Niagara Tunnel Project
Hydraulic Aspects

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Introduction

Hydropower is essential for covering growing global energy demand. Hydropower of Niagara River is used by Canada and the United States of America in accordance to a bilateral agreement from 1950 (Niagara Diversion Treaty). Until 2013, Canada had capacity for diverting 1,800 m³/s from Niagara River to be used at Sir Adam Beck Generating Station for power generation [3][6].

The aim of the Niagara Tunnel Project was to design and construct a conveyance system diverting an additional amount 500 m³/s from Niagara River to an approximately 10 km distant reservoir. From there the additional flow should be available at Sir Adam Beck Generating Station for power generation. The system includes a tunnel of a bored diameter of 14.44 m, constructed 150 m under the City of Niagara Falls using the world's largest hard-rock tunnel boring machine (TBM) [3]. This paper describes the hydraulic aspects and challenges of this project.

Meeting the mandatory contractual requirement of diverting a guaranteed flow amount (GFA) of 500 m³/s using the very small hydraulic head of 5.6 m through a 10.34 km long tunnel was challenging for the designers at ILF as well as for the design build contractor STRABAG.

Due to the extreme dimensions of the tunnel, conventional hydraulic calculation methods like the formula of Prandtl-Colebrook were not applicable anymore and other ways for dimensioning the hydraulic system had to be taken into consideration. The formula of Manning/Strickler turned out to be appropriate for calculating the system. For modelling the system in steady flow condition HEC-RAS 4.1.0 (US Army Corps of Engineers) has been used [5].

The operation of opening and closing the gate in the outlet structure was investigated for unsteady flow conditions in the conveyance system. Calculations of unsteady flow conditions have been made using software developed by ILF - Consulting Engineers, proven in numerous earlier projects. The calculations of this software are based on the "method of characteristics" [4].

After construction of the conveyance system flow measurements have been conducted by Alden Research Laboratory, Inc. to determine the as-constructed flow amount of the tunnel. The flow was measured using a multipath ultrasonic transit-time flowmeter with redundant transducers. In addition the water levels at the intake and the outlet were measured to mathematically determine the reference tunnel flow with the reference hydraulic head of 5.6 m [1].

1. Background

Niagara River provides a flow up to 6,000 m³/s. The for power generation available amount of the total discharge is equally shared between the two neighboring countries Canada and the United States of America according to Niagara Diversion Treaty from 1950. Canada put the first part of Sir Adam Beck Station (SAB 1) in service in 1922 to use hydropower of Niagara River. In 1954 the hydro power plant was extended with SAB 2 to the Sir Adam Beck Power Generating Complex as it can be seen in the foreground of Figure 1 [6]. Until 2013 Canada had capacity for diverting 1,800 m³/s from Niagara River to the approximately 10 km distant reservoir for Sir Adam Beck Station. The reservoir is marked in Figure 1 as well. The diversion system consisted of one open-cut channel build in the 1920s and two tunnels built in the 1950s [3]. An additional tunnel was designed by ILF-Consulting Engineers and constructed by STRABAG to enhance an additional capacity of 500 m³/s.

Diverting 500 m³/s using the very small available hydraulic head of 5.6 m through a 10.34 km long tunnel was challenging for the designers as well as for the constructors. To achieve the required small friction losses a tunnel of a bored diameter of 14.44 m was constructed using the world's largest hard-rock tunnel boring machine.

The new Niagara Tunnel follows the same basic route as the existing two tunnels. The alignment of the tunnel is marked in Figure 1 and in the longitudinal section in Figure 2. The intake of the tunnel is located at the International Water Control Dam, 1.6 km upstream the Horseshoe Falls. From there the tunnel starts with 7.15% decline over the first 1,200 m. Then the tunnel proceeds approximately 7,800 m with a relatively horizontal plane. Afterwards the tunnel falls again with a decline of 7.15% down to its deepest point, 150 m below the city of Niagara Falls. This great depth is necessary in order to pass under Buried St. Davids Gorge. Before its end the tunnel climbs with an incline of 7.82% to its outlet on the OPG property at Queenston. The curves of the tunnel have a radius of 1,000 m [3].

Figure 1: overview of the alignment of the Niagara Tunnel and Sir Adam Beck Station

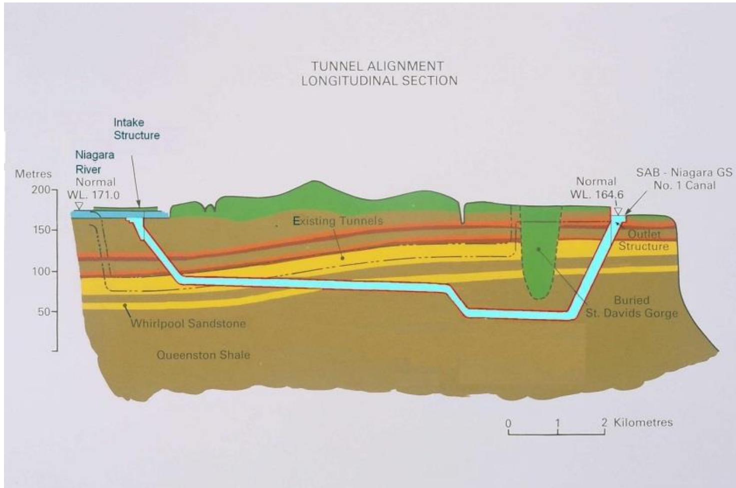

Figure 2: longitudinal section of the Niagara Tunnel [2]

2. Hydraulic Calculations

2.1 Steady Flow Condition

The system includes both, surface flow in open channels (intake channel and outlet channel) and the tunnel itself which is the longest part of the system. The transition between the tunnel and the surface flow takes place in the intake and outlet structure. Those structures were carefully designed in order to avoid energy losses through e.g. vortex at the intake structure or hydraulic jump at the outlet structure. A gradually dilatation of the outlet structure to additionally avoid local losses at this part of the system was not considered profitable.

The friction losses of each section were calculated on the bases of the formula of Manning/Strickler (formula 1). The more precise formula of Prandtl-Colebrook however was not applicable any more. The reason for this might be that the hydraulic conditions in the tunnel of an inner diameter of about 12.7 m are not in the range of application of this formula which is usually applied to calculate conventional pipes of much smaller dimensions [5].

The whole conveyance system was divided into five sections listed in Table 1. A Strickler coefficient k_{st} was applied to each section. The values of those roughness coefficients were taken from both, the literature and experiences from selected existing hydropower plants in Europe. The assumed values of the roughness coefficients are listed in Table 1 as well. The flow in some sections is 3-dimensional which is not considered in the formula of Manning/Strickler. To consider additional losses caused by take 3-dimesnional flow in the model, a higher roughness coefficient is applied at the intake channel [5].

To also consider changing cross section areas within the sections the sections additionally were divided into more than 30 subsections. The different areas of the cross sections are the result of changing bed slope in sections with surface flow or different lining thickness of the tunnel. The losses for transitions at thickness changes of tunnel lining have been calculated for an exemplary dilatation according to [7]. The resulting friction losses were very small and it has been concluded that if the dilatation was gradually and slow (e.g. angle of dilatation ≤8°), as it is the case in the Niagara Tunnel, the losses at the transition could be neglected.

The calculation of the possible flow through the tunnel at a given hydraulic head was done by summing up the friction losses of the subsections and sections of the diversion system. The total friction loss was compared with the available hydraulic head of $H_{ref} = 5.6\,\mathrm{m}$ and the reference elevation of the energy grade line at the outlet gauge ($EGL_{2ref} = 165.2\,\mathrm{m}$) in accordance to the principle of energy conservation (formula 2) [5]. The hydraulic head is the difference of the elevations of the energy grade line between the intake and the outlet channel.

For modelling the system HEC-RAS 4.1.0 (US Army Corps of Engineers) has been used. The model is based on the formula of Manning/Strickler.

An additional plausibility check with ILF - Consulting Engineers' own calculation tools in MS Excel, based on formula of Manning/Strickler as well has been executed to proof the results of HEC-RAS 4.1.0. For the crosscheck the input data and other relevant flow conditions had to be modified slightly.

$$Q = k_{st} \cdot J^{\frac{1}{2}} \cdot R^{\frac{2}{3}} \cdot A \Leftrightarrow J = \left(\frac{Q}{k_{st} \cdot R^{\frac{2}{3}} \cdot A}\right)^2 \tag{1}$$

$$EGL_1 = EGL_{2ref} + \sum_{i \epsilon I} (l_i \cdot J_i) < EGL_{1ref} = EGL_{2ref} + H_{ref} = 170.8\,\mathrm{m} \tag{2}$$

Q $[Q] = \frac{\mathrm{m}^3}{\mathrm{s}}$ .... discharge

k_{st} $[k_{st}] = \frac{\mathrm{m}^{1/3}}{\mathrm{s}}$ .... Strickler's roughness coefficient

J $[J] = 1$ .... friction slope

R $[R] = \mathrm{m}$ .... hydraulic radius

A $[A] = \mathrm{m}^2$ .... flow area

l_i $[l_i] = \mathrm{m}$ .... length of section i

EGL_1 $[EGL_1] = \mathrm{m}$ .... elevation of the energy grade line at the intake gauge at a specific flow

EGL_{2ref} $[EGL_{2ref}] = \mathrm{m}$ .... reference elevation of the energy grade line at the outlet gauge

H_{ref} $[H_{ref}] = \mathrm{m}$ .... available hydraulic head (5.60 m): difference of the elevations of the energy grade line between the intake and the outlet channel;

Results [5]:

The calculations indicated that the entire conveyance system should be capable of delivering 500 m³/s with a predicted hydraulic head of 5.32 m. Furthermore the calculations of the steady flow analyses indicated that 513.5 m³/s could be conveyed with the given reference hydraulic head of 5.6 m. The friction loss of each section is listed in Table ***1***.

Table 1: Sections of the conveyance system [5]

section	length		Strickler coefficient	friction loss at a flow of 513.5 m³/s
intake channel	165 m	surface flow	25 $\mathrm{m}^{1/3}$/s	0.07 m
intake structure	23 m		80 $\mathrm{m}^{1/3}$/s	0.02 m
tunnel	10,340 m		85 $\mathrm{m}^{1/3}$/s	5.21 m
outlet structure	38 m		80 $\mathrm{m}^{1/3}$/s	0.06 m
outlet channel	307 m	surface flow	30 $\mathrm{m}^{1/3}$/s	0.24 m
Sum	**10,873 m**			**5.60 m**

2.2 Unsteady Flow Conditions

The operation of lowering and raising the gate in the outlet structure was investigated for unsteady flow conditions in the water conveyance system of the Niagara Tunnel Facility Project. The transient analyses are performed using software developed by ILF - Consulting Engineers. The efficiency of the software has been proven in numerous earlier projects, including large water transmission systems, hydropower stations as well as crude oil and products pipelines. The computer software is based on the “method of characteristics”. A numerical model of the hydraulic active system conduces as input for the software. The numerical model describes all necessary parameters like tunnel diameter, liquid properties, levels and stations as well as characteristics for all hydraulic relevant equipment like surge shafts and valves [4].

Starting from a pre-defined steady state condition, the behavior of the hydraulic system in case of events which can disturb the steady state condition was modelled. The investigated load cases have been selected for the calculation because they result in minimum or maximum pressures or result in some other kind of special behavior of the system under transient conditions. The computations always comprised the entire hydraulic system [4].

To cover also unfavorable conditions with the calculations of unsteady flow conditions a maximum discharge through the tunnel of $550\,\mathrm{m^3/s}$ has been assumed for the analyses [4]. The water level at the intake channel has been assumed to be 173.17 m which is the probable maximum flood (worst case). The roughness of the tunnel lining has been set to $k_s = 0.03\mathrm{mm}$ according to Prandtl-Colebrook. This is equivalent to $k_{st} = 85$ according to Strickler which has been used in section 2.1 for steady flow analyses. The type of the valve is a gate valve with an inside diameter of 15.88 m. Since no vendor data was provided by the manufacturer of the valve COH Inc. the valve loss characteristics have been assumed on the bases of standard gate loss characteristics [4].

The following four critical load cases have been selected for investigation for lowering the gate [4]. In order to avoid overflow and to keep closing times as short as possible two-speed closing patterns with a lower closing speed starting at 12% remaining open position have been investigated:

- powered lowering (max. speed 0.16 m/min) → one speed
- powered lowering (max. speed 0.16 m/min, 0.025 m/min) → two speed
- unpowered lowering (max. speed 0.31 m/min) → one speed
- unpowered lowering (max. speed 0.31 m/min, 0.025 m/min) → two speed

In addition one load case for raising the gate was investigated:

- raising the gate (max. speed 0.16 m/min)

For all load cases the first 10,000 seconds after the lowering or raising procedure has started have been modelled.

Using the model, the following parameters have been calculated for each of the load cases:

- history of water level at the surge shaft
- maximum water level at the surge shaft
- minimum water level at the surge shaft
- history of surge tank overflow and total amount of water overflow
- history of pressures
- maximum pressures for lowering the gate and minimum pressures for raising the gate in the entire longitudinal section of the tunnel
- history of flowrate

The computation results were presented in two different kinds of computer plots [4]:

- hydraulic profiles at a “Situation at Time” (e.g.: minimum/maximum pressure along the longitudinal section of the tunnel)
- “History of Events” which show the development in time of the preselected values (e.g.: flow rate, pressure, valve positions, …)

Results [4]:

The calculations showed that using one-speed closing patterns at the given rates of 0.16 m/min or higher resulted in an overflow volume of approximately 100,000 m³. The overflow could be avoided using the two-speed opening patterns with a speed change at 12% remaining open position. This way the total closing time for power gate host lowering is 9,900 seconds (approximately 165 minutes). The total closing time for unpowered gate host lowering is 7,364 seconds (approximately 123 minutes). The time for raising the gate is 5,955 seconds (approximately 99 minutes).

As mentioned above, the gate valve characteristics were not provided by the manufacturer and standard gate valve characteristics have been assumed for the calculations. It has to be pointed out, that during the closure process standard gate valves are not hydraulically active until they reach a point where only 10-15% of the cross section remains open. This means that during the first stage of approximately 85% gate closure, the gate could be operated at faster closing speeds than 0.16 m/min [4].

Due to the slow lowering and raising of the valve the other investigated parameters did not turn out to show unsteady flow specific reactions. The maximum pressure resulting when closing the gate turned out to be the hydrostatic pressure with the valve fully closed, when the lowering procedure has ended.

3. Flow Verification

3.1 Flow Measurement Method and Test Procedure [1]

Since there are monetary incentives for increased flow (> 510 m³/s) and disincentives for decreased flow (< 490 m³/s) the flow measurement was conducted by ALDEN Research Laboratory, Inc. as a third party, mutually accepted by the owner OPG/NPG and the contractor STRABAG.

The flow was determined using an 8-path ultrasonic transit time flowmeter with redundant transducers and cable (32 in total). In addition the water levels at the intake and the outlet were measured using pressure transducers [1].

The temporary water elevation measurement instrumentation was installed by Alden on July 22-23, 2013. Once the sensors were deployed the data acquisition started and continued until the sensors were shutdown to control the water levels at a steady state. The official test started on July 24, at 12:32am EST when levels had stabilized to the satisfaction of Alden and the average differential head was constant over a period of 30 minutes within a frame of +/-1.0%. During the test the water levels at the intake and the outlet where maintained at a steady state [1].

<u>Determination of the reference tunnel flow [1]:</u>

Since it is hardly possible to maintain a steady state with exact reference conditions as mentioned in section 1 the reference tunnel flow has to be determined mathematically. The calculation of the reference tunnel flow is based on the formula of Bernoulli and the energy grade line. The measured energy grade line at the intake gauge (EGL_{1m}) and the outlet gauge (EGL_{2m}) can be calculated as presented in formula 3 and 4. The velocity head at the intake gauge was assumed to be 0. The discharge Q_{ref} that would have been obtained if EGL_{1ref} and EGL_{2ref} would have been obtained can be calculated as shown in formula 5.

$$EGL_{1m} = HGL_{1m} \tag{3}$$

$$EGL_{2m} = HGL_{2m} + \frac{\left(\frac{Q_m}{A_{outlet}}\right)^2}{2g} \text{ with } A_{outlet} = (HGL_{2m} - invert_2) \cdot w_{outlet} \tag{4}$$

$$Q_{ref} = Q_m \cdot \sqrt{\frac{H_{ref}}{EGL_{1m} - EGL_{2m}}} \tag{5}$$

EGL_{xm} $[EGL_{xm}] = \mathrm{m}$ .... measured energy grade line at intake gauge (1) and outlet gauge (2)

HGL_{xm} $[HGL_{xm}] = \mathrm{m}$ .... measured hydraulic grade line at intake gauge (1) and outlet gauge (2)

A_{outlet} $[A_{outlet}] = \mathrm{m}^2$ .... cross-sectional area at the outlet gauge (rectangular)

w_{outlet} $[w_{outlet}] = \mathrm{m}$ .... width of the channel at the outlet gauge

$invert_2$ $[invert_2] = \mathrm{m}$ .... invert elevation at the outlet channel

Q_m $[Q_m] = \frac{\mathrm{m}^3}{\mathrm{s}}$ .... measured discharge with EGL_{1m} and EGL_{2m}

Q_{ref} $[Q_{ref}] = \frac{\mathrm{m}^3}{\mathrm{s}}$ .... reference tunnel flow determined from the measured tunnel flow

H_{ref} $[H_{ref}] = \mathrm{m}$ .... available hydraulic head (5.60 m): difference of the elevations of the energy grade line between the intake and the outlet channel;

g $[g] = \frac{\mathrm{m}}{\mathrm{s}^2}$ .... acceleration due to gravity (9.80665 m/s²)

3.2 Results and Precision of the Flow Measurement

Upon review, Alden was able to use data recorded from 12:32am EST until 2:12am EST on July 24, 2013. During the 100 minutes of the measurement 1,208 data points were recorded. The mean values of are listed in Table 2. In addition the data of the flow measurement is plotted in Figure 3.

In order to draw conclusions about the precision of the flow measurement the uncertainty of the test results has been estimated. Estimates of precision indices were made from standard deviations of the test data. Bias uncertainties were estimated from comparative tests and experiences. The two components were propagated separately from individual measurements to the final results. Elementary error source uncertainties for each component were combined by the root sum square method and the precision uncertainty was estimated as the precision indices multiplied by the Student t factor. The overall uncertainty of the result is reported as the sum of the bias and the precision uncertainties at 95% confidence level [1].

The following uncertainties were calculated using the method explained above [1]:

- water level measurement: 0.15%
- flow measurement: 1.24%
- reference tunnel flow calculation: 1.65%

Table 2: Results of the flow measurement [1]. The guaranteed flow amount for the reference hydraulic head was calculated according to section 2.1

Date: July 24, 2013 12:32am EST to 2:12am EST	
average upstream water level HGL_{1m}=EGL_{1m}	170.99 m
average downstream water level HGL_{2m} EGL_{2m}	164.78 m 165,19 m
average flow meter reading Q_m	503.94 m³/s
guaranteed flow amount for the reference hydraulic head $H_{ref} = 5.6$m, Q_{ref}	**495.1 m³/s**

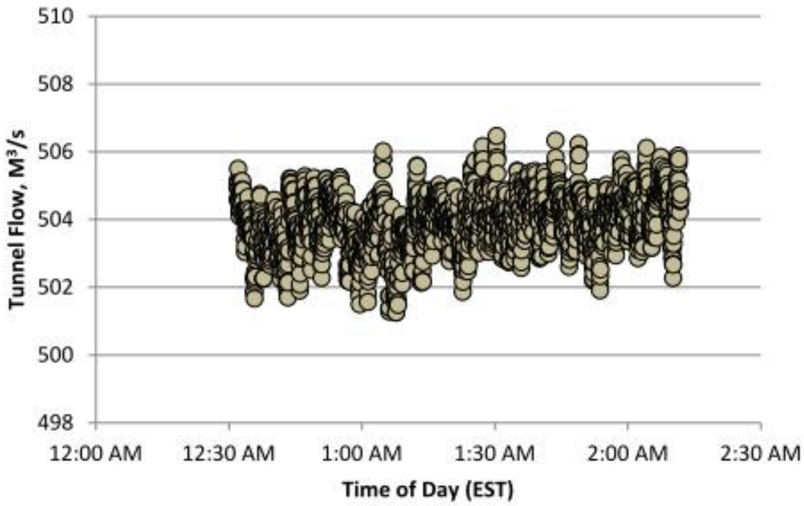

Figure 3: tunnel flow measurement data: 1,208 recorded points during 100 minutes of measurement from 12:32am to 2:12am EST [1]

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The Authors

R. Fritzer is a 1992 graduate of the Civil Engineering Faculty of Innsbruck University and began working in the Water & Environment Department of ILF Consulting Engineers in 1993. He has been involved in numerous hydropower projects as design engineer for the civil and hydraulic engineering structures. For the last ten years Reinhard Fritzer has been responsible mainly as project manager for ILF's large hydropower and pumped storage plants in central Europe and abroad. Reinhard Fritzer was responsible for the steady flow calculations for the Niagara Tunnel Project.

E. Gschnitzer was born in 1958 in Italy and graduated from the University of Innsbruck, Austria with a Ph.-D. in Geology/Mineralogy in 1986. Before joining STRABAG in 2004, he was employed as a Contract Manager with ILBAU Underground Construction in Spittal, Austria, worked as Deputy Manager on the Second Manapouri Tunnel for the Fletcher Dillingham Ilbau Joint Venture in Te Anau, New Zealand and as Technical Director for Prader Ltd. Tunneling in Zurich, Switzerland. After one year as Area Director with STRABAG AG in Vienna, Austria he moved to, Canada in 2005 as Area Director and Project Manager for Strabag Inc., Ontario, Canada. There he was responsible for the execution of the Design Build Agreement for the Niagara Tunnel Project as well as for business development of the North American markets. Between January 2003 and Febraury 2005 Ernst Gschnitzer was Managing Director for STRABAG S.p.A. in Bologna, Italy. Since March 2015 Mr. Gschnitzer works in his current function as a Managing Director of STRABAG AG in Vienna, Austria.

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Niagara Tunnel Project – Hydraulic Aspects

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Niagara Tunnel Project – Hydraulic Aspects
Overview

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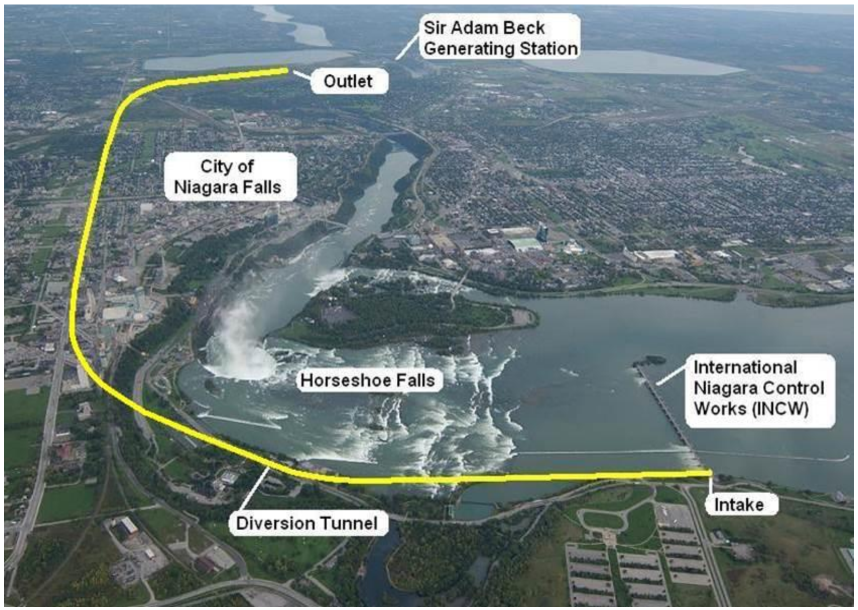


source: Strabag, 2014

Niagara Tunnel Project – Hydraulic Aspects
Description of the Niagara Tunnel Project

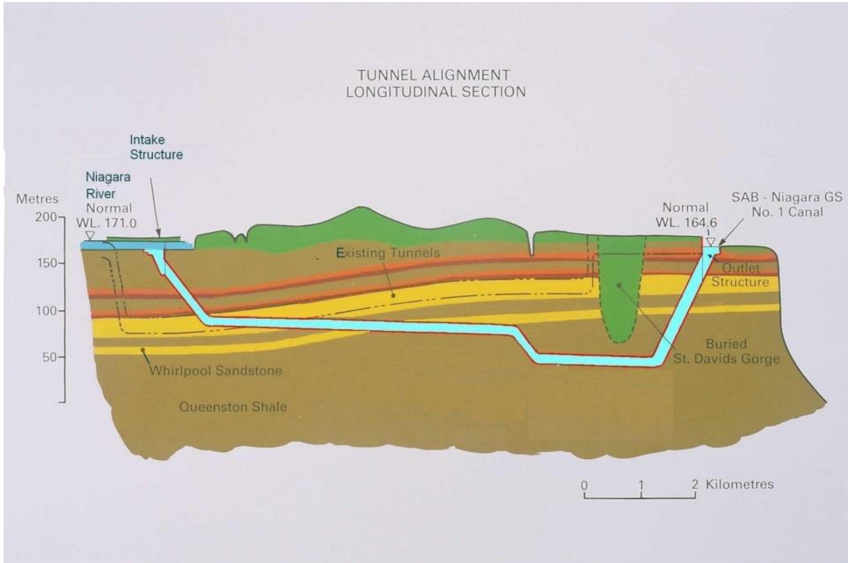
available flow for power generation Canada and U.S.A.	up to 6,000 m^3/s
previous diversion capacity for power generation in Canada	1,800 m^3/s
additional capacity to be supplied by the Niagara Tunnel in Canada	500 m^3/s
new total diversion capacity of Canada	**2,300 m^3/s**

Ontario Power Conduit 2 Final Inspection 1905

Niagara Tunnel Final Inspection 2013

Niagara Tunnel Project – Hydraulic Aspects
Technical Details

- **Length:** 10.4 km
- **Hydraulic Head:** 5.60 m
  - Difference of the elevations of the energy grade line between intake and outlet channel
- **Max Overburden:** 150 m
  - Operating Pressure: 15 bar
- **TBM Diameter:** 14.44 m
  - lined tunnel diameter: 12.7
- **Worlds largest hard rock TBM**
- **Design Life:**
  - 90 years without maintenance



source: Grunicke and Ristić, 2012

Niagara Tunnel Project – Hydraulic Aspects
Intake

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Niagara Tunnel Project – Hydraulic Aspects
Intake flooding

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Niagara Tunnel Project – Hydraulic Aspects
Outlet

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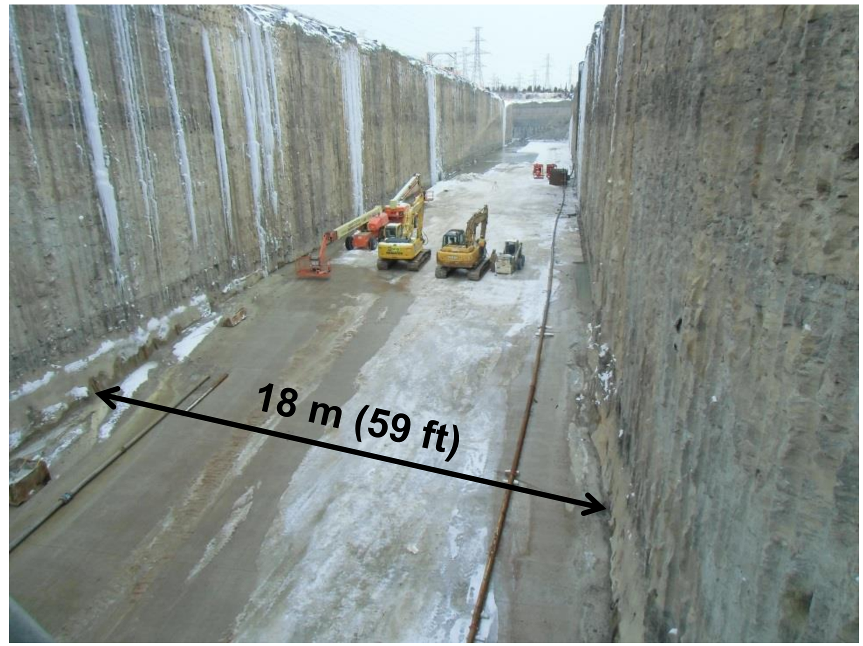

Hydraulic Design – steady flow condition

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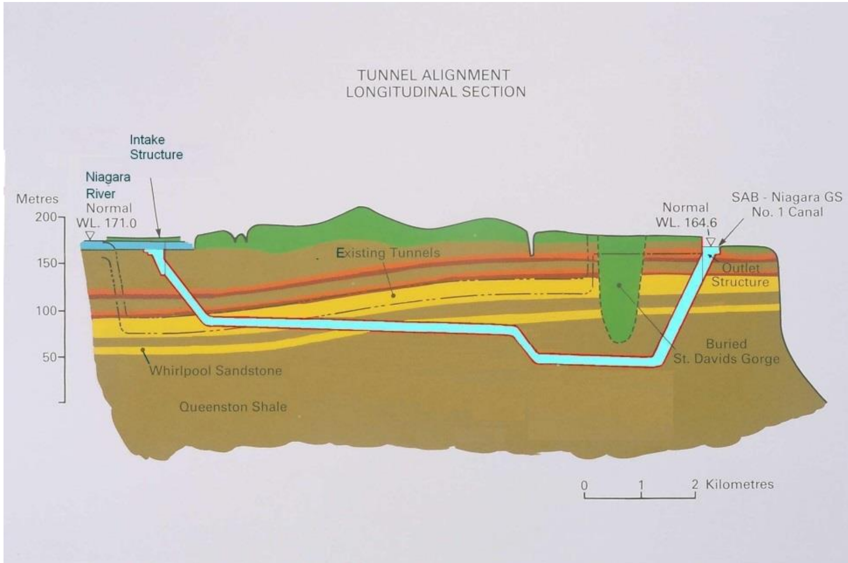
Calculation of discharge:

- HEC-RAS 4.1.0 => Manning / Strickler
  - calculation of total friction losses and comparison with the reference hydraulic head of 5,6 m
  - different roughness coefficients based on intense literature studies and experiences from several similar hydropower projects in Europe are applied to 5 sections
  - increased roughness coefficients to consider additional losses through 3-dimensional flow
- formula of Prandtl-Colebrook reaches its limit due to the extreme dimensions of the tunnel

Niagara Tunnel Project – Hydraulic Aspects
Steady flow condition - Results

- **Summation of assumed friction losses**
  - intake channel: $k_{st} = 25 \rightarrow$ free surface
  - intake structure: $k_{st} = 80$
  - diversion tunnel: $k_{st} = 85$
  - outlet structure: $k_{st} = 80$
  - outlet channel: $k_{st} = 30 \rightarrow$ free surface
  - divided into more than 30 subsections with different diameters

- **Results**
  - Predicted total hydraulic losses of the tunnel at a flow of 500 m³/s are 5,32 m
  - 513 m³/s can theoretically be conveyed at available head of 5,6 m

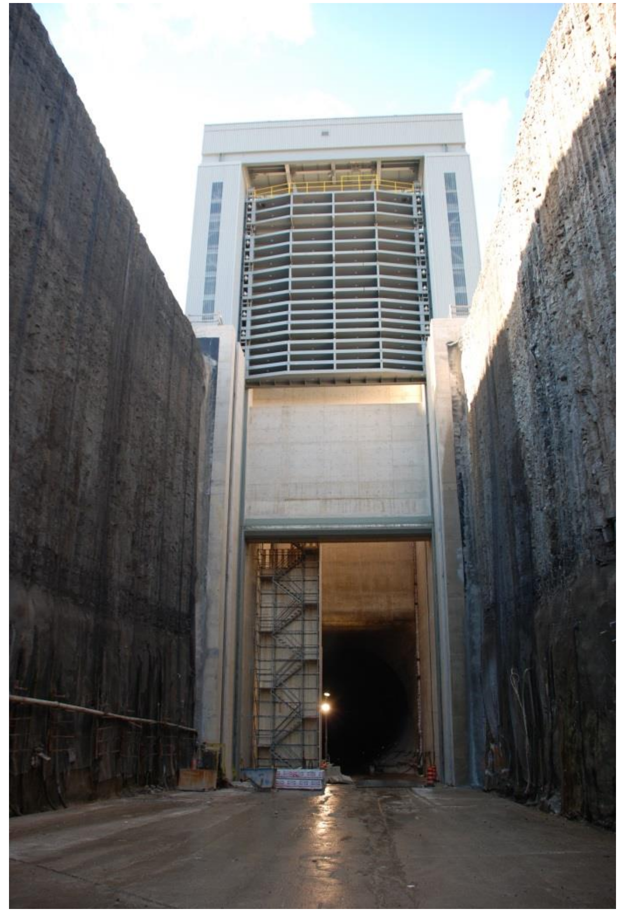


source: Grunicke and Ristić, 2012

Niagara Tunnel Project – Hydraulic Aspects
Unsteady Flow Condition

- **Computation Method**
  - software developed by ILF, proven in numerous earlier projects
  - based on the “method of characteristics”
- **Investigated Operations of downstream Gate**
- **closing time 165 min (9,900 sec)**

	speed m/min	
	100% – 12%	12% - 0%
powered lowering	0.16	0.16
	0.16	0.025
unpowered lowering	0.31	0.31
	0.31	0.025
raising	0.16	0.16



Unsteady flow condition – Results

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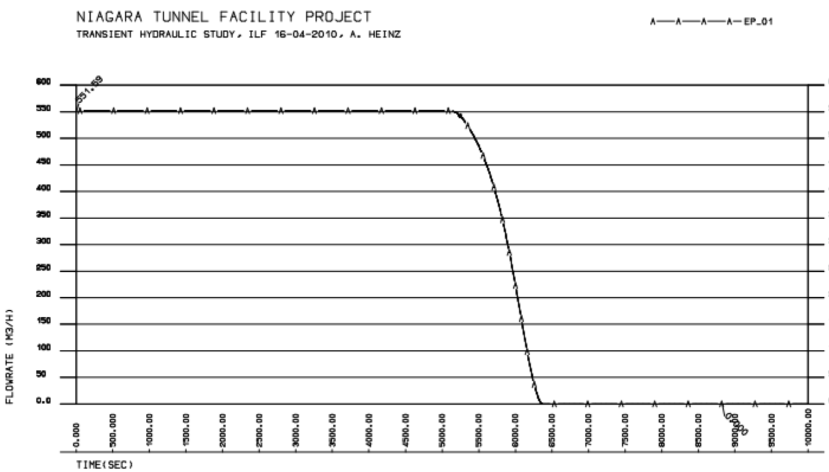
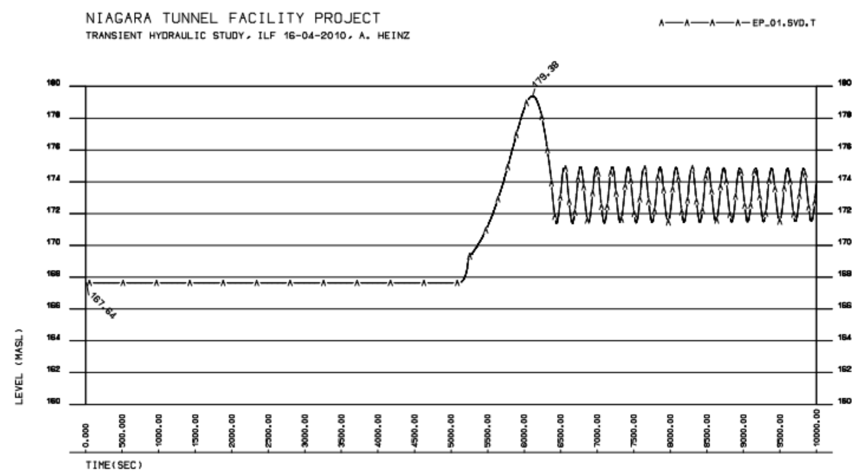
Closing Patterns

- predicted overflow using constant-speed closing patterns,
- overflow can be avoided by lowering the gate with a decreased speed of 0.025 m/min at 12-0% remaining open position



HISTORY OF FLOWRATES

CASE 02: GATE HOIST LOWERING POWERED (MAX. 0.16M/MIN) - TWO SPEED



HISTORY OF SURGE TANK LEVEL

CASE 02: GATE HOIST LOWERING POWERED (MAX. 0.16M/MIN) - TWO SPEED

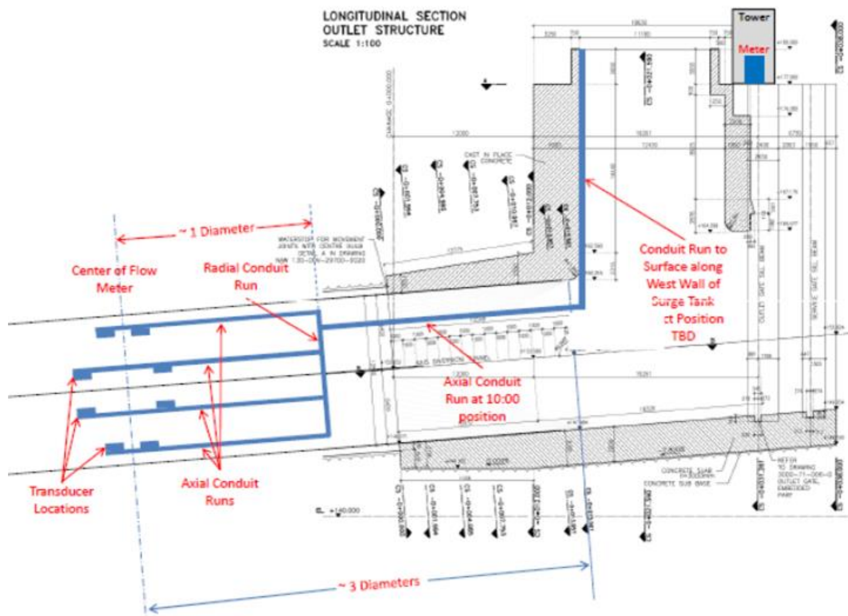
Niagara Tunnel Project – Hydraulic Aspects

Contractual Obligations

- **Flow verification test to determine:**
  - as-constructed capacity
  - at a reference hydraulic head of H_{ref} = 5.6 m (difference of the elevation of the energy grade line between intake and outlet channel)
- **Test**
  - Conducted by Alden Research Laboratory Inc.
  - as an independent party

Flow Measurement Method

- **Multipath Ultrasonic Transit-Time Flowmeter**
  - 8-path flowmeter system with redundant transducers and cable (32 total)
  - maximum uncertainty (including random and systematic errors) shall be +/- 2 %
- **Water levels measured using pressure transducers**
  - intake water level gauge located at the bend in the approach wall
  - outlet water level gauge located at the radius at the end of the outlet canal
  - approximately 45 m upstream from the PGS canal



source: Alden, 2013

Test Procedure

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- **Installation**
  - of temporary water elevation measurement instrumentation by Alden
- **Start of official test**
  - after upstream and downstream water levels had stabilized
  - average differential head is constant over a period of 30 minutes within +/- 1.0 %
- **mathematical determination of reference tunnel flow**

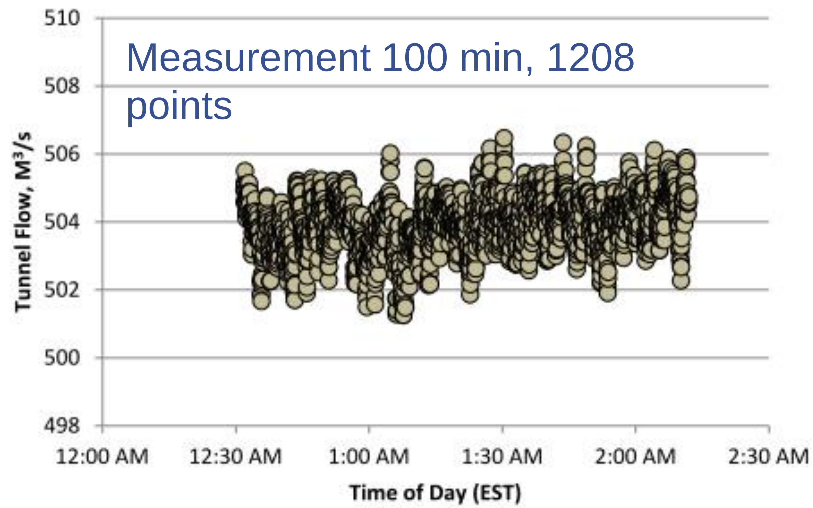
$$EGL_{1m} = HGL_{1m}$$

$$EGL_{2m} = HGL_{2m} + \frac{\left(\frac{Q_m}{A}\right)^2}{2g} \text{ with } A = (HGL_{2m} - inv.) \cdot w$$

$$Q_{ref} = Q_m \cdot \sqrt{\frac{H_{ref}}{EGL_{1m} - EGL_{2m}}}$$

Niagara Tunnel Project – Hydraulic Aspects
Final Results

date: July 24, 2013 12:32am EST to 2:12am EST	
average upstream water level	170.99m (EGL_{1m} = 170.99m)
average downstream water level	164.78m (EGL_{2m} = 165.19m)
average flow meter reading	503.94m³/s
guaranteed flow amount specified for the reference hydraulic head H_{ref} = 5.6 m, Q_{ref}	**495.1m³/s**



source: Alden, 2013

Niagara Tunnel Project – Hydraulic Aspects

Thank you for your attention!