Hogg’s Hollow Highway 401 Bridge Widening, Toronto, Canada

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ABSTRACT
Highway 401 stretches in Ontario, Canada, from Windsor (at Michigan, U.S.A. boarder) to Quebec border. Part of Highway 401 is presently the busiest highway in North America, carrying about a half million vehicles per day along its sixteen lanes (core-collector system) in the Yonge Street and the central Toronto area. In the early 2000’s the Ministry Transportation of Ontario (MTO) decided to widen the existing east-bound collector lanes of Highway 401 at Yonge Street over the Hogg’s Hollow Valley in Toronto. The Hogg’s Hollow is an approximately 500m long, 40m deep valley, incised by the West Don River, prior to its joining the East Don River further south and forming the Don Valley. This paper describes the geology of the site along a summary of the available geotechnical and structural information. The selection of the most suitable widening alternative and its implementation are also discussed.

RÉSUMÉ
L’autoroute 401 s’étire en Ontario, au Canada, de Windsor (au Michigan, frontière américaine) à la frontière du Québec. Partie de l’autoroute 401 est actuellement l’autoroute la plus achalandée en Amérique du Nord, transportant environ un demi-million de véhicules par jour sur ses seize voies (système à collecteur principal) dans la rue Yonge et le centre de Toronto. Au début des années 2000, le ministère des Transports de l’Ontario (MTO) a décidé d’élargir les voies de circulation existantes de l’autoroute 401 en direction est de la rue Yonge sur la Hogg’s Hollow Valley à Toronto. La Hogg’s Hollow est une vallée d’environ 500 m de long et de 40 m de profondeur, incisée par la rivière West Don, avant de rejoindre la rivière East Don plus au sud et de former la vallée du Don. Cet article décrit la géologie du site avec un résumé des informations géotechniques et structurelles disponibles. Le choix de la variante d'élargissement la plus appropriée et sa mise en œuvre sont également abordés.

1 INTRODUCTION

The inner under deck truss of the Hogg’s Hollow Bridge, originally known as the Yonge Boulevard Viaduct, was constructed in 1928 and this provided a much-needed bypass of the deep ravine. In 1952, the Toronto Bypass (today’s Highway 401) was routed over the bridge after adding a south truss bridge. Thus in 1964, before the construction of the east bound and west bound collector lanes’ bridges, Highway 401 crossed the Hogg’s Hollow via two (twin) bridges, currently referred to as the ‘core lanes. Subsequently, two additional bridges were constructed: one on the north side to carry the west bound collector lanes and one on the south side to carry the east bound collector lanes (see Figures.1, 2 &3).

Figure 1. Site Map

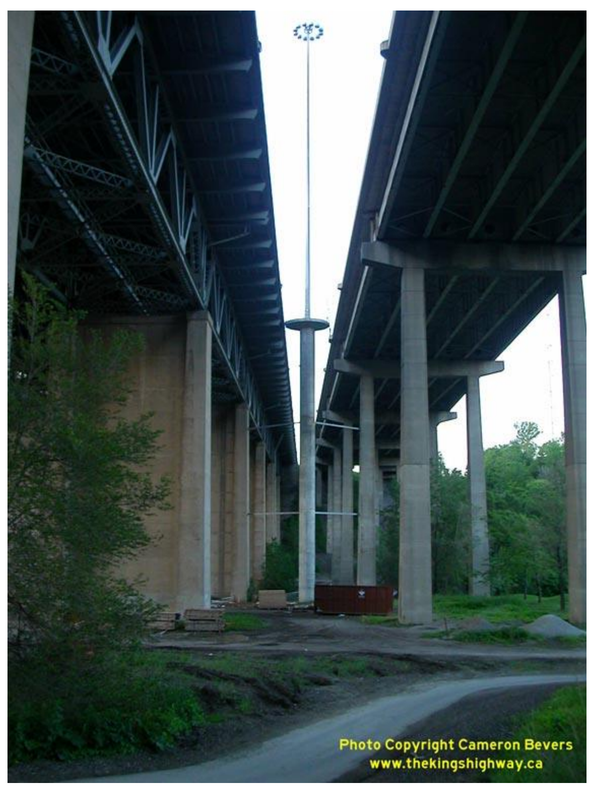

Figure 2. Underneath of the EB Core/Collector Bridges, Photo by Cameron Bevers, www.thekingshighway.ca. Reprinted with permission

To reduce the congestion on one of North America's busiest highways, in the early 2000's Ministry of Transportation Ontario (MTO) proposed to construct an additional lane along the east bound collectors. The proposed widening was on the south side of the existing EB collector bridge.

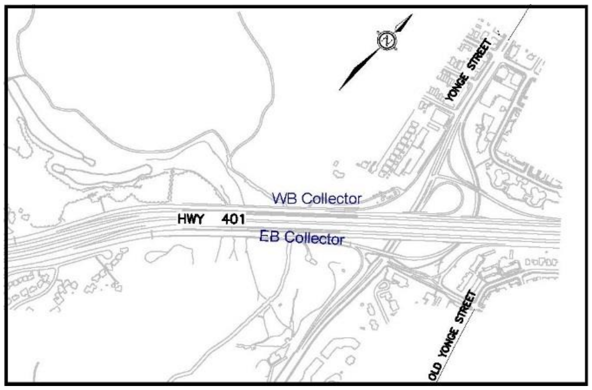

Figure 3. Key Plan

A literature search for available information showed the presence of glacial and interglacial deposits along the east and west banks, with post glacial soils within the floor of the river valley. The depth to the surface of the bedrock in the Hogg's Hollow valley is more than 40m. In general, near the west bank, sandy silt tills with sand and some silt layers are prevalent. Within the river's valley, sands with silt zones predominate, with occasional glacial till layers, while towards and on the east bank, the presence of clayey to sandy silt tills with silt and sand seams was indicated.

Building a new bridge to support a single lane traffic was an option but a prohibitively expensive one for this 453 m long bridge, especially in view of the difficult access to the valley and equipment movement on the slopes along with challenges and high construction cost for the construction of tall piers in excess of 30 m high at some pier locations. The modification of the existing foundations was another option to support the additional loading caused by the proposed widening. However, similar difficulties and cost concerns were presented for this option. Subsequent to the initial assessment, consideration was given by the structural engineers for this project to widen the existing bridge by about 0.5 to 0.6 m each side rather than widening only on the south side to avoid building a new structure or altering the existing foundations.

2 SITE DESCRIPTION AND GEOLOGY

The topography along this section of Highway 401 in Toronto is characterized by two major river valleys; namely West Don River and East Don River. In general, the grade along this section of the highway starts falling west of the subject bridges from about El. 183 m towards the West Don River valley to about EL. 137 m. It then rises toward east of the subject bridges to about El. 169 m, dropping again to about El. 135 m along the East Don River. Within this general area, the overburden consists of Pleistocene or glacial deposits, which were laid down under a vast thickness of ice or else within glacial rivers and lakes associated with them. Soils were deposited by the ice itself from the glacial till deposits which are mainly unsorted by water action, while those formed by melt waters are stratified deposits. Near and within the present river valleys, these deposits were modified by rivers emptying into the glacial Lake Iroquois (fore-runner of the present Lake Ontario) and sand, gravel, silt and minor amounts of clay were deposited. Finally, further modifications took place in modern times by the West Don River and East Don River. In summary, below some man-made fill and/or modern post-glacial deposits, the stretch of land along Highway 401, east and west of the subject bridge is underlain by glacial tills with some sand and silt layers,

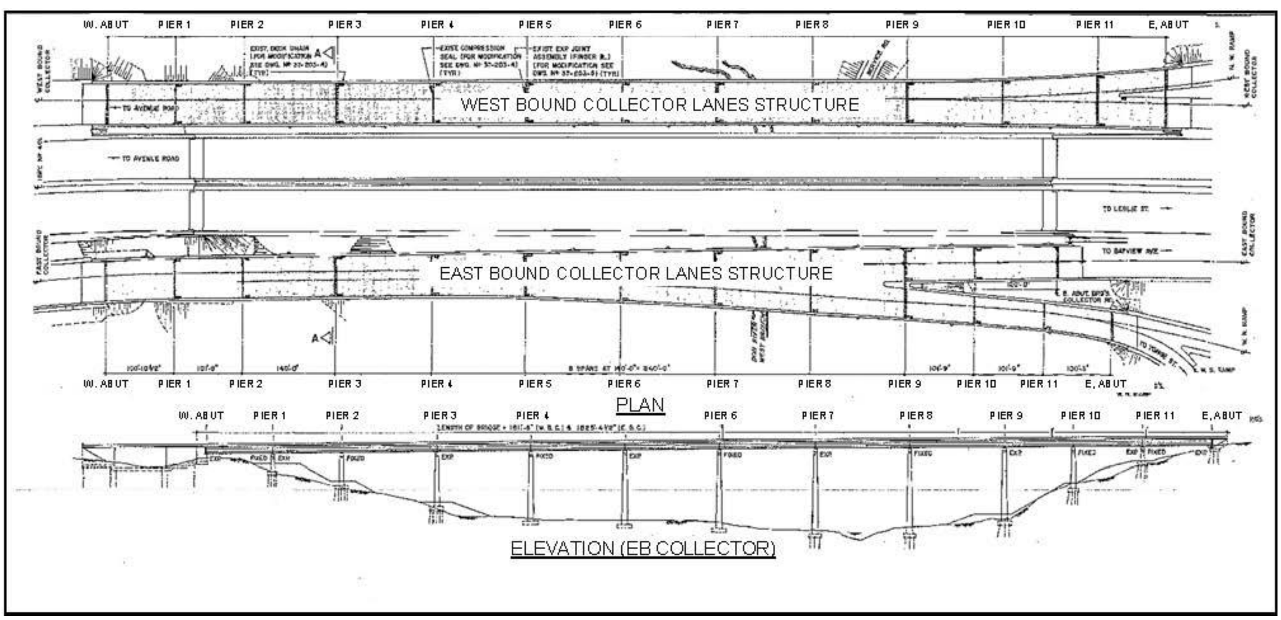

Figure 4. Hogg's Hollow Collector Road Bridge Plan and Elevation

except near the river where fine-grained granular deposits comprised of sands and silts are more prevalent.

The depth of the overburden in the general area can be expected to be more than 40 m. The overburden, which consists of glacial and interglacial deposits, is underlain at about El. 95 m to 100 m by the Georgian Bay shale bedrock which contains limestone and siltstone interbeds. The surface of the bedrock within the Hogg's Hollow valley generally dips from about El. 90 m to as low as El. 60 m. This formation belongs to the Upper Ordovician Periods of the Paleozoic Era and is approximately 440 million years old.

Hogg's Hollow valley is some 0.5 km wide with ground surface elevations cresting at about El, 168 m and 171 m at the west and east ends respectively. The west slope of the valley is relatively flatter with ground elevations ranging from about El. 143 m at Pier 3 location, gradually dropping to El. 134 m at Pier 7 (Figure 4). The river bottom is at about El. 129 m, while the river banks are at El. 134 m. The east slope rises more sharply compared to the west slope with ground surface elevations being 139 m, 154 m and 168 m at the locations of Piers 9, 10 and 11, respectively, (see Figure 5).

From visual observations it appears that the east slope was steepened at some locations from its original slope of 2H:1V to about 1.5H:1V.

3 SUBSURFACE CONDITIONS

The subsurface conditions at the bridge locations were collected from the previous investigations available through MTO Foundation Library. The historical data showed that between 1958 and 1964, a number of foundation investigations was carried out and boreholes were drilled for the proposed (presently existing) structures. In 1999, AGRA Earth & Environmental Limited (AGRA) conducted a foundation investigation for proposed high mast lighting (now constructed) at the site. In 2004, Shaheen & Peaker drilled two boreholes to supplement the available information. In 2005, Shaheen & Peaker conducted additional borehole investigation at the top of the river valley on the east side.

The borehole information from the aforementioned investigations is shown on the profile drawing presented in Figure 5.

The available information shows that in general, near the west bank of the valley, sandy silt tills with sand and some silt layers prevail, while within the valley itself, sands and silts predominate, with occasional till layers. On the east bank, the soil consists of clayey to sandy silt till with silt and sand layers. At the west bank, the groundwater level was recorded at about El 156 m or about 12 m below the ground surface, dropping to about El. 135 m near Pier 3 with a further drop to El. 133 m at Piers 4 and 5. At Piers 6, 7, and 8, near the bottom of the valley, the groundwater level was recorded at El. 132 to 133 m. The water level in the River was typically at El. 131 m. Further west up the valley hill, groundwater level was recorded at El. 133 m or about 6 m below the ground surface. Near Pier 11 and near the top of the valley, the groundwater level was recorded at about El. 136 m, with a perched water table at about 2 m below the ground surface.

4 EXISTING FOUNDATIONS

The existing structural and geotechnical data were examined. Design drawing in some cases were available and in other cases some assumptions had to be made.

Based on the existing data, the west abutment is supported on a spread footing at El. 162.2 m. Piers 4, 5, and 6 are also supported on spread footings at El. 136.2 m, 134.7 m, and 133.2 m, respectively. The available drawing indicated that the piers are trestle type, i.e., there are two columns on a single footing which provide support for a reinforced concrete beam on which the steel girders rest. The trestle is a rigid-frame type structure which would be normally sensitive to differential settlements.

Piers 1, 2, and 3 to the west of the river bed and Piers 10 and 11 and the East Abutments, located to the east of the river bed are supported on 12BP53 (HP 310 X 79) steel H-Piles. The design working load for all the piles is 50 tons or 445 kN per pile. The design pile length generally ranged from 15 to 30 ft. or 4.6 m to 9.1 m. The following was extracted from 'Construction Note' on a drawing: "(1) All H-Piles shall be driven to practical refusal and shall be controlled in the field by means of the Hiley Formula according D.H.O. Standards DD1218 and DD1219.". Piers 1, 2, and 3 are trestle-type. There are 32 piles supporting Pier 1 while Piers 2 and 3 are on 42 and 50 piles, respectively. Some of the piles are battered at 4V:1H to resist lateral loads.

Near Pier 9, the eastbound collector lanes start to separate into two roadways; on the north side, the reduced collector lanes extend further eastward while on the south side, one lane, Ramp W.N. connects one collector lane to Yonge Street. At Pier 10, two northerly footings support the eastbound collector lanes while the southerly footing supports Ramp W.N. The substructures are trestle-type. At about 37 m east of Pier 10, the bridge structure carrying the east bound collectors ends, while Ramp W.N. extends further east over Pier 11. The abutment for the collectors is an L-shaped element with Southeast Retaining Wall I on the south side of the abutment and Southeast Retaining Wall II on the north side. Pier 11 and the east abutment are supported on driven 12BP53 (HP310X79) steel H-Piles with a design working load of 50 tons or 445 kN per pile. There are 20 piles and 19 piles supporting Pier 11 and the east abutment, respectively. Some piles are battered at 4V:1H.

Ramp W.N. leads to Yonge Street. East abutment of Ramp W.N. structure is supported on 17 12BP53 (HP310X79) steel H-Piles, with a design load of 50 tons (445 kN) per pile. Some of the piles are battered at 3V:1H.

Piers 7, 8, and 9 are supported on cast-in place concrete piles with 22-inch (560 mm) stem diameter. The following ‘Construction Notes’ were shown on one of the available design drawings regarding these piles; “(1) The reinforcement of the 22” case-in-place concrete piles are to be supplied and placed by the contractor… (2) Pre-boring of piles may be required if found necessary by the engineer to prevent excessive vibration in the vicinity of existing structure.” At piers 7 and 8, a large cap supports two reinforced concrete columns, which in turn support a reinforced concrete beam, carrying steel girders. Pier 7 is on the west side of the West Don River, and Pier 8 is on the east of the river. The design length of each pile is 20 ft (6.1 m). Pier 7 is supported by 33 piles (11 rows, 3 piles per row) while Pier 8 is supported on 45 piles (12 rows, 3 piles per row). The design load is 110 tons (980 kN) per pile. Some piles are batted at 4:1. Pier 9 also has a rigid-frame trestle-type support with three reinforced concrete columns supporting a reinforced concrete beam on which rest. The pile cap is 26.2 m long, 5.5 m wide and 1.8 m thick and founded at El. 135.6 m. It is supported by 48 cast-in-place concrete piles of 22” (560 mm) stem diameter. The design length of the piles is 20 ft (6.1 m). the design working load of each concrete pile is 110 tons (980 kN). Some piles are battered at 4V:1H.

Table 1 presents a summary of the available foundation information.

Table 1. Existing Foundation Information

Spread Footing Support Elements (Allowable Design Pressure 290 kPa)		
Location	Footing Size (m)	Founding Elevation (m)
West Abutment	3.7x18.3	162.2
Pier 4	6.4x17.4	136.2
Pier 5	6.4x18.6	134.7
Pier 6	7.0x20.0	133.2
12BP53 (310X79) Steel H-Pile Supported Elements (Design Load 445 kN/Pile)		
Location	Pile Length (m)	Number of Piles
Pier 1 (EB)	6.1	16
Pier 1 (WB)	4.6	16
Pier 2 (EB)	6.1	21
Pier 2 (WB)	4.6	21
Pier 3 (EB)	9.1	25
Pier 3 (WB)	7.6	25
Pier 10 (EB)	9.1	16
Pier 10 (WB)	7.6	16
Pier 10 (Ramp WN)	9.1	32
Pier 11 Ramp WN (EB)	9.1	10
Pier 11 Ramp WN (WB)	7.6	10
East Abut.	9.1	19
East Abut. Ramp WN (EB)	7.6	9
East Abut. Ramp WN (WB)	4.6	8
22” Stem Dia. Concrete Pile Supported Elements (Design Load 980 kN/Pile)		
Location	Pile Length (m)	Number of Piles
Pier 7	6.1	33
Pier 8	6.1	45
Pier 9	6.1	48

5 DISCUSSION

The feasibility of rehabilitation of the existing bridge structure, involving widening of the existing bridge by about 0.5 m to 0.6 m on each side as well as the replacement of the existing concrete deck, was examined. The concrete slab thickness needed to be increased to meet the more recent traffic load requirements in accordance with Canadian Highway Bridge Design Code (CHBDC). In order to further study the feasibility, the structural engineering consultants provided a list of the foundation loads estimation with consideration of loads increasing due factors affecting the foundation loads. The primary author was asked to make an assessment based on the existing subsurface and structural information if the existing foundations are adequate for the proposed widening/rehabilitation scheme. Table 2 presents the existing foundation loads as well as the anticipated foundation loads resulting from the proposed rehabilitation.

Table 2. Existing and Anticipated Foundation Loads

Spread Footing Support Elements

Location	Loads (kPa) Serviceability Limit State (SLS)		Load Increase (%)	Loads (kPa) Ultimate Limit State (ULS)		Load Increase (%)
	After Rehab	Existing		After Rehab	Existing	
West Abutment	270	240	12.5	290	260	11.5
Pier 4	290	260	11.5	350	320	9.4
Pier 5	310	280	10.7	370	330	12.1
Pier 6	240	220	9.1	310	280	10.7

12BP53 (310X79) Steel H-Pile Supported Elements

Location	Pile Axial Force (kN) Serviceability Limit State (SLS)		Load Increase (%)	Pile Axial Force (kN) Ultimate Limit State (ULS)		Load Increase (%)
	After Rehab	Existing		After Rehab	Existing	
Pier 1	510	460	10.9	680	610	11.5
Pier 2	610	550	10.9	850	770	10.4
Pier 3	530	480	10.4	690	620	11.3
Pier 10	720	650	10.8	1000	900	11.1
Pier 10 (Ramp WN)	600	540	11.1	850	770	10.4
Pier 11 Ramp WN	670	600	11.7	890	800	11.3
East Abut.	510	460	10.9	690	620	11.3
East Abut. Ramp WN	420	380	10.5	590	530	11.3

22” Stem Dia. Concrete Pile Supported Elements

Location	Pile Axial Force (kN) Serviceability Limit State (SLS)		Load Increase (%)	Pile Axial Force (kN) Ultimate Limit State (ULS)		Load Increase (%)
	After Rehab	Existing		After Rehab	Existing	
Pier 7	1200	1080	11.1	1600	1440	11.1
Pier 8	1100	990	11.1	1450	1310	10.7
Pier 9	710	640	10.9	920	830	10.8

Recommendations regarding the design of the existing foundations were contained in a letter-report issued in November 1963 by DHO (Department of Highways of Ontario). The recommended geotechnical allowable bearing value was 3 tsf or 287 kPa. This value can be considered to approximately correspond to geotechnical bearing at SLS with the current limit states design. Based on the 2000’s standards, this value was considered conservative. At most footing locations, due to the primarily granular nature of the founding soils, typical geotechnical resistances available would have been in excess of 800 kPa for factored geotechnical resistance at ULS. At SLS typical geotechnical resistance would be 400 kPa. As shown in Table 2, the loads on the spread footings would typically increase by 10 to 12%. The additional settlements due to increased loads were estimated to be less than 6 mm at the west abutment and less than 10 mm at pier locations. The recorded SPT N-values below the founding elevations are generally in the range of 50 to 80 blows per 0.3 m penetration, except at Pier 6 location, where an N-value of 20 blows/0.3m was recorded. This could be possibly be attributed to soil disturbance at a level slightly above the groundwater table due to the fact that in those days wash-boring methods were used rather than augering. Fortunately, at this location both the existing and the increase loads were considerably below the available resistance values. It is of interest to note that a sheet pile enclosure was used around Pier 6 footing for scour protection, as was common in the 1950’s and 1960’s for bridge constructions near water courses in Toronto area.

Piers 1, 2, 3, 10, 11 and East Abutment are supported on driven 12BP53(HP310X79) steel H-Piles using a design working load of 50 tons (445 kN) per pile. In accordance with the early 2000’s design standards, it was assessed that typical axial resistance of HP 310X79 steel piles, driven to practical refusal into dense to very dense soil was in the order of 650 kN/pile for SLS and 1,000 kN/pile for ULS. As can be seen from Table 2, the calculated existing loads on the piles were at or below the assessed available SLS value (which roughly equals to the allowable design values used in the 1960’s) of 650 kN/pile. The increase in loading due to rehabilitation would be at about 10 to 12 %, bringing the total loads to between 420 kN/pile and 720 kN/pile. At two locations the loads would exceed the assessed SLS value of 650 kN by a margin of 20 kN and 70 kN. The loads were found to be safe, but some settlements could be expected. These settlements were determined to range from less than 6 mm at some foundation locations to possibly as much as 12 mm at other locations where insufficient subsurface information was available (e.g., no borehole information at Pier 2).

Piers 7, 8, and 9 were supported on 22-inch (560 mm) stem diameter cast-in-place concrete piles. Based on the correspondence found in the historical data, piles supporting the Piers 7 and 8 were believed to be “Franki”-type piles. The Franki pile is a driven, cast-in-place concrete pile with an enlarged base. The enlarged base is formed within the founding soil to maximize capacity of piles. Assuming the piles at Piers 7 and 8 were installed to their full design length, the bottom of the piles was estimated to be at El. 125.3 m. The soils generally consisted of compact to very dense silty find sand to sandy silt below groundwater table. These conditions are particularly well-suited for Franki piles, in comparison with more conventional drilled and cast-in-place concrete piles (i.e., drilled caissons), as Franki piles are not adversely affected by the presence of high water table, unlike drilled caissons. In spite of this, as the project progressed, the most difficult decision of the assessment evolved to be whether the Franki-type piles, as constructed in the early 1960’s, would be capable of sustaining the additional loads, especially since the casing was left permanently in place (i.e., very limited frictional side support). Franki piles themselves undergone several changes since their inception in 1909. Initially, the casing was top driven, being equipped with a sacrificial steel plate at the bottom until 1926, when the dry concrete plug was introduced. Installation with a vibrated shaft was not introduced until 1960 and hydraulic vibrating hammer was unavailable until 1971. While the latter was definitely not available at the time of the construction of the bridge, even the vibrated shaft was probably not available, as historically innovations were introduced in Europe and took some years to arrive in Ontario, usually through Quebec; primarily due to legal concerns, civil engineers in Ontario tend to take a rather conservative approach. In the 1960’s Franki had a larger presence than present in Ontario, including a branch, which was subsequently closed and operations, albeit few, were dealt with through their Montreal Office. The main concern in this instance was the size of the bulb formed, which could not be verified. Another concern was why a permanent steel casing was utilized, as this would largely reduce the side friction. A phone conversation with a former Franki Canada employee from Toronto operations indicated that the use of permanent steel casing was not uncommon in those days, which helped to formulate a decision.

The piles supporting Piers 7 and 8 were designed and constructed to provide a working load capacity of 110 tons or 980kN per pile. This can be considered to be approximately equivalent of axial resistance at serviceability limit states (SLS). For Pier 7, the calculated existing design value exceeded the original design value by 100kN or by 10.2%. Additional loadings due to the proposed widening would add another 120kN and this would lead to an increase of 22.5% in excess of the original design values.

For Pier 8, the calculated existing load per pile was 990kN i.e., only marginally higher than the original design value of 980kN. The additional load due to widening was calculated by the structural engineers to be 110kN, that is, about 11.2% higher than the original design. The loads supporting Pier 9 were below the design capacity and marginally above the original design value of 980kN per pile, even with the additional loads. Nevertheless, as the additional loads (70kN) represented an increase of about 11% over the existing, some settlements could be anticipated, albeit minor.

6 CONCLUSIONS

Building a new bridge for the proposed widening, which was originally considered by MTO to be prohibitively costly. Supplementing the capacity of the existing foundations was considered but found to be very costly (in the range of many millions), especially in view of the very difficult access conditions for the construction. In addition, the consulting structural engineer was not in favour of this option. For example, consideration was given to augment the existing spread footing using steel mini-piles, but the structural engineer was of the opinion that it was difficult to predict the behavior of the footings when supplemented with mini-piles (how these two systems would work together, load transfers, etc. being indeterminate at that time).

In every geotechnical engineer's career some occasions arise where difficult decisions need to be made, considering the risks involved. Using the somewhat inadequate information available and the best engineering analyses within the capacity of the geotechnical engineering firm, the final decision rested with the engineer's gut feeling whether to accept that the additional loads would safely be carried by the existing foundations of this important structure. The final recommendations were that the existing foundations would safely carry the additional loads provided some possible settlements, typically less than 10mm, would be acceptable. This was found to be acceptable by the structural engineers and by MTO. However, as Highway 401 is considered the lifeline of Ontario's economy, it was decided to monitor the settlements of the foundations immediately before, during and for several months after the construction. Corrosion of the steel (e.g. H-piles) and sulphate attack on concrete (e.g. concrete piles) were also considered but the risk was found to be acceptable.

7 EPILOQUE

The construction started in 2006 and settlements recorded were generally less than 6 mm, except for one Friday afternoon when high settlements at one of the spread footing foundations were recorded on one side which could be attributed to a possible overturning or possible surveying error. Contacting the surveyors to go back for a check on this particular footing proved to be futile, as they were gone for the weekend. That weekend was a stressful one for the primary author, in case a failure was becoming imminent. A revisit by the surveyors the following Monday thankfully revealed that the reading made on that particular footing was in error and the project continued to proceed without any major settlements, showing that with good cooperation between parties involved, large saving can be had for taxpayers.

8 REFERENCES

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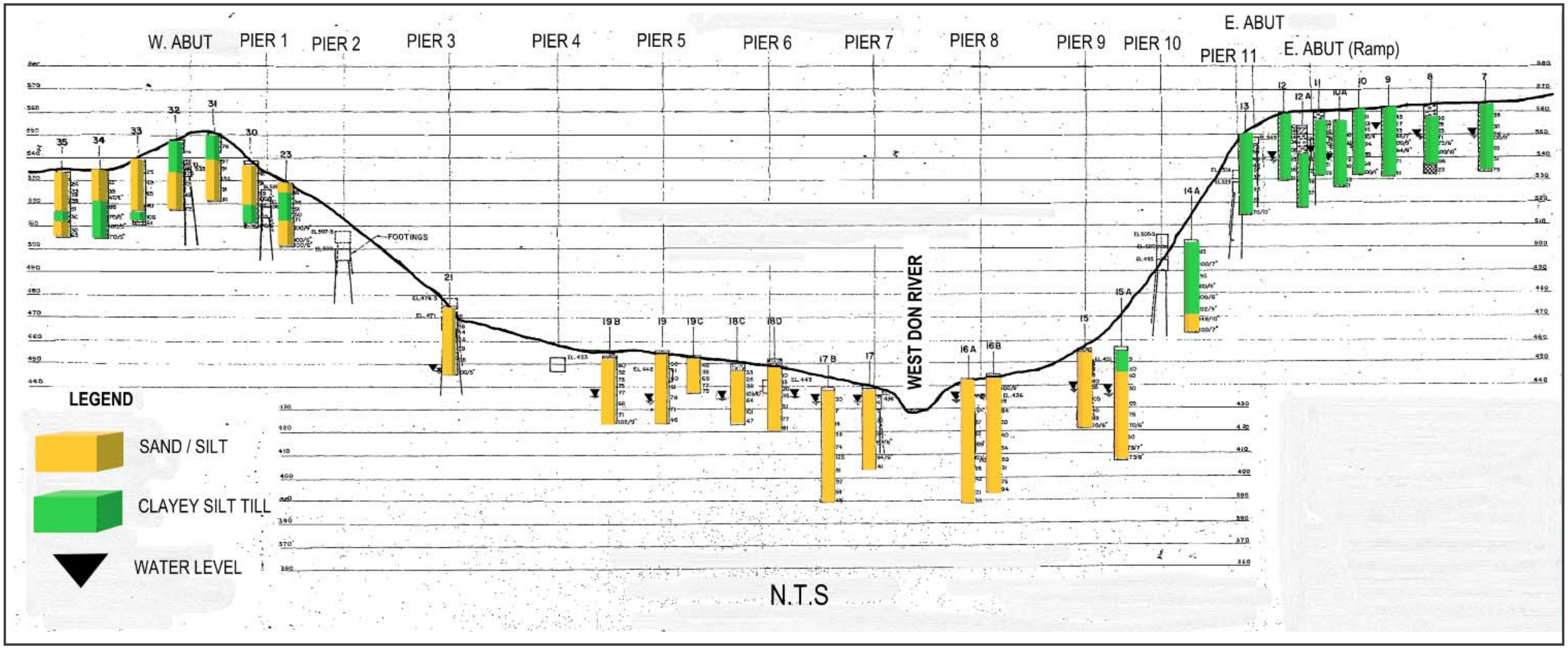

Figure 5. Subsurface Profile along Eastbound Collector Road Bridge (reproduced from Foundation Investigation Report, August 2006)