First Korea-Japan Joint Geotechnical Seminar
October 18-20, Seoul, Korea

TWO TUNNEL PROJECTS IN SWELLING ROCKS

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SUMMARY

This paper describes the importance of incorporating the time-dependent deformation behaviour in the design and construction of tunnels in swelling rocks. Two tunnel projects, in which authors got involved in Canada, are chosen to demonstrate the importance. In diversion tunnels for Oldman River Dam Project, time-dependent deformation characteristics of the mudrocks obtained from test tunnel program were neglected in the design and construction of the tunnels and several sections of concrete lining in tunnels were cracked extensively. In SABNGS No.3 Project, an extensive experimental program was carried out to study time-dependent deformation behaviour of highly swelling Queenston shale, with the aim of establishing the constitutional relationship for the rock-structure time interaction analysis.

INTRODUCTION

It is well established in North America that time-dependent deformation behaviour of swelling rocks is important in the design and construction of underground structures in swelling rocks. Many geological materials exhibit time-dependent deformation behaviour upon excavation of underground openings in these materials, but their behaviours are attributed to different mechanisms; rock salt due to movement of dislocations and intercrystalline gliding, anhydrite due to hydration with volume increase, and shale and marl due to ion concentration differences between the double layer water and the free water[1],[3],[4].

Time-dependent deformations in the from of heave and/or lateral inward movement have been observed in tunnels constructed in different parts of the world; e.g. 30-45 cm of floor heave in tunnels in Europe. Tunnels constructed in rocks exhibiting time-dependent deformation behaviour have been reported to suffer various degrees of distress, resulting in expensive remedial measures in some tunnels. Short and long-term problems in open excavation and tunnels built in swelling rocks have been summarized by Lo and Yuen [7].

This paper describes two different approaches taken for incorporating time-dependent deformation behaviour of swelling rocks (shale and claystone) in the design and construction of tunnel. In one project (Oldman River Dam), displacements of claystone and siltstone layers were measured in the test tunnel using multi-point borehole extensometer (MPBX). Based on the results of time-displacement measurement , it was decided not to take time-dependent deformation behaviour into account for the design and construction of two diversion tunnels. In the other project(Sir Adam Beck Niagara Generating Station, No.3), the importance of the time-dependent deformation behaviour of swelling rock was recognized and an extensive experimental study has been carried out to investigate swelling behaviour of Queenston shale formation in which twin tunnels are to be constructed.

DIVERSION TUNNELS IN OLDMAN RIVER DAM PROJECT

Project description

A reservoir for irrigation and recreational purposes is under construction on the Oldman River located approximately 250 km south of Calgary, Canada. The excavation of twin tunnels were required to bypass the dam construction site prior to the construction of the main dam over the river. Two parallel concrete-lined tunnels are approximately 1.0 km long each (Figure 1). Shapes of both tunnel cross sections are circular and the finished diameter of each tunnel is 6.5 m.

Geology

The Oldman River Dam site area consists of nearly flat lying Tertiary sedimentary rocks mantled by Pleistocene and Recent glacial and alluvial deposits which vary widely in thickness and type. The diversion tunnels were driven in the lower part of the Upper Mudrocks Sequence and the upper part of the Basal Sandstone, both in the Porcupine Hills Formation. The Upper Mudrocks Sequence comprises a sequence of interbedded claystone and siltstones with subordinated sandstones and the thickness of the sequence is greater than 82 m. The Basal Sandstone is a uniform massive, fine to medium grained, cemented sandstone with occasional partings of siltstone, claystone or carbonaceous material, and the thickness of this unit is about 37 m.

Test tunnel

A 100 m long test tunnel was excavated on the left bank of the Oldman River Dam site close to the outlet portal of the proposed diversion tunnels to obtain geotechical data for tunnel design and construction purposes(Figure 1). The tunnel was driven at a 10 percent grade down through a horizontally bedded sequence of siltstones, sandstones and claystones. In general, sandstones are strong(uniaxial strength of mostly over 100 MPa), while claystones are weak(uniaxial strength of 10 - 23 Mpa). Uniaxial strength of siltstones are in between the above two extremes. Thick sandstone forms the invert and lower 4 m of the test chamber ; the siltstones and claystones predominate elsewhere. Joint spacing varies widely throughout the tunnel, but is related to lithology.

The tunnel as driven comprised a straight access section of 3.5 m diameter and 50 m length ; a curved access section of 3.5 m diameter and

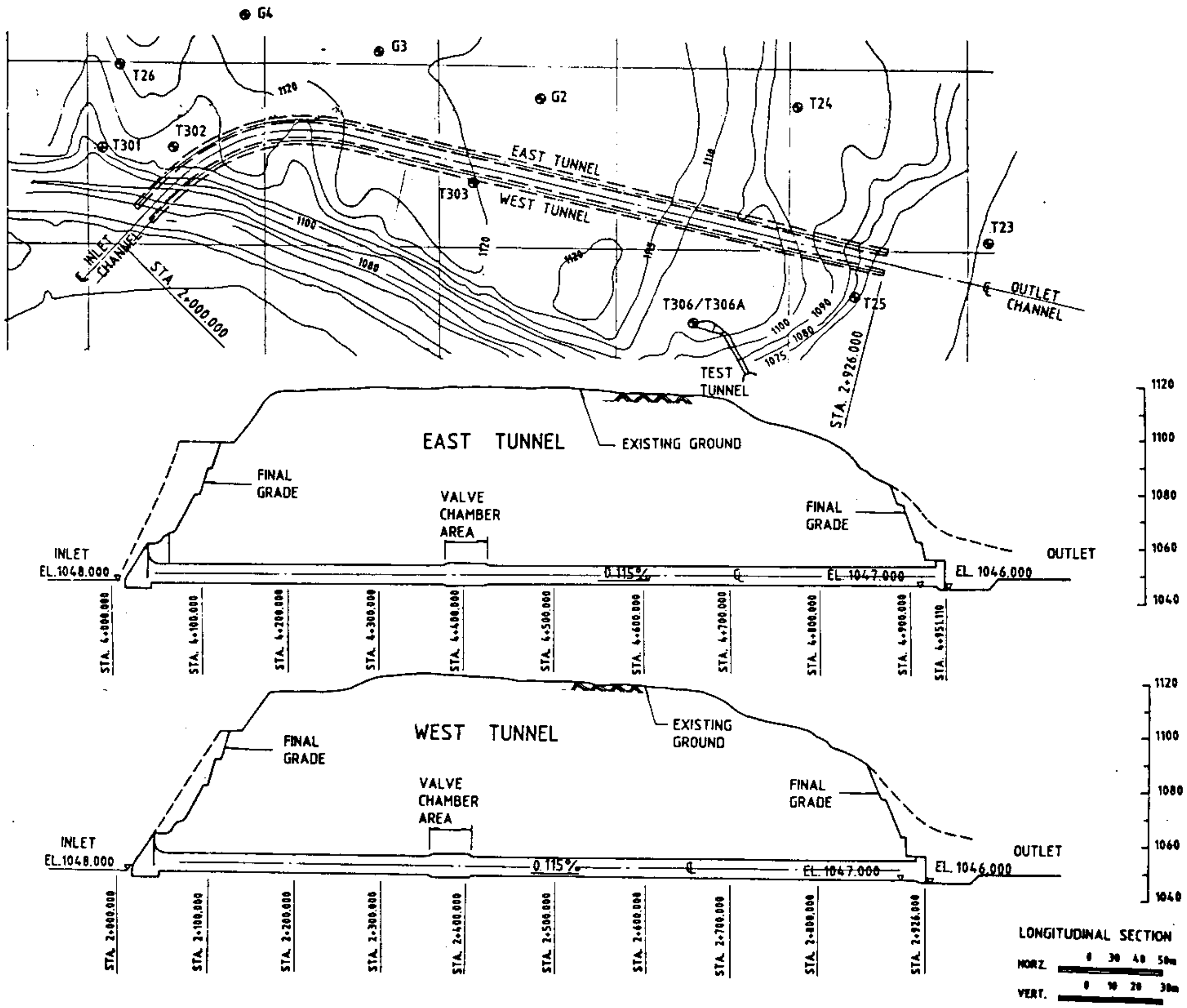
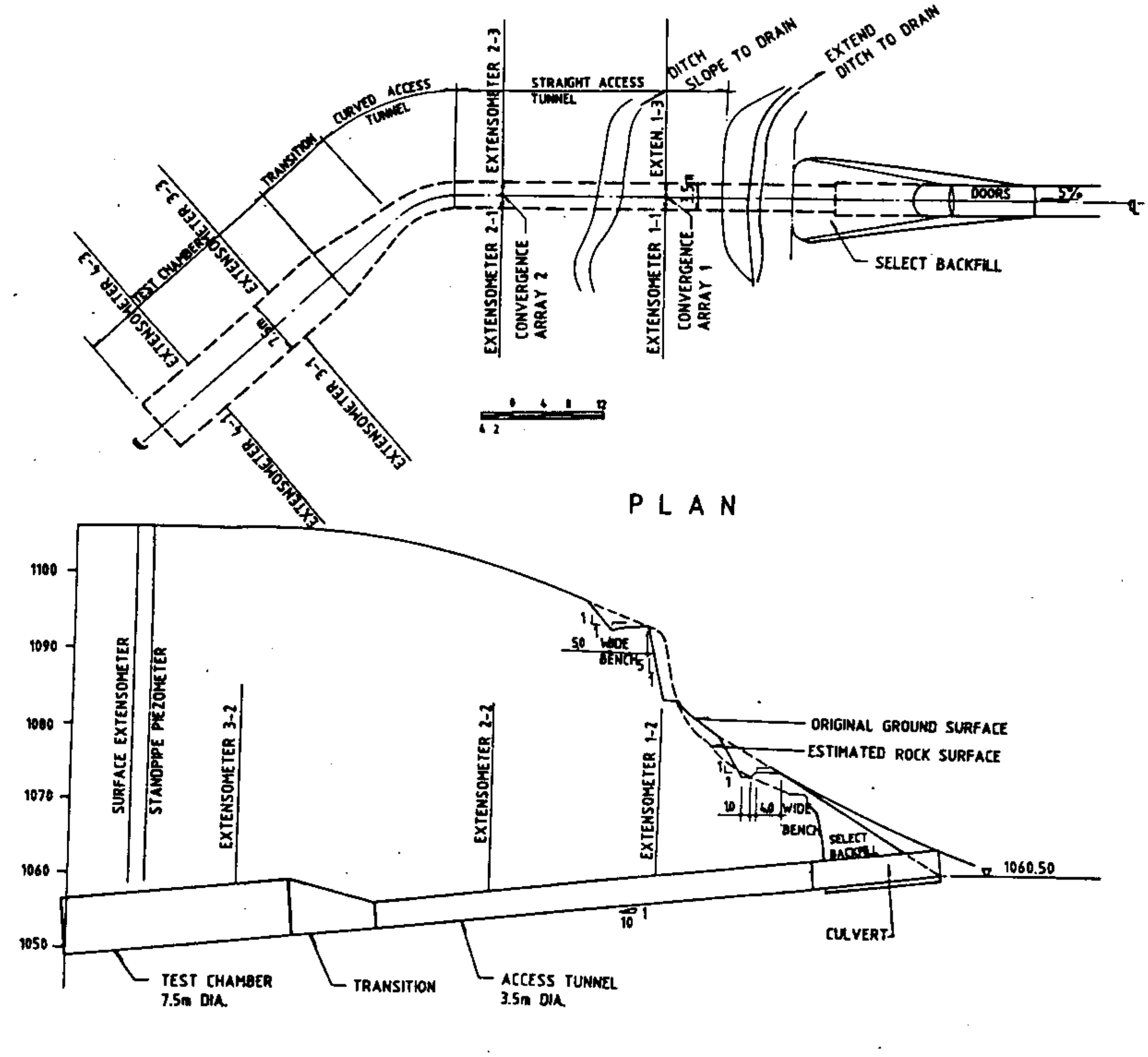


Fig. 1. Diversion tunnels for Oldman River Dam, Canada ; layout plan and profile

9.3 m length ; a transition zone varying from 3.5 m diameter to 7.5 m diameter over a 11.2 m length ; and a 7.5 m diameter chamber of 30 m length (Figure 2). Excavation was full face in the access tunnel and top heading and bench in the transition and test chamber. Drill-and-blast techniques were used to achieve an inverted U section throughout. The tunnel was supported by rock bolts and dowels with a 50 mm layer of shotcrete which also served to prevent slaking of the mudrocks.



PROFILE ALONG TEST TUNNEL ALIGNMENT

Fig. 2. Test tunnel; plan, profile and instrumentation layout

Four sections of the test tunnel were instrumented with multipoint borehole extensometers and convergence points to determine the extent of rock deformation after excavation (Figure 2). The extensometers were installed within approximately 2 meters of the advancing face. Two sections were within the access tunnel and two within the test chamber. Prior to excavation, a vertical extensometer was installed from the surface above the test chamber crown at the fourth array. A 5-anchor multipoint rod extensometer was installed from ground surface to the test tunnel chamber. Displacement data was monitored and analyzed by Golder Associates and the results were reported in the Contract Specifications for Oldman River Dam - Diversion Tunnels, Volume II[8]. The essential features of each type of instruments used and installation details may be found from the above Contract Specifications.

<u>Results of time-displacement measurement</u>

Surface displacements monitored by six extensometers in the access tunnel are plotted against time in Figure 3. All movements plotted in

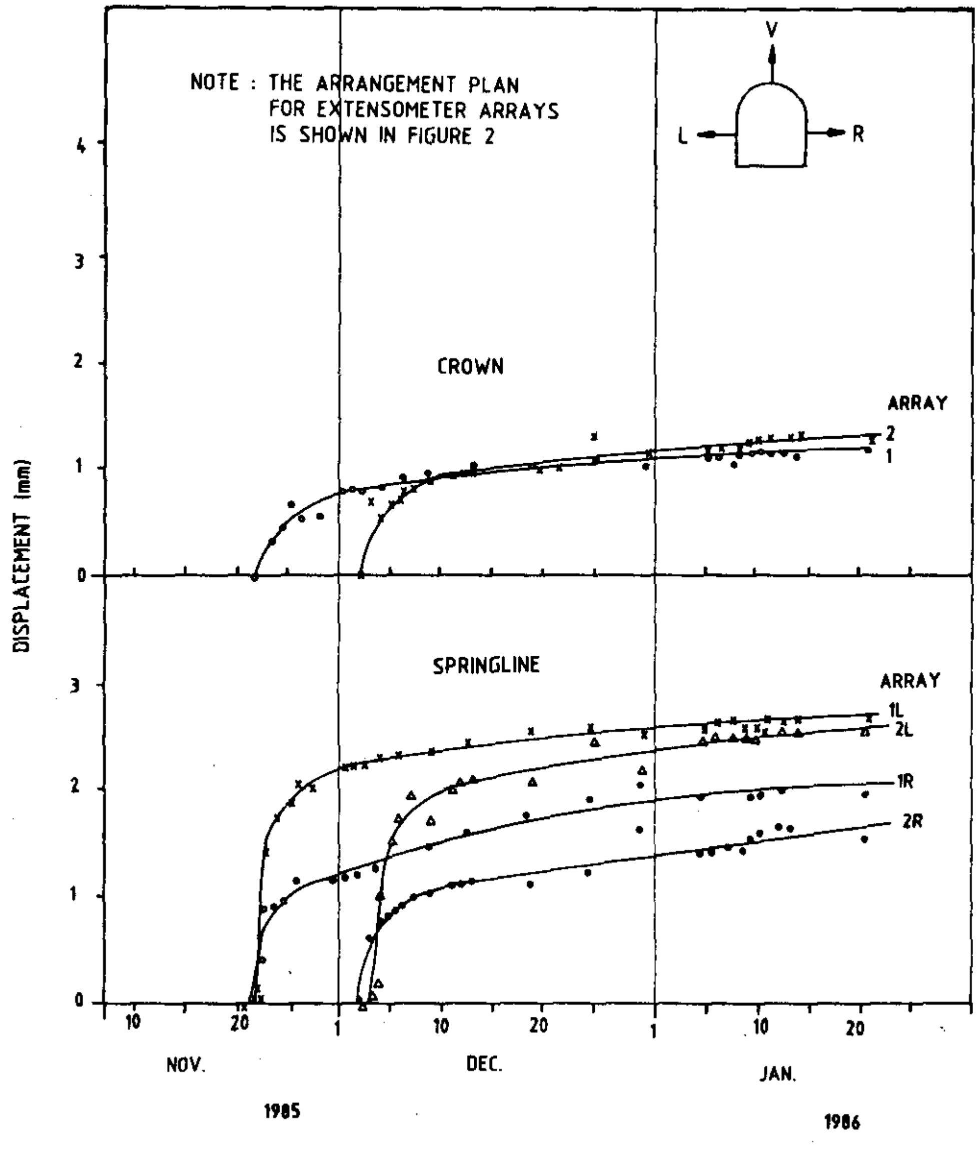


Fig. 3. Time-displacement curves recorded by extensometers installed in access tunnel

this figure are with reference to the deepest anchor (generally 10 m from the tunnel perimeter) which is assumed to be stationary. From the examination of the results shown in Figure 3, the following observations may be made ;

1) The results monitored for the period of 50 to 60 days show rapid inward movements between four and eight days after installation of extensometers. After the initial rapid movements, the deformation gradually decreases with time.

2) Displacements at both sides of the springline are between 1.4 to 2.7 mm at 50 - 60 days after the instrumentation, while those at the crown are about 1.3 mm.

3) For six extensometers installed in the access tunnel, the rate of time-dependent deformation subsequent to initial stage of rapid movement ranges from 0.2 mm/month to 0.35 mm/month, which is still significant.

From the above observations, it is clear that the rocks around the tunnel exhibit the time-dependent deformation behaviour. However, it was reported by the consultant that the mudrocks from this site did not appear to have time-dependent deformation characteristics and hence no special measures against the swelling of the rock were taken in the design and construction of the tunnels. It is interesting to note that displacements at the springline are higher than those at the crown, suggesting that there may be high initial horizontal stresses in this area. The progress and retrieval of glacial ice in the site area resulted in the loading and unloading of mudrocks[9], leaving high residual stress in horizontal plane.

The results of all the extensometers in the test chamber were not analyzed, mainly because these results show the complex behaviour due to stage excavation (top heading and bench) adopted.

Performance of diversion tunnels after construction

a) Construction of tunnels

The excavation of two diversion tunnels started from the outlet channel (downstream) side on February, 1987 and excavation of the inlet channel(upstream) side started from May, 1987. Once the upstream side excavation reached valve chamber area, concrete lining work started and proceeded to the inlet channel area. A similar concrete lining sequence was adopted for the downstream side, ie. excavation to the valve chamber area and then concrete lining from valve chamber toward the outlet portal area. This concrete lining sequence yielded small elapsed time between excavation and lining installation for the sections close to valve chamber.

The overall excavation was completed on July, 1987 and concrete lining of tunnel sections was finished on April, 1988. The excavation was carried out using drill-and-blast technique with the full face advance. The tunnel was stabilized temporarily by shotcrete and rockbolt before the installation of permanent lining. A 144m section of lining from the valve chamber to upstream and the last 54m section near outlet portal were reinforced and the rest of tunnel was not reinforced. Since most of concreting work was executed during the cold winter season (ambient temperature, -12 ^{o}C to -28 ^{o}C), several special measures were taken; special mix design, admixture control and heating of materials.

b) Post-construction performance

After the completion of tunnels, visual inspection was made to check the integrity of concrete lining. There was no visible crack up to Station 4+116 (Figure 1). A horizontal crack at about the level of the springline extended from Station 4+116 to 4+240 on both sides of the east tunnel. There were also numerous circumferential cracks from Station 4+120 onwards varying in width between 1 and 2mm. Beyond the valve chamber at Station 4+448, many circumferential cracks were present mostly spaced at 3.5m to 6m intervals and some horizontal cracks were also observed. The crack width was in the order 1 to 1.5mm. Some of the circumferential cracks formed at the construction joints which were spaced at 18.29m(60'-0"). Typical crack patterns surveyed are shown in Figure 4. The west tunnel showed similar crack formation and crack widths as the east tunnel.

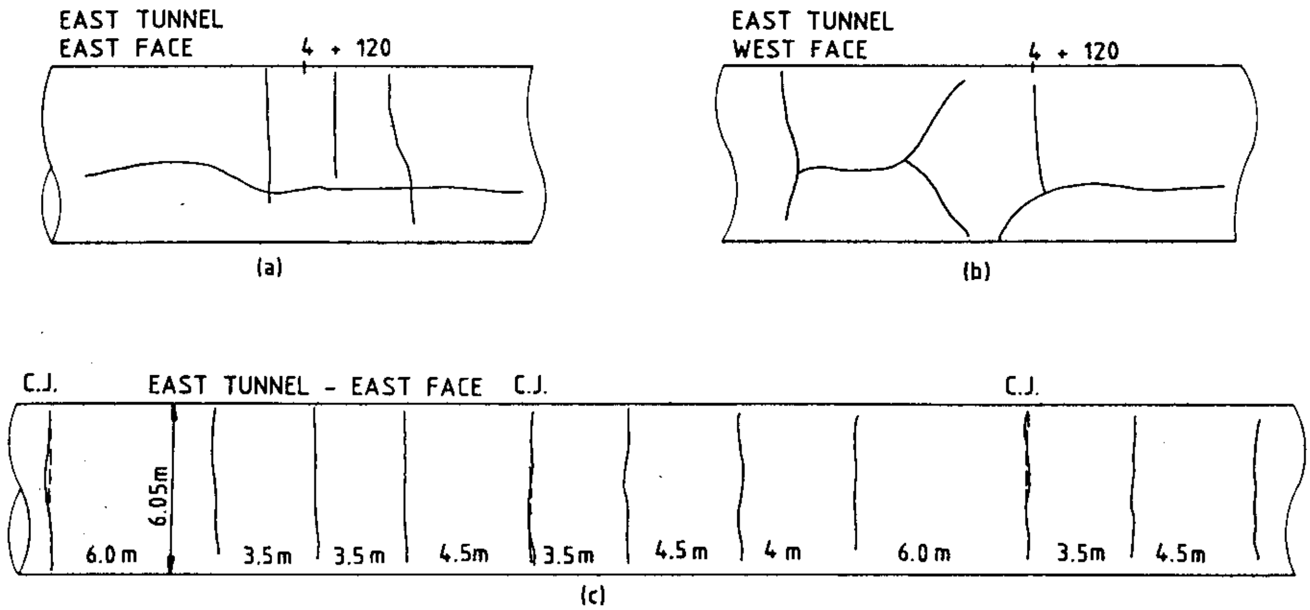


Fig. 4. Typical crack patterns in tunnel lining

Most of these cracks were visible starting from 10 days after pouring of the concrete. It is interesting to note that most of these cracks were formed on the sections where concrete lining was not reinforced.

c) Discussion

For a better discussion, a case history very similar to the diversion tunnels is described first. The formation of longitudinal cracks at the level of springline in the swelling rock under high horizontal stress has been reported for the Heartlake tunnel in Ontario by Lo and Yuen[7]. A detailed rock-lining interaction analysis was performed and the process of tunnel cracking was explained.

When a lining of some rigidity is placed against the rock at some elapsed time after excavation, the time-dependent deformation is partially restricted by the concrete lining. The structural constraint causes stress to build up with time at the rock-lining interface. Deformation of the lining tends to occur inwards, thus producing tensile stress in the concrete lining at the inner face at the springline level. The magnitude and rate of increase in tensile stress at the springline depend on the initial stresses, the time-dependent behaviours of rock and the lining, and the elapsed time between excavation and installation of lining.

It is not feasible to perform rock-lining time interaction analysis for the diversion tunnels due to lack of the critical information such as the magnitude of initial stresses and the exact construction details, but it is possible to explain the process of cracking in the concrete lining, using the concept developed by Lo and Yuen. It has been already established that the hosting rocks for tunnels show the time-dependent deformation behaviour and there is strong evidence for high initial horizontal stress. For the upstream side, the elapsed time between excavation and lining installation was rather short, allowing only small amount of swelling strain to occur. This led to the gradual build-up of stresses at the back of concrete lining, causing the tensile stress to develop at the inner face of the lining. Once the tensile stress developed reached the tensile strength of concrete, longitudinal cracks were then formed at the level of springline. However, the provision of reinforcements in the lining prohibited the formation of cracks, where the lining was reinforced.

Circumferential cracks were formed by the combination of the drying shrinkage and thermal contraction[2]. However, these cracks may also have been partially affected by the stress build-up at the back of concrete lining due to the swelling of the mudrocks.

SABNGS NO.3 PROJECT

Description of the project

Ontario Hydro is currently carrying out the feasibility study for the construction of a new hydraulic power plant, known as Sir Adam Beck Niagara Generating Station No.3(SABNGS No.3). The basic layout of the power plant will be similar to the 1400 MW SABNGS No.2 built in the 1950s. The new powerhouse will be constructed in the vicinity of the existing generating station near the town of Queenston, as shown in Figure 5. Water for power generation will be diverted from Grass Island Pool, located some 1.7 km upstream of Horseshoe Falls, to the powerhouse through a system of tunnel(s) and canal (Figure 6). The tunnel(s) for the proposed SABNGS No.3 will be constructed in the Queenston Formation at depths between 90 to 140 m below ground surface. A twin tunnel system with a finished diameter of 10.7 m is favored.

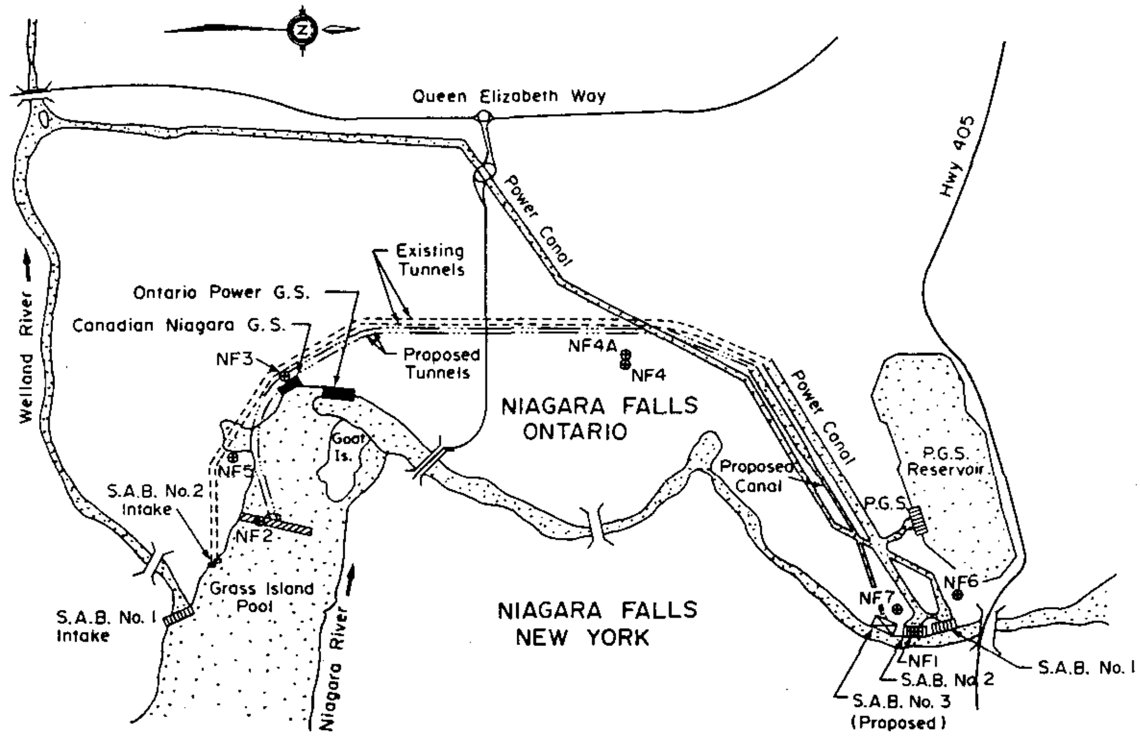


Fig. 5. General plan of Sir Adam Beck Niagara Generating Station complex [8]

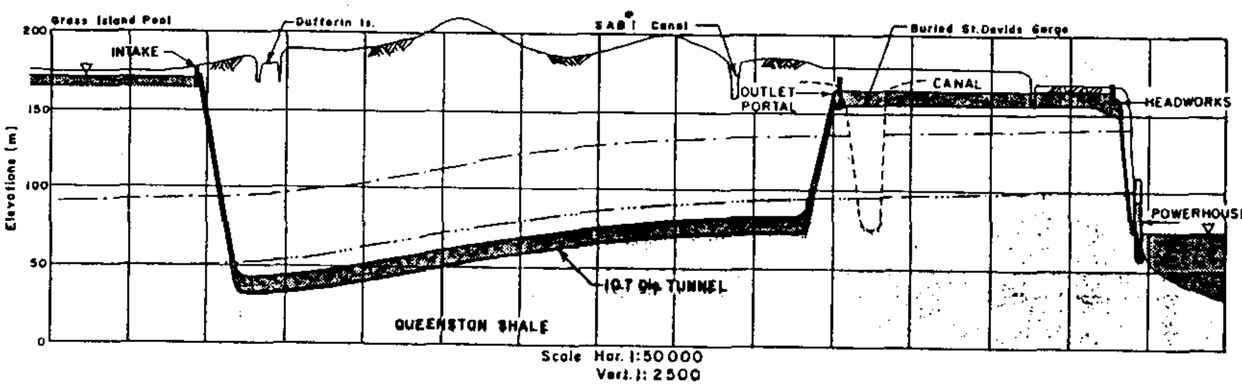


Fig. 6. Longitudinal section of proposed tunnel system[8]

Description of Queenston shale tested

a) General Properties

The proposed tunnels are to be excavated in the Queenston shale formation of Upper Ordovician period of the Paleozoic era. The Queenston shale consists of reddish brown and locally greenish grey very fine grained calcareous shale. It may be more accurately classified as a mudstone since it lacks the fissility found in shales [8].

Over the depths of investigation from 80 m to 122 m where the twin tunnels may be located, the water content lies between 2.0 % to 2.9 % with an average value of 2.6 %. The unit weight averages 26.7 KN/m^3, ranging from 26.4 KN/m^3 to 26.8 KN/m^3. The specific gravity is 2.82. The porosity is approximately 7 %. The calcite content varies from 3 % to 7 % which is lower than value obtained at other sites of the Queenston shale formation in the Niagara Peninsula. The salinity of the pore fluid is in the range of 108 mg/l to 265 mg/l. There was no significant trend of variation with depth of water content, unit weight, calcite content or salinity.

b) Strength and deformation properties

The uniaxial strength from vertical specimens is approximately 25 MPa and from horizontal specimens 26 MPa. The vertical and horizontal elastic moduli are approximately 9 GPa and 13 GPa respectively. The Poisson's ratio for the effect of vertical stress on horizontal strain is 0.35, while the Poisson's ratio for the effect of horizontal stress (in HM direction) on vertical strain is 0.40. The Poisson's ratio for the effect of horizontal stress on horizontal strain is 0.26. It is evident therefore that while the strength behaviour appears isotropic,the deformation behaviour is moderately anisotropic. The compressional wave velocities of vertical and horizontal specimens are approximately 3.6 km/sec and 4.0 km/sec respectively. The vertical and horizontal dynamic moduli from these wave velocity measurements are approximately 22 GPa and 30 GPa, resulting in a comparable ratio of horizontal to vertical moduli to the corresponding ratio of static moduli.

The tensile strengths determined from splitting (Brazilian) tests are 4.6 MPa and 3.4 MPa respectively for fracture across and along bedding planes.

c) Mineralogy

X-ray diffraction analysis was carried out to identify minerals present in Queenston shale. For the quantitative determination of mineralogical composition, chemical analyses were performed. From the results of X-ray diffraction analyses, it is clear that the fines of Queenston shale contain abundant illite and chlorite along with an assemblage of interlayered clays. The results of semi-quantitative analysis show that the Queenston shale has the following mineral composition ;

Calcite	2 %	Illite	40 %
Dolomite	2 %	Chlorite &	
Quartz	26 %	Interlayered clay	27 %
Feldspar	2 %	Vermiculite	1 %

Results of time-dependent deformation tests

a) General

In general, test specimens for time-dependent deformation tests were prepared in three principal stress directions, using the oriented rockcores as shown in Figure 7. Based on the known orientation of scriber mark (or chisel mark) on the core and the "known" direction (N45E) of the secondary major principal stress from borehole NF-4, the line of the major principal stress direction was drawn on rockcores. After marking the line on cores, free swell test specimens were prepared by mounting gauge points in three principal stress directions. All nine modified semi-confined swell test specimen were recorded from HQ3 size (nominal diameter of 61.1 mm) rockcores in three principal stress (vertical and two horizontal ; major, HM and minor, HN) directions.

To study time-dependent deformation behaviour of the Queenston shale, four different swell tests were performed between 1985 and 1987 and the results of two important tests, free swell test and modified semi-confined swell test, are reported and discussed in this paper.

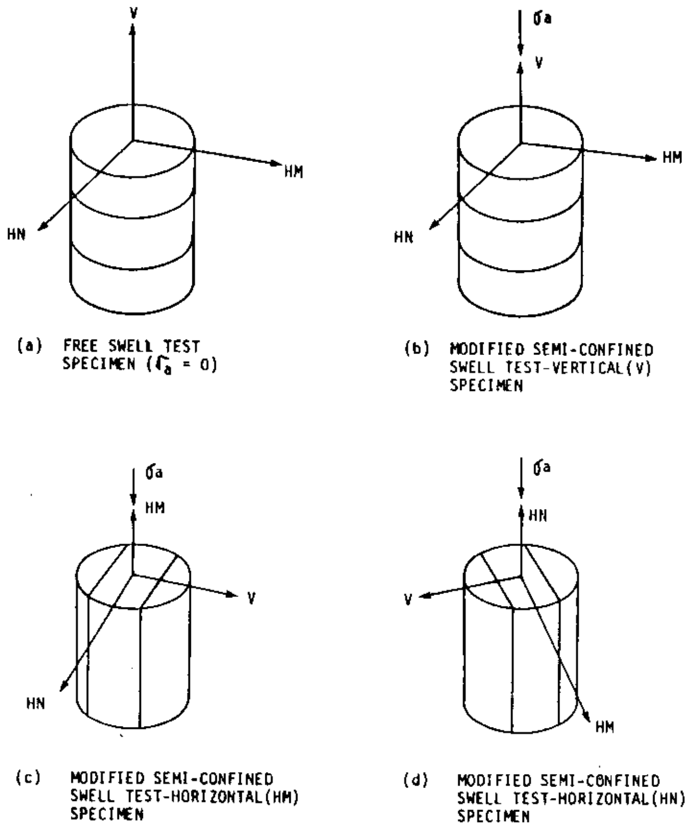


Fig. 7. Various specimen orientations with respect to the applied stress

b) Results of free swell test

In this test, the swelling deformations of test specimen under no applied stress are measured in all three orthogonal directions. This test simulates the stress condition of an element where majority of the stresses in three orthogonal directions are relieved due to excavation.

A total of nine free swell tests were carried out on oriented rockcores from depths between 94 to 113 m below ground surface. Typical free swell test results are shown in Figure 8. The results plotted in semi-logarithm show that the test specimen exhibits swelling strains in three orthogonal directions and swelling behaviour in horizontal plane is isotropic as observed in the other shales from this region.

It may be also observed that swelling potential, defined as swelling strain per log cycle of time, in the vertical direction is moderately higher than those in the horizontal directions, though the stress relieved in the vertical direction (about 2.5 MPa) is smaller than that in the horizontal direction (5.2 - 7.9 MPa). The results on the other specimens of the Queenston shale show the same trend and support the above observation[5]. This has been further extended to other shales from this region[6].

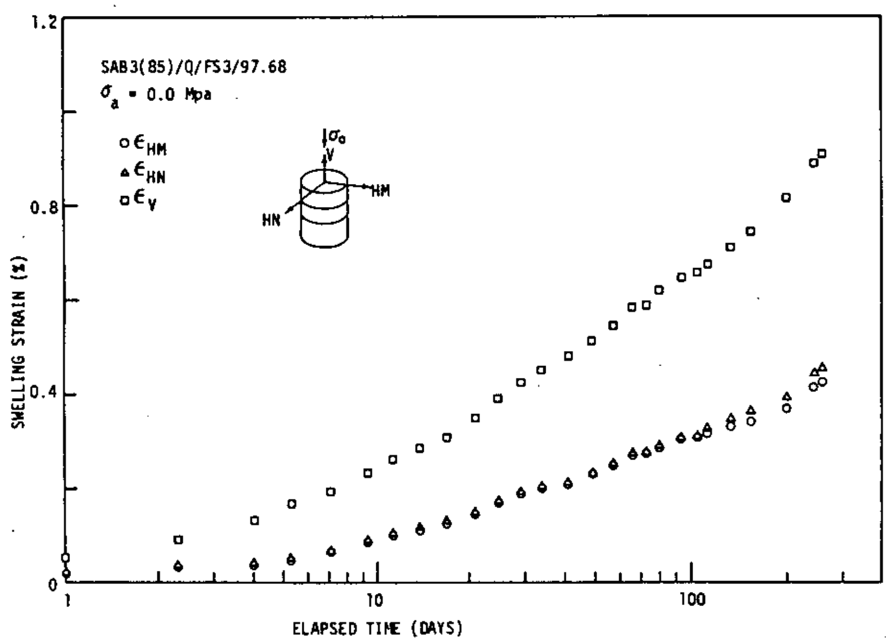


Fig. 8. Typical results of free swell test for the Queenston shale, Niagara Falls, Canada

The vertical swelling potential of the Queenston shale, showing random fabric, is only about 1.6 times higher than the horizontal swelling potential. The Blue Mountain shale, having a very strong clay fabric, gives the average ratio of 6.3, indicating that the vertical swelling is about 6.3 times higher than the horizontal swelling. The Georgian Bay shale, exhibiting a fabric intermediate between the above two extremes, shows an average of 4.3, revealing the effect of the intermediate fabric.

c) Results of modified semi-confined swell test

In this test, specimen under the applied stress in the axial direction is allowed to swell and time-dependent deformation in three orthogonal directions are measured. This test represents the stress condition of the element where the dominant stress is the uniaxial compressive stress.

It has been shown, using the test results presented in semi-logarithm scale as in Figure 8, that the applied stress in one direction reduces the swelling strain in the direction of applied stress, compared to that of free swell test[5]. The reduction effect of applied stress on swelling potential in that direction is shown in Figure 9. In this figure, the swelling potentials under applied stress are plotted against logarithm of the applied stress, σ_a for the Queenston shale. Swelling potentials of specimens prepared in the vertical and horizontal directions decrease linearly with increasing applied stress in the semi-log plot. The rate of decrease in swelling potential of vertical specimen is slightly higher than that of horizontal specimen.

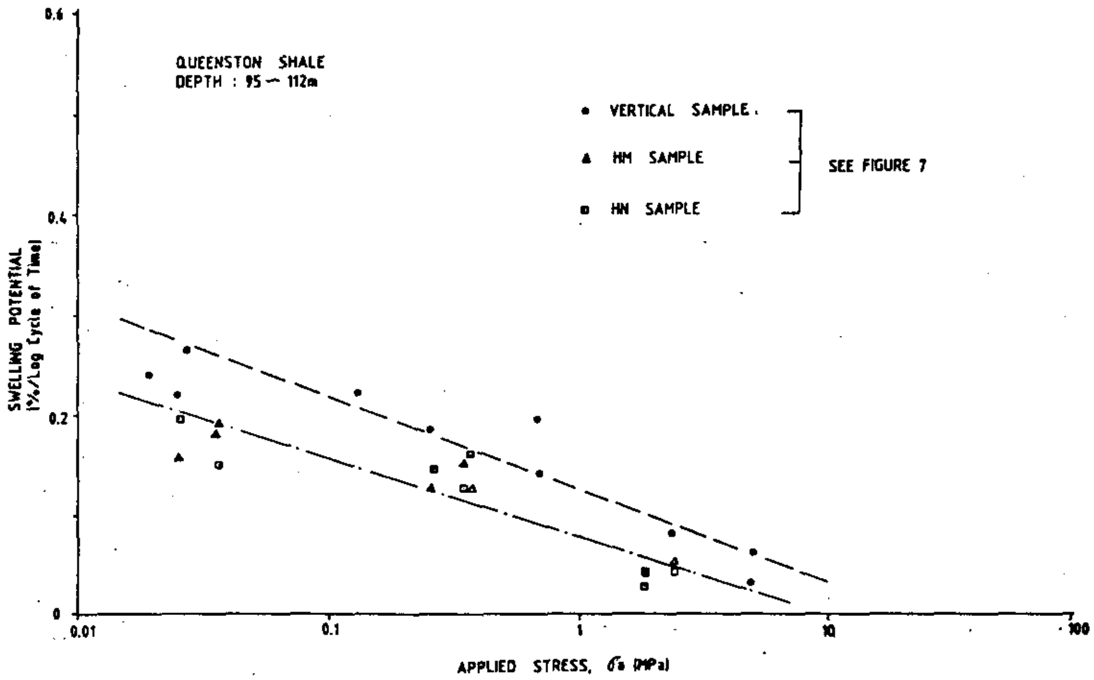
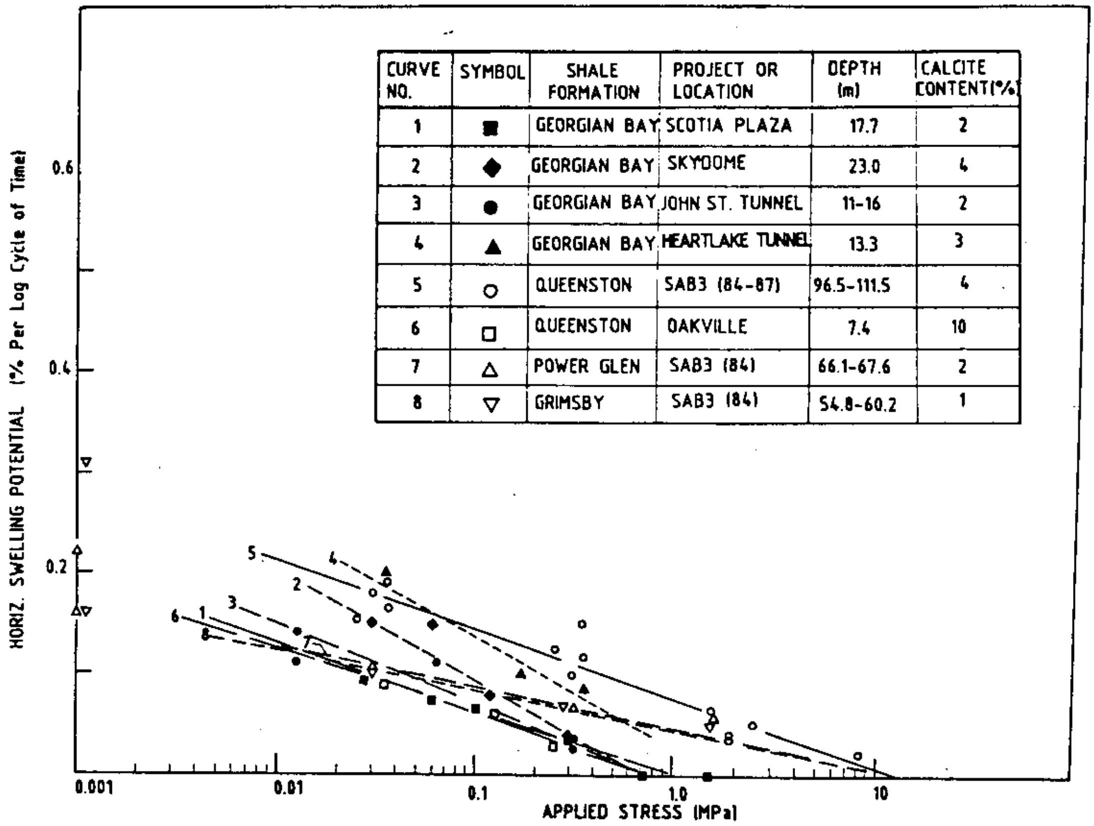


Fig. 9. Effect of applied stress on reduction of swelling potential in the direction of applied stress, Queenston shale

This observation is further extended for the shales tested in the region of Niagara Peninsula, as shown in Figure 10. The suppression effect of the applied stress on both axial and lateral strains have been illustrated by Lee[5].

Implications on design of tunnels

The horizontal swelling potential of the Queenston shale measured from



CURVE NO.	SYMBOL	SHALE FORMATION	PROJECT OR LOCATION	DEPTH (m)	CALCITE CONTENT(%)
1	■	GEORGIAN BAY	SCOTIA PLAZA	17.7	2
2	◆	GEORGIAN BAY	SKYDOME	23.0	4
3	●	GEORGIAN BAY	JOHN ST. TUNNEL	11-16	2
4	▲	GEORGIAN BAY	HEARTLAKE TUNNEL	13.3	3
5	○	QUEENSTON	SAB3 (84-87)	96.5-111.5	4
6	□	QUEENSTON	OAKVILLE	7.4	10
7	△	POWER GLEN	SAB3 (84)	66.1-67.6	2
8	▽	GRIMSBY	SAB3 (84)	54.8-60.2	1

Fig. 10. Effect of applied stress on horizontal swelling potential of shales from Southern Ontario

free swell test averages 0.29 % per log cycle of time(days), which is highest among all the shales tested in Niagara Peninsula region. Since it was expected from the past experiences accumulated in this region that substantial "rock squeeze" problem may occur in the tunnels to be excavated in the highly swelling Queenston shale, an extensive investigation program was established to determine the constitutional relationship for the rock-structure time interaction analysis and to find out the swelling mechanism. The stress-strain-time relationship of the Queenston shale was established and reported in Lee [4], together with the relevant swelling mechanism.

For the design of the proposed tunnels, it would be essential to perform rock-structure time interaction analysis using the established constitutional relationship of the Queenston shale. The results of this analysis will allow the designer to determine design measures against the "rock squeeze" problem in the proposed tunnels, such as the time lapse between excavation and lining installation and provision of stress relief slots. It will be equally important to develop a new flexible lining material which can withstand the large amount of time-dependent deformation without going to failure.

SUMMARY AND CONCLUSIONS

The time-dependent deformation behaviours of swelling rocks from two tunnel projects have been studied to demonstrate the importance of the swelling characteristics of the rock in the design and construction of tunnels. In diversion tunnels for Oldman River Dam, the time-dependent displacement was monitored, but neglected in the design and construction of tunnels. Longitudinal cracks at the level of the springline are mainly attributed to the swelling of the rock, while circumferential cracks are considered to be affected mainly by drying shrinkage and thermal contraction. In SABNGS No. 3 project, an extensive experimental study was performed to investigate the swelling behaviour of highly swelling Queenston shale with an aim of establishing stress-strain-time relationship for the rock structure time interaction analysis.

From the results of the present study, the following conclusions may be drawn:

1) For the design and construction of underground structure in swelling rock, it is essential to perform laboratory tests to determine the time-dependent deformation characteristics.

2) It would be beneficial for the design and construction of tunnel to carry out rock-structure time interaction analysis using the stress-strain-time relationship developed from the results of laboratory tests.

3) The findings from laboratory tests and analysis are to be incorporated in the design and construction of tunnels to determine the possibility of adopting special measures such as time lapse between excavation and lining installation, stress relief slots and flexible lining.

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