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ABSTRACT

At CAD $1.54B, the Nouvelle Autoroute 30 project is the largest and second Public Private Partnership (PPP) transportation project procured in the province of Québec, Canada. It will operate as a tolled two-lane 42km divided highway with 31 bridges including two major bridge crossings of the St Lawrence River and Beauharnois Canal and a short tunnel. The project is located approximately 30km south-west from downtown Montréal and will relieve traffic congestion on Montréal Island by providing the final section of an alternative southern bypass route.

The project is located in a seismic cold climate region and is underlain by deep deposits of soft sensitive Champlain Clay along much of the route. The ground engineering solutions engineered for the project include driven steel piles, drilled shafts and micro-piles, pile load testing, spread footings, earthworks (cuttings and embankments) including lightweight fill and surcharged embankments on soft clays with vertical drains as well as instrumentation and monitoring.

This paper describes details of the geotechnical solutions designed and constructed on this major self-certification infrastructure project, and how by combining local and international experience the ground investigation, geotechnical design and construction certification were successfully delivered under this procurement method.

INTRODUCTION

The Nouvelle Autoroute 30 (A30) Public Private Partnership (PPP) project is located south of the island of Montréal, between Vaudreuil-Dorion and Châteauguay in Québec, Canada. This is only the second transport infrastructure PPP project so far in the province of Québec and with net costs of $1,538.8 million, the largest to date in Québec.

This A30 route is approximately 35 kilometres in length with an additional seven kilometre section (A530) connecting it to Route 201 in the municipality of Salaberry-de-Valleyfield for a total of 42 kilometres of two lane divided highway. The project has a short tunnel under the Soulanges Canal and 31 bridges including, major lifeline bridges over the St. Lawrence River and the St. Lawrence Seaway at the Beauharnois Canal.

It will help to relieve traffic congestion on Montréal Island by providing the final section of the A30, as shown in Figure 1, providing a southern bypass route and improve regional economic development along the periphery of the highway.

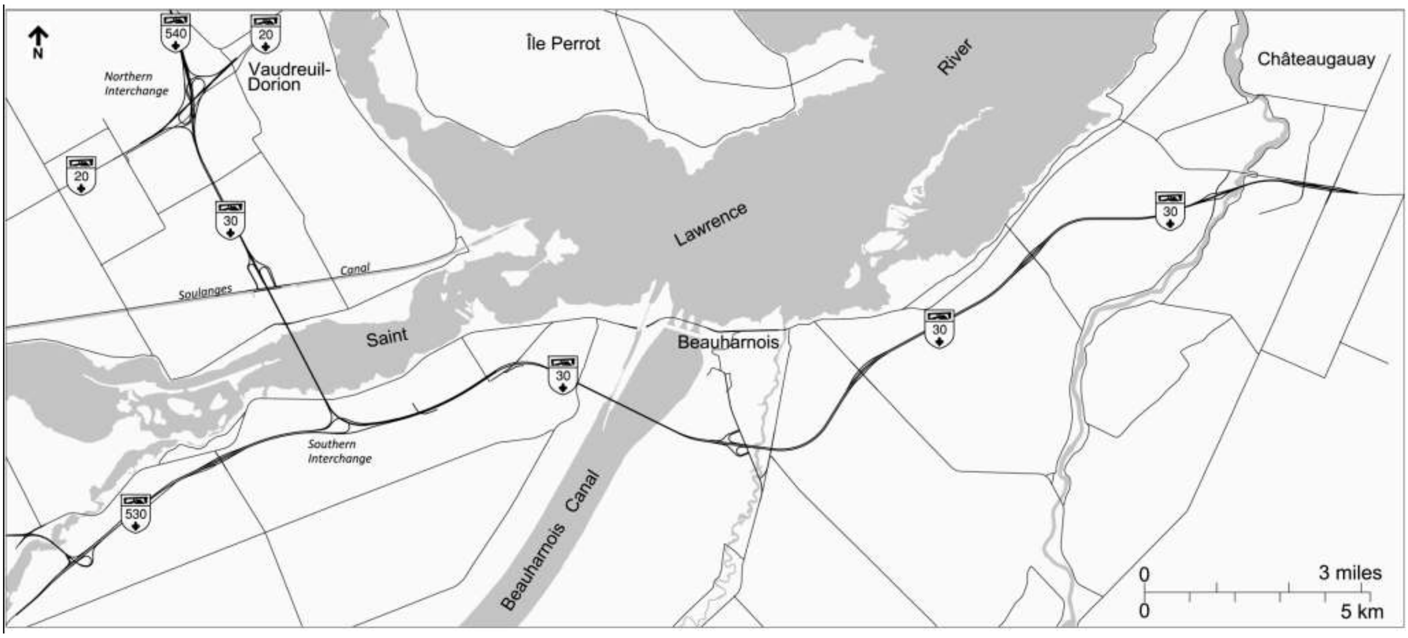

Fig. 1. Autoroute 30 route alignment

As illustrated in Figure 2, the concession to finance, build, maintain and operate the road for 35 years was awarded to Nouvelle Autoroute 30 S.E.N.C (Private Partner), a partnership between Acciona and Iridium on September 25th 2008.

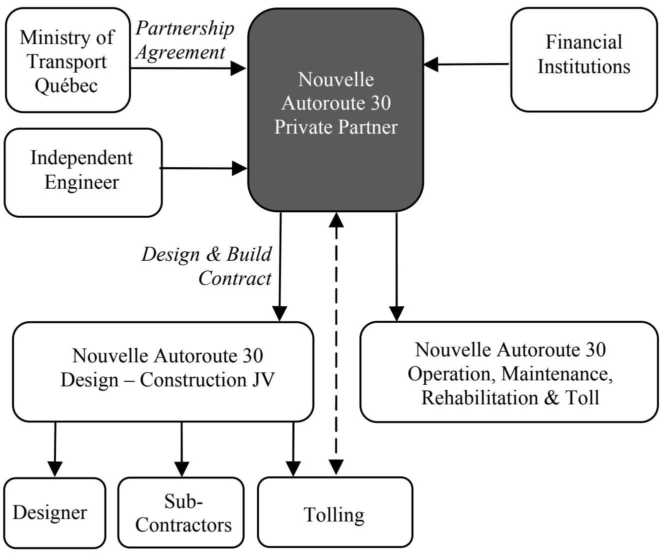

Fig. 2. Project organogram

The design and construction was carried out over four years, on a design-build basis by the Nouvelle Autoroute 30 Construction Joint Venture (CJV), comprising Dragados Canada, Acciona Infrastructures Canada, Aecon and Verreault. Lead design services are being provided by Arup, with AECOM as design sub-consultant.

GEOLOGY

The area through which the A30 passes is relatively flat with ground level ranging from a maximum of 46 m above sea level (asl) at the Northern Interchange to a minimum of 34 m asl at the Southern Interchange and it is typically at around 40 m asl. The land use along the route is predominantly rural and used for agriculture.

The ground conditions for the route generally comprise deep deposits of soft sensitive Champlain Clay up to approximately 25m thick, overlying granular glacial till which in turn overlie a quartzite and dolomitic sandstone bedrock. The upper 1m to 3m of Champlain Clay usually consists of a stiffer, weathered brown clay “crust” that overlies softer and more brittle unweathered grey Champlain Clay.

The Champlain Sea Basin is one of the major Quaternary basins in Canada and the geology of the marine clay and underlying deposits of eastern Canada is well documented in the literature (Gadd 1988 and Quigley 1980).

The project is located in a seismic cold climate region. The ground engineeering solutions for these ground conditions include driven steel piles, drilled shafts and micro-piles, pile load testing, spread footings, earthworks (cuttings and embankments) including light weight fill and surcharged embankments on soft clays with vertical drains as well as instrumentation and monitoring. Details of the geotechnical solutions designed and constructed on this major self-certification infrastructure project are described below.

GROUND INVESTIGATION

Nearly CAD$8 million of ground investigation was carried out for the geotechnical design of the A30 project. The total ground investigation cost was about 0.5% of the total capital project cost of CAD$ 1.5 billion, which is at the lower end of the range for highway infrastructure projects, reviewed by Rowe, 1972. These ground investigations (GI) were procured in three broad stages, namely during the initial development of the project prior to tender stage by the Ministry of Transport Québec (MTQ); by the CJV for the detailed design; and during the construction stage to support value engineering initiatives. These three stages are discussed below.

During the pre-tender development of the project, MTQ procured several phases of ground investigation (summarized in Table 1) and also commissioned studies on the most critical areas of the project. Most of these GIs were carried out between 2002 and 2006. Almost all of the boreholes had "Nilcon" field vane data, a test which is widely used in Québec and was the main test used to investigate the *in situ* strength of the Champlain Clay along the route. Trial pits, piezocones and hand augers were also undertaken together with associated laboratory testing. Groundwater instrumentation was installed and monitored for a limited period. MTQ made this GI information available to tenderers through an electronic data room.

Table 1: Summary of A30 Ground Investigations

Stage	Pre-Tender	Detailed Design	Construction	Total
Boreholes	229	223	85	537
Test Pits	418	530	78	1026
Piezocone CPTs	23	214	31	268
Hand Augers	28	-	-	28
Nilcon Vane Boreholes	-	-	18	18
Dynamic Probe	-	-	8	8
Proportional Split	43%	41%	16%	100%
Approx Value (CAD)	$3.4M	$3.2M	$1.3M	$7.9M

Also, during tender stage MTQ procured an 8m high advance embankment at the Haute Rivière junction at Chateauguay. This was built over a layer of Champlain Clay and vertical drains were installed to accelerate the primary settlements. The embankment was constructed with stabilizing berms and surcharge. Instrumentation was installed and the settlements were monitored for 12 months.

During the award of the concession to the NA30 Private Partner, detailed design commenced in July 2008. As is common with large infrastructure projects, the pre-tender GI was supplemented with further investigation for the detailed design stage. This detailed design GI was scoped by the designers, Arup/Aecom, and procured by the CJV. The factual reports were delivered in the spring of 2009. The scope of this investigation is summarized in Table 1.

The third stage of GI was procured by the CJV to support value engineering initiatives (see Table 1). The costs of this stage of GI demonstrated considerable returns when compared against the resultant cost and programme savings. An example of this was extra GI to justify savings in lightweight fill volumes.

To make the multiple design phases efficient, this vast amount of hard copy ground investigation, laboratory testing and monitoring information from the outset was collated and entered into an electronic database for ease of access, management, and use. From detailed design phase, the designer's required the GI contractor to additionally supply this ground investigation data in electronic format. This was combined with digital gINT® data provided directly by the GI contractor, Inspecsol, from the detailed design GI to provide a very successful and efficient routewide geotechnical database for the project design.

EARTHWORKS

The project earthworks volumes comprise a total of 6,347,000m^3 excavation and 5,400,000m^3 of bulk fill for both structural (i.e. highway embankments) and non-structural (e.g. temporary surcharges, berms and landscape mounds). Additionally, 747,000m^3 of geofoam lightweight fill and 1,900,000m^3 imported granular pavement material was also used.

The predominant structural earthworks material used was 2,300,000m^3 of site won surficial weathered brown clay crust. To a lesser extent, site won and locally quarried glacial till, road planings and sand from removal of existing embankments were also used.

Expanded polystyrene (EPS) geofoam lightweight fill was used to meet the four year construction programme and constrained landtake in the existing interchanges. Lightweight fill has been used as highway embankment fill to minimize settlement in North America for more than 20 years. It has established large volume highway applications as described in the largest geofoam project in the U.S.A., the I-15 Salt Lake City Utah, that used some 100,000m^3 lightweight fill (Geofoam, 2001).

Embankments

Most of the main line of A30 is on low-height embankments typically up to about 2-3m high, including pavement layers. However, a total of 64 higher embankments were required for

Fig. 3. Northern Interchange construction, July 2012

highway geometry requirements at interchanges, side roads and on the approaches to bridges over the motorway. The finished heights of these embankments are between 3.9m and 10.7m in height with an average of 8.2m.

The main constraints on the embankment design generally were:

(i) predominance of deep deposits of soft sensitive Champlain Clay;
(ii) the short construction schedule did not allow more than one consolidation period for embankments built over vertical drains;
(iii) concurrent construction required embankment solutions that allowed bridges to be built concurrently;
(iv) traffic management required existing interchanges to be kept open to traffic throughout the construction period;
(v) landtake permitted was typically very constrained.

With these constraints the solutions for these higher embankments were: 26 were entirely constructed of lightweight fill and 13 were constructed part earthworks part lightweight fill for stability. The remaining 25 embankments were constructed conventionally with earthworks. Also, of these 64 embankments 22 required vertical drains and temporary surcharge for consolidation of the underlying soft clay.

Low height embankments. The Champlain Clay is a structured and highly compressible marine clay. As such, conventional earthworks embankments were typically limited to less than 2.5m height so that the pre-overburden pressure (i.e. preconsolidation stress – effective stress) through the grey Champlain Clay was not exceeded as once the preconsolidation stress is exceeded settlements increased significantly. At the Northern Interchange the pre-overburden pressure was typically 50 to 60kPa.

Surcharged embankments with vertical drains. Where sufficient clearance from existing trafficked roads and the construction schedule could accommodate, surcharged embankments were adopted. These involved installing prefabricated vertical drains on a triangular grid at 0.9m or 1.2m centers through the Champlain Clay and into the underlying Glacial Till and then constructing the embankment with a temporary surcharge and usually stabilizing berms to accelerate consolidation.

The original design was to use multi-stage construction with surcharge, berms and vertical drains to achieve the required vertical alignment. During the design phase an alternative solution involving a single stage of construction, with fewer vertical drains and topping off the embankment with lightweight fill was developed. This solution was cheaper, quicker to construct, less complicated, and less risky to construct than multi-stage construction.

The results from one dimensional consolidation tests carried out for the design suggested that the relation proposed by Leroueil *et al* (1983) provided a conservative estimate of the coefficient of compressibility (Cc). An average value for the data collected at the Northern Interchange was used for design. A review of the coefficient of secondary settlement (Cαe) concluded that the relationship with Cc was closer to 0.015 than the usual 0.04 value quoted by Mesri and Castro (1987), as shown in Figure 4.

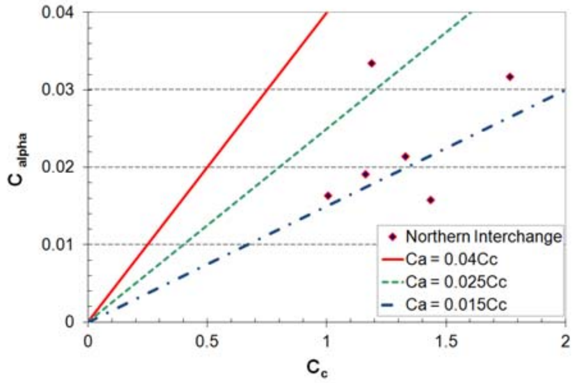

Fig.4. Variation of Cα with Cc at Northern Interchange.

There are issues of strain compatibility between the weathered brown clay crust and the unweathered grey Champlain Clay beneath. This was illustrated by Lefebvre *et al* (1987) who investigated and described the causes of failures of a number of embankments on Champlain Clay. The underlying brittle grey clay fails at low strains, typically less than 1%, while the ductile weathered brown clay crust deforms plastically at higher strains. Consequently the shear strength in the overlying weathered crust, which achieves its maximum shear strength at a higher strain, has to be factored down for design.

At the Northern Interchange, the groundwater regime is governed by two separate profiles. The upper groundwater profile is hydrostatic from about 1m below ground level before being underdrained by the lower groundwater profile in the glacial till and rock. The reduction in the pore water pressure between the upper and lower groundwater profiles is reflected in an increase in the preconsolidation stress profile in the Champlain Clay.

By connecting the upper and lower groundwater profiles at the Northern Interchange with the vertical drains, the pore water pressures in the Champlain Clay were reduced, and this increased the effective stress and embankment settlements.

The stability of the embankments was analyzed using Oasys® Slope software. The soil was divided into zones with various shear strength characteristics and several failure surfaces were checked. Circular slips were automatically checked within a grid while non circular surfaces were specified and modified by the user to identify the worse case. Due to the brittle nature of the Champlain Clay a minimum factor of safety of 1.4 was used. Embankment slide slopes were typically 1(V):2(H) constructed from either lightweight fill, site won granular Glacial Till or the brown Champlain Clay crust fill surfaced with a 600mm thick granular layer for shallow stability. Where unconstrained by landtake, the embankments built from the brown Champlain Clay crust material were not surfaced with a granular layer and sloped at 1(V):2.5(H).

The embankment fill thickness was calculated to include an allowance for the settlement of the ground of up to 3.5m (for the highest embankments) and also to provide some surcharge to the embankment to reduce post construction secondary settlements. The embankments were monitored using settlement plates, piezometers and inclinometers. The settlement plate results indicated that the consolidation rate was lower than expected and extra surcharge was placed on some of the embankments to accelerate the rate of consolidation. The final settlements were predicted based on the Asaoka (1978) method and the surcharge was removed once the specified settlement had been reached.

Lightweight Fill Embankments. Where high embankments were required but construction time constraints prevented a surcharged embankment solution and immediately behind bridge abutments, lightweight fill embankments were used, as shown in Figure 5. These embankments were designed to not exceed 85% of the pre-overburden pressure. The upper two layers of lightweight fill were constructed with expanded polystyrene strength 140kPa with the remainder comprising blocks of strength 100kPa. Particular detailing was done in the embankment design to accommodate, crash barrier foundations, lighting column foundations, manholes and carrier pipes.

Fig.5. Construction of lightweight fill embankment.

Cuttings

Two major cuttings in the Champlain Clay were constructed for the A30 project. One up to 11.4m deep for the approaches to Soulanges Canal tunnel and the other up to 5.5m deep at Chateauguay. These were designed with side slopes of 1(V):3.5(H) and 1(V):3(H), respectively. Where the Soulanges cutting depth exceeded 6m, a mid height bench was introduced.

ST LAURENT BRIDGE

The bridge over the St. Laurent River is one of two major bridges required for the A30 project (the other being the Beauharnois Canal Bridge). The bridge has a total length of 1.8 km and includes two separated decks, each supporting a two lane highway, see Figure 6.

Fig. 6. Southern Interchange and St Laurent River Bridge

Each deck is supported by 43 piers with a typical span length of 45m, which includes a single abutment on each bank of the river, resulting in a total of 84 individual foundation units. The decks are supported on single columns which are in turn supported by isolated pad footings bearing directly onto rock. Each footing is anchored to the rock with drilled and grouted micropiles (between 8 and 28 micropiles at each footing, depending upon water depth and column height) to resist sliding due to ice loading and provide overturning resistance in the event of an earthquake. More than 1400 150mm diameter micropiles were installed and consisted of a central 65mm nominal diameter Grade 1035 MPa threaded and galvanized steel bar, which was double corrosion protected (i.e. minimum 5mm grout encapsulation thickness contained within a corrugated HDPE sheath). A grout tube was installed full-length along the encapsulated rebar to ensure full grout contact along the entire bond length in the underlying rock, which was typically 6m, but increased locally up to 9m at a total of 8 footing units as a consequence of a zone of disturbed rock at these locations.

The six historical borings at the St. Laurent River Bridge site were supplemented with a further site investigation for detailed design in August and September 2008, together with later supplementary phases to ensure a minimum of one cored borehole at every foundation location.

Pre-production uplift testing was performed on sacrificial micropiles to validate the values of unit side resistance for the micropile design. In addition, two micropiles at each footing were selected for proof testing during production installation.
In the shallower water closer to the shorelines, excavation of riverbed sediments and surficial fractures rock was typically advanced prior to the placement of pre-fabricated square-shaped cofferdams, each encompassing a single footing. Access to these cofferdams was made in the form of a temporary access bund constructed in shallower water. In the deeper water, rectangular-shaped driven sheet pile cofferdams encompassing pairs of adjacent footings were first installed using construction equipment on barges, with the excavation of the riverbed sediments and surficial fractured rock subsequently advanced from inside the cofferdams.

A third party inspector was responsible for approving the quality of the exposed rock prior to the placement of the footing rebar cage and concrete. These inspections involved visual inspection of the rock surface in cofferdams where water could be effectively lowered and where footings were formed under water, the use of a diver. The general requirements for the rock on which the footings were founded included a minimum Rock Quality Designation (RQD) of 75% and minimum Total Core Recovery (TCR) of 85%. The structural engineer's requirement to control vertical settlement of the piers to less than 10mm (under a bearing pressure of approximately 12 MPa) resulted in a field assessment of the depth, thickness and frequency of infilled horizontal joints under the bearing surface during construction.

BEAUHARNOIS CANAL BRIDGE

The 2.5km long bridge over the Beauharnois Hydroelectric Canal and St. Lawrence Seaway, see Figure 1, is a major structure along the new Autoroute 30 alignment. Its length is dictated by the need to provide nearly 40m of vertical clearance above the Seaway channel, along with a maximum gradient of 3.5% along the approaches, see Figure 7.

Fig. 7. Beauharnois Canal Bridge crossing of the St. Lawrence Seaway, July 2012.

Each deck of this dual carriageway bridge is supported by single columns, with pairs of adjacent columns being tied together at the waterline to form a single foundation element. The 44 foundation elements for the bridge include pad footings bearing directly on rock (8 locations), piers supported on groups of 96 324mm diameter concrete-filled driven steel tube piles (at 16 locations, for a total of 1,536 driven piles), and piers supported by groups of 1.85m diameter drilled shafts socketed a minimum of 4m into rock (20 locations).

The typical drilled-shaft supported pier consists of 6-shaft groups (four piers on land and 13 over water), with an 8-shaft group supporting the eastern abutment and 14-shaft groups supporting piers at each end of the navigation span over the Seaway channel, resulting in a total of 138 individual rock-socketed drilled shafts for the bridge. The required ultimate axial compression and uplift load capacities for each driven pile were 4400 kN and 75 kN, respectively. For the typical 4m long rock-socketed drilled shaft, the required ultimate axial compressive and uplift load capacities were 33.5 MN and 17.2 MN, respectively.

Pile Load Test Program

A comprehensive pile load testing program was performed to validate the foundation design parameters for the bridge. Full-scale static compression and uplift tests were performed on instrumented driven steel tube piles (subsequently filled with concrete) to verify end bearing and side resistance in the lower till, along with side resistance in the upper grey Champlain Clay. In addition, two loading tests were performed on sacrificial, heavily-instrumented 1.18m diameter test shafts using Osterberg load cells to validate unit end bearing and side resistance values for the rock-socketed drilled shaft foundations (Cushing, *et al*., 2011; Hee, *et al*., 2011). The results of the pile load testing program were used in the final design of the driven steel pile and drilled shaft foundations, and permitted the use of slightly higher resistance factors pursuant to the Canadian Highway Bridge Design Code (S6).

The land-based western approach piers are supported on concrete-filled driven steel piles which are deeply embedded in a thick pile cap to achieve a fixed-head condition under lateral loading. Under normal service conditions, the driven piles are generally under compression and lateral loading only, but some of the piles also rely on nominal uplift resistance in the seismic condition. The full scale uplift test on the driven steel pipe pile successfully validated the unit side resistance of the Champlain Clay adopted in the seismic foundation design.

Field Testing and Inspection

Pad footing inspections of the rock formation was carried out for all eight footings bearing directly onto rock to confirm the design bearing capacity had been achieved, and to instruct the removal of any softer/weaker rock areas as required. Quality control testing was performed during the installation of both the driven piles and the drilled shaft foundations.

For the foundations supported on driven piles, the blow counts and final pile set were recorded for each pile to ensure that the refusal criteria had been achieved. After adjacent piles were installed, each pile was redriven to ensure that the refusal criterion was still achieved. Three driven piles from each pier were selected for dynamic testing to ensure conformance with the design requirements. CAPWAP analyses were also provided for selected piles to ensure that the required penetration and pile end bearing was achieved within the underlying till (Deakin and Sartain, 2011).

After the boring of the drilled shafts was completed, the rock sockets were cleaned using a bucket to dredge material from the base, a rotary brush to remove sediment and loose fragments from the side walls, a gravel pump to remove larger fragments, and finally an airlift to extract any remaining fines. Visual inspection of each socket was performed by a third party inspector with the aid of an underwater camera. Within 24 hours of the inspector's approval, the rebar cage was lowered and concrete was tremied under water from the bottom up. Each production shaft was integrity tested with the crosshole sonic logging (CSL) technique. Base coring was also required to check the interface between the base of the drilled shaft and bedrock to ensure intimate contact. While none of the verification cores revealed sediments or loose rock at the toe of the pile, a number of cores revealed a zone of loose unbound concrete aggregate (typically 50mm thick, up to 150mm thickness) at the base near the start of the tremie. As the shaft design relied on both side resistance and end bearing, base grouting was required as a remedial measure to fill up the voids in the unbound aggregate.

ROUTE BRIDGES

There are 29 route bridges along the route supporting crossings highway interchanges, local roads and rivers. There is also a 90m long cut and cover tunnel to carry the A30 beneath the Soulanges Canal. The choice of foundations for these structures was determined primarily by the thickness of the Champlain Clay Deposits at each location. Where competent strata, such as glacial till or rock, was present within 6m of ground surface, then spread footings were considered. In situation where the competent strata was encountered at depth then piled foundations were adopted.

Piled Structures

Twenty of the bridges within the project, are situated in areas with thick deposits of soft Champlain Sea Clay. Following local practice, these bridges were designed with foundations comprising 320mm diameter steel tubular piles, which are driven, closed ended either vertically or raked to resist horizontal forces, see Figure 8.

A total of 4650 tubes were installed which had a wall thickness of either 10mm or 16mm and were formed with a straight weld, using ASTM 500 Grade steel with a minimum yield strength of 345MPa (50,000psi).

Typically the piles are driven to the bedrock, which is very stiff and strong. In this case, it is the structural strength of the piles that is the limiting criterion for their capacity. However, this becomes less certain with increasing thickness of glacial till. Local experience indicated that once the till becomes thicker than 3m it is unreasonable to assume that the piles penetrate to the rock surface, and this limit was used to determine if the piles were to be designed to be founded in the glacial till. In this situation, the capacity of the pile is governed by the available geotechnical resistance of the soil at the toe. Generally cross head toes were used for the tubes. However occasionally Oslo type shoes were used to assist penetration and achieve construction tolerances.

Fig. 8. Driving 320mm diameter steel tubular piles

A programme of preliminary pile drives and tests was scheduled in order to assess the driveability of the piles in various geological strata and locations across the route. A number of variables were considered in the assessment, including, tube thickness, pile spacing, and whether the piles were driven open or closed ended. Twenty-two dynamic and four static load tests were undertaken in order to better understand the available resistances, particularly the tension capacity within the soft clay. This was of particular interest in order to assess the tension capacity of pile which were typically end bearing on rock with minimal penetration into the glacial till. The results also served to help assess potential downdrag loads induced by the consolidating soft clay.

The pile design needed to take account of additional effects due to the concurrent construction of large approach embankments on the soft deposits. These effects included negative skin friction and downdrag loads and lateral pressures acting on the piles due to embankment loading. It was necessary to take a co-ordinated design approach between earthworks and bridge design in order to provide an optimised solution that would not be detrimental to either the abutment or approach embankment design. This typically involved the design of a lightweight fill transition zone immediately behind the abutment within the approach embankment to minimise differential settlement between the structure and embankment and to minimise lateral earth pressures acting on the pile due to embankment loading. Accelerated consolidation through surcharging embankment before construction of the structure or excavate and replace of soft deposits in the approaches to the structure were also used.

In some locations pile rebound was observed during the driving of the piles. While rebound was generally in the order of 100-300mm in some locations rebound well in excess of 1m was observed. The rebound was considered to be due to a piston effect, where excess porewater pressure within discrete soil layers was unable to dissipate. Production rates for driven piles were thus significantly reduced.

During the installation of the piles it was noted that where thick deposits of till were present (6-7m), it was possible for the piles to penetrate in excess of 3m. However, where there were moderate thicknesses of till deposits, refusal was often achieved just above the rock surface, possibly on very dense deposits or on boulders within the till. Back analysis of the working pile tests suggests that where piles were driven to within 1m of the rock the available base resistance was significantly increased by the close proximity of the rock (Deakin and Sartain 2011).

The three route bridges crossing the St Louis River used a total of 42 drilled shafts, with diameters of 1.35m and 2m. The piles were constructed using the same techniques employed on the Beauharnois Canal Bridge.

Shallow Foundations

For seven of the bridges where the Champlain Clay was thin spread footings were used bearing directly on to rock or glacial till. For two structures the founding stratum was considered too deep for pads but too shallow for piles. Therefore, unsuitable material was excavated down to glacial till or rock and replaced with a mattress of compacted granular fill.

During construction all formations were inspected by an experienced geotechnical engineer. For rock formations the same procedure discussed for foundations on the St Laurent crossing were employed.

For some foundations, like the bridge crossing of the Haute Rivière, where very strong competent rock was present at shallow levels and it wasn't practicable to recess the footing into the rock, dowels were incorporated into the foundation in order to increase the sliding resistance.

SOULANGES TUNNEL

The historic Soulanges Canal sits several metres above ground level, contained within two parallel earth embankments, see Figure 9. An 80m long four lane tunnel was formed beneath the canal using a concrete cut and cover box structure. The box is founded on the Champlain Clay. This effectively floating foundation solution was possible by balancing the weight of the structure with the excavation of a significant thickness of clay.

Fig. 9. Soulanges Canal Tunnel construction, July 2012.

The design of the tunnel was further complicated by the need to reconstruct the canal embankments without inducing excessive differential settlement. This was achieved by incorporating high modulus basal reinforcement and part construction using EPS light weight fill within the embankments immediately adjacent to the tunnel sides. In order to ensure water tightness of the canal embankments a combination of mineral liner, geocomposite and HDPE liner was used to cover the newly constructed embankments.

CONSTRUCTION CERTIFICATION

The Public Private Partnership (PPP) procurement of the project came with a self certification requirement, in that the Private Partner is required to demonstrate that the project complies with the quality of works and scope of service requirements of the Partnership Agreement, see Figure 2.

The Private Partner is responsible for audit and quality control activities, necessary for the adequate implementation of the design, construction and supervision of work, operations, maintenance and repair. To ensure that the technical obligations are being met at each stage of the works, the Private Partner established, implemented and maintained a quality management system to International ISO Standard 9001:2000 requirements.

The Private Partner was also required to appoint an independent consultant to act as Independent Engineer. The Independent Engineer had the right to review and object with respect to the nature and contents of the construction site surveillance, material quality control program, shop drawings and completion certificates.

Also as part of the technical obligations the Designer was required to examine the works and declared that the works have been completed in a manner that is entirely compliant with the Detailed Design.

The Partnership Agreement required all three parties to sign the completion certificates which totaled some 250 design and 250 constructions certificates.

During the four year construction period, the project raised more than 4000 requests for information (RFI) and non-conformance reports (NCR). At its peak, 25 senior civil, structural and geotechnical engineers were co-located with the contractor in Montréal managing design delivery and optimisation as well as auditing construction works for compliance.

CONCLUSION

The Nouvelle Autoroute 30 project is a major technical undertaking which brought numerous geotechnical challenges to its designers. The main challenge is the presence, along a great portion of the alignment, of a sometimes deep soft sensitive clay layer which required the use of special construction techniques such as preloading preceded by installation of vertical drains, important use of light weight fill, adaptation of foundation design methods and installation of a network of monitoring instruments. Moreover the use of the stiff crust clay layer in the construction of temporary and permanent fills following several test embankments, allowed to give a second life to this abundant excavated material.

The design and construction of two major bridges required the use of advanced foundation techniques. The St Laurent Bridge footings were anchored to the bedrock with more than 1 400 150mm diameter drilled and grouted micropiles; the design was confirmed by extensive testing on sacrificial micropiles as well as through extensive inspection of the foundation rock. The Beauharnois Canal Bridge is a major structure with nearly 40m clearance supported by single columns tied together in pairs at the waterline level. A combination of shallow foundations, more then 1 500 driven metal piles and several groups of 1.85m diameter drilled shafts constitute the bridge foundation elements.

All these innovating geotechnical accomplishments had also to be carried out in a record time ahead of construction and under the scrutiny of several approval levels always with the concern for safety, optimization and sound engineering techniques.

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