Geotechnical Issues Associated With the Design and Construction of the Middle River Bridge

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GEOTECHNICAL ISSUES ASSOCIATED WITH THE DESIGN AND CONSTRUCTION OF THE MIDDLE RIVER BRIDGE

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ABSTRACT

The existing short single span, single lane, concrete bridge constructed in 1930 across the Middle River in the Town of Mc Kellar, Ontario required replacement as a result of age and sub-standard approach roadway geometrics. In 1986, a geotechnical investigation was undertaken to upgrade the existing bridge. This investigation was undertaken primarily within the river since the new bridge structure was contemplated on an alignment with improved geometrics. The proposed structure was designed but was never constructed. In 2009, the need for a new structure was revived with the idea of placing it on the existing alignment with minor alignment modifications. However, as a result of buried timber being encountered during the geotechnical investigation, conventional drilling had to be abandoned and an air track rig used to ascertain the nature of the subsurface conditions. Thirty two (32) probe holes/boreholes were undertaken to define the preferred location of the new abutment piles. During bridge construction, the designed batter piles slipped along the steeply dipping rock surface at the west abutment and had to be retained by a massive deadman anchor. This paper addresses the issues encountered and concerns raised during the geotechnical investigations and how these were addressed and resolved. This site although small exhibited challenging ground conditions resulting from the lack of historic evidence, physical constraints imposed on the geotechnical investigation by the river and surroundings, and overall expenditures.

INTRODUCTION

The proposed replacement of the existing single span, single lane Middle River Bridge in the Town of Mc Kellar, Ontario required a geotechnical investigation to determine the extent, nature and characteristics of the subsurface soils/materials at the proposed abutment locations. The findings of this investigation and a previous investigation in 1986 were used to facilitate the geotechnical and structural designs, and the construction of the foundations for the new bridge structure.

SITE LOCATION AND DESCRIPTION

The existing bridge was constructed in 1930. However, it is understood that prior to the construction of the existing bridge, the river crossing consisted of a two-span wooden structure resting on wooden piles situated at about midspan of the river. Remnant wooden piles from this structure can still be noticed below water level from the deck of the existing bridge.

As shown in the aerial photograph of the site taken from Google Maps, Fig.1, the river was constricted from both abutments with a larger constriction from the east abutment.

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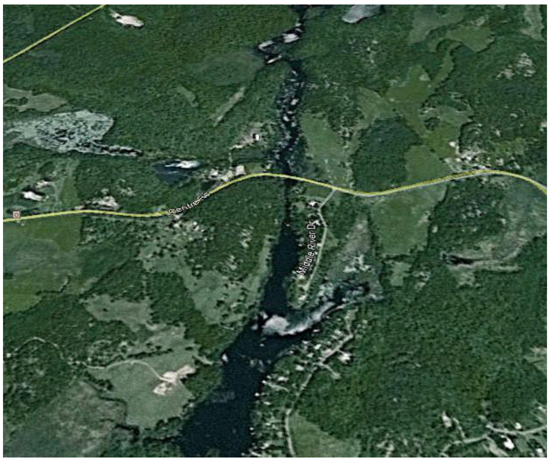

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Fig.1. Aerial Photograph of Site

This river constriction was likely done to reduce the span of the existing bridge. Whether this constriction was undertaken during the initial construction or the construction in 1930 is unknown.

On both sides of the crossing and in the vicinity of the existing bridge bedrock outcrops are evident to the north and south of the crossing with the outcrops more pronounced at the west abutment location. The roadway alignment is substandard resulting from the rising hillside to the southwest consisting of massive bedrock. The present alignment suggests that this bedrock was likely avoided because of the expenditure that would have been incurred to construct a satisfactory vertical and horizontal alignment.

FIELD INVESTIGATIONS

Four (4) distinct geotechnical investigations were undertaken. These investigations were undertaken on May 4, May 11-12, June 8, and July 20 and 21. The details of the investigations are provided below.

May 4, 2009 Investigation

As a result of the narrow roadway width and to ensure unimpeded commuter traffic flow, only two boreholes BH-1 and BH-2 could be undertaken. This investigation was undertaken with a CME truck mounted drill rig. These two boreholes could only be drilled at approximately 22 m east of the east abutment and on the inside of the guardrails demarcating the roadway pavement width. The locations of these boreholes are shown on Fig.2. Since it was proposed to maintain the crossing on or close to the existing alignment, the offset of 22 m was considered too far away to be useful for bridge foundation design.

Boreholes (BH-1 and BH-2) were drilled to depth of 7.4 and 6 m, respectively, below the existing ground level at the borehole locations. The subsoil stratigraphy at both borehole locations consisted of a 25 mm thick layer of asphalt surface treatment representing the paved roadway surface. Below this surficial layer the stratigraphy differed somewhat in the two boreholes.

This surficial layer was followed in BH-1 by a 440 mm thick poorly graded damp sand and gravel fill in a loose state of compactness. This layer was absent in BH-2. Poorly graded damp sand fill in a loose to compact state of compactness was encountered below the sand and gravel fill for a depth of 1.1 m in BH-1. Sandfill in a similar state with a thickness of 0.8 m was encountered in BH-2.

In contrast, the sandy silt layer was absent in BH-2, but instead a 2.9 m thick soft silty clay layer was encountered. Pieces of wood were also encountered within the clay layer at around a depth of 2.3 m but was absent below 3.1 m. No wood was encountered in BH-1. Underlying the silty clay, sand and gravel till in a compact state of compactness was encountered at approximately the same elevation in both boreholes and at a depth of about 3.6 m below ground level. This till layer was about 300 mm thick in BH-1 and 700 mm in BH-2.

Bedrock was encountered in BH-1 immediately below the silty sand and gravel till layer and two core runs were undertaken providing RQD values of 85 % and 100 % in core runs 1 and 2 indicating that the bedrock was of good to excellent quality.

No bedrock was cored in BH-2 but the SPT uncorrected "N" value of 50 plus obtained for the last 75 mm of the SPT testing was indicative that bedrock was encountered. Since the holes were outside of the required abutment location and BH-1 was cored, it was decided not to undertake any further coring in the interest of minimizing the cost of the investigation.

Groundwater levels were recorded during the drilling at 3.2 and 3.4 m below ground level with soil sloughing in the boreholes reported at around 3.1 m below ground level.

May 11-12, 2009 Investigation

To be able to investigate closer to the existing bridge abutments, arrangements were made with the Town of McKellar for closure of the roadway for 2 days. An additional six (6) boreholes BH's 3 - 8 were undertaken between May 11 and May 12. These boreholes were located closer to the existing bridge abutment and near to the locations anticipated for the proposed abutments of the new bridge. Except for BH 7 which was drilled around the middle of the river from the existing bridge deck through a hole in the deck, the remaining holes were drilled on either side of the existing bridge abutments. The locations of the boreholes are shown in Fig.2.

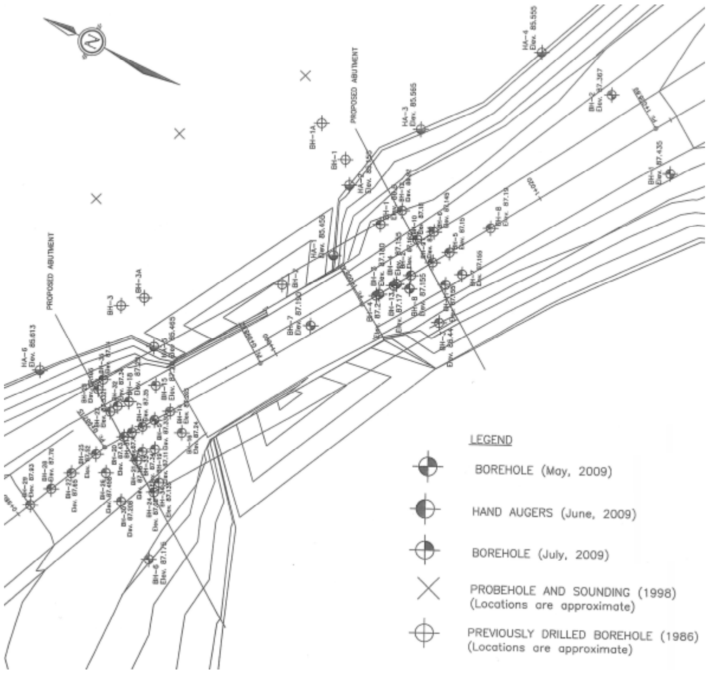

Fig.2. Locations of Boreholes (1986 and 2009)

Except for BH's 6 and 7 which were drilled to 9 and 15 m respectively, the remaining boreholes were drilled to depths varying from 2 to 4m. The short depth of investigation was not intended but resulted from non-soil materials encountered during the drilling of these holes preventing further penetration of the boreholes. The material frequently encountered was wood which could not be penetrated even using a diamond coring bit. This wood was generally encountered at about 3 m depth below the ground surface. As a result, the subsurface soils could not be explored to the depth of bedrock, as required.

Sand fill or sand and gravel fill was encountered in all boreholes, except BH-7, below the 25 mm asphalt surface treatment, with thickness varying from 2 to 3 m. All boreholes excepting BH-6 and 7 encountered wood below the fill sand. In BH-6, bedrock was encountered below a depth of 6 m and was investigated by two core runs to a depth of 3 m in the bedrock. The RQD values obtained varied from 77% for the first core run to 99% in the second core run. Each core run was about 1.5 m in length. These values indicate the rock encountered to be of good to excellent quality.

On the other hand, in BH-7, the soils encountered were generally of very soft consistency below the top 5 m to the depth of investigation of 14.5 m since the SPT uncorrected "N" values were zero throughout the 9 m thickness of this layer. Above this depth, to the bottom of the riverbed, soil samples could not be recovered and hence the material was inferred as being of very soft consistency as well. The water level of the river was approximately 1.8 m below the top of the bridge deck and with the river bed a further 2.2 m below.

Groundwater was recorded in BH's 5, 6, and 7 at a depth of around 1.8 m below ground while sloughing was observed in BH's 3 and 4 at about 1.52 m below ground. In general, the groundwater level was about the same as that of the river water level at the time of the investigation.

June 8, 2009 Investigation

On June 8, several hand auger probes were undertaken outside of the existing guardrail locations to assess the ground conditions to examine the feasibility of construction of a detour bridge. These probes were limited to a depth of 0.76 m mm as a result of encountering rock fill which was visible along the east and west shorelines. These probe holes encountered sand fill varying from 0.2 to 1 m in thickness before encountering refusal on rockfill.

Further Geotechnical Investigations

The overall findings of the three geotechnical investigations resulted in uncertainty of the nature of the subsoil in the location of the existing bridge abutments. The presence of wood in some of the boreholes led to the assumption that corduroy may have been used in fording the river to allow a single span bridge to be constructed. Several assumptions were made regarding the possible layout of the corduroy.

Concepts on whether this consisted of a single or multiple layers were envisaged along with a possible crib construction with rock in-fill since there were some instances where rockfill was encountered below wood. All of these concerns caused much debate on the types of foundations that could be used for the new bridge. The preferred type of foundation was envisaged to be a piled one since the soils encountered in the boreholes were loose or very soft and would not be suitable for shallow foundations.

At one point, consideration was given to utilize the existing foundations since the bridge appeared not to have undergone any distress at the abutments. However, this was ruled out in discussion with the Structural Engineer since there was uncertainty of the type and condition of the substructure under the existing bridge abutments.

Finally, after much brain storming it was decided to investigate the extent of the corduroy at the abutment locations in an attempt to define locations on either side of the crossing where H-piles could be driven without encountering wood. This was done as a result of uncertainty whether the piles could be driven through this material readily and as well through rock fill if this existed in considerable thickness.

Discussions were held with local geotechnical drilling companies regarding their capabilities in penetrating wood. However, no satisfactory feedback was obtained but rather they all advised that the core barrels irrespective of type would become stuck once wood was encountered.

The only alternative left was to engage the advice of owners of Air track drilling equipment since these were capable of rapid penetrating most materials overlying bedrock, and in fact these rigs are often used in proofing the location and depth of bedrock. This type of rig uses compressed air for advancing a percussion type drill bit with depth.

One of the two local air track rig companies who indicated an interest in the project was taken on a tour and review of the site to ensure that the conditions would be suitable for his rig to undertake the desired probes. On site, the issues encountered were discussed and the areas that were required to be probed identified. The locations were based on a decision made by the Structural Engineer in consort with the Geotechnical Engineer to limit the proposed bridge to a particular length which would allow for an economical design for the proposed single span structure.

Since it was important that the nature and depth of subsurface soils be investigated to facilitate the design of the new bridge the additional geotechnical investigation was proposed using the air-track drill rig. This rig has the capability of penetrating the wood encountered at both abutment locations and of investigating the subsoil below the wood to the depth of bedrock which appears to be about 14 to 16 m below the existing ground. The depth to bedrock was obtained from a previous geotechnical investigation and a report made available to during our May 11-12 geotechnical investigation.

This 1986 geotechnical investigation was undertaken by an Ontario Consulting firm and was entitled "Geotechnical Investigation and Stability Analysis, Middle River Bridge Township of McKellar". This investigation was undertaken for the design of a two-lane bridge to be located close to alignment as the existing bridge. Boreholes were done within the river using a barge. These holes showed the stratigraphy in the river down to ground that was inferred to be bedrock. No boreholes were done along the roadway and in the vicinity of the existing abutments.

In order to undertake this additional investigation as quickly as possible, the roadway was required to be closed to public traffic for a minimum of two (2) days and a maximum of four (4) days. This request was received with some concern by the Town since minimal closure or no closure of the road was preferred. However, following discussions between the Town and Trow, closure of the roadway was approved for the start date of July 20. It was also agreed that the work would be expedited to ensure that the road would be opened as quickly as possible.

From our geotechnical investigations done so far, it appears that the existing river crossing was constricted to provide for a short bridge using corduroy construction from the existing banks. The results of the investigations done so far indicate that rock fill is above the corduroy but what lies below the corduroy was unknown. The findings of this investigation are outlined below.

July 20 – 21, 2009 Investigation

A total of thirty-two (32) probes were undertaken on both sides of the existing bridge abutments at locations shown in Fig.2. The only setback in using the air track rig was that samples of soils could not be obtained except that soils and materials that were blown out of the hole were visually observed as the probe was advanced. Materials encountered were also identified by the rig operator based on his experience and examination of the cuttings as the drilling advanced with depth. Figure 3 shows the air track rig in operation.

During this probing wood, steel (chain) and rock were encountered at various locations resulting in several holes being drilled to define the most suitable location for the abutments where any obstructions if present would be minimal and would not result in serious problems in driving of piles to bedrock. Bedrock was encountered in several probe holes on both sides of the crossing. The probing operation went smoothly, and as a result the investigation was completed in two days.

Fig.3. Air Track Rig Drilling Probe/Boreholes

Presentation of Subsoil Information

In addition to presenting the subsoil information obtained from each borehole and probe hole on borehole logs, it was decided to present the information graphically in terms of stratigraphic sections to allow for easier interpretation and evaluation of the subsurface soils to aid in the design and construction of foundation elements.

Several stratigraphic cross-sections and a longitudinal stratigraphic profile were evaluated. The longitudinal profile is shown in Fig. 4. These results were used to set the locations of the new bridge abutments. These locations are shown on the longitudinal profile. Also shown on the longitudinal profile is the bedrock line which also played an important role in the decision of where the abutment was to be located. As can be seen, the bedrock profile is steep on the west side beyond the existing abutment with the slope at almost 45 degrees while on the east side the slope is slightly flatter.

FOUNDATION DESIGN RECOMMENDATIONS

General

A deep foundation system was considered to be the only suitable foundation type for supporting the proposed bridge structure as a result of the relatively "weak" nature of the subsurface soils reflected by soft to very soft soils and/or loose to very loose soils below the riverbed level and overlying the bedrock. This condition was evident both within the river channel and in the approach roadway areas.

Driven open and closed toe pipe piles and H-piles were considered suitable foundation systems. However, as a result of the possibility of encountering obstructions during driving such as rockfill and wood, and to ensure that the piles were satisfactorily seated into the underlying bedrock as a result of its sloping nature, the preference was given to the use of an H-pile system to support the bridge abutments.

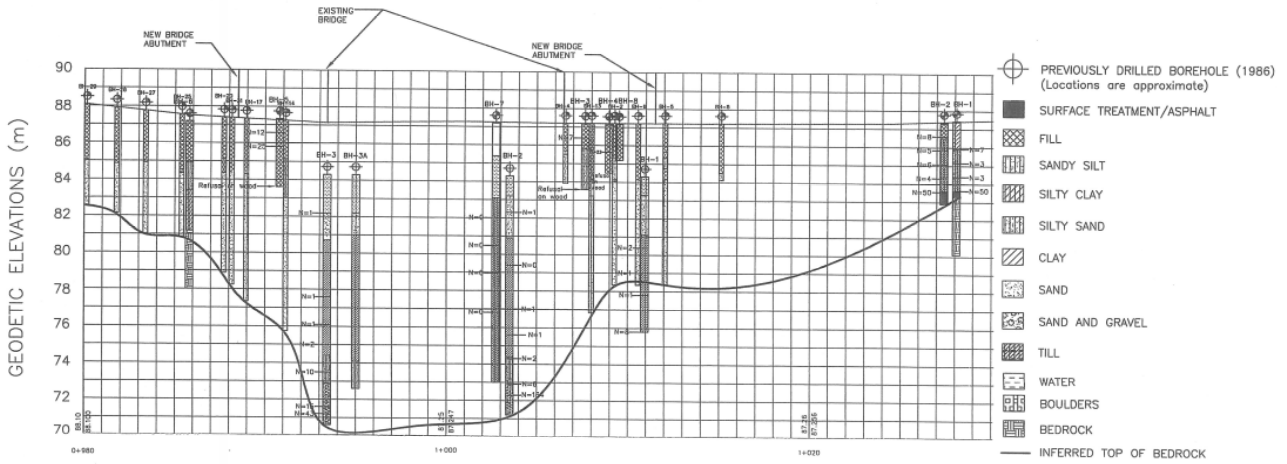

Fig.4. Longitudinal Stratigraphic Profile

The conceptual design of the abutment foundations proposed by the Structural Engineer consisted of both vertical and raking/batter piles to resist axial and horizontal loading, respectively, that would result from superimposed bridge loadings from traffic, and including other external feasible loadings.

Geotechnical Design Considerations

The geotechnical considerations for the design of the abutment piles followed the guidelines of the Canadian Foundation Manual 4th Edition, 2006 and the Canadian Highway Bridge Design Code. In both these Manuals, the Limit States Design approach is recommended instead of the conventional Allowable Stress Design (ASD) or Working Stress Design approach.

Since the piles were to be terminated on bedrock, the piles would be essentially achieving their geotechnical resistance from end bearing on rock Very little resistance was expected from the soil surrounding the shaft as a result of the weak nature of the surrounding soils and short length of pile (approximately 7 m) involved for mobilizing shaft resistance.

As a result of the piles terminating on bedrock the design of the pile would be based on the structural resistance of the pile since the rock was of good to excellent quality. There is the possibility that as a result of raising the grade of the existing roadway about 1 to 1.5 m, this load can result in some consolidation of the soil at the abutment locations. However, based on the variable nature of the ground consisting of corduroy and relatively compact upper soil layers above the top of the abutment footing at El 85.0 the soils above El 85 are expected to act raft like and result in minimal transfer of pressures to the soils surrounding the piles. As a result, the downdrag would be minimal and the drag load would be of little consequence in affecting the capacity of the piles in axial compression. Similarly, the inclined/batter piles would be subjected to minimal lateral squeezing pressures.

Generally, downdrag and attendant drag loads become an issue when long piles, greater than 20 m in length, are used in settling ground.

Limit States Design-Ultimate and Factored Resistances. The ultimate structural resistance of the HP 310 x 110 steel sections in CSA–G40.21 Grade 350 steel can be determined using the yield stress of the steel as 350 MPa and the area of steel when used as a structural column subjected to axial compression

In terms of the factored structural resistance, a resistance factor of 0.9 is normally applied to the ultimate structural resistance. However, when the member is used as a pile a further reduction factor of 0.75 is recommended to be applied to the factored structural resistance of the column to account for pile installation difficulties and uncertainties in subsurface conditions.

Hence, an overall desirable resistance factor of 0.67 would be applied to the ultimate structural resistance of a column in axial compression when used as a pile. While it can be argued that a higher resistance factor can be used, for this site and its subsurface conditions the 0.67 factor is considered appropriate. For steel of yield stress less than 350 MPa, the same resistance factor is applicable.

For the raking/batter piles, the factored structural resistance would be determined using the factored axial compressive resistance resolved along the alignment of the rake/batter.

The factored structural resistance for the pile in question is determined to be 3306 kN. This resistance is far in excess of the design factored load of 1140 kN per pile for ULS design requirements for each abutment. While not significant for this site as a result of the magnitude of the design factored loads, the factored structural resistance depends not only on the strength of the steel but also on the characteristics of the rock on which it bears. While this rock is very competent, the steeply sloping nature of the bedrock where the abutment piles are to be located would be of concern for higher factored loads. Intuitively, one would likely reduce the factored structural resistance by a further 50% to cater for inclined bedding etc which may not be penetrated deeply by the rock points required for pile installation. Hence, the factored structural resistance at ULS of 1658 kN would be appropriate from a practical viewpoint unless rock cores are examined to determine bedding planes, their orientation etc. This was not undertaken as a result of site constraints.

Limit States Design – Serviceability Limit States Design Considerations. The factored resistance at the Serviceability Limit State (SLS) would result in a resistance of 3701kN using a resistance factor of 1.0 instead of 0.9 but maintaining the 0.75 reduction factor for uncertainties in subsurface condition and pile installation difficulties. The factored resistance at the SLS is also far in excess of the design unfactored load of 670 kN provided for the SLS design for both abutments. The same argument regarding the Ultimate Limit States factored resistance holds as well for the factored resistance at the Serviceability Limit State. Hence, the advisable factored resistance at the SLS would be 1850 kN.

Since settlement is one of the considerations for performance of the structure at the Serviceability Limit State, this requirement would also dictate the determination of an acceptable design and as such influence the magnitude of superimposed loadings. In relation to settlement considerations, it is expected that the settlement would result from the elastic compression of the pile with and elastic compression of the rock

For the design unfactored load, the pile elastic compression is determined through the load, elastic modulus of the pile section, length of pile, and area of pile using the relationship-PL/AE. This elastic compression of the pile has been determined to be approximately 2 mm. A further 2 mm of elastic compression is obtained for the same load transferred by the pile tip to the bedrock. Hence, under the unfactored load, an estimated settlement of 4mm is anticipated using the elastic settlement relationship similar to that used for shallow foundations.

Pile Lateral Loading at Ultimate and Serviceability Limit States. The abutment piles for the proposed structure will consist of vertical and raking/batter piles. The batter piles are required to resist primarily the lateral loads resulting from the horizontal braking forces resulting of traffic loadings, the lateral earth pressures of the backfill behind the abutment, horizontal component of seismic loads and other forces or loads that will result in horizontal loads on the piles. The batter piles will also assist in accommodating axial loads and moment combinations that may occur as a result of superimposed dead and live loads. The vertical piles also contribute to the lateral load resistance.

The ultimate lateral resistances of the piles have been evaluated in terms of the ultimate and factored geotechnical resistance that can be accommodated by a single pile using the Broms approach. The Broms method of analysis as outlined in the Canadian Foundation Manual 4th Edition, 2006 takes into consideration horizontal loads only and is suggested to be satisfactory for loading conditions associated with small structures and hence fits the scenario of the proposed structure.

The analyses were undertaken assuming the soils were cohesive and also cohesionless to cater for the mixed subsurface soil conditions. The results are shown in the Table below for the values obtained when the analyses considered cohesive and cohesionless soils and for the short and long pile categories.

On the basis of the results obtained, the results of the long pile cohesive are recommended for design. Since there are batter piles in the foundation system, the horizontal resistance afforded by these piles would provide for additional lateral restraint. It should be noted however that the contribution of these piles would depend on how well they are fixed to the rock since these would be founded on the sloping bedrock as well. In this case the reduced axial compressive resistance both at the ULS and SLS along with their factored resistances should be used in the determination of the additional horizontal component of resistance that will be afforded by the pile toe.

In relation to the Serviceability Limit States, a Serviceability Limit State Reaction of 80 kN results in a deflection of 25 mm at the head of the pile.

Table 1.Summary of Lateral Load Resistances

Pile Size	**Ultimate Lateral Geotechnical Resistance**	**Factored Lateral Geotechnical Resistance**	**Pile Category**
HP 310 x 110	590 kN	295 kN	Short Pile Cohesionless
HP 310 x 110	280 kN	140 kN	Long Pile Cohesionless
HP 310 x 110	290 kN	145 kN	Short Pile Cohesive
HP 310 x 110	180 kN	90 kN	Long Pile Cohesive

Pile Installation Considerations

Piles driven to competent bedrock require the use of rock points fitted at their toe. Rock points shall conform to OPSS 3000.201 or shall be Titus H bearing pile point, rock injector model. The piles shall be driven into bedrock in accordance with OPSS 903. It should be noted that as a result of possible rockfill and /or wood being encountered during driving of the piles, it has been decided that pile lengths would serve as an additional criterion in determining whether piles have reached the desired toe elevation on the bedrock.

For the east and west abutments, the estimated minimum pile length for a vertical and batter piles at the abutment locations would be about 6.6 m minimum below the pile top cut off at around El 84.3. Hence, in relation to the existing ground level at El 88/87, the minimum pile length to be installed to reach the bedrock surface would be around 11 m below ground. Piles achieving refusal before this length has been achieved would be suspect of encountering subsurface obstructions. This situation would require that the pile be extracted and the obstruction be removed by excavation. If wood is encountered and cannot be penetrated then a trench excavation may be warranted across the width of the abutment to ensure that the obstruction is removed. The logs of the boreholes and probe holes and the stratigraphic cross sections and longitudinal profile all provide information on ground conditions encountered during the geotechnical investigation.

Pile Driving Analyzer (PDA) testing is not recommended for this site since it may not prove to be of practical value, rather pile driving operations should preferably be monitored visually to ensure that the each pile has been installed to bear onto bedrock. This monitoring is also required to check on whether piles are suitably embedded in the bedrock or whether piles are slipping along the sloping rock surface. However, if the Contractor desires to monitor the piles using the PDA test, the Contractor and PDA testing Company need to be fully aware of the site conditions so that testing can be undertaken appropriately.

In addition to the requirements in OPSS 903, pile driving shall at no time exceed 30 blows per inch (absolute refusal) so as to avoid damage of the pile toe. A resistance to driving of about 20 blows per inch is considered practical refusal. These resistances would apply to when the prescribed minimum length of pile has been installed to attain the bedrock surface. Where a diesel hammer is used a hammer with a manufacturer rated energy of 40-50 kilo joules would be desirable.

Groundwater Construction Related Considerations

With the proposed pile to cutoff elevation at around EL 83.4, the proposed footing or pile cap would be below the ground water level at the site which is influenced by the water level of the River. This high ground water could pose problems to construction of the abutment footings and bridge seat requiring dewatering or keeping water out. The methodology to be used to allow satisfactory construction will depend on when the construction is likely to take place - spring, summer or winter. This methodology to be used is the Contractor's responsibility. However, the Contractor is required to provide his scheme for review by the Bridge Design Engineer. Where steel sheet piles may be considered for use, attention has to be paid to the stream hydraulics where the flow of water within the river may be impacted. Relevant permits for undertaking work within the river environment and the regulatory requirements for this work have to be obtained from the appropriate jurisdictions and would be the Contractor's responsibility.

GRADE RAISE, BACKFILL, AND WING WALLS

As mentioned and discussed previously, there will be some grade raise in the order of 1 to 1.5 m at the approach to the proposed bridge and as well backfill of the abutments. Both these operations can be undertaken with granular material meeting the specifications for Granular "A" and Granular "B" materials (OPSS 1010).

However, for backfill at the abutment location, Granular "A" is preferred for its free draining characteristics. This backfill should meet the requirements of Granular "A" and placed in layers not exceeding 150 mm in thickness for the full width of the abutment and each layer should be compacted to 95% of the Standard Proctor Maximum Dry Density below the bottom 300 mm from the finished subgrade and 100 % within the top 300 mm of finished subgrade.

The perforated pipe shown just above the pile cap/footing can also be used behind the wall but its effectiveness would depend on its location with respect to the prevailing water level in the river. Alternatively, the upper level perforated pipe can be constructed to discharge outside of the wing walls of the abutments toward the river. The need for one or both pipes is best resolved on site during the actual construction when site conditions are better visualized.

Geotextile filter fabric should be placed between the wall and the surface of the existing excavation to prevent loss of retained soil in the soft riverbed soils. A non-woven geotextile (Terrafix 270 R or equivalent) can be used where the site conditions warrant as instructed by the Client's representative.

Wing walls beyond the bridge abutments are required to contain the roadway fill within the existing roadway footprint on either side of the existing bridge. As a result of likely minor settlement of the fill, wing walls constructed of gabion baskets were recommended as these would be flexible enough to accommodate minor movements and would not be subject to cracking in the case of conventional reinforced concrete constructed wing walls. In addition, the gabion wing walls can be constructed to accommodate small changes in alignment. Since these walls will be less than about 2 m or less in height, standard modular configurations provided by the Gabion suppliers can be used. The gabion wing walls were, however, not used.

Lateral Earth Pressures

For the design of the abutment backwall/stem, the backwall will be subjected to lateral earth pressures and stresses due to traffic loading as well as compaction stresses.
The abutment walls should be designed for lateral earth pressures according to the following expression, assuming a triangular pressure distribution:

$$p = k (g h + q)$$

Where

p = the pressure in kPa acting against the wall surface at depth, h, below the ground surface
k = lateral earth pressure coefficient;
g = the bulk unit weight of the retained backfill;
h = depth below the ground surface at which the pressure, p, is to be computed; and
q = the value of any adjacent surcharge in kPa which may act close to the wall (including traffic loads).

The above equation assumes that sub-drainage is provided at the founding level, together with free-draining granular backfill adjacent to the wall, to prevent the build-up of hydrostatic pressure behind the wall. Backfill should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation (MTO) Standards.

The effects of compaction surcharge should be taken into account in the calculations of active and at rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing. Vibratory compaction equipment for use behind the abutment walls should be restricted in size as per current MTO practice. Traffic load on the road should be taken into account as well.

The above equation assumes that a sub-drain is provided at the founding level, together with free-draining granular backfill adjacent to the wall, to prevent the build-up of hydrostatic pressure behind the wall. However, where the wall is likely to be subjected to unbalanced hydrostatic forces this should be taken into account in the lateral pressures for the wall design.

Backfill should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation (MTO) Standards. For design purposes, the following physical characteristics of the materials can be used.

DETOUR BRIDGE CONSIDERATIONS

A detour bridge or closure of the roadway would be required to facilitate the reconstruction of the proposed bridge. Closure of the roadway was met with much concern during the geotechnical investigation as a result of the inconvenience roadway closure places on business, community residents and travelling public in general. As a result, the provision of a detour bridge was requested by the Town despite preliminary indications that this would be costly as a result of site constraints.

The proposed detour bridge will be a Bailey Type/Modular bridge structure accommodating single lane traffic. This structure is planned to be constructed on the south side of the existing bridge as shown on the Contract Drawings.

The location of this bridge and its construction was given much thought regarding its alignment, height above the water level in the river and the fact that it would require an embankment since its support at the east side of the river would be in the river environment. This would require that fill be placed in the river to facilitate the detour bridge and its approach roadway construction. Site observations and hand auger probes as well as limited air track probes indicated that rockfill and/or sandy and gravelly fill was used previously to widen the roadway on the west abutment.

The proposal for constructing the east approach consists of backfilling/infilling within the river to create an embankment to support the detour bridge and its foundations. In order to contain the embankment because of the soft nature of the river bed soils above the bedrock, it was decided to recommend provision of a sheetpile retaining wall as shown on Contract Drawing S-14. This sheet pile retaining wall would have to be sturdy enough to resist the pressures from the river infill material. The infill would theoretically be about 2.5 m which was determined would not result in slope instability issues if the infill was contained. Without containment, the infill would lead to slope instability as the Factor of Safety would be less that 1.0. This proposed retaining wall system would prevent infill material from spreading and hence less material placement which would prevent any lateral displacement of the surficial riverbed soils and less height of surcharge required on the riverbed soils. .

A recommended requirement for the design of the sheet pile in addition to its ability to resists lateral pressures was for it to be driven to the rock surface for embedment. This can be achieved by the use of proprietary shoes fitted to the bottom of the sheetpiles. It is anticipated that some deflection of the sheetpiles would occur but because of its temporary nature then some small deflection can be accommodated.

To ensure that the bridge would be functional during the period of the construction the single span proposed Bailey/Modular Bridge would require to be founded on piles driven to the underlying bedrock in the same fashion as for the permanent bridge structure. The only concern with this approach is the need to ensure that there is no failure of the sheetpiles in front of the abutment piles or along the sideslopes. Large movement of the steel sheet piles at the headslope can result in the abutment pile movement and possible failure of the detour bridge.

To guard against this from occurring it is recommended that the sheetpiles be anchored into the river infill behind the abutment. This can be achieved by the use of tie rods attached to deadman anchorages. These anchorages can be constructed with steel channels sections, concrete blocks or some other system that would result in restraint to outward movement of the sheet piles. For the design of the sheet piles against lateral pressure, the triangular pressure distribution and the recommendations made for the design of the abutment walls can be used to determine the lateral pressures for sizing the sheetpiles and designing of the tie rod anchorages. A typical location of the tie rods on the sheet piles would be about 1 to 1.5 m below the top of the piles.

For the sheet piles along the side slopes a similar design should be used to prevent the sideslopes from deforming and creating instability in the roadway section.

ROADWAY RECONSTRUCTION

With the construction of the new bridge structure, reinstatement of the approach roadways in the locations disturbed by the construction activities was required. From the Contract Drawings the limit of roadway construction extends over a length of about 130 m extending from around Sta 0+930 to Sta 1+063. The existing roadway at the bridge approaches consisted of an asphalt surface treatment overtop of granular material, which presumably consisted of Granular "B" type material.

Proposed for the pavement reconstruction was 90 mm of asphalt concrete pavement consisting of 50 mm of HL 3(OPSS 1101) surface course overlying 40 mm of binder course. This asphalt pavement overlies 150 mm of Granular "A" placed overtop of a minimum of 300 mm of Granular "B". In comparison with the existing pavement structure and the relatively good appearance of the existing roadway, this proposed pavement design would be more than adequate. It is presumed that the excavated sand and gravel fill material can be reused, if carefully removed and stockpiled. This material can also be utilized in the construction of the detour roadway.

CONSTRUCTION ISSUES

The major issue that occurred during construction was the slippage of the H-piles when driven onto the bedrock despite being fitted with Titus points which is used for gripping to hard rock surfaces.

As reported by the Structural Engineer because of the steep rock profile, "the piles could not seat even with rock points, they just slipped once the rock was encountered"

At the west abutment where the rock was inclined approximately 45 degrees the batter of the piles had to be reversed and driven toward the rock face. As a result of loosing horizontal resistance by virtue of reversing the batter a massive concrete dead man had to be constructed and tied to the piles with four dywidag bars. Although some piles slipped at the east abutment, the situation did not require the same treatment as in the case of the west abutment in the case treatment No photos are available showing the remedial work undertaken.

The photos in Fig. 5 thru Fig. 10 show the existing bridge and varying aspects of the replacement bridge construction.

Fig.5. Existing Bridge Looking East

Fig.6. Existing Bridge Looking West

Fig.7. Installation of Abutment Piles some with Reverse Batter

Fig.10. Completed Abutment and Bridge Seat

Fig.8. Bridge Girders Being Installed

Fig.10. Completed Bridge

Fig.4. Completed Bridge and Reinstated Abutments

CONCLUSIONS

The replacement of the existing bridge constructed in 1930 across the Middle River at Mc Kellar, Ontario posed significant geotechnical and structural problems that were largely unsuspected prior to the geotechnical investigation, structural design and proposed bridge construction.

Considerable additional geotechnical investigation had to be undertaken to define the locations of previously installed infrastructure. An Air track rig had to be used to penetrate timber that was buried below the ground, which prevented conventional drilling to be undertaken. This investigation allowed the bridge abutment locations to be properly defined.

As a result of the constraints posed by the soft soils within the river environment and the perceived expenditures for realignment it was decided to construct the new bridge along the existing alignment.

Overall, this case study demonstrates that despite the small size of the existing and replacement bridges in comparison to larger bridges the ground conditions encountered were complex and posed difficulties that very often are not encountered with larger structures. This situation brings to light the importance of undertaking a geotechnical investigation of the ground conditions irrespective of the size and cost of the structure to be constructed.

REFERENCES

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