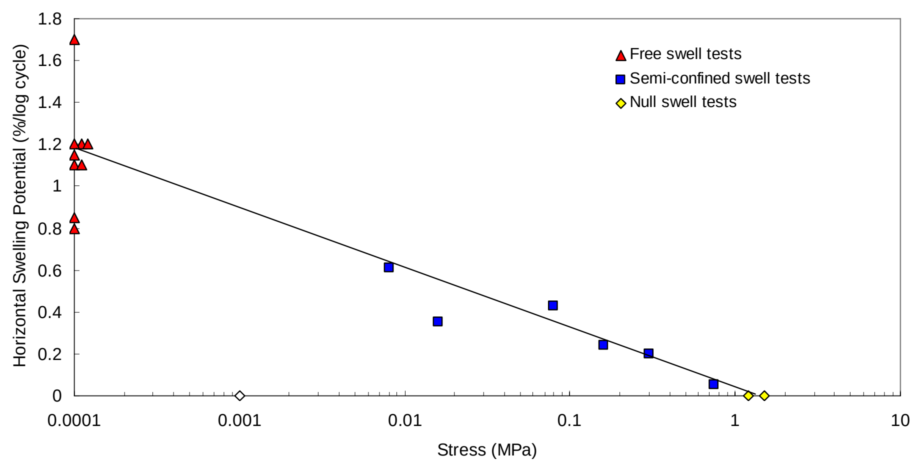


Figure 5. Suppression pressures from results of semi-confined and null swell tests (Shale, Kentucky)

4.3 Suppression Pressure After Long Term Swelling

Many case histories of severe cracking and structural distress of underground structures in shaly rocks have been reported. Some of these structures required rehabilitation. The design of remedial measures would involve the evaluation of the residual swelling potential to which the rehabilitated structure would be subjected to. The following case history, involving a long period of observed performance of 27 years is of interest.

The Heart Lake Tunnel was constructed in Georgian Bay shale during 1973-1975 using three excavation technologies (see Fig.7 and 8). Construction problems and subsequent structural distress of the tunnel in the earlier years were described by Lo, Devata and Yuen (1979) [12] where the results of laboratory tests and field stress measurements in the rock were also reported. The performance of the tunnel was analyzed in detail by Lo and Yuen (1981) [10]. The progress of cracking in the upstream direction was predicted with final extension to approximately St. 123+50 ft (37+64 m) (Fig. 8). The summary of post construction performance of the tunnel based on site visit observations is shown in Table 2. The progressive deterioration in the Drill and Blast (D&B) section, particular under Tomken Road was noted. The propagation of continuous springline cracks upstream in the TBM section are illustrated and updated in Fig. 8. In May 2001 the cracking progressed to approximately 126+00 ft (38+40 m). Therefore, the prediction made in 1981 using the methodology described is satisfactory.

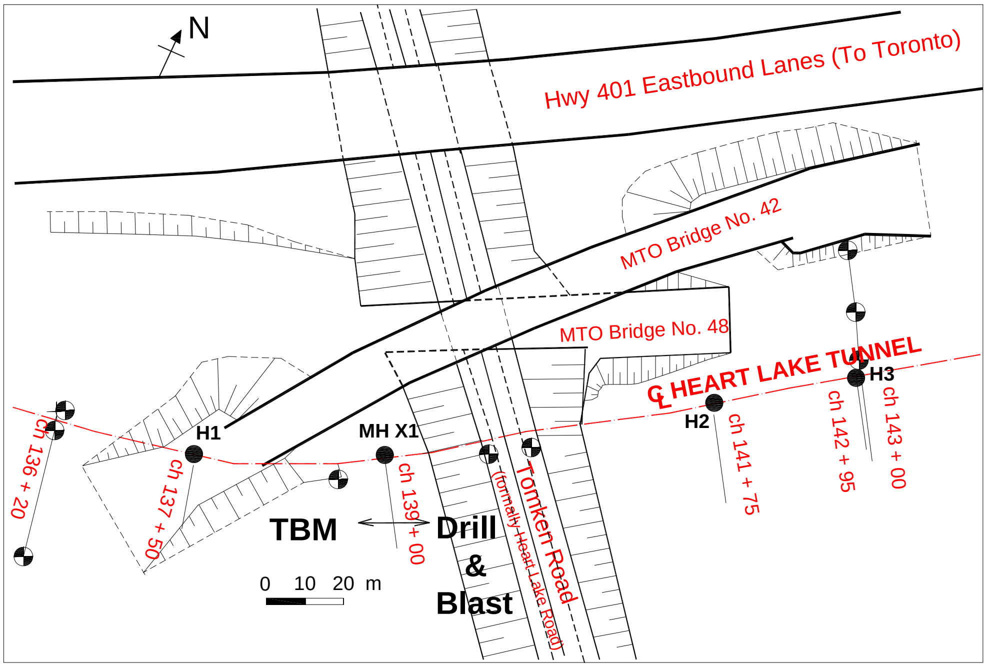


Figure 7. Location of investigated parts of Heart Lake Tunnel

Table 2. Summary of post construction performance (as observed by the first author)

Date of Inspection	Time Elapsed After Completion of Permanent Lining		Chaining Inspected	Principal Observations
	D&B Section (Comp. July 1974)	TBM Section (Comp. April 1975)		
April, 1977	2 years, 9 months	2 years	136+50 to 152+61 (outfall)	Cracking at springline for entire D&B section. No crack in TBM section
Sep., 1977	3 years, 2 months	2 years, 5 months	136+50 to 152+61 (outfall)	Deterioration in section under Heart Lake Road. No crack in TBM section.
Oct., 1977	3 years, 3 months	2 years, 6 months	136+25 (MH X2) to 152+61 (outfall)	Cracking in TBM section first appeared between 135+00 to 138+40.
May, 1978	3 years, 10 months	3 years, 1 months	104+55 (bulkhead) to 152+61 (outfall)	Slight deterioration under Heart Lake Road. Cracking extended slightly. Continuous cracking extended upstream to 128+25.
March, 1983		~8 years		Extended 10 m upstream.
May, 1986		~11 years		Extended to 127+77.
Nov., 1999		24 years, 7 months	Inspected section below Tomken Road (formally Heart Leak Road)	Serious deterioration between MH X1 and C&C section. Continuous crown spalling and crack widening at springline.
May, 2001		~27 years		Continuous serious deterioration in section below Tomken Road. Cracking in C&C and TBM sections (progressed to 126+00).

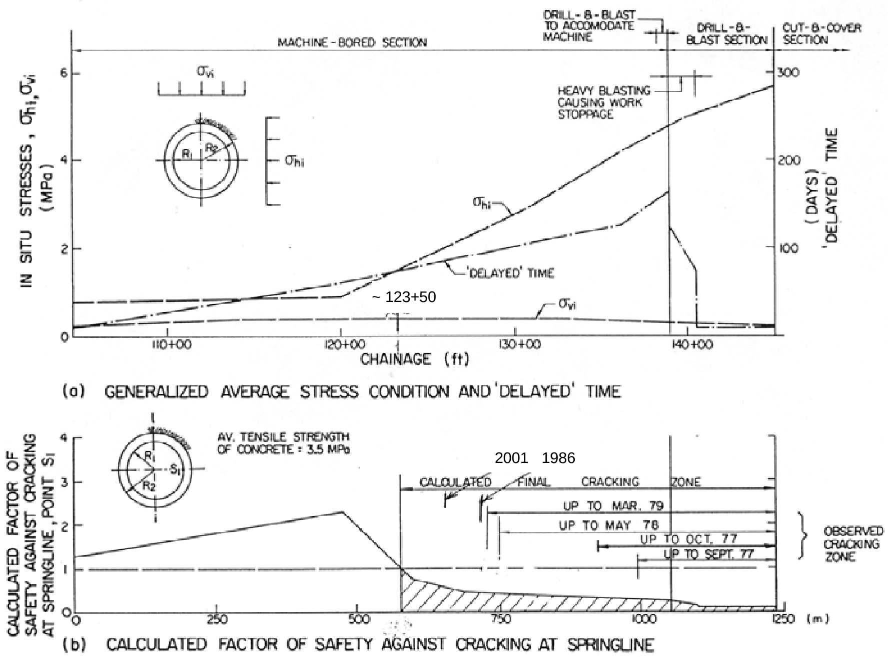


Figure 8. History of cracking along tunnel (Heart Lake Tunnel, Ontario; adopted from Lo and Yuen, 1981 [10])

From the results of investigation, it is clear that the tunnel lining has undergone severe structural distress arising from the swelling of the rock. It is expected that from construction to 2001, the majority of time-dependent deformation occurred. The key information to be obtained for the design of rehabilitation measures is the residual swelling that may take place in the future, and the impact on the selection and design of remedial measures.

As part of investigation and design for the rehabilitation of the tunnel, horizontal holes were drilled and rock samples recovered for swell testing of the rock in 2001. The results of the null swell tests were compared with those obtained during the 1977 investigation in Table 3.

There was no measurement of suppression pressure for the design of the tunnel. However, from results of tests in Georgian Bay shale at other sites, it is likely that the original suppression pressure is about 1.2 to 1.8 MPa. It can be seen from Table 3 that both from the TBM section and the Drill and Blast section, there is a decrease in the suppression pressure after 27 years. The decrease in the Drill and Blast section is tending towards barely measurable value. The behavior is consistent with the concept that with time, the dilution of salt concentration reduced the potential to swell, and thus decreases the suppression pressure. It is also obvious from the photos that the water flow for salt concentration dilution in the Drill and Blast section is more severe. The tunnel was rehabilitated with a design taking into account residual swelling.

Table 3. Summary of swell tests results from the 1977 and 2001 investigations

Tunnel Section	Time After Construction (year)	Distance from Extrado (m)	Horizontal Suppression Pressure (MPa)
TBM	3.5	0.1	0.43
	3.5	1.2	1.2
	27	1.4	0.38
	27	4.0	0.42
Drill and Blast	3.5	2.0	~0.16
	27	3.6	~0.06

5. TEST METHODOLOGY FOR COMPLETE EVALUATION OF SWELLING CHARACTERISTICS

The following test methodology and protocol should be adopted, using well preserved samples from the field:

a. Free swell tests, together with measurements of water content, salt concentration and calcite content. The results of these tests give an assessment of the importance of swelling potential and the horizontal and vertical variations of swelling along the tunnel alignment.
b. Semi-confined swell tests (in series of 4 tests) on selected samples with measurements of water content, salt concentration and calcite content. The results provide swelling parameters for design analysis and the suppression pressure.
c. Null swell tests on selected samples to measure the suppression pressure. The test is usually carried out in cases where few samples can be obtained and information is required quickly. Results are generally available in a few days.
d. X-ray diffraction tests to identify swelling clay minerals, bonding agents and other unusual mineralogical features, in formations where no existing information is available.

6. CONCLUDING REMARKS

In the past 30 years, investigation, design and field monitoring of underground structures in shaly rock formations, as well as analysis of case histories of performance, have indicated the test methodology (Lo et al., 1978 [1]) together with the design approach (Lo and Yuen, 1981 [10]) are satisfactory for design purpose. Accumulated experience indicated that for a more complete evaluation of the possible mechanisms of swelling, additional simple tests should complement the main test methods. This extended test methodology has benefited the investigation and design of underground structures in rock formations where no previous information was available.

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