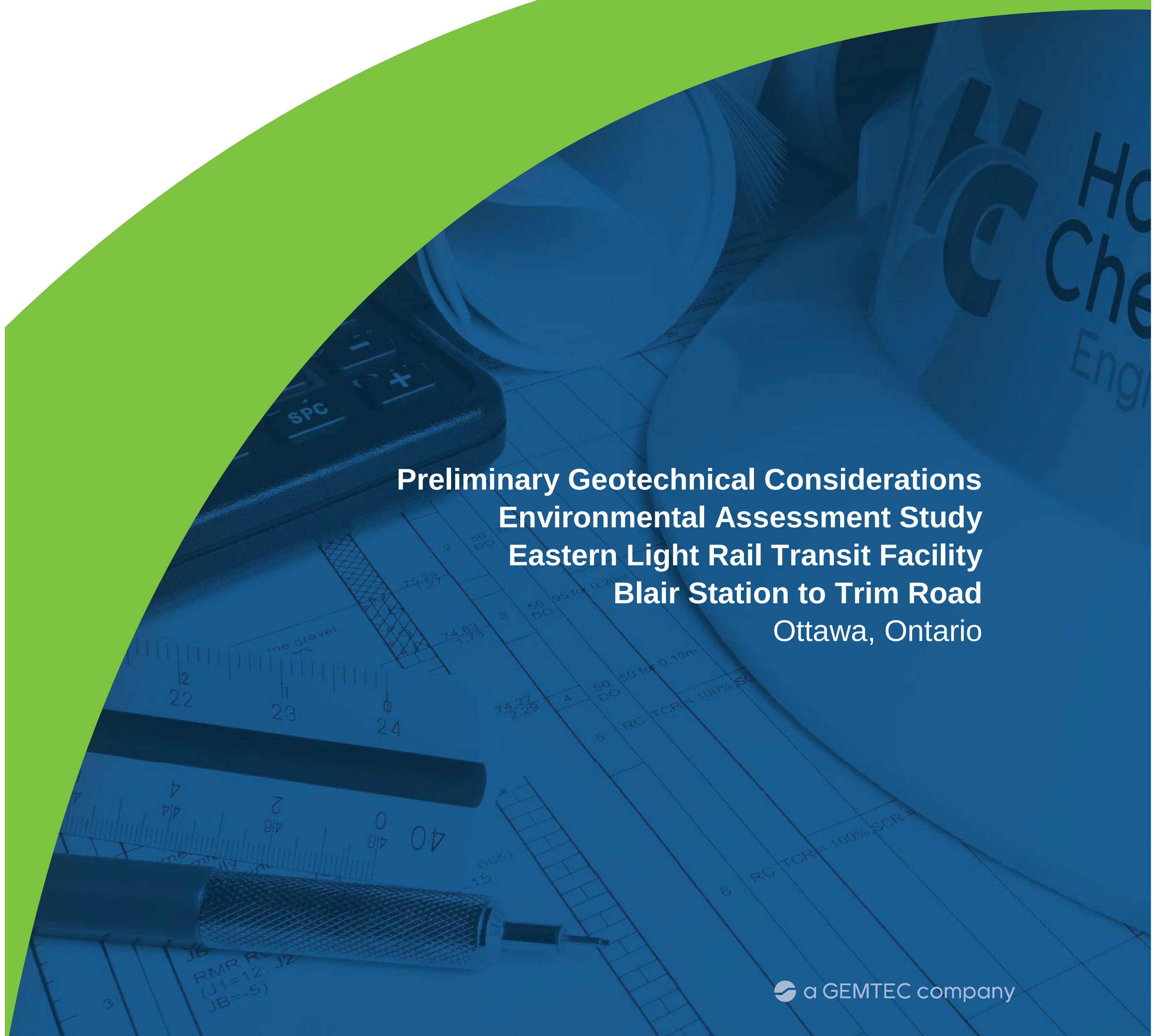
Appendix F

Preliminary Geotechnical Considerations

Houle Chevrier Engineering



Preliminary Geotechnical Considerations
Environmental Assessment Study
Eastern Light Rail Transit Facility
Blair Station to Trim Road
Ottawa, Ontario

a GEMTEC company

Houle Chevrier Engineering

Submitted to:

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September 10, 2015
Project: 14-275

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1.0 INTRODUCTION

As identified in the 2013 Transportation Master Plan (TMP), consideration is being given to extending the proposed Light Rail Transit (LRT) facility further east to Orleans. The alignment for the Eastern LRT is envisioned along the Ottawa Road 174 (OR 174) corridor between Blair Station and Trim Road. Consideration has been given to locating the Eastern LRT north of OR 174, south of OR 174, or within the median (i.e., between the eastbound/westbound lanes). An inventory of the expected soil, bedrock and groundwater conditions for the study area is provided in our draft report titled: “Geotechnical Inventory, Environmental Assessment Study, Eastern Light Rail Transit Facility, Blair Station to Trim Road, Ottawa, Ontario”, dated October 31, 2014.

Following an evaluation of the various alternatives, it is understood that the preferred location for the Eastern LRT is within the median between Green’s Creek and Trim Road. Between Green’s Creek and Blair Station, the preferred location for the Eastern LRT is north of OR 174. The proposed alignment is provided on Sheets 1 to 22 in Appendix A.

The optimal solution for the Eastern LRT will include construction of railway infrastructure, transit stations, culvert/bridge structures and grade separation of existing roadways. Relatively high embankments and deep cuts will be constructed.

This report provides an overview of the possible construction and operational impacts, including preliminary design considerations and mitigation measures, from a geotechnical point of view, based on our interpretation of the available information and project requirements. It is stressed that the information in the following sections is provided for preliminary planning and costing purposes only. It should be noted that the proposed Eastern LRT is in the same corridor as the proposed widening of OR 174; however, this report addresses the proposed Eastern LRT only as this project is expected to proceed first. Impacts and preliminary geotechnical considerations for the OR 174 widening will be provided under a separate cover.

For the purposes of this report, we have divided the proposed Eastern LRT alignment into the following five (5) segments:

Segment 1 – Blair Road Interchange (Station 300+000 to 301+100)

Segment 2 – Beacon Hill South (Station 301+100 to 302+700)

Segment 3 – Montreal Road Interchange (Station 302+700 to 303+800)

Segment 4 – Green’s Creek to Trim Road (Station 303+800 to 312+200)

Segment 5 – Trim Road Interchange (Station 312+200 to 312+900)

An overview of the possible construction and operational impacts, including preliminary design considerations and mitigation measures, for each segment of the alignment are provided in the following sections.

2.0 BLAIR ROAD INTERCHANGE (STATION 300+000 TO 301+100)

2.1 Project Requirements

The proposed Eastern LRT alignment between Blair Station (Station 300+000) and Station 301+100 is located north of OR 174. The proposed alignment will cross below the following existing roadways/embankments (from west to east):

- Westbound on-ramp to OR 174 from southbound Blair Road;
- Blair Road;
- Westbound on-ramp to OR 174 from northbound Blair Road; and
- OR 174 off-ramp to Blair Road.

As part of the proposed plans, the existing transitway on-ramp and off-ramp will be decommissioned.

This portion of the Eastern LRT alignment is generally in cut, with the exception of the grades approaching Blair Station where the Eastern LRT will be at or slightly above existing grade. In general, the proposed grades are located about 2 to 5 metres below existing grade, increasing to between 7 and 10 metres below the surface of the current embankments.

In order to implement the Eastern LRT between Blair Station (Station 300+000) and Station 301+100, construction of new underpass structures will be required, including permanent retaining walls and, where possible, open cut sections with suitable graded side slopes.

2.2 Overview of Subsurface Conditions

As part of a previous geotechnical investigation for the east transitway extension, twelve (12) boreholes were advanced along the proposed Eastern LRT alignment. Details of the boreholes are provided in the Golder Associates Ltd. report titled: “Geotechnical Investigation, Subsurface Conditions, East Transitway, Station 12+680 to 15+150, Regional Municipality of Ottawa-Carleton”, dated January 1988.

In general, the boreholes encountered fill material over glacial till and shale bedrock. In the area of the Blair Road interchange, the overburden thickness was found to range between 2 and 5 metres, excluding the current embankment fills. Based on the results of the boreholes, the bedrock surface in the area of the Blair Road interchange is located between elevation 72 and 74 metres.

East of the Blair Road interchange (i.e., east of about Station 300+500), the overburden deposits thicken and are composed of fill material over native deposits of silty sand, silty clay, silt, and glacial till.

The bedrock encountered during the investigation consisted of laminated dark grey shale of the Billings formation. The upper portion of the bedrock is weathered and/or fractured.

Along the proposed Eastern LRT alignment, the groundwater level was found to be within about 2 metres of the ground surface (elevation 71 to 78 metres).

The approximate locations of the relevant boreholes previously advanced by Golder Associates Ltd. are shown on Sheets 1 to 3 in Appendix A. A copy of the Record of Borehole sheets for the relevant boreholes are provided in Appendix B for reference.

2.3 Geotechnical Considerations

2.3.1 Excavations

2.3.1.1 Overburden

Excavations for the structures and walls will be carried out through topsoil, fill materials, glacial till and, in some areas, the underlying dark grey shale bedrock. Comments regarding excavation of the overburden are provided below:

- For open cut excavations, allowance should be made for temporary excavation side slopes of 1 horizontal to 1 vertical, or flatter, extending from the bottom of the excavation (in accordance with Ontario Regulation 213/91 under the Occupational Health and Safety Act for Type 3 soil).
- In areas where space constraints dictate, the sides of excavation in overburden could be supported during construction using a shoring system, such as a pile and lagging shoring wall, driven interlocking steel sheet piles, secant concrete pile wall, or a concrete diaphragm wall. For all cases, lateral support to the shoring, such as tensioned rock anchors, will be required. It should be noted that the glacial till contains cobble and boulder obstructions which could affect the shoring installation. An allowance should be made to socket the soldier piles for a pile and lagging wall into the bedrock using predrilled holes.
- The shoring system should be designed to resist lateral earth pressures imposed on the shoring from the weight of the retained soil and any other surcharge loads. The lateral earth pressures acting on the shoring system will depend on the type of shoring system used and on the type of lateral support.
- The type of shoring used on this project should be based on the permissible movement behind the shoring as well as space constraints. Some unavoidable inward horizontal movement and settlement of the ground behind the retaining walls should be anticipated,

which could affect existing structures and services located behind the retaining walls. The amount of movement will depend on the type of shoring system used and will be much less for secant concrete pile and diaphragm walls (relative to pile and lagging and steel sheet pile walls).

2.3.1.2 Bedrock

Comments regarding excavation of the shale bedrock are provided below:

- Bedrock removal at this site could be carried out using drill and blasting, hoe ramming techniques in conjunction with line drilling on close centres, or a combination of both. In areas where an upper layer of fractured bedrock is encountered, rock removal could likely be carried out using hydraulic excavation equipment. Any blasting should be carried out in a manner that maintains vibration levels below the threshold values established for the project.
- The shoring should be setback from the edge of the adjacent bedrock excavation. The setback requirements will depend on the type of shoring, socket depth, and depth of bedrock excavation.
- Provided that good bedrock excavation techniques are used, the bedrock could be excavated using near vertical side walls.
- The bedrock contains near vertical joints and bedding planes and, therefore, some vertical and horizontal over break of the bedrock should be expected (i.e., the bedrock will likely break at a horizontal bedding plane below the planned excavation depth).
- The shale bedrock at this site has the potential to swell due to the oxidation of pyrite and the subsequent formation of jarosite and gypsum between the shale laminations. The shale bedrock below any proposed structures should be covered immediately following exposure with a protection layer of sulphate resistant concrete (50 to 75 millimetre thick). Any vertical surfaces should be protected using shotcrete.

2.3.1.3 Groundwater Inflow during Excavation (Short-Term)

Based on the results of the previous boreholes advanced in the area of the Blair Road interchange, excavation in overburden and bedrock below the groundwater level will likely be required. Comments regarding groundwater inflow into the excavations are provided below:

- Based on our previous experience, no unusual constraints are anticipated during excavation of the glacial till and shale bedrock below the groundwater level.
- Groundwater inflow should be relatively small and controlled by pumping from filtered sumps within the excavation.

- It is not expected that short term pumping during construction will have a significant effect on nearby structures and services.
- Allowance should be made for a Permit to Take Water (PTTW) in accordance with Ministry of the Environment and Climate Change (MOECC) requirements.

2.3.2 Underpass Structures

As previously indicated, construction of new underpass structures will be required in order to implement the Eastern LRT. Preliminary considerations for design of the underpasses are provided below:

- Based on the results of the previous boreholes advanced in the area of the Blair Road interchange, new underpass structures will likely be founded on spread footing foundations bearing directly on competent shale bedrock.
- Spread footing foundations bearing on or within competent shale bedrock could be sized using a factored bearing resistance at Ultimate Limit States (ULS) of between about 1,500 to 3,000 kilopascals. These bearing pressures assume that all soil and fractured/weathered bedrock is removed from the bearing surfaces, and that no significant soil filled seams exist in close proximity to the bearing surface. In accordance with the Canadian Highway Bridge Design Code (CHBDC), the geotechnical resistance at ULS was factored using a resistance factor of 0.5. The geotechnical reaction at SLS will be greater than the factored geotechnical resistance at ULS; as such, ULS conditions will govern.
- Based on available information, seismic Site Class C could be used for preliminary design purposes. Shear wave velocity testing could be carried out to evaluate whether a more favourable Site Class (i.e., A or B) can be specified.
- Shear resistance of the footings against lateral sliding could be calculated using an unfactored angle of friction of 25 degrees between the shale bedrock and the underside of the foundations. In accordance with the CHBDC, the geotechnical resistance to sliding should be factored using a resistance factor of 0.8.
- The depth of the spread footings should be at least 1.8 metres below finished grade to provide adequate frost protection of the footings. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation.
- The abutments should be backfilled with imported, free-draining, non-frost susceptible granular material meeting Granular B Type I or II requirements. The abutment and wing walls should be design to resist the static and seismic lateral thrusts imposed on the structures as a result of earth pressures.

2.3.3 Roadway Cuts

As previously indicated, the proposed grades for the Eastern LRT in the area of the Blair Interchange are located about 2 to 5 metres below existing grade, increasing to between 7 and 10 metres below the surface of the current embankments. It is noted that the proposed grades for the Eastern LRT are below the reported groundwater levels.

The following alternatives could be considered for treating the sides of the roadway cuts:

Alternative 1 - Open cut with permanent excavation side slopes of about 2.5 horizontal to 1 vertical, or flatter, within overburden. In bedrock, near vertical side walls would likely be suitable, assuming a suitable setback between the toe of the overburden slope and the bedrock face. A swale would be required at the toe of the rock slope since there will be some unavoidable degradation of the bedrock with time, and subsequent rock falls.

Alternative 2 - Provide permanent retaining walls, such as a cast in place concrete wall, Mechanically Stabilized Earth (MSE) wall, pile and lagging shoring wall, secant concrete pile wall, or a concrete diaphragm wall.

Comments on various types of permanent retaining walls are summarized in Table 2.1.

Table 2.1 – Summary of Permanent Retaining Wall Options

Support Option	Comments
Option 2A: Cast in Place Concrete Wall	• To allow for construction, excavation side slopes of 1 horizontal to 1 vertical, or flatter, are required. Alternatively, temporary shoring could be considered (e.g., socketed pile and lagging wall). • To prevent buildup of hydrostatic pressure behind the retaining wall, the backfill material should be drained with a perforated drain at the base of the wall. • In areas where bedrock cuts are required, the walls should be constructed at least 1.8 metres below finished grade for frost protection purposes. • Minor groundwater seepage from the overburden and bedrock should be anticipated. Long-term groundwater handling/disposal will be required.

Table 2.1 – Summary of Permanent Retaining Walls Options (Continued)

Support Option	Comments
Option 2B: MSE Walls	• The excavation should be suitability sized to allow for placement of the reinforcement behind the wall. As a general guide, the reinforcement typically extends a horizontal distance equal to about 80% of the ultimate height of the wall. • To allow for construction, excavation side slopes of 1 horizontal to 1 vertical, or flatter, are required. Alternatively, temporary shoring could be considered (e.g., socketed pile and lagging wall). • To prevent buildup of hydrostatic pressure behind the MSE wall, the backfill material should be drained with a perforated drain at the base of the wall. • In areas where bedrock cuts are required, the walls could be constructed on the bedrock surface. For this case, the walls should be suitably set back from the edge of the bedrock excavation (the setback requirements will depend on the stability and frost susceptibility of the bedrock). In bedrock, near vertical side walls would likely be suitable assuming that a catch bench is provided at the bottom of the rock slope. Due to the presence of swelling shale, protection of the bedrock face will be required (e.g., shotcrete). • Minor groundwater seepage from the overburden and bedrock should be anticipated. Long-term groundwater handling/disposal will be required.
Option 2C: Pile and Lagging Wall	• The overburden and bedrock could be permanently supported using a pile and lagging wall (with or without a facing applied). • The soldier piles should be socketed into the underlying bedrock below the depth of excavation using predrilled holes. • Lateral support to the shoring, such as tensioned rock anchors, will be required. • The overburden contains cobble and boulder obstructions which could affect the shoring installation. • Some unavoidable inward horizontal movement and settlement of the ground behind the retaining walls should be anticipated, which could affect existing structures and services located behind the retaining walls. • Minor groundwater seepage from the overburden and bedrock should be anticipated. Long-term groundwater handling/disposal will be required.

Table 2.1 – Summary of Permanent Retaining Walls Options (Continued)

Support Option	Comments
Option 2D: Secant Concrete Pile Wall or Concrete Diaphragm Wall	• The sides of the excavation could be supported permanently using rigid secant concrete pile walls or concrete diaphragm walls (with a facing applied) socketed into the underlying bedrock below the level of the Eastern LRT. • Depending on the height of the wall, some form of lateral support to the shoring, such as tensioned rock anchors, may be required. • Although the most costly of the options discussed, the main advantages of using rigid secant concrete pile walls or concrete diaphragm walls are: ○ Ground movement behind the walls will be much less, relative to pile and lagging and steel sheet pile walls. ○ Secant concrete pile walls or concrete diaphragm walls have a relatively low permeability and, as such, groundwater inflow from the overburden and bedrock will be much less, relative to Options 2A to 2C. In other words, secant concrete pile walls or concrete diaphragm walls would essentially “cut-off” horizontal groundwater inflow from the overburden and bedrock. Some minor groundwater seepage from the base of the excavation, and surface water that infiltrates into the LRT ballast material, will require long-term handling and disposal.

2.4 Impacts and Mitigation Measures

A summary of the possible construction and operational impacts in the area of the Blair Road interchange are summarized in Table 2.2.

Table 2.2 – Impacts and Mitigation Measures (Blair Road Interchange)

Type	Impact(s)	Mitigation Measure(s)
Operational Impact(s)	• The handling and disposal of groundwater, and surface water, entering the cut sections of the Eastern LRT could present some constraints. • Long-term seepage will result in permanent groundwater lowering in the vicinity of the alignment. Excessive groundwater level lowering within the bedrock may result in swelling/heaving of unprotected shale bedrock due to drying and exposure to oxygen. Basement floor slabs may be susceptible to heaving if founded on unprotected (i.e., without adequate soil cover or a concrete protection layer) shale bedrock within the zone of influence of groundwater drawdown.	• Construction of rigid secant concrete pile walls or concrete diaphragm walls (i.e., “cut-off” walls) along cut sections of the Eastern LRT.
Construction Impact(s)	• No significant construction impacts are anticipated.	• No significant construction impacts are anticipated.

3.0 BEACON HILL SOUTH (STATION 301+100 TO 302+700)

3.1 Project Requirements

The proposed Eastern LRT alignment between Stations 301+100 and 302+700 is located north of OR 174. At about Station 302+125, the Eastern LRT alignment crosses the Jasmine Park Ravine.

In general, the proposed grades along this portion of the Eastern LRT will be about 1 to 2 metres above existing grade. In some areas, the proposed grades match the existing grades. At the Jasmine Park Ravine crossing, the proposed grades are about 5 metres above existing grade.

In order to implement the Eastern LRT between Stations 301+100 and 302+700, widening of the existing OR 174 embankment will be required, including extension of the existing culvert structure at the Jasmine Park Ravine.

3.2 Overview of Subsurface Conditions

Based on the available subsurface information, the subsurface conditions between Stations 301+100 and 302+700 are expected to consist primarily of deposits of silty clay, increasing in thickness from about 5 to 100 metres from west to east. Bedrock is mapped as interbedded limestone and shale (Lindsay formation), limestone (Bobcaygeon formation), and interbedded limestone and dolostone (Gull River formation).

3.3 Geotechnical Considerations

3.3.1 Embankment Widening

Along this portion of the Eastern LRT (Station 301+100 to 302+700), widening of the existing OR 174 embankment is required. Preliminary considerations for embankment widening are provided below:

- All topsoil, organic material, and unsuitable fill material should be removed from the base of the embankment, and all topsoil and organic material should be removed from the existing embankment slope.
- The embankment fill material could consist of compacted material meeting OPSS Granular B Type I or II, or OPSS Select Subgrade Material.
- Final fill slopes constructed with OPSS Granular B Type I or Select Subgrade Material could be sloped at 2.5 horizontal to 1 vertical, or flatter; for Granular B Type II, fill slopes of 2.0 horizontal to 1 vertical, or flatter would be suitable. If steeper embankment slopes are required due to space limitations, a geogrid reinforced slope or a mechanically stabilized earth retaining wall could be considered.
- To avoid abrupt differential heaving at the joint between the new and existing embankment, the embankment fill material used within the zone of frost penetration below the embankment widening should be frost compatible with the fill materials below the existing OR 174 embankment; alternatively, a gradual frost heave transition should be provided.
- The proposed Eastern LRT alignment between Stations 301+100 and 302+700 is underlain by deposits of sensitive silty clay which have a reduced capacity to support loads imposed by grade raise fill material. Based on the settlement history of previous embankments constructed on sensitive silty clay deposits in the Ottawa area, the following long-term settlements (i.e., after about 10 to 20 years) are estimated for the proposed embankments within this portion of the Eastern LRT:
    - Less than 50 millimetres of settlement for embankments less than 2 metre high.
    - 100 to 150 millimetres of settlement for 5 metre high embankments (i.e., at the Jasmine Park Ravine crossing).
- The settlement below the widened portions of the embankments will likely be greater than the settlement of the existing embankment. Differential settlement could be

problematic for buried structures (i.e., culverts) located along the base of the embankment.

- The overall stability of the embankment side slopes will depend on the ultimate height of the approach embankments and inclination of the side slopes. However, to assess the feasibility of embankment construction in this area, preliminary slope stability analyses were carried out. The results of the stability analyses indicate that a 5 metre high embankment, with side slopes constructed at 2 to 2.5 horizontal to 1 vertical, would have an adequate factor of safety against a deep seated shear failure through the silty clay. Furthermore, the preliminary slope stability analyses also indicate that a 5 metre high retaining structure would be feasible.

3.4 Impacts and Mitigation Measures

A summary of the possible construction and operational impacts between Blair Station (Station 300+000) and Station 301+100 are summarized in Table 3.1.

Table 3.1 – Impacts and Mitigation Measures (Beacon Hill South)

Type	Impact(s)	Mitigation Measure(s)
Operational Impact(s)	• Differential settlement of any buried structures (i.e., culverts) located along the base of new and existing embankment (e.g., at the Jasmine Park Ravine crossing). Differential settlement may negatively impact the long-term performance of the structure. • Long-term settlement along the length of the newly constructed embankment may also result in long-settlement of Eastern LRT infrastructure supported on the embankment.	• Track re-ballasting could be a viable alternative to address the long-term settlements. • If the estimated settlements cannot be tolerated, the use of lightweight fill material (e.g., expanded polystyrene, Isofill, slag fill, etc.) as part of the embankment construction could be considered in order to limit the stress increase on the underlying silty clay and reduce post-construction settlement. The estimated settlement of the embankments will be refined during detailed design.
Construction Impact(s)	• Removal and replacement of any topsoil, organic material, and unsuitable fill material encountered below the base of the embankment.	• Evaluation of topsoil, organic material, and unsuitable fill material thicknesses during detailed design. • If the thickness of any unsuitable material is excessive, some form of temporary shoring may be required in order to allow for its removal.

4.0 MONTREAL ROAD INTERCHANGE (STATION 302+700 TO 303+800)

4.1 Project Requirements

At the Montreal Road interchange, the proposed location of the Eastern LRT transitions from north of OR 174 to within the median of OR 174 (east of about Station 303+250). The proposed alignment will cross over the following roadways (from west to east):

- Realigned Westbound on-ramp to OR 174 from southbound Montreal Road;
- Montreal Road;
- Westbound on-ramp to OR 174 from northbound Montreal Road; and
- Westbound OR 174.

As part of the proposed plans, the existing transitway on-ramp and off-ramp will be decommissioned.

The profile for the Eastern LRT at the Montreal Road interchange is well above existing grades (up to about 12 metres at Montreal Road). In order to implement the Eastern LRT, an overpass structure will be required (spanning a distance of 700 to 900 metres), along with approach embankments and permanent retaining walls.

4.2 Overview of Subsurface Conditions

4.2.1 Previous Investigations by Others

As part of the geotechnical investigation carried out in 1956 by G.C. McRostie Consulting Civil Engineers for the Montreal Road interchange, two (2) boreholes were advanced in the area of the Eastern LRT at Montreal Road. In addition, the Ministry of Transportation (MTO) advanced one (1) borehole just west of Green's Creek as part of a feasibility study for Highway 17 in 1978.

In general, the boreholes encountered about 32 to 33 metres of silty clay underlain by about 6 to 7 metres of glacial till over shale bedrock. The total overburden thickness was found to be about 39 metres.

The approximate locations of the relevant boreholes previously advanced by G.C. McRostie Consulting Civil Engineers and the MTO are shown on Sheets 6 and 7 in Appendix A. A copy of the Record of Borehole sheets for the relevant boreholes are provided in Appendix C for reference.

4.2.2 Preliminary Borehole Investigation

4.2.2.1 Methodology

In order to supplement the existing geotechnical information, two (2) boreholes, numbered 15-1 and 15-2, were advanced in the area of the Montreal Road interchange and Green's Creek crossing, respectively (i.e., in areas where major infrastructure improvements are anticipated). The boreholes were advanced using a truck mounted drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-Rouge, Quebec. Details for the boreholes are provided in Table 4.1.

Table 4.1 – Borehole Details

Borehole	Location	Program of Investigation
15-1	Montreal Road Interchange	• Borehole 15-1 was advanced through the shoulder of the westbound Transitway On-Ramp to OR 174 at Montreal Road to about 30.5 metres below ground surface. Dynamic cone penetration testing was carried out in borehole 15-1 between 16.8 and 30.5 metres below ground surface.
15-2	Green's Creek Crossing	• Borehole 15-2 was advanced through the shoulder of the westbound OR 174 to about 30.5 metres below ground surface. Dynamic cone penetration testing was carried out in borehole 15-2 between 16.8 and 30.5 metres below ground surface.

Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler. In situ vane shear testing was carried out in the boreholes to measure the undrained shear strength of the silty clay deposits.

The field work was supervised throughout by a member of our engineering staff, who located the boreholes, logged the samples and observed the in-situ testing. Following the field work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content and Atterberg limits.

The borehole locations were selected by Houle Chevrier Engineering Ltd. personnel and positioned at the site relative to existing site features. The ground surface elevations and locations of the boreholes were determined using a Trimble R8 GPS survey instrument. The elevations are referenced to Geodetic datum.

The approximate locations of the boreholes are shown on Sheets 6 and 7 in Appendix A. Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix C.

4.2.2.2 Subsurface Conditions

Granular Shoulder Material

Boreholes 15-1 was advanced through the shoulder of the westbound Transitway On-Ramp to OR 174 at Montreal Road and encountered about 2.1 metres of grey, crushed sand and gravel (base/subbase material). Borehole 15-2 was advanced through the shoulder of the westbound OR 174 and encountered about 1 metre of grey, crushed sand and gravel (base/subbase material). The water content of the base/subbase material is about 2 to 4 percent.

Silty Clay

Native deposits of silty clay were encountered in boreholes 15-1 and 15-2 beneath the base/subbase material at 2.1 and 1.0 metres below ground surface, respectively. The boreholes were terminated within inferred silty clay deposits at 30.5 metres below ground surface.

The upper part of the silty clay is weathered grey brown. Standard penetration tests carried out in the weathered, grey brown silty clay gave N values varying from 2 to 7 blows per 0.3 metres of penetration. Field vane shear strength tests carried within the weathered silty clay in boreholes 15-1 and 15-2 gave shear strengths of 69 to 96 kilopascals, which indicates a stiff consistency. At borehole 15-1, the weathered, grey brown silty clay has a thickness of about 2.5 metres and extends to a depth of about 4.6 metres below ground surface (elevation 55.6 metres). At borehole 15-2, the weathered, grey brown silty clay has a thickness of about 2.0 metres and extends to a depth of about 3.1 metres below ground surface (elevation 47.4 metres). The water content of the weathered silty clay varies from 28 to 61 percent.

Below the weathered zone at boreholes 15-1 and 15-2, the silty clay is grey in colour. Standard penetration tests carried out in the grey silty clay gave N values varying from 1 to 4 blows per 0.3 metres of penetration. Field vane shear strength tests carried within the grey silty clay in boreholes 15-1 and 15-2 gave shear strengths of 67 to 96 kilopascals, which indicates a stiff consistency. The water content of the grey silty clay varies from 60 to 76 percent.

Dynamic cone penetration tests were carried out in boreholes 15-1 and 15-2 between about 16.8 and 30.5 metres below ground surface. At borehole 15-1, the blow counts generally increased from 4 to 36 blows per 0.3 metres of penetration. At borehole 15-2, the blow counts generally increased from 9 to 57 blows per 0.3 metres of penetration. It should be noted that the results of the dynamic cone penetration testing carried out in boreholes 15-1 and 15-2 may have been influenced by friction acting along the length of rods. The dynamic cone penetration tests were terminated at 30.5 metres below ground surface, likely within the silty clay deposits.

The results of an Atterberg limit tests carried out on samples of the silty clay are provided on Figure C1 in Appendix C. The results are summarized in Table 4.2.

Table 4.2 – Summary of Atterberg Limit Test Results

Borehole	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Figure
15-1 (Sample No. 9)	76	53	27	26	C1
15-2 (Sample No. 5)	69	56	27	29	C1

This testing indicates that the silty clay has high plasticity. The water content of the sample tested is above the measured liquid limit values.

Groundwater Levels

Piezometers were not installed as part of this preliminary borehole investigation. However, based on our observations during drilling, the groundwater level is likely at 1.5 to 3.0 metres below ground surface.

4.3 Geotechnical Considerations

4.3.1 Overpass Structure

As previously indicated, construction of an overpass structure will be required in order to implement the Eastern LRT at the Montreal Road interchange. In this area, the Eastern LRT alignment is underlain by thick deposits of sensitive silty clay and deep foundations will be required (i.e., it will not be possible to achieve the required capacities from conventional spread footings founded within overburden). It is noted that the bedrock surface is located about 39 metres below ground surface. The following deep foundation alternatives could be considered:

Alternative 1 - Driven end bearing piles on sound bedrock

Alternative 2 - Socketed piles or caissons into sound bedrock

Comments on the deep foundation alternatives are summarized in Table 4.3.

Table 4.3 – Summary of Deep Foundation Alternatives at Montreal Road

Foundation Alternative	Comments
Alternative 1: Driven end bearing piles on sound bedrock	• The proposed structure could be supported on steel piles driven to refusal on the underlying bedrock. • Buoyancy (uplift) issues are anticipated if large diameter closed ended driven steel pipe piles are used. Therefore, consideration should be given to using steel H piles. The tips of H-piles should be reinforced with steel plates to reduce the potential for damage. • Cobbles and boulders should be anticipated in the glacial till. Some of the piles may be bent, driven off plumbness or location tolerance, may break, or may terminate on cobbles/boulders in the glacial till. Any defective piles should be replaced, as required. • Pile capacities at this site will depend on pile type, pile dimensions, pile material and the end bearing material. As an example, the following preliminary pile capacities could be used for H piles driven to refusal within sound bedrock: ○ The factored geotechnical reaction at Serviceability Limit States (SLS) could be taken as 850 kilonewtons. In accordance with the CHBDC, the geotechnical reaction at SLS was factored using a resistance factor of 0.8. ○ The factored geotechnical resistance at ULS could be taken as 1,200 kilonewtons. In accordance with the CHBDC, the geotechnical resistance at ULS was factored using a resistance factor of 0.4. • Post-construction settlement of the silty clay deposits is likely. For this case, the resulting downward movement of the soil around the piles will induce downdrag forces on the piles through negative skin friction. The downdrag load will depend on pile type, pile dimensions, and pile material and should be considered during design. • Re-striking all of the piles should be carried out in order to confirm the permanence of the pile set. It should be noted that achieving permanence of the pile set on or within the shale bedrock could be problematic and several rounds of re-striking may be required in order to achieve pile load capacities. • Pile caps should be provided with at least 1.8 metres of earth cover for frost protection purposes. • For uplift resistance (i.e., resistance to overturning), the tensile capacity of the driven piles should be ignored. Therefore, some form of uplift resistance will be required (e.g., tensioned rock anchors).

Table 4.3 – Summary of Deep Foundation Alternatives at Montreal Road (Continued)

Foundation Alternative	Comments
Alternative 2: Socketed piles or caissons into sound bedrock	• Consideration could be given to founding the proposed structure on steel piles socketed and then grouted into bedrock (e.g., using the ODEX drilling system). Alternatively, large diameter concrete caissons socketed into bedrock could be considered. • For preliminary design and costing purposes, the geotechnical resistance at ULS of socketed piles or caissons that derive support only in shear within the bedrock could be calculated using an unfactored average shear resistance along the socket of about 1,000 kilopascals. The geotechnical reaction at SLS will be greater than the factored geotechnical resistance at ULS; as such, ULS conditions will govern. For socketed piles, the structural capacity of the pile will govern if it is less than the capacity derived from socket shear. • The capacity derived from socket shear could be used to provide both compression and uplift (tension) resistance. In accordance with the CHBDC, the geotechnical resistance at ULS should be factored using resistance factors of 0.4 and 0.3 for compression and tension, respectively. • Socketed micropiles or caissons that derive support in shear within the bedrock should have a nominal socket length to diameter ratio of at least 2 to 3. • Post-construction settlement of the silty clay deposits is likely. For this case, the resulting downward movement of the soil around the piles/caissons will induce downdrag forces on the piles/caissons through negative skin friction. The downdrag load will depend on pile/caisson type, pile/caisson dimensions, and pile/caisson material and should be considered during design. • Cobbles and boulders should be anticipated in the glacial till. As such, allowance should be made to break boulders, where necessary, within a temporary steel casing using churn drilling techniques or to remove any boulders encountered by the caissons. Any voids created during removal of boulders should be filled with concrete. • Pile caps (and caisson caps, if required) should be provided with at least 1.8 metres of earth cover for frost protection purposes.

For comparison purposes, it is noted that socketed piles/caissons (Alternative 2) can provide uplift resistance whereas with driven piles, some form of uplift resistance will be required (e.g., tensioned rock anchors). Furthermore, cobbles and boulders obstructions within the glacial till are less problematic for Alternative 2 relative to Alternative 1. Higher capacities could be achieved using socketed concrete caissons relative to driven/socketed piles.

Based on the results of shear wave velocity testing that we have carried out in similar deposits in the Ottawa area, seismic Site Class E could be used for preliminary foundation design purposes. Site specific shear wave velocity testing should be carried out during the design stage to confirm the seismic Site Classification and evaluate whether a more favourable Site Class (i.e., Site Class D) can be specified. There is no potential for liquefaction of the overburden material at this site.

4.3.2 Approach Embankments

Approach embankments to the proposed overpass structure will be required. The west approach embankment will be located north of OR 174. The east approach embankment will be located within the median of OR 174. Although the height of the approach embankments are not presently known, preliminary considerations for embankment construction are provided below:

- All topsoil, organic material, and unsuitable fill material should be removed from the base of the embankments.
- The embankment fill material could consist of compacted material meeting OPSS Granular B Type I or II or OPSS Select Subgrade Material.
- Final fill slopes constructed with OPSS Granular B Type I or Select Subgrade Material could be sloped at 2.5 horizontal to 1 vertical, or flatter; for Granular B Type II, fill slopes of 2.0 horizontal to 1 vertical, or flatter would be suitable. Where steeper embankment slopes are required due to space limitations (e.g., for the east approach), some form of retaining structure, such as a geogrid reinforced slope, a cast in place concrete wall, or a Mechanically Stabilized Earth (MSE) wall could be considered. The cast in place walls could be a cantilever wall with the base extending below the ELRT, and below the zone of seasonal frost penetration. MSE walls could bear on a narrow strip footing below the zone of seasonal frost penetration.
- The approach embankments will be underlain by deposits of sensitive silty clay which have a reduced capacity to support loads imposed by grade raise fill material. Based on the settlement history of previous embankments constructed on sensitive silty clay deposits in the Ottawa area, the long-term settlement of embankments that are 4 to 5 metres high could be in the order of 100 to 200 millimetres. For embankments greater than 4 to 5 metres in height, long-term settlements in excess of 100 to 200 millimetres should be expected.
- The settlement of the embankments would be entirely differential to the rigidly supported overpass structure. Furthermore, the design of the embankments must also consider the location of OR 174 and the impact that the settlements may have on that roadway.
- The overall stability of the embankment side slopes will depend on the ultimate height of the approach embankments and inclination of the side slopes. However, to assess the feasibility of embankment construction in this area, preliminary slope stability analyses

were carried out. The results of the stability analyses indicate that 6 metre high approach embankments, with side slopes constructed at 2.5 horizontal to 1 vertical, would have an adequate factor of safety against a deep seated shear failure through the silty clay. Furthermore, the preliminary slope stability analyses also indicate that a 4 metre high vertical retaining structure would be feasible.

4.4 Impacts and Mitigation Measures

A summary of the possible construction and operational impacts in the area of the Montreal Road interchange are summarized in Table 4.4.

Table 4.4 – Impacts and Mitigation Measures (Montreal Road Interchange)

Type	Impact(s)	Mitigation Measure(s)
Operational Impact(s)	• Differential settlement of the approach embankments will be entirely differential to the rigidly supported overpass structure. • Depending on the location of OR 174 relative to the proposed embankments, construction of the approach embankments may result in localized post-construction settlement of OR 174.	• Construction of approach slabs at the transition between the structure and embankments. • Track re-ballasting could be a viable alternative to address the long-term settlements • If the estimated settlements cannot be tolerated, the use of lightweight fill material (e.g., expanded polystyrene, Isofill, slag fill, etc.) as part of the embankment construction could be considered in order to limit the stress increase on the underlying silty clay and reduce post-construction settlement. Site pre-loading (with or without wick drains) could also be considered to reduce post-construction settlements; however, pre-loading would not mitigate the potential for settlement related impacts to OR 174. The estimated settlement of the embankments will be refined during detailed design.
Construction Impact(s)	• Deep foundations, which derive their capacity from the underlying bedrock, are required for support of the proposed overpass structure. The bedrock surface is reported to be located about 39 metres below ground surface.	• Various foundation alternatives are available.

5.0 GREEN'S CREEK TO TRIM ROAD (STATION 303+800 TO 312+200)

5.1 Project Requirements

The proposed Eastern LRT alignment between Green's Creek and Trim Road is located within the median of OR 174. In general, the proposed grades along this portion of the Eastern LRT will match the existing grades.

In order to implement the Eastern LRT between Green's Creek and Trim Road, regrading of the toe of the existing approach embankments for the Sir-George-Étienne Cartier underpass is required. This will likely require the construction of some form of retaining structure along the toe of the embankment to allow for widening of the corridor below the existing structure.

It is noted that replacement of the Champlain Street underpass will likely be required in order to accommodate the proposed widening of OR 174. As previously indicated, the impacts and geotechnical considerations for the proposed widening of OR 174 are beyond the scope of this report.

5.2 Overview of Subsurface Conditions

Based on the available subsurface information, the subsurface conditions between Green's Creek and Trim Road are expected to consist primarily of deposits of silty clay. Shallow bedrock is mapped near the Place D'Orléans Shopping Centre (bedrock outcrops exist at the Champlain Street overpass and the pedestrian bridge at Place D'Orléans Station). Deposits of glacial till should be expected at the transition areas between shallow bedrock and deposits of silty clay. The overburden thickness is highly variable, ranging from about 3 to 100 metres, being thinnest near areas where shallow bedrock is mapped (i.e., Place D'Orléans Shopping Centre). Bedrock is mapped as dolostone (Oxford formation), limestone (Bobcaygeon formation), and interbedded limestone and dolostone (Gull River formation).

5.3 Geotechnical Considerations

5.3.1 Regrading Existing Embankments (Sir-George-Étienne Cartier Underpass)

As previously indicated, regrading of the toe of the existing approach embankments for the Sir-George-Étienne Cartier underpass is required. This will likely require the construction of some form of retaining structure along the toe of the embankment to allow for widening of the corridor below the existing underpass. The retaining structure could consist of a geogrid reinforced slope, a cast in place concrete wall, or a Mechanically Stabilized Earth (MSE) wall. To allow for construction, excavation side slopes of 1 horizontal to 1 vertical, or flatter, are required.

The overall stability of the regraded embankment side slopes will depend on the ultimate height of the retaining structure (i.e., the amount of "trimming required at the toe of the existing embankment side slope). However, to assess the feasibility of embankment regrading in this area, preliminary slope stability analyses were carried out. The results of the stability analyses indicate that 3 metre high retaining structure constructed along the base of an 8 metre high embankment, with side slopes of 2 to horizontal to 1 vertical, would have an adequate factor of safety against a deep seated shear failure through the silty clay.

5.4 Impacts and Mitigation Measures

No significant construction or operational impacts are anticipated along this portion of the Eastern LRT.

6.0 TRIM ROAD INTERCHANGE (STATION 312+200 TO 312+900)

6.1 Project Requirements

The proposed Eastern LRT alignment in the area of the Trim Road interchange is located within the median of OR 174. In general, the proposed grades along this portion of the Eastern LRT will match the existing grades.

In order to implement the Eastern LRT in the area of the Trim Road interchange, construction of a new overpass structure, and approach embankments, will be required to carry Trim Road over the Eastern LRT and OR 174. In addition, on and off ramps to OR 174 from Trim Road will be constructed; however, the geotechnical impacts and design considerations for the proposed ramps are beyond the scope of this report.

6.2 Overview of Subsurface Conditions

Based on our experience in the area, the area of the Trim Road interchange is underlain by thick deposits of firm to stiff, sensitive silty clay. The overburden thickness in this area is presently unknown. Drift thickness maps indicate that the bedrock surface is located between 15 and 25 metres below ground surface; however, based on our experience in the area, it is anticipated that the bedrock surface is located more than 30 metres below ground surface. Furthermore, based on our review of MOECC water well records, the bedrock surface could be located between about 60 and 80 metres below ground surface. Bedrock is mapped as interbedded limestone and dolostone (Gull River formation).

6.3 Geotechnical Considerations

6.3.1 Underpass Structure

As previously indicated, construction of an underpass structure will be required in order to implement the Eastern LRT at the Trim Road interchange. In this area, the Eastern LRT alignment is underlain by thick deposits of sensitive silty clay and deep foundations will be required (i.e., it will not be possible to achieve the required capacities from conventional spread footings founded within overburden). It is noted that the bedrock surface in this area is presently unknown, but could be located up to between 60 and 80 metres below ground surface. The following deep foundation alternatives could be considered:

Alternative 1 - Driven end bearing piles on sound bedrock

Alternative 2 - Socketed piles or caissons into sound bedrock

Comments on the deep foundation alternatives are summarized in Table 6.1.

Table 6.1 – Summary of Deep Foundation Alternatives at Trim Road

Foundation Alternative	Comments

Alternative 1: Driven end bearing piles on sound bedrock	• The proposed structure could be supported on steel piles driven to refusal on the underlying bedrock.
	• Buoyancy (uplift) issues are anticipated if large diameter closed ended driven steel pipe piles are used. Therefore, consideration should be given to using steel H piles. The tips of H-piles should be reinforced with steel plates to reduce the potential for damage.
	• Pile capacities at this site will depend on pile type, pile dimensions, pile material and the end bearing material. As an example, the following preliminary pile capacities could be used for H piles driven to refusal within sound bedrock:
	  ○ The factored geotechnical reaction at Serviceability Limit States (SLS) could be taken as 850 kilonewtons. In accordance with the CHBDC, the geotechnical reaction at SLS was factored using a resistance factor of 0.8.
	  ○ The factored geotechnical resistance at ULS could be taken as 1,200 kilonewtons. In accordance with the CHBDC, the geotechnical resistance at ULS was factored using a resistance factor of 0.4.
	• Post-construction settlement of the silty clay deposits due to embankment construction will result in downward movement of the soil around the piles induce downdrag forces on the piles through negative skin friction. The downdrag load will depend on pile type, pile dimensions, and pile material and should be considered during design.
	• Pile caps should be provided with at least 1.8 metres of earth cover for frost protection purposes.
	• For uplift resistance, the tensile capacity of the driven piles should be ignored. Therefore, if some form of uplift resistance will be required (e.g., tensioned rock anchors).

Table 6.1 – Summary of Deep Foundation Alternatives at Trim Road (Continued)

Foundation Alternative	Comments
Alternative 2: Socketed piles or caissons into sound bedrock	• Consideration could be given to founding the proposed structure on steel piles socketed and then grouted into bedrock (e.g., using the ODEX drilling system). Alternatively, large diameter concrete caissons socketed into bedrock could be considered. • For preliminary design and costing purposes, the geotechnical resistance at ULS of socketed piles or caissons that derive support only in shear within the bedrock could be calculated using an unfactored average shear resistance along the socket of about 1,000 kilopascals. The geotechnical reaction at SLS will be greater than the factored geotechnical resistance at ULS; as such, ULS conditions will govern. For socketed piles, the structural capacity of the pile will govern if it is less than the capacity derived from socket shear. • The capacity derived from socket shear could be used to provide both compression and uplift (tension) resistance. In accordance with the CHBDC, the geotechnical resistance at ULS should be factored using resistance factors of 0.4 and 0.3 for compression and tension, respectively. • Socketed micropiles or caissons that derive support in shear within the bedrock should have a nominal socket length to diameter ratio of at least 2 to 3. • Post-construction settlement of the silty clay deposits due to embankment construction will result in downward movement of the soil around the piles/caissons induce downdrag forces on the piles/caissons through negative skin friction. The downdrag load will depend on pile/caisson type, pile/caisson dimensions, and pile/caisson material and should be considered during design. • Pile caps (and caisson caps, if required) should be provided with at least 1.8 metres of earth cover for frost protection purposes.

It is noted that the bedrock surface could be located up to 80 metres below ground surface. For this case, caisson construction may not be a viable alternative for the proposed underpass.

Based on the results of shear wave velocity testing that we have carried out for developments south of the Trim Road interchange, seismic Site Class E could be used for preliminary foundation design purposes. Site specific shear wave velocity testing should be carried out during the design stage to confirm the seismic Site Classification and evaluate whether a more favourable Site Class (i.e., Site Class D) can be specified. There is no potential for liquefaction of the overburden material at this site.

6.3.2 Approach Embankments

Approach embankments to the proposed underpass structure will be required. Although the height of the approach embankments is not presently known, preliminary considerations for embankment construction are provided below:

- All topsoil, organic material, and unsuitable fill material should be removed from the base of the embankments.
- The embankment fill material could consist of compacted material meeting OPSS Granular B Type I or II, OPSS Select Subgrade Material, or suitable, well graded and shattered blast rock.
- Final fill slopes constructed with OPSS Granular B Type I or Select Subgrade Material could be sloped at 2.5 horizontal to 1 vertical, or flatter; for Granular B Type II or well graded and shattered blast rock, fill slopes of 2.0 horizontal to 1 vertical, or flatter would be suitable. Where steeper embankment slopes are required due to space limitations, some form of retaining structure, such as a geogrid reinforced slope, a cast in place concrete wall, or a Mechanically Stabilized Earth (MSE) wall could be considered.
- The approach embankments will be underlain by deposits of sensitive silty clay which have a reduced capacity to support loads imposed by grade raise fill material. Based on the settlement history of previous embankments constructed on sensitive silty clay deposits in the Ottawa area, the estimated long-term settlement of 2 to 3 metre high embankments could be in the order of 100 to 200 millimetres. Since the approach embankments will likely be in the order of 7 metres high, the long-term settlement is expected to exceed settlement tolerances. Consideration could be given to:
  - Pre-loading the site could be considered to allow for the majority of the primary consolidation settlement of the silty clay deposits to occur prior to construction of the proposed embankments. Pre-loading could be carried out in conjunction with the use of wick drains to accelerate the rate of settlement. It is noted that secondary consolidation and settlement of the silty clay deposits will continue after the pre-loading material is removed from the site. This will result is some future settlement of the embankments.
  - The use of lightweight fill material (e.g., expanded polystyrene or slag fill) as part of the embankment construction in order to limit the stress increase on the underlying silty clay and reduce post-construction settlement.
- The settlement of the embankments would be entirely differential to the underpass structure. Furthermore, the design of the embankments must also consider the location of OR 174, the proposed Trim Road Station, and any adjacent services, and the impact that the settlements may have on that infrastructure.
- The overall stability of the embankment side slopes will depend on the geometry and composition of the approach embankments. However, to assess the feasibility of

embankment construction in this area, preliminary slope stability analyses were carried out. The results of the stability analyses indicate that 6 metre high approach embankments, with side slopes constructed at 2 to 2.5 horizontal to 1 vertical, would have an adequate factor of safety against a deep seated shear failure through the silty clay.

6.4 Impacts and Mitigation Measures

A summary of the possible construction and operational impacts in the area of the Trim Road interchange are summarized in Table 6.2.

Table 6.2 – Impacts and Mitigation Measures (Trim Road Interchange)

Type	Impact(s)	Mitigation Measure(s)
Operational Impact(s)	• Differential settlement of the approach embankments will be entirely differential to the overpass structure. • Depending on the location of OR 174, and any existing services, relative to the proposed embankments, construction of the approach embankments may result in post-construction settlement of OR 174, the proposed Trim Road Station and any adjacent services. • Embankments that are greater than 2 to 3 metres in height could have long-term settlements in excess of 100 to 200 millimetres.	• Construction of approach slabs at the transition between the structure and embankments. • Track re-ballasting could be a viable alternative to address the long-term settlements. • If the estimated settlements cannot be tolerated, the use of lightweight fill material (e.g., expanded polystyrene or slag fill) as part of the embankment construction could be considered in order to limit the stress increase on the underlying silty clay and reduce post-construction settlement. Site pre-loading (with or without wick drains) could also be considered to reduce post-construction settlements; however, pre-loading would not mitigate the potential for settlement related impacts to OR 174 or any adjacent services. The estimated settlement of the embankments will be refined during detailed design.
Construction Impact(s)	• Deep foundations, which derive their capacity from the underlying bedrock, are required for support of the proposed overpass structure. The location of the bedrock surface is presently unknown, but could be located up to 80 metres below ground surface.	• Various foundation alternatives are available.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Eastern LRT Stations

Stations will be constructed at various locations along the Eastern LRT alignment. For preliminary design and costing purposes, our comments on the proposed stations are provided in Table 7.1.

Table 7.1 – Preliminary Design Information for the Transit Stations

Station	Anticipated Subsurface Conditions	Most Probable Foundation Type	Factored Net Geotechnical Reaction at SLS (kPa)	Factored Net Geotechnical Resistance at ULS (kPa)	Comments
Montreal	Silty Clay	Deep Foundations	Refer to Table 4.3	Refer to Table 4.3	• The station will be located about 4 to 10 metres above existing grade. • It is likely that a portion of the station will be founded on the approach embankment and a portion carried by the overpass structure. • Given that significant settlement of the approach fills should be anticipated, deep foundations are likely for the west portion of the station structure.
Jeanne D'Arc	Silty Clay	Spread Footings	100 to 125	200 to 250	• No significant grade raise required. It is anticipated that the footings will be founded directly on native soil.
Orleans Boulevard	Silty Clay	Spread Footings	100 to 125	200 to 250	• No significant grade raise required. It is anticipated that the footings will be founded directly on native soil.
Place D'Orleans	Glacial Till or Bedrock	Spread Footings	150 (Till)	300 (Till) 1500 (Bedrock)	• No significant grade raise required. It is anticipated that the footings will be founded directly on native soil or bedrock.

Table 7.1 – Preliminary Design Information for the Transit Stations (Continued)

Station	Anticipated Subsurface Conditions	Most Probable Foundation Type	Factored Net Geotechnical Reaction at SLS (kPa)	Factored Net Geotechnical Resistance at ULS (kPa)	Comments
Orleans Town Centre	Silty Clay	Spread Footings	100 to 125	200 to 250	• No significant grade raise required. It is anticipated that the footings will be founded directly on native soil.
Tenth Line Station	Silty Clay	Spread Footings	100 to 125	200 to 250	• No significant grade raise required. It is anticipated that the footings will be founded directly on native soil.
Trim Station	Silty Clay	Spread Footings	100 to 125	200 to 250	• No significant grade raise required. It is anticipated that the footings will be founded directly on native soil or bedrock.

The silty clay deposits along the Eastern LRT alignment are highly susceptible to frost heaving when exposed to freezing temperatures. The footings/caps should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Furthermore, in order to avoid differential frost heaving between the station platforms and tracks, the subgrade surface below the tracks, within the stations, could be protected from frost using extruded polystyrene insulation. Transitions tapers will be required where the extruded polystyrene insulation is terminated beyond the stations.

7.2 Culvert Crossings

The proposed Eastern LRT alignment will cross structural culverts at the Jasmine Park Ravine, Green's Creek, Bilberry Creek, and Taylor Creek. Any culverts that are reaching the end of their service life should be rehabilitated/replaced prior to the Eastern LRT construction.

7.3 Track Bed

The track ballast and subballast should be appropriately designed for the sensitive silty clay deposits along the proposed alignment. Frost tapers will be required where the track bed transitions between different subgrade materials (e.g., from silty clay to glacial till). Other transition treatments may be required; for example, where the subgrade abruptly changes from overburden to bedrock, it may be necessary to "cushion" the transition by replacing the upper part of the bedrock in the area of the transition with granular material.

Adequate drainage of the ballast/subballast materials is important for the long term performance of the track bed. In order to provide drainage of the subballast, it is suggested that subdrains be installed in areas where the Eastern LRT will be in cut section and in areas where the Eastern LRT will be located between the east and westbound lanes of OR 174.

7.4 Vibrations

Rail lines can produce levels of ground vibrations that may be perceptible at nearby buildings. The vibration levels felt at nearby buildings will depend on the subsurface conditions in the area. If significant, vibrations could affect the liveability of adjacent buildings and some form of vibration isolation may be required. Vibration impact studies should be carried out early in the planning process for the Eastern LRT.

7.5 Noise Wall and Lighting Foundations

It is anticipated that any standard low height noise wall/lighting foundations will consist of augered concrete piers completed in native deposits of silty clay or glacial till. It is noted cobble and boulders obstructions may be encountered within the glacial till. As such, it may be necessary to remove/break any boulders encountered during augering. To minimize disturbance to the sensitive silty clay deposits, foundation construction within silty clay using vibratory methods should be avoided. Where bedrock is encountered, socketed piers or spread footing foundations (with or without anchors to resist overturning) may be required.

High mast lighting, where required, will likely require some form of deep foundation to resist overturning (e.g., concrete caissons).

7.6 Additional Investigation and Comments

This report provides preliminary geotechnical guidelines, and our interpretation of the construction/operations impacts, for the project based on available sources of information. Detailed geotechnical investigations should be carried out as part of the final design and detailed costing.

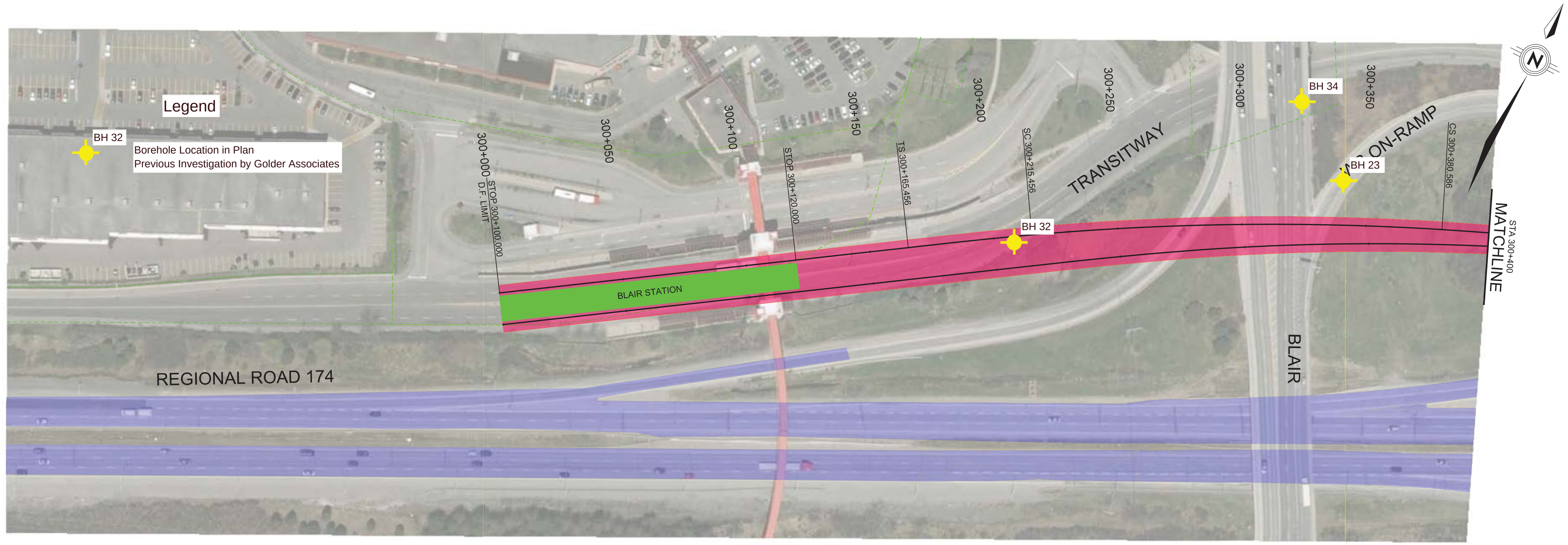
We trust that this report is sufficient for your current requirements. If you have any questions concerning this information or if we can be of further assistance to you on this project, please call.

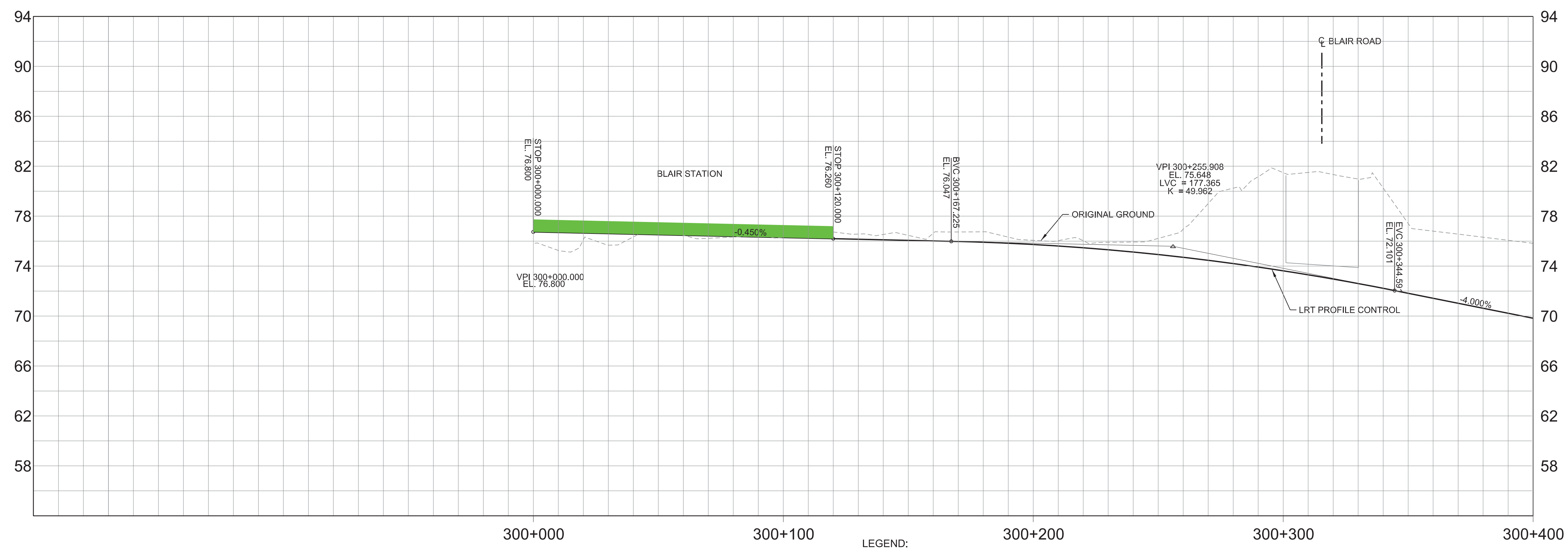
Johnathan A. Cholewa, Ph.D., P.Eng.

Andrew Chevrier, M.Eng., P.Eng.
Principal

APPENDIX A

Proposed Eastern LRT Plan and Profile (Sheets 1 to 22)





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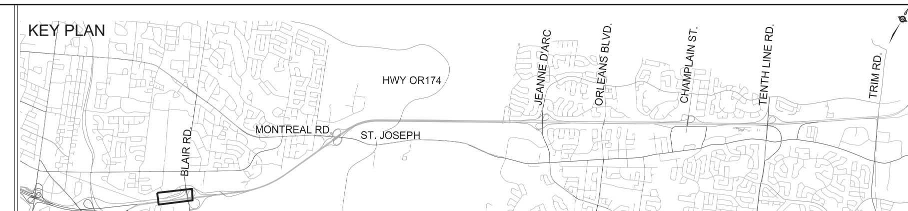
KEY PLAN



LEGEND:

PROPOSED LRT CORRIDOR	WATERCOURSES
PROPOSED LRT	PROPERTY LIMITS
PROPOSED LRT STATIONS	WATERMAIN
PROPOSED WIDENING	
PROPOSED LANE ALIGNMENT	
EXISTING ROADWAY	

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

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CAD File Name: TO3175TOC- 01-ELRT-01.DGN

Plot Date: XX/XX/XXXX

EASTERN LRT
STA. 300+000 to STA 313+000

Drawings No.:

Revision 00

Sheet No. 01

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Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:25:48 AM

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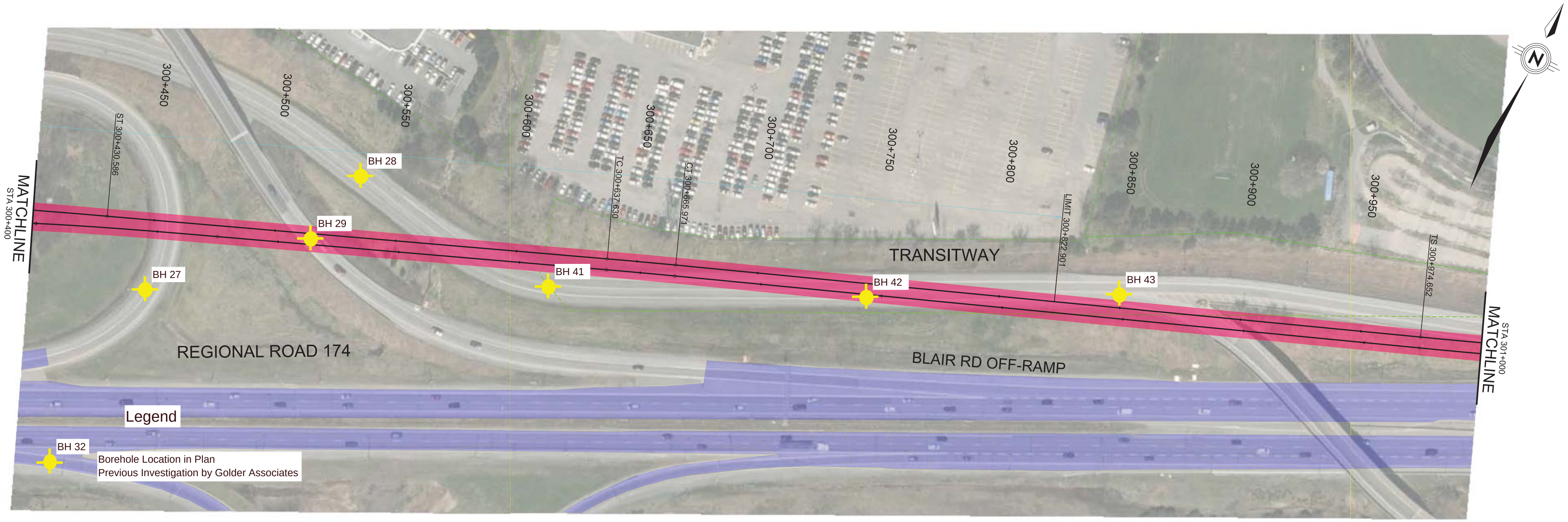

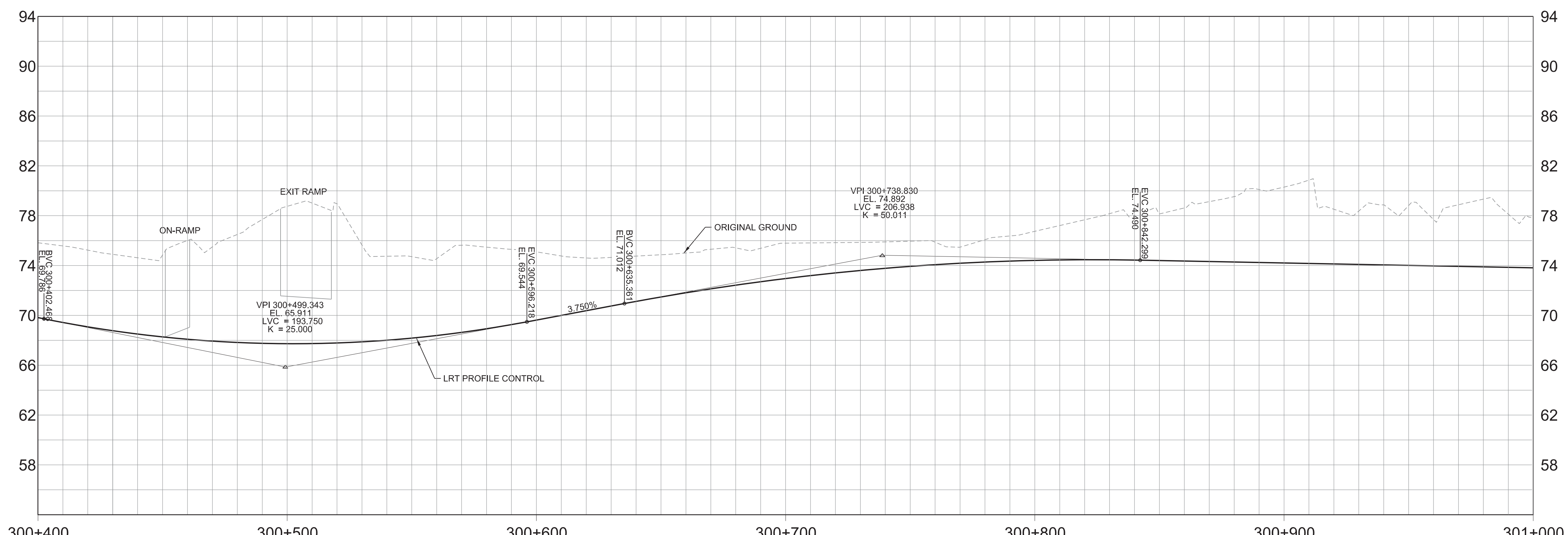

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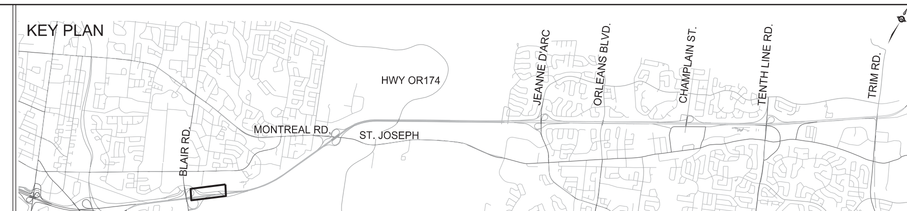


LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

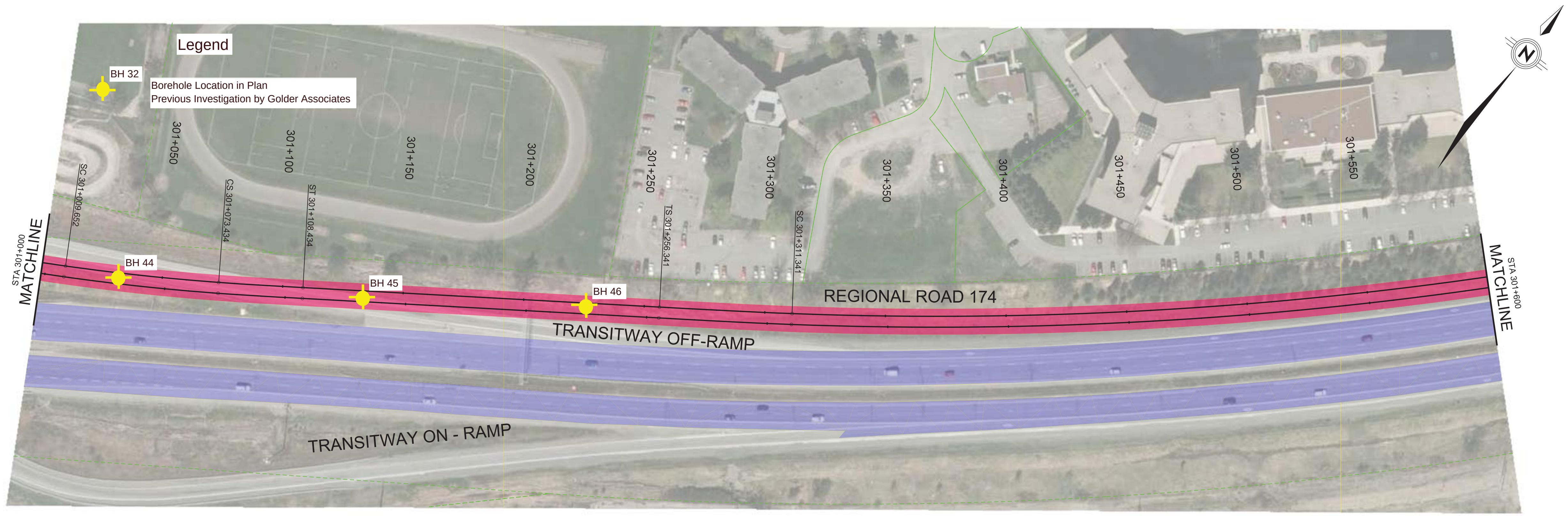
PARSONS

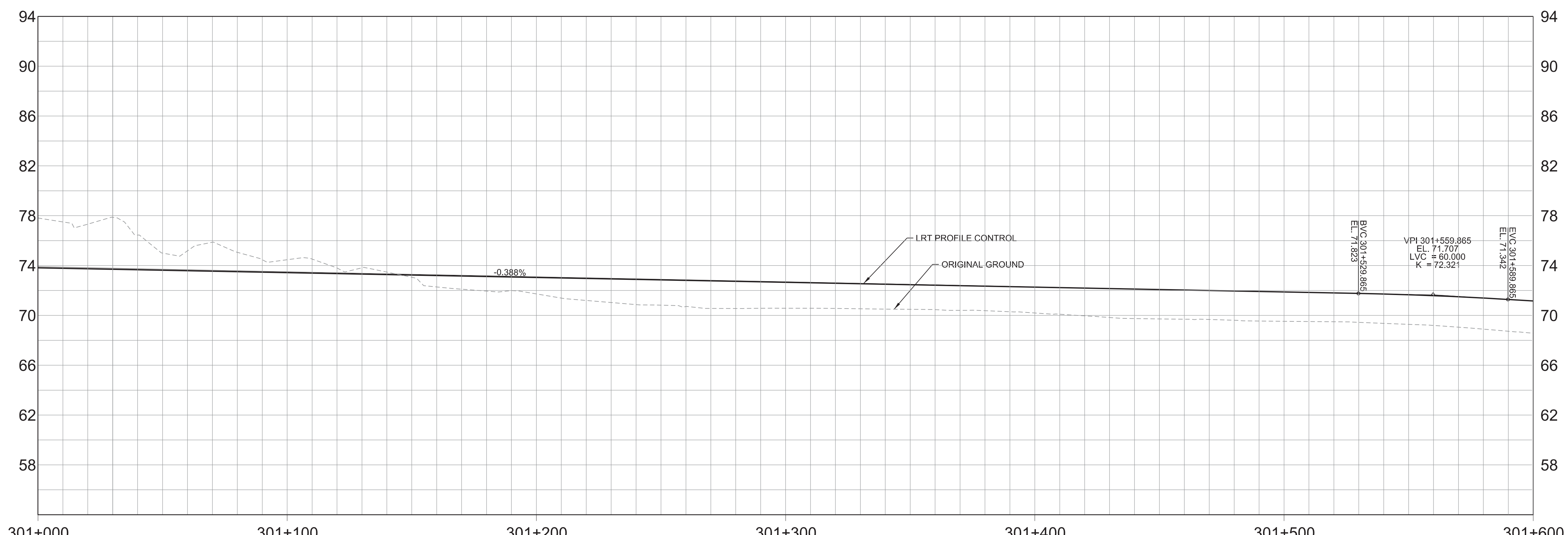
Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-02.DGN		Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 300+400 to STA 301+000

Drawings No.:	Revision	Sheet No.
	00	02





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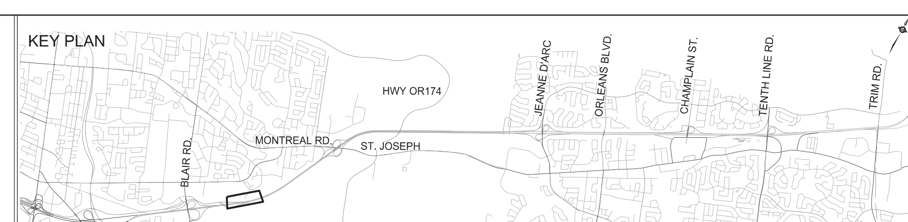

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-03.DGN

Plot Date: XX/XX/XXXX

EASTERN LRT
STA. 301+000 to STA 301+600

Drawings No.:

Revision 00

Sheet No. 03

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:29:15 AM

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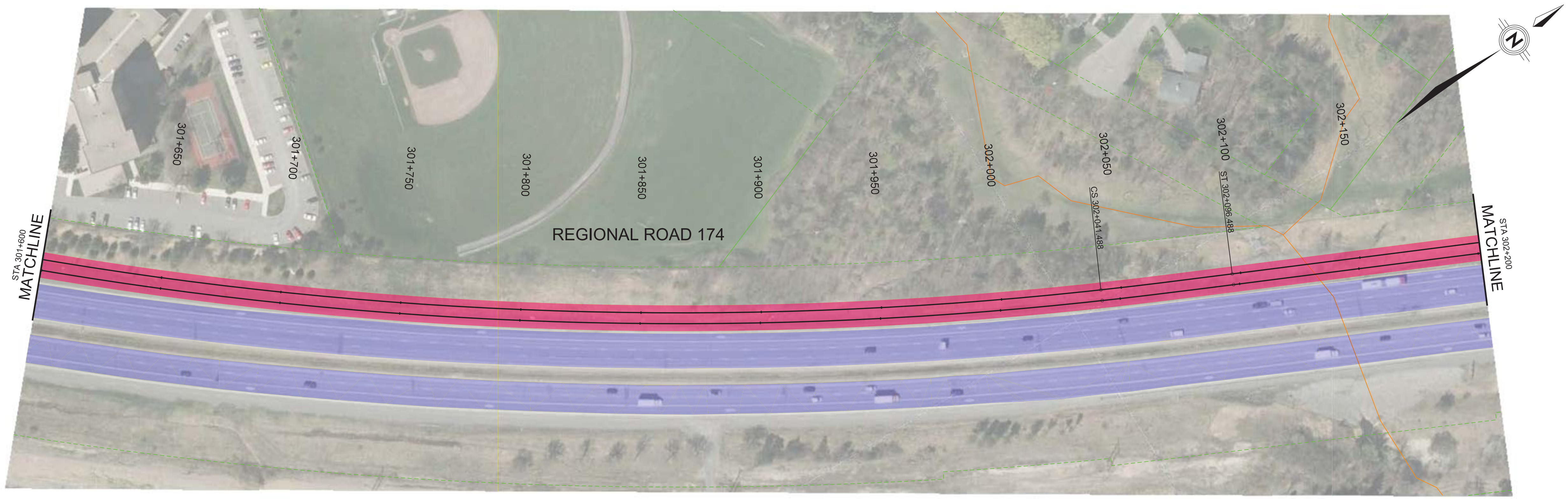

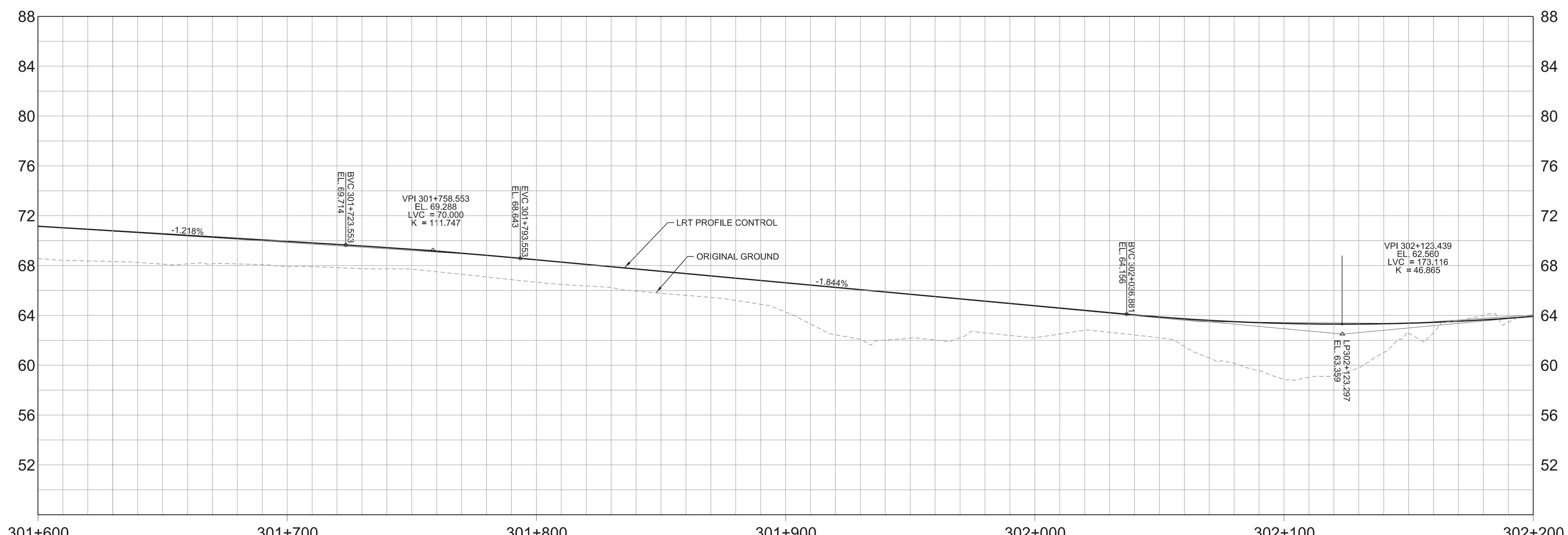

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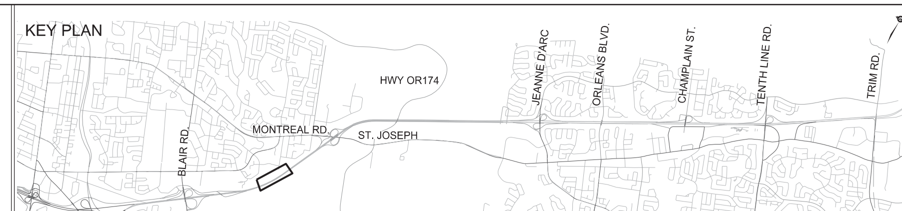

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-04.DGN		Plot Date: XX/XX/XXXX

EASTERN LRT
STA. 301+600 to STA 302+200

Drawings No.:	Revision	Sheet No.
	00	04

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:30:17 AM

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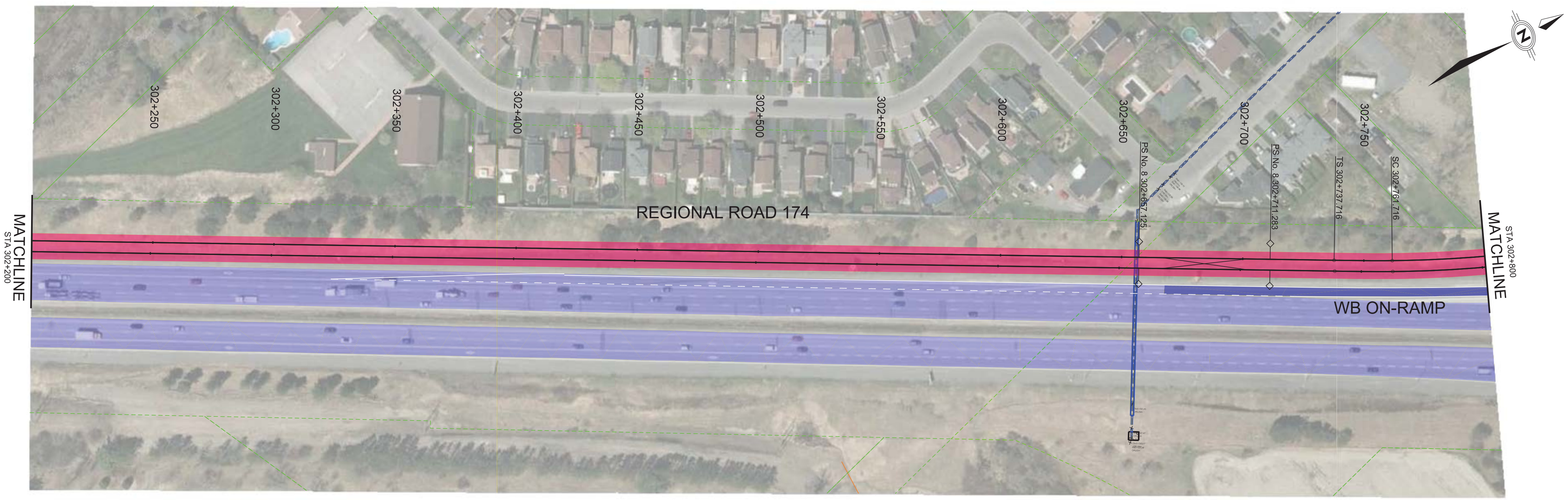

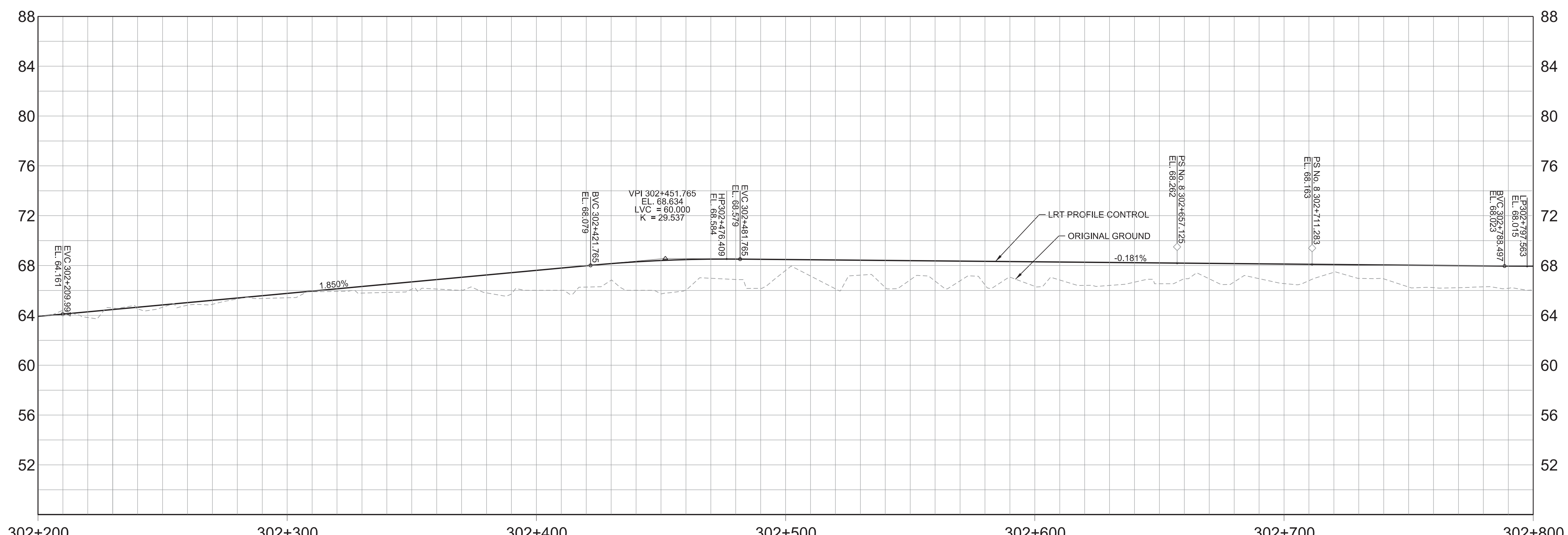

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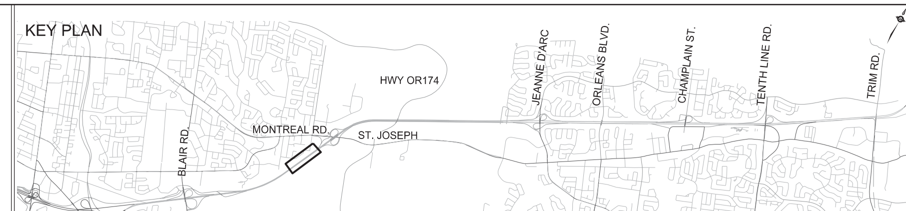

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-05.DGN

Plot Date: XX/XX/XXXX

EASTERN LRT
STA. 302+200 to STA 302+800

Drawings No.:

Revision 00

Sheet No. 05

Plot Date: $DATE$ $TIME$

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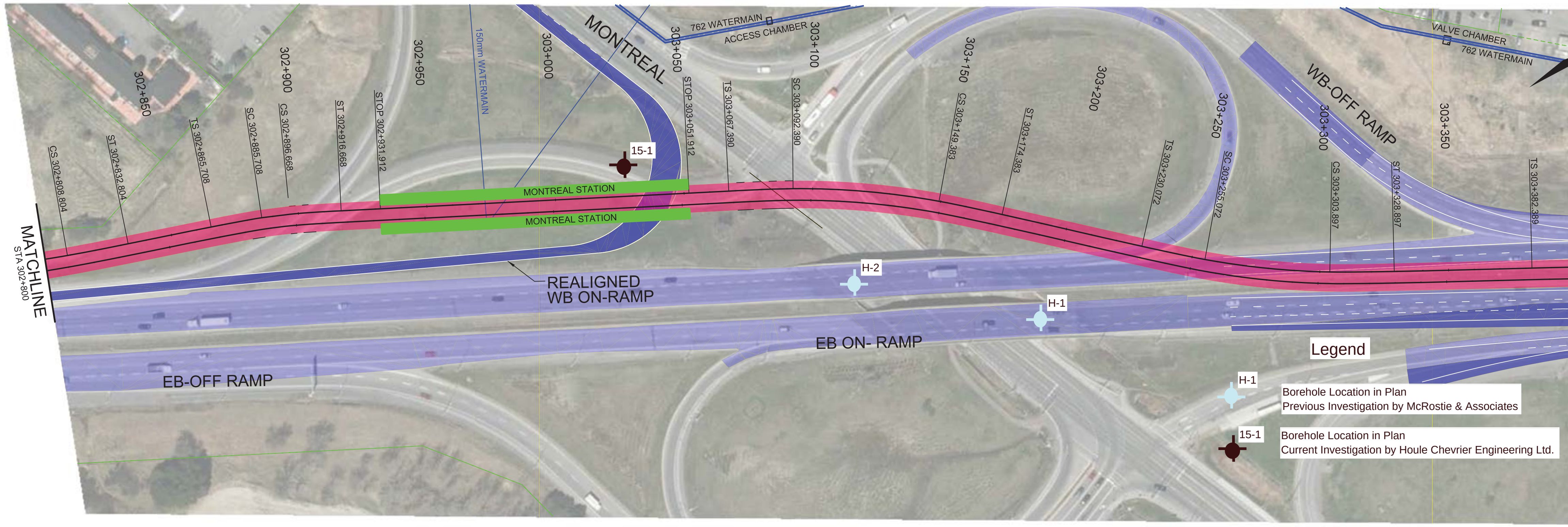

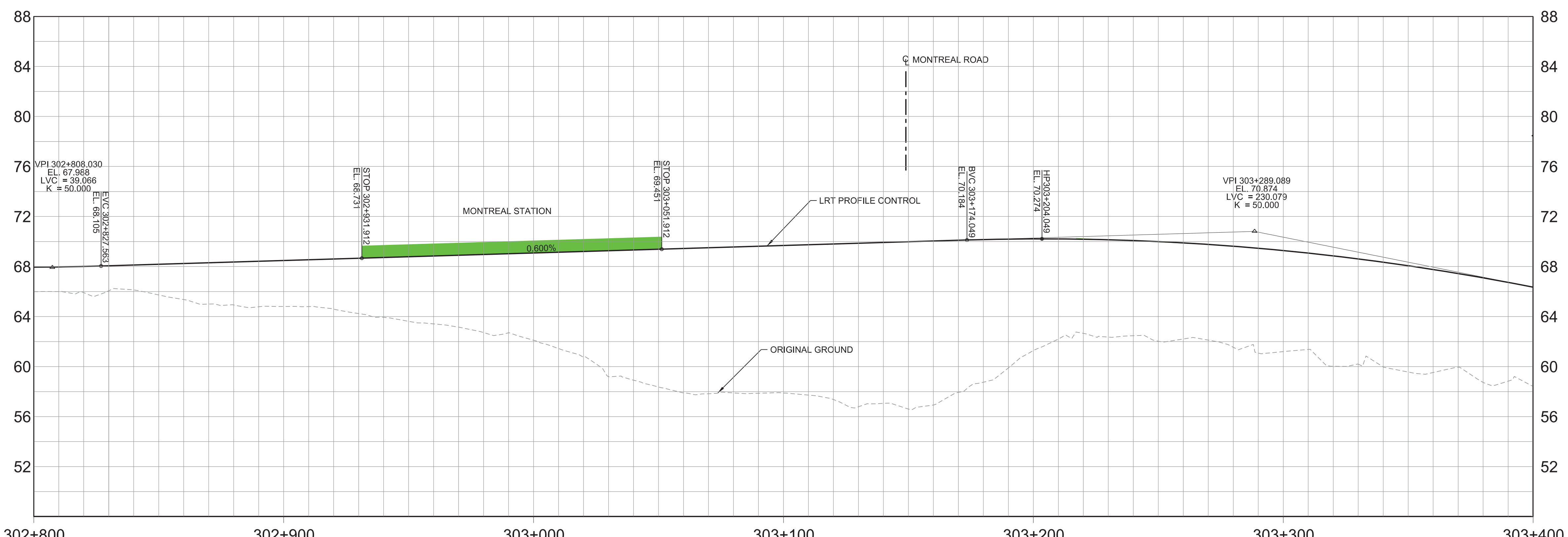

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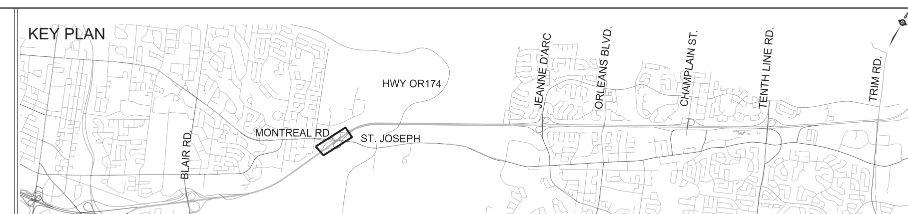

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-06.DGN		Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 302+800 to STA 303+400

Drawings No.:

Revision 00 | Sheet No. 06

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:32:59 AM

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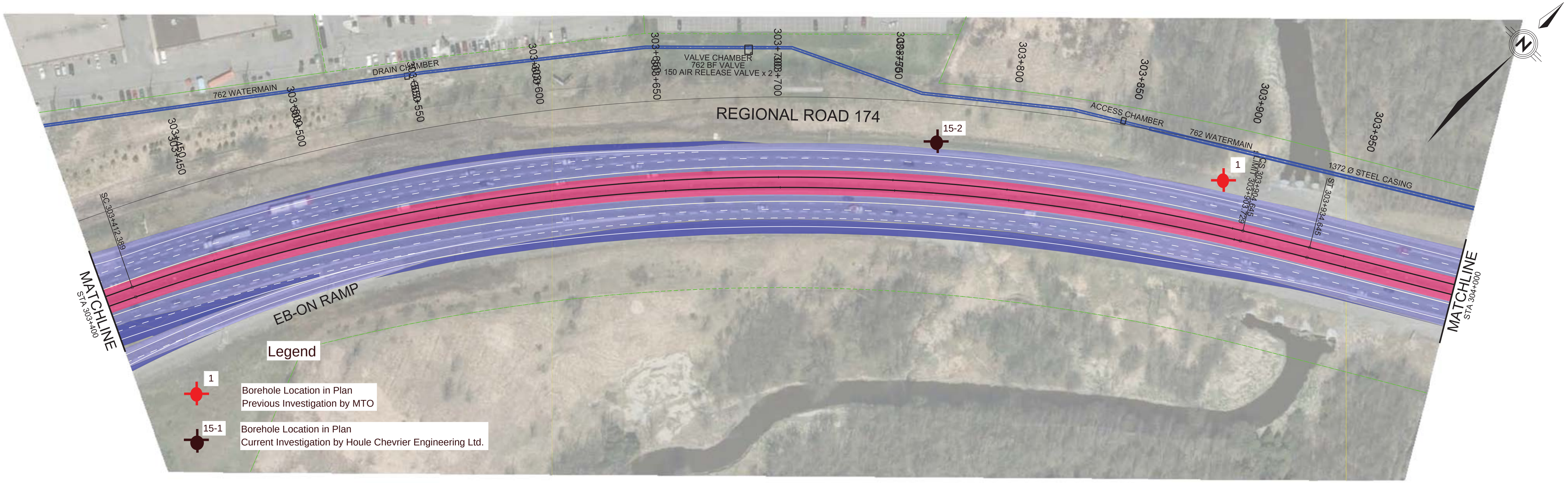

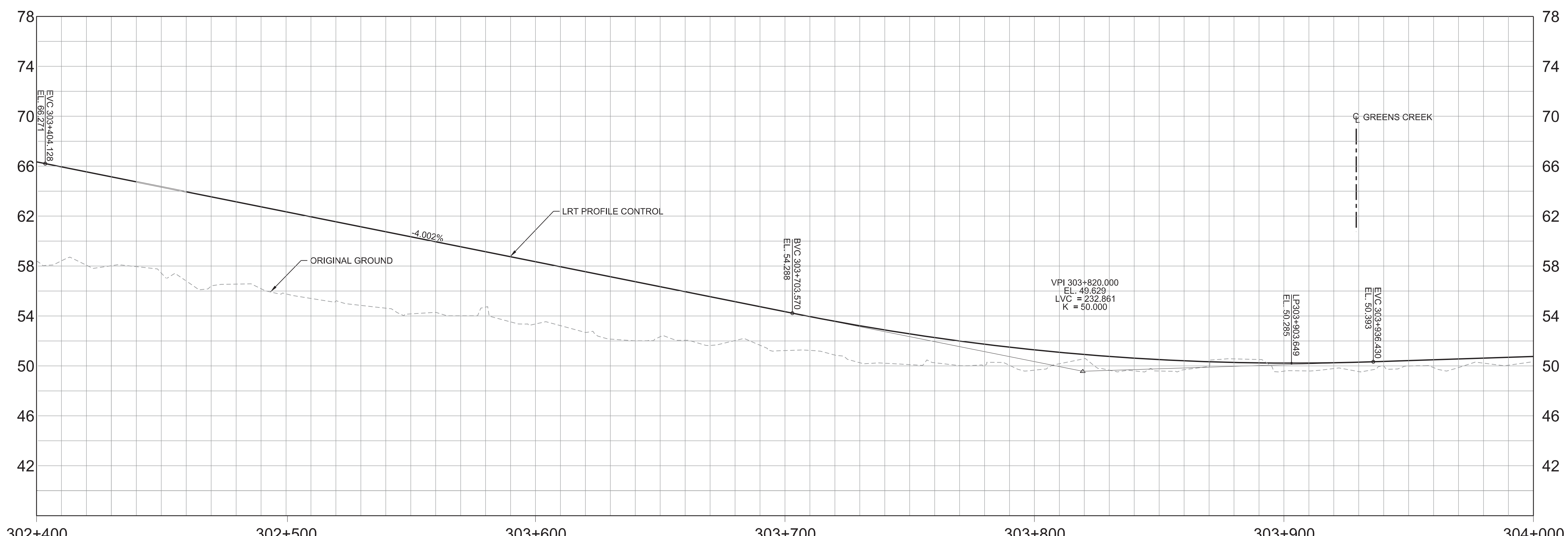

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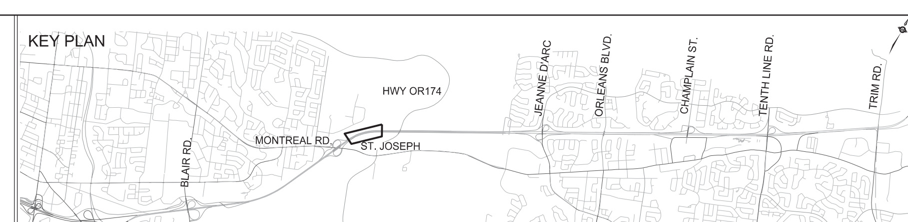

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-07.DGN | Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 302+400 to STA 304+000

Drawings No.: | Revision 00 | Sheet No. 07

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:34:01 AM

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TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:46:35 AM

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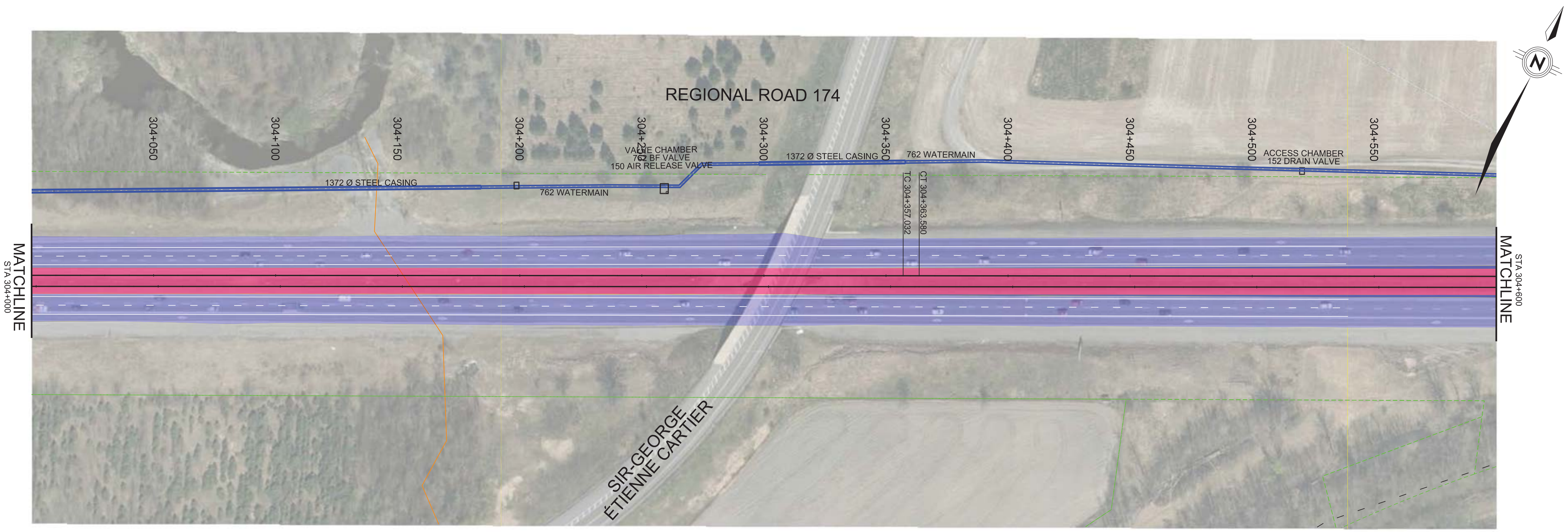

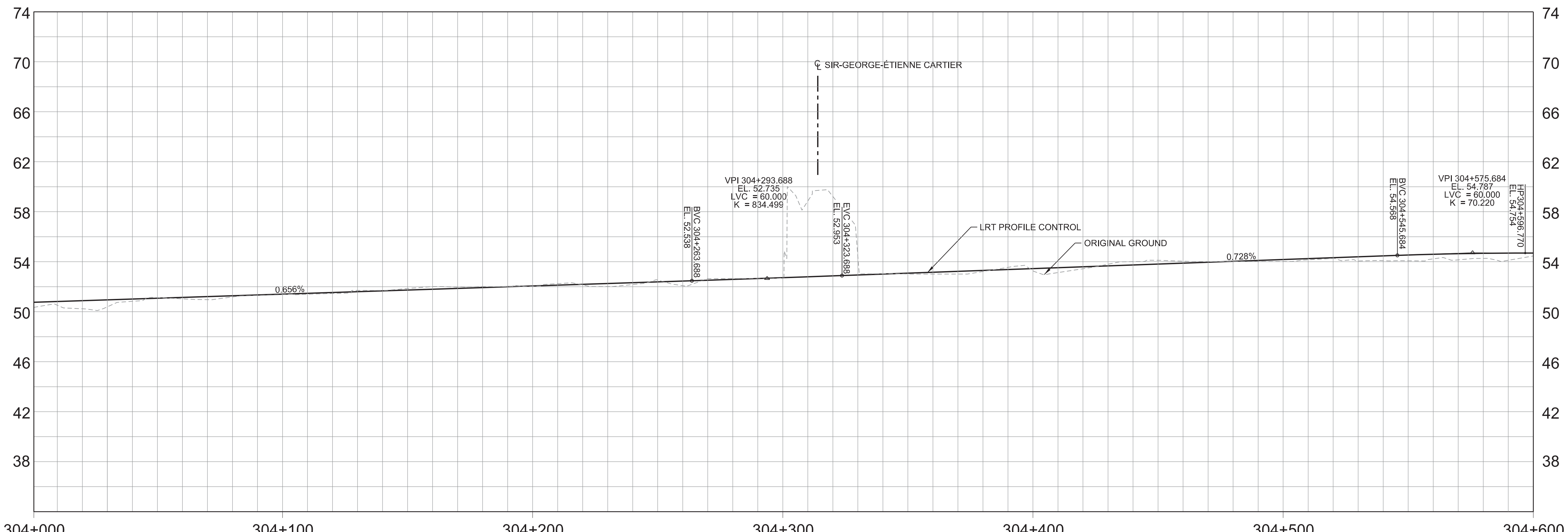

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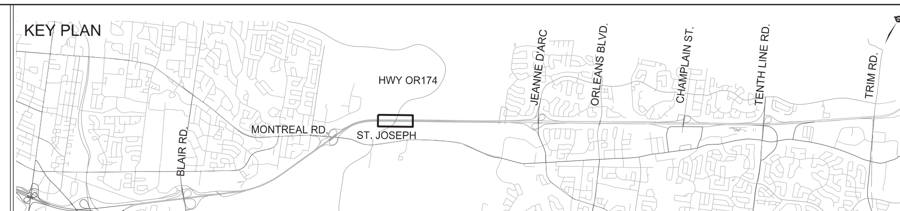

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN

PARSONS

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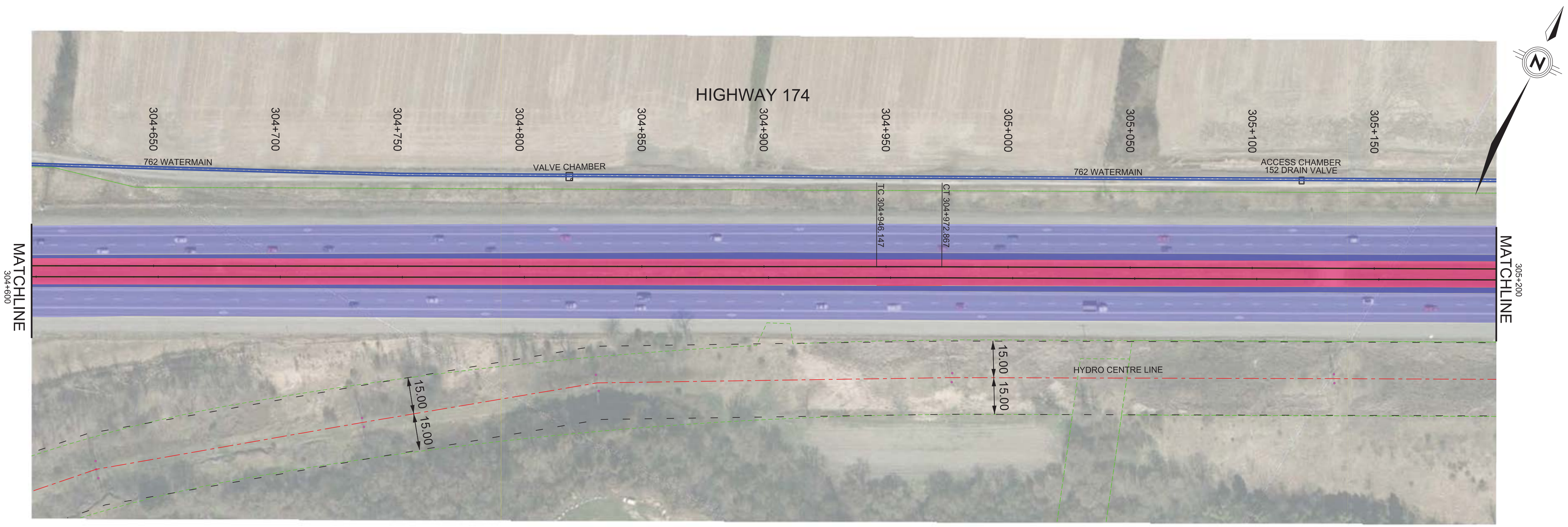
Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-08.DGN		Plot Date: XX/XX/XXXX

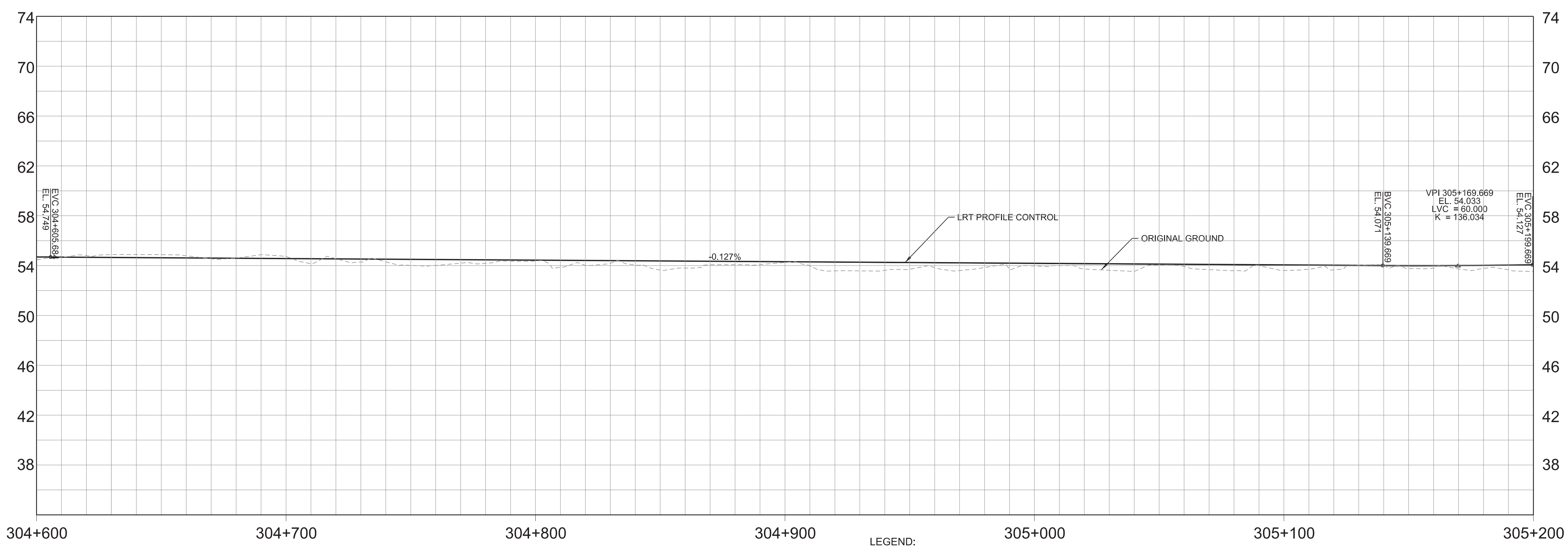
EASTERN LRT
STA. 304+000 to STA 304+600

Drawings No.:

Revision 00

Sheet No. 08





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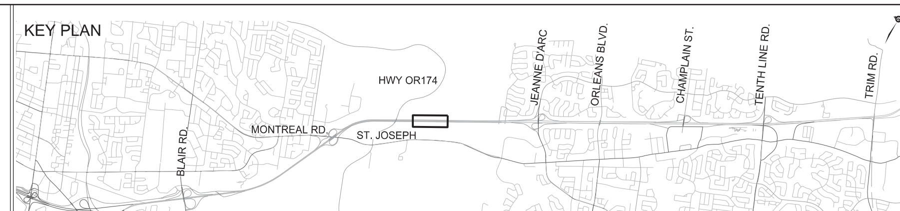

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-09.DGN	Plot Date: XX/XX/XXXX	

Ottawa

EASTERN LRT
STA. 304+600 to STA 305+200

Drawings No.:	Revision	Sheet No.
	00	09

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 8:46:14 AM

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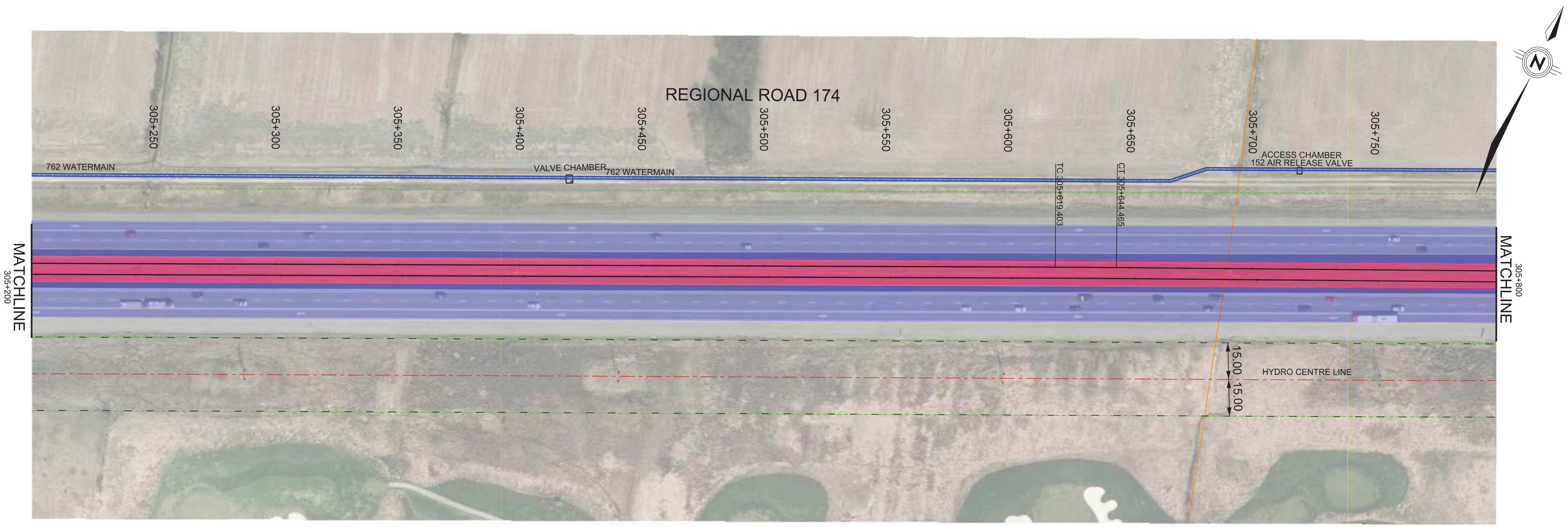

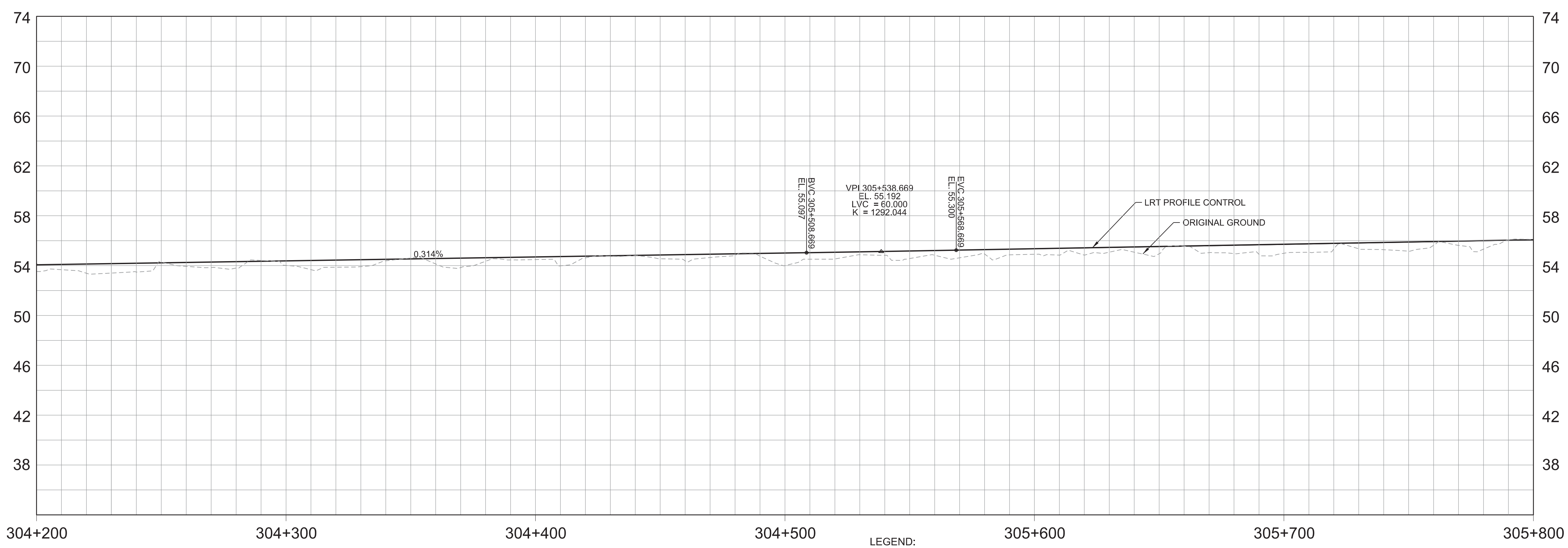

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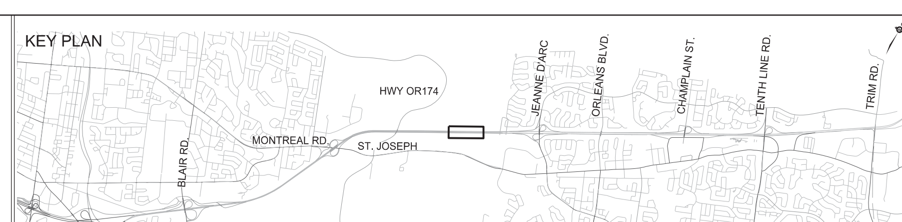

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-10.DGN		Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 305+200 to STA 305+800

Drawings No.:	Revision	Sheet No.
	00	10

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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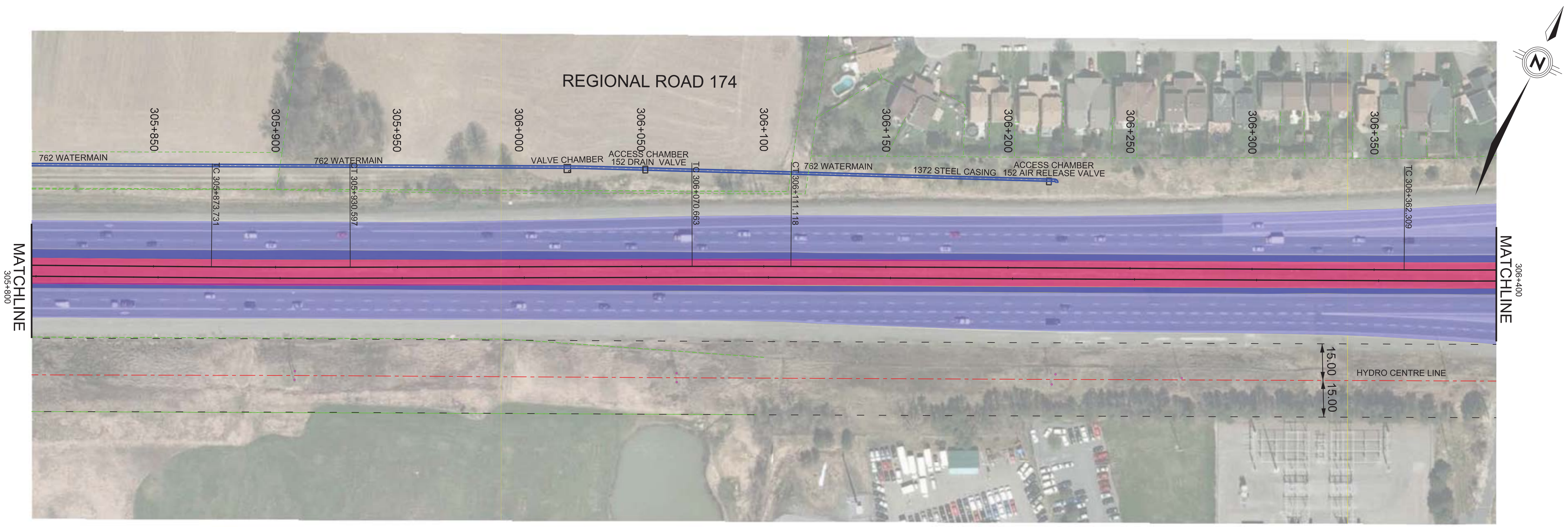

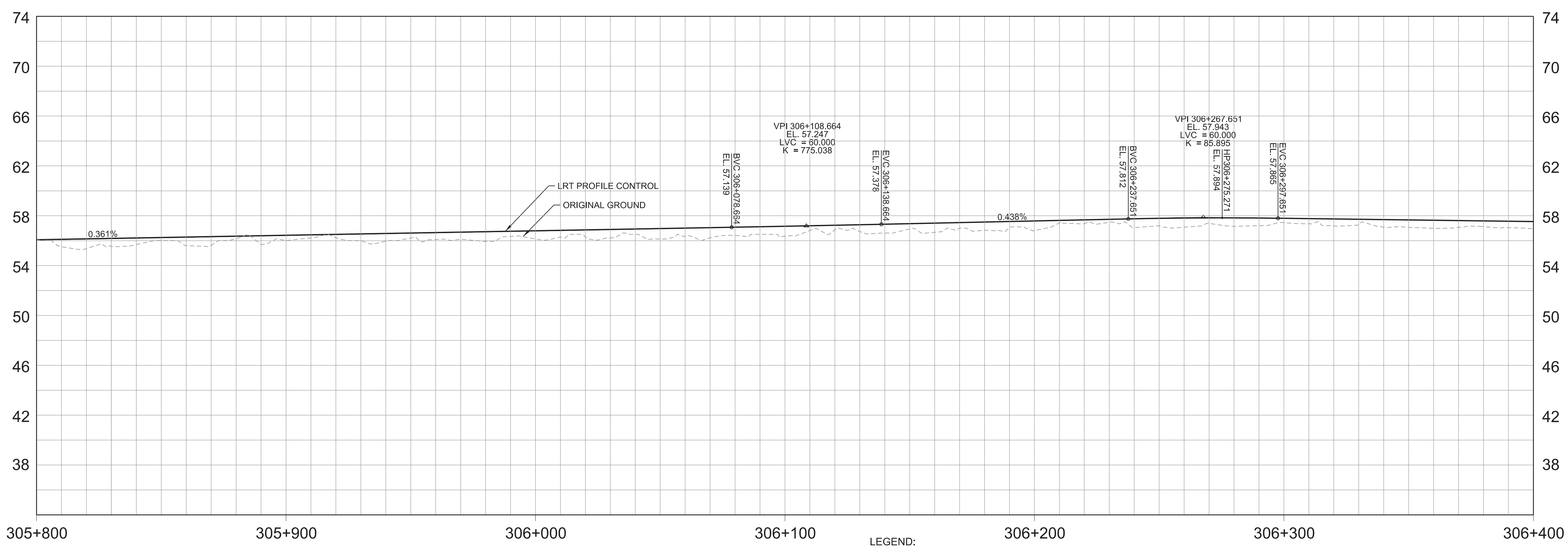

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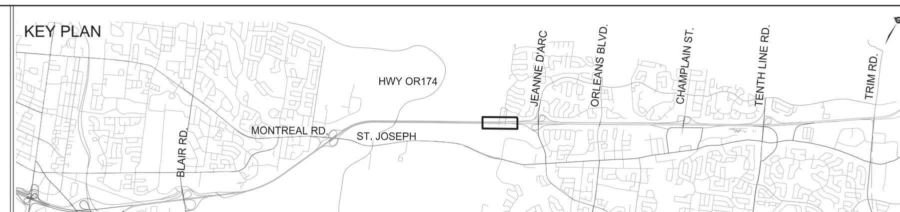


LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-11.DGN

Plot Date: XX/XX/XXXX

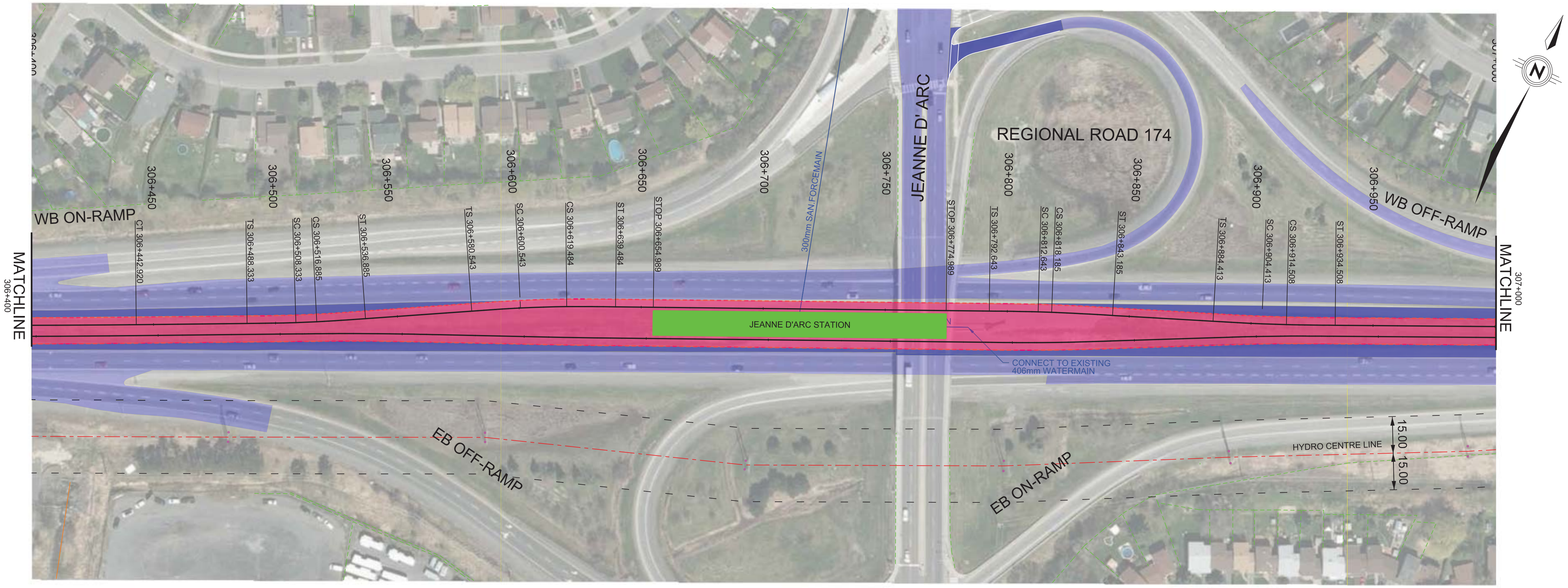
Ottawa

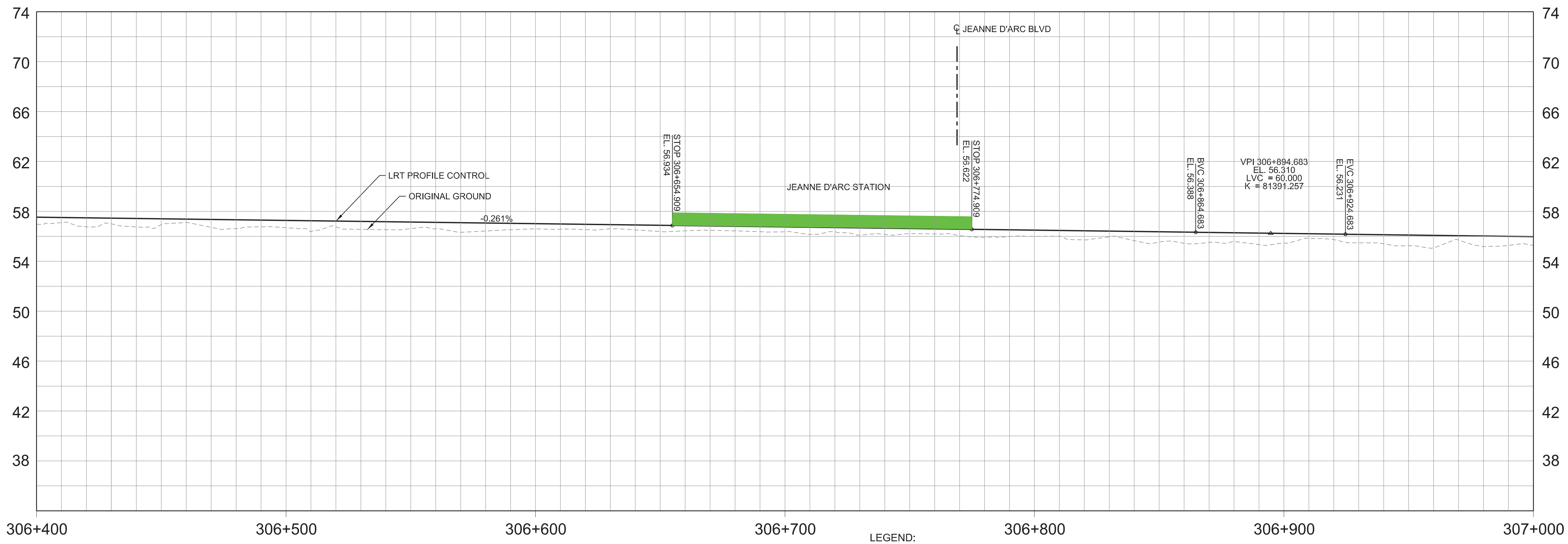
EASTERN LRT
STA. 305+800 to STA 306+400

Drawings No.:

Revision 00

Sheet No. 11





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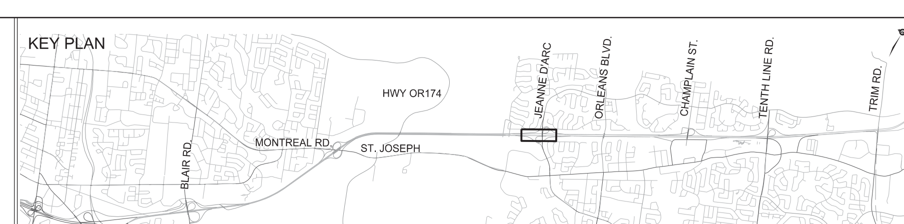

LEGEND:

PROPOSED LRT CORRIDOR	EXISTING ROADWAY
PROPOSED LRT	WATERCOURSES
PROPOSED LRT STATIONS	PROPERTY LIMITS
PROPOSED WIDENING	WATERMAIN
PROPOSED LANE ALIGNMENT	HYDRO CENTRELINE
	HYDRO CORRIDOR
	EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-12.DGN		Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 306+400 to STA 307+000

Drawings No.: | Revision 00 | Sheet No. 12

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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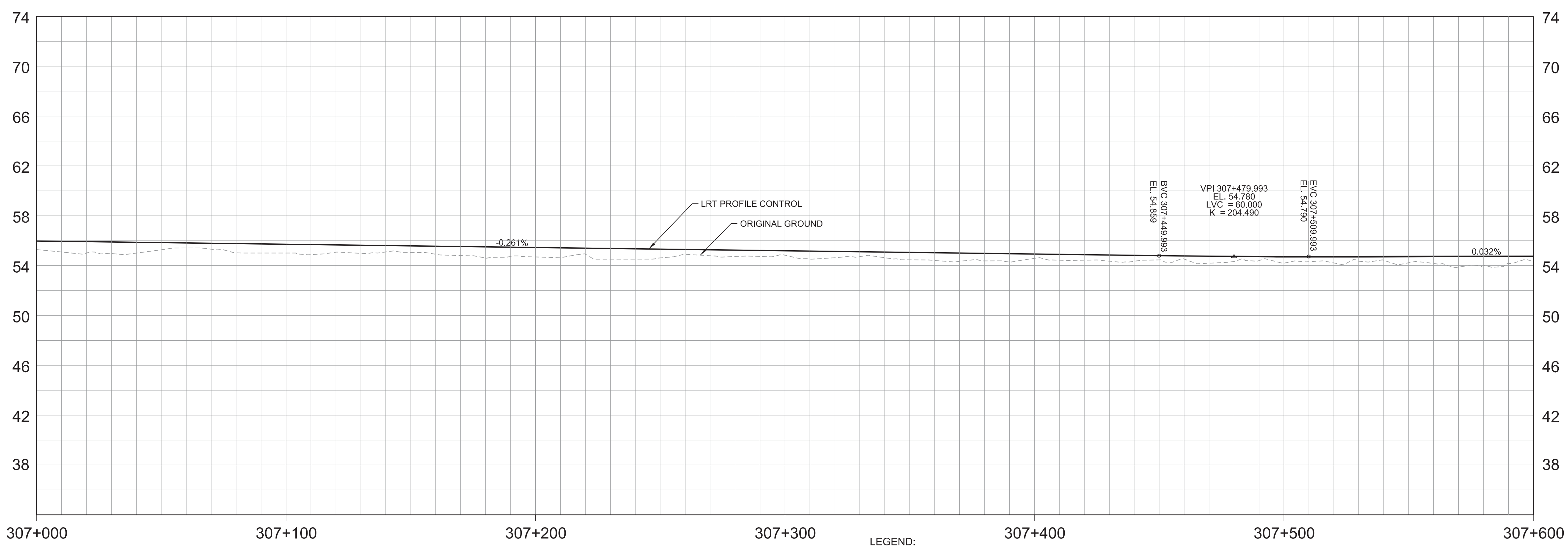

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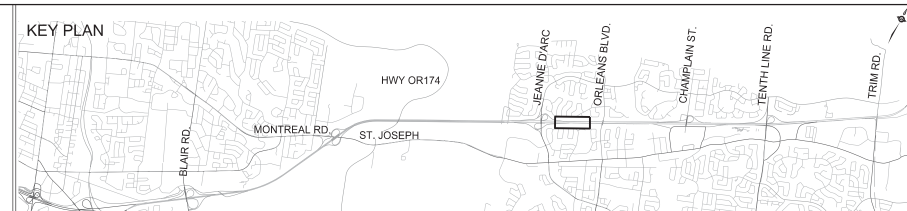

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-13.DGN	Plot Date: XX/XX/XXXX	

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EASTERN LRT
STA. 306+400 to STA 307+600

Drawings No.:	Revision	Sheet No.
	00	13

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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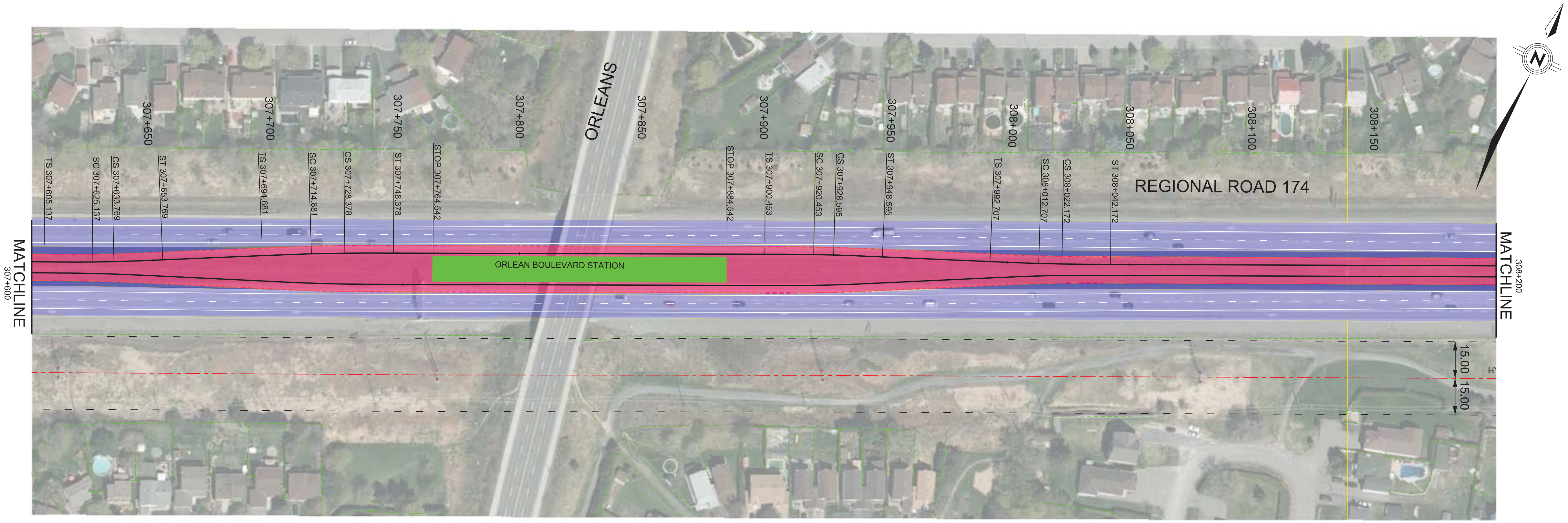

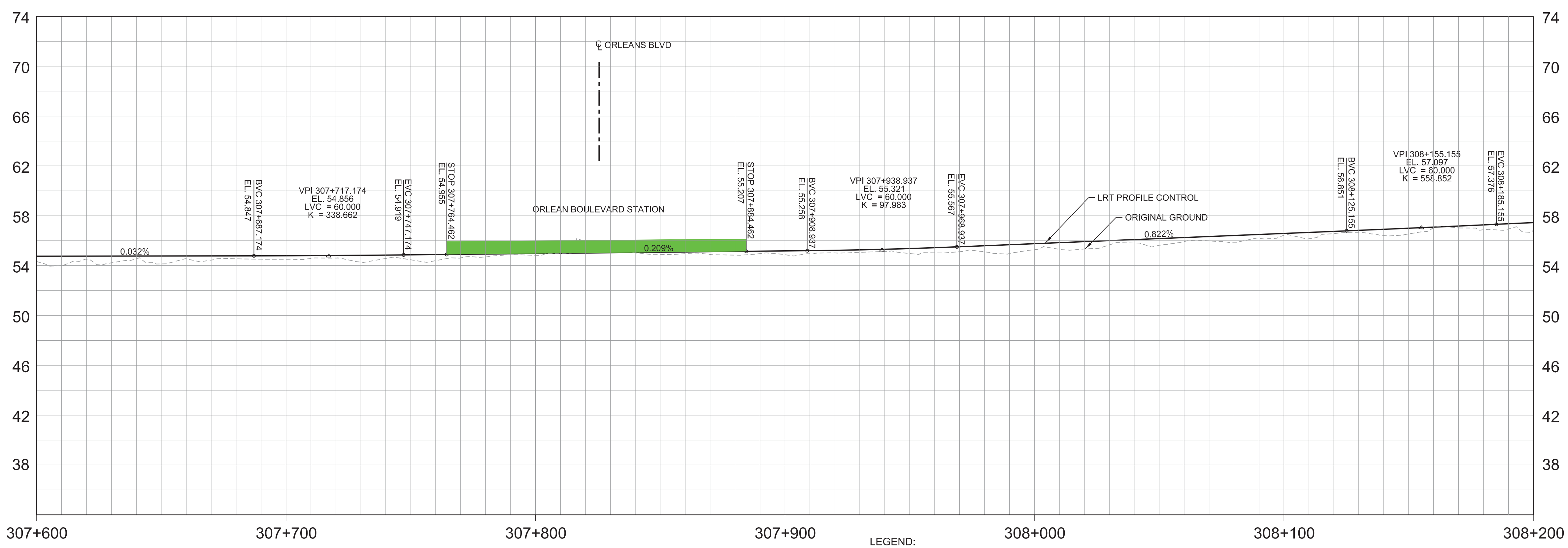

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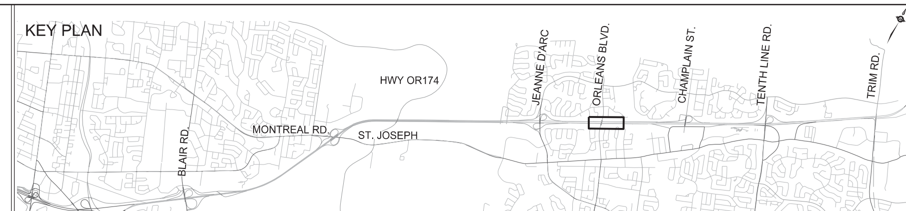

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-14.DGN	Plot Date: XX/XX/XXXX	

EASTERN LRT
STA. 307+600 to STA 308+200

Drawings No.: | Revision 00 | Sheet No. 14

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 9:27:23 AM

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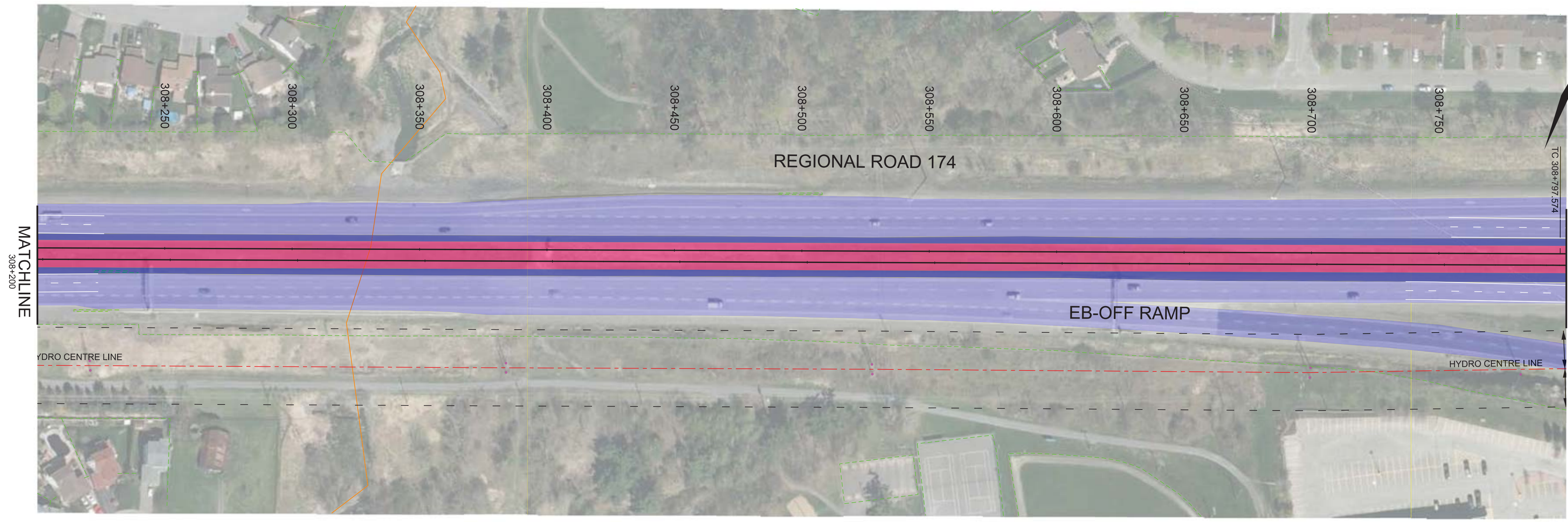

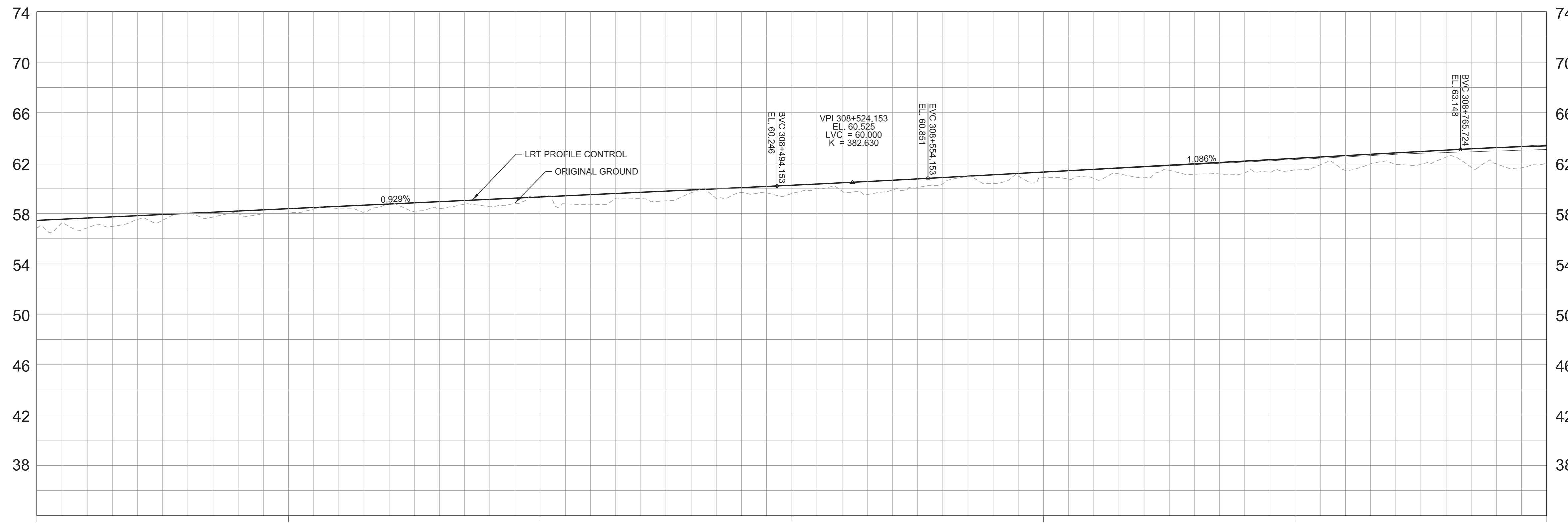

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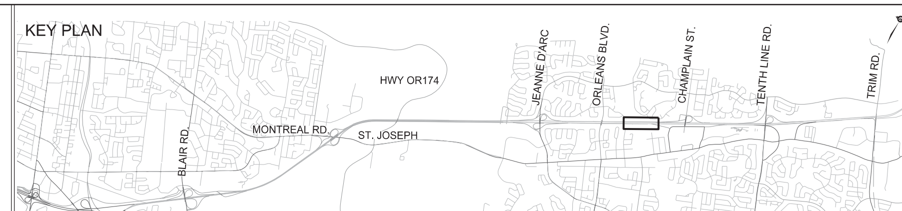

LEGEND:

PROPOSED LRT CORRIDOR	EXISTING ROADWAY
PROPOSED LRT	WATERCOURSES
PROPOSED LRT STATIONS	PROPERTY LIMITS
PROPOSED WIDENING	WATERMAIN
PROPOSED LANE ALIGNMENT	HYDRO CENTRELINE
	HYDRO CORRIDOR
	EXISTING HYDRO POLES

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-15.DGN		Plot Date: XX/XX/XXXX

EASTERN LRT
STA. 308+200 to STA 308+800

Drawings No.: | Revision 00 | Sheet No. 15

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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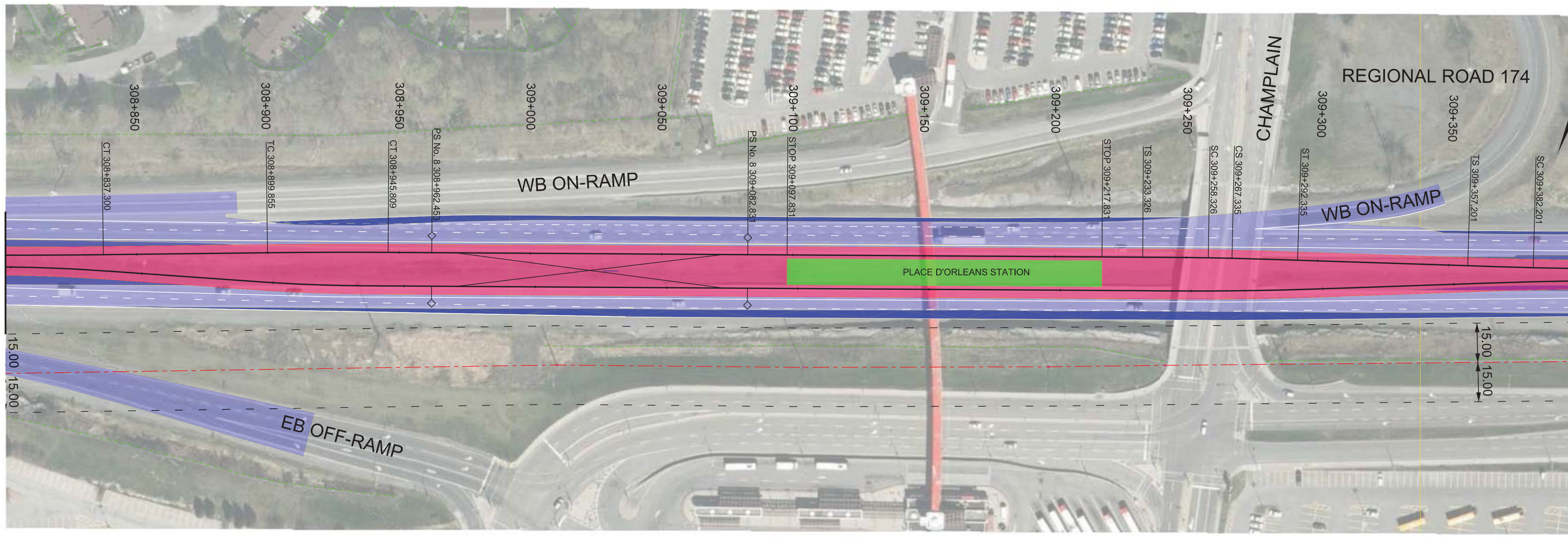

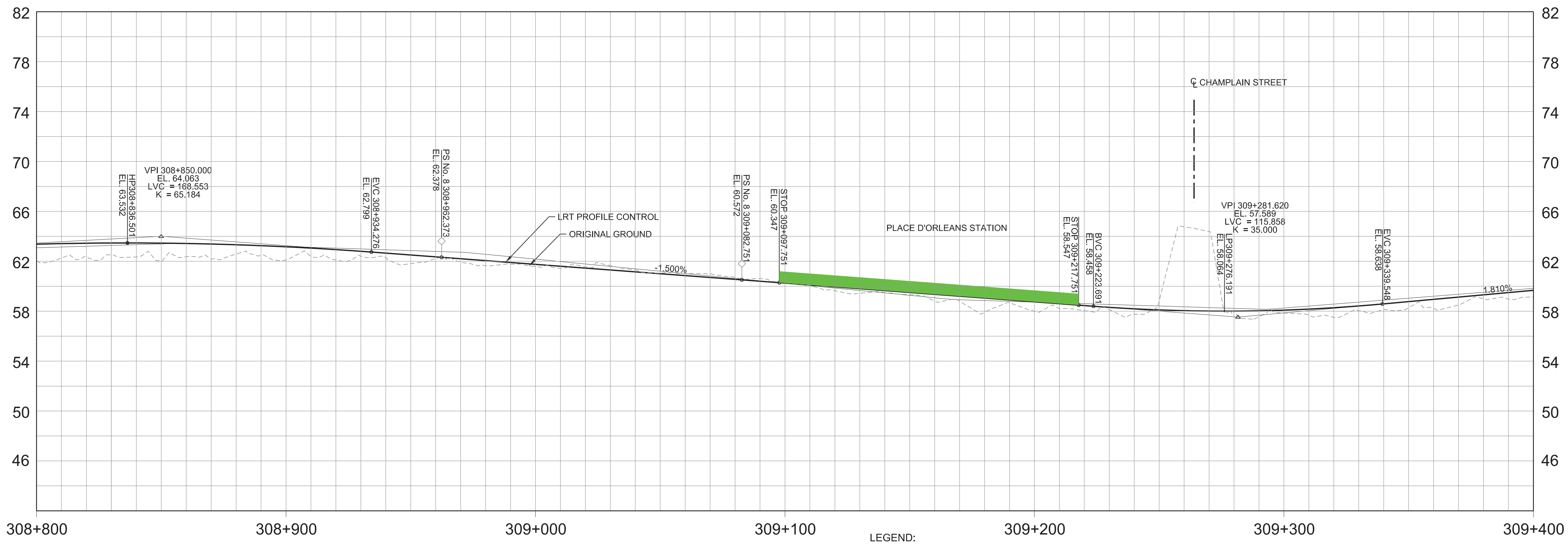

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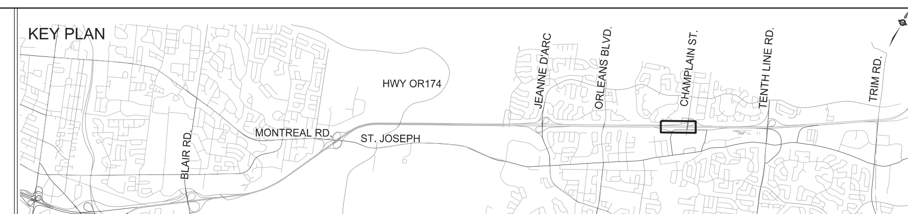
KEY PLAN



LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-16.DGN

Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 308+800 to STA 309+400

Drawings No.:

Revision: 00

Sheet No.: 16

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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Plot Date: $DATE$ $TIME$

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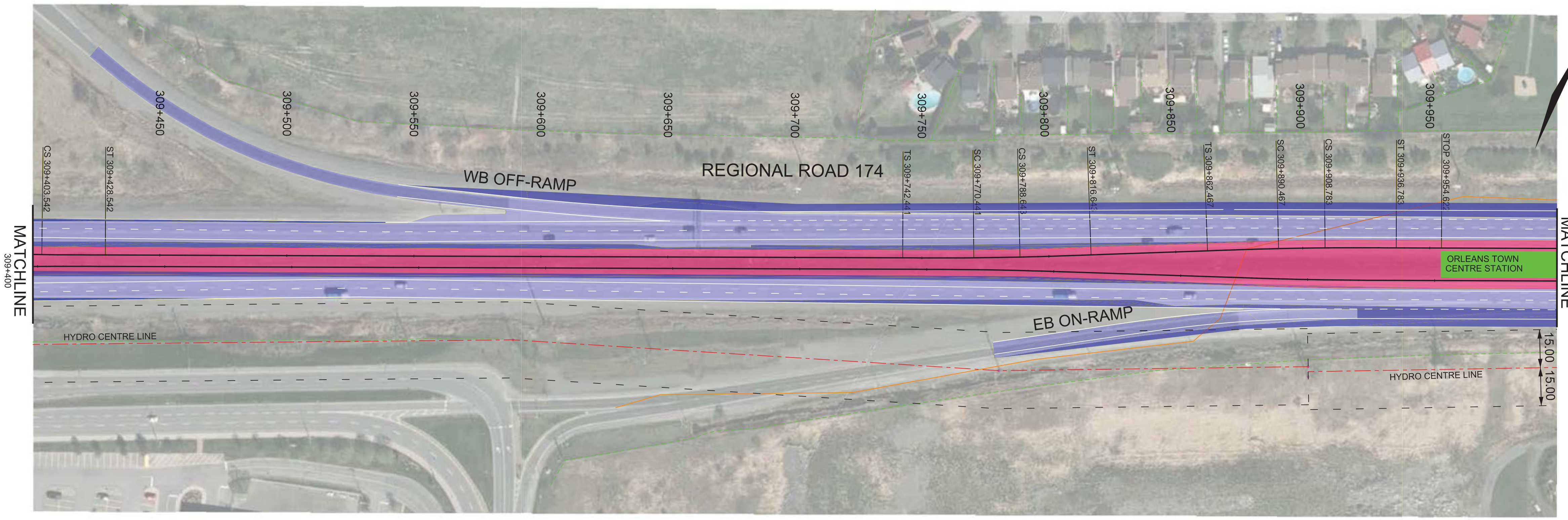

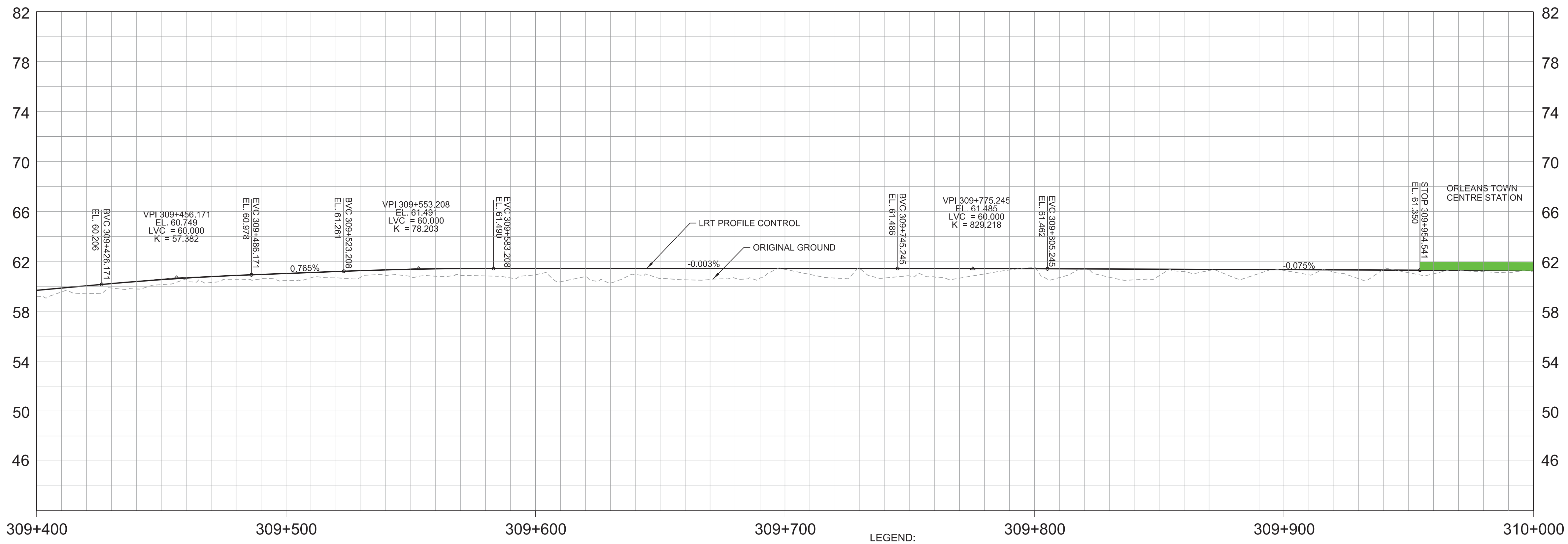

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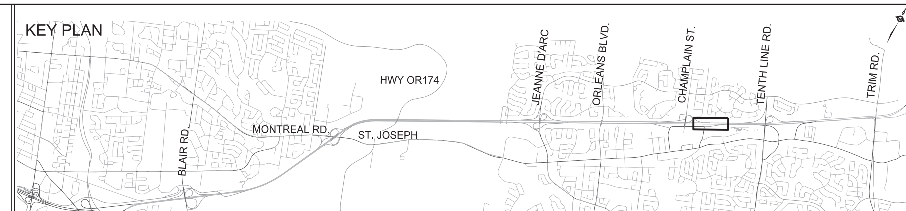

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-17.DGN

Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 309+400 to STA 310+000

Drawings No.:

Revision 00

Sheet No. 17

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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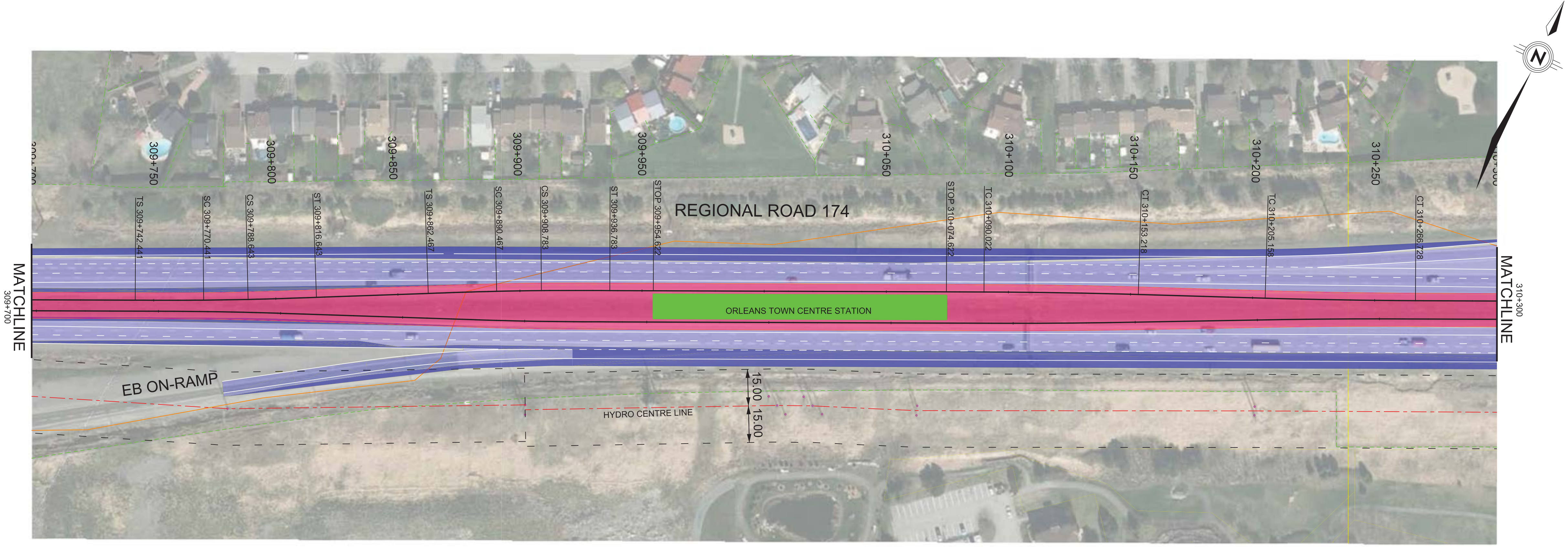

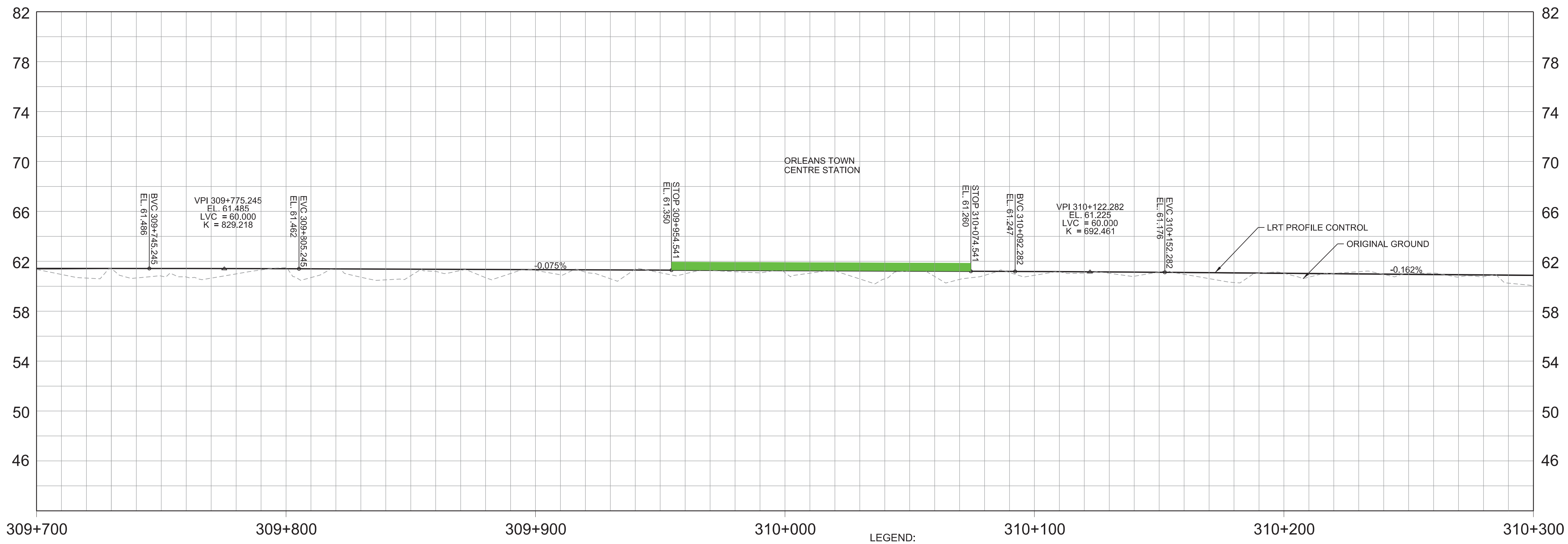

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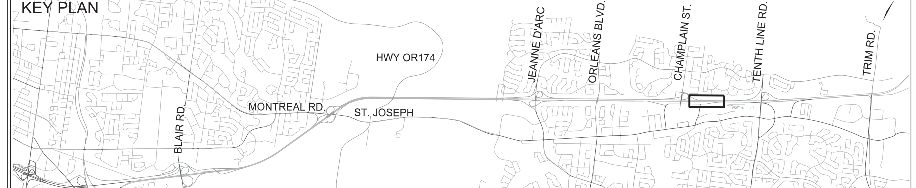

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100 — VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-17A.DGN

Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 309+400 to STA 310+000

Drawings No.:

Revision 00

Sheet No. 17A

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

Last Saved: 16/06/2015 9:46:58 AM

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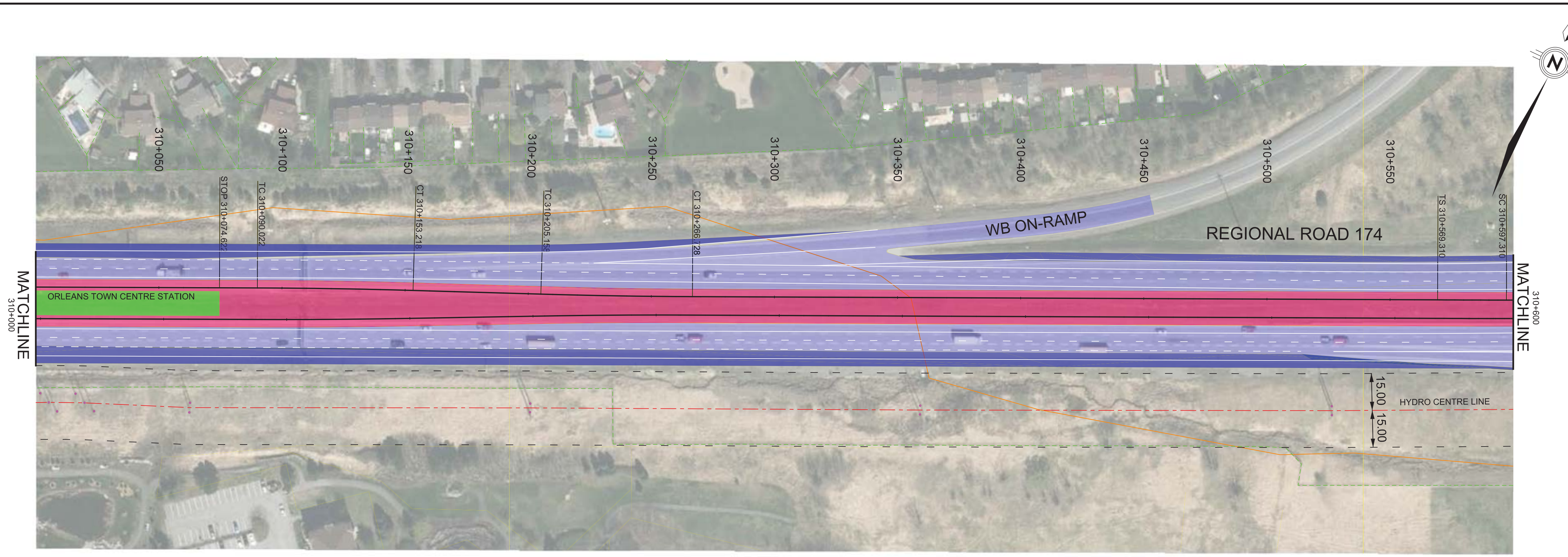

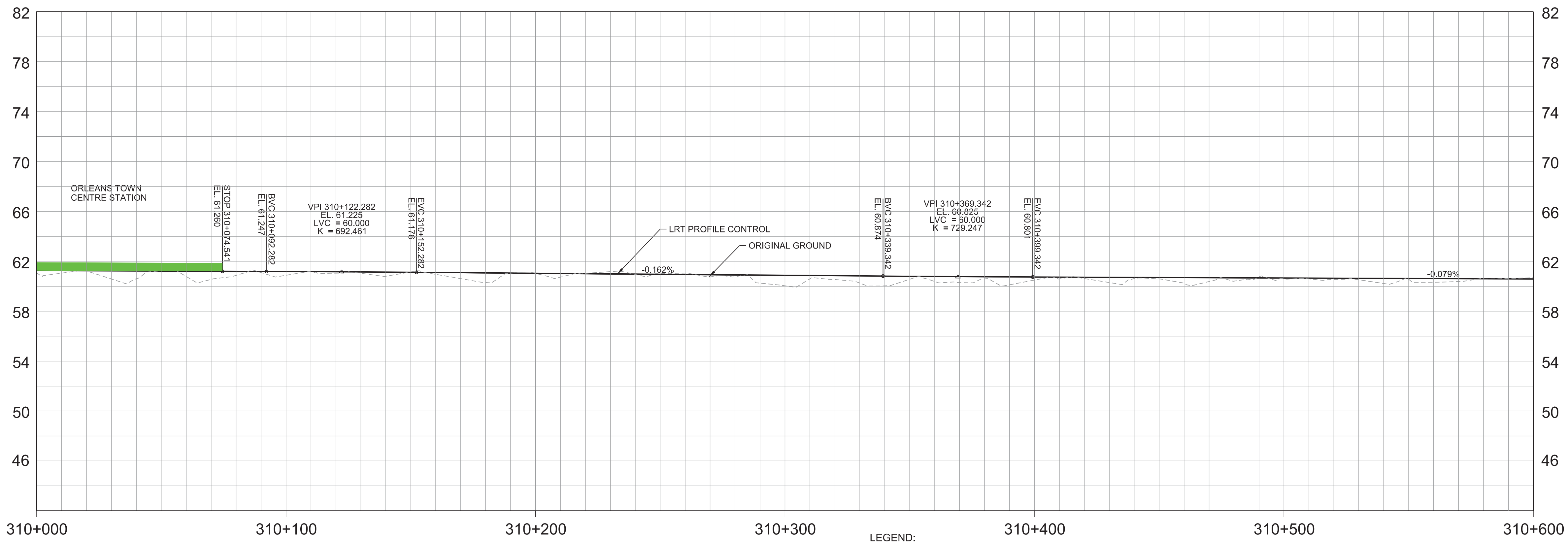

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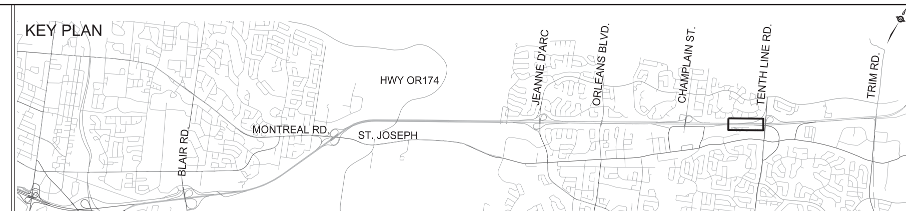


LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-18.DGN

Plot Date: XX/XX/XXXX

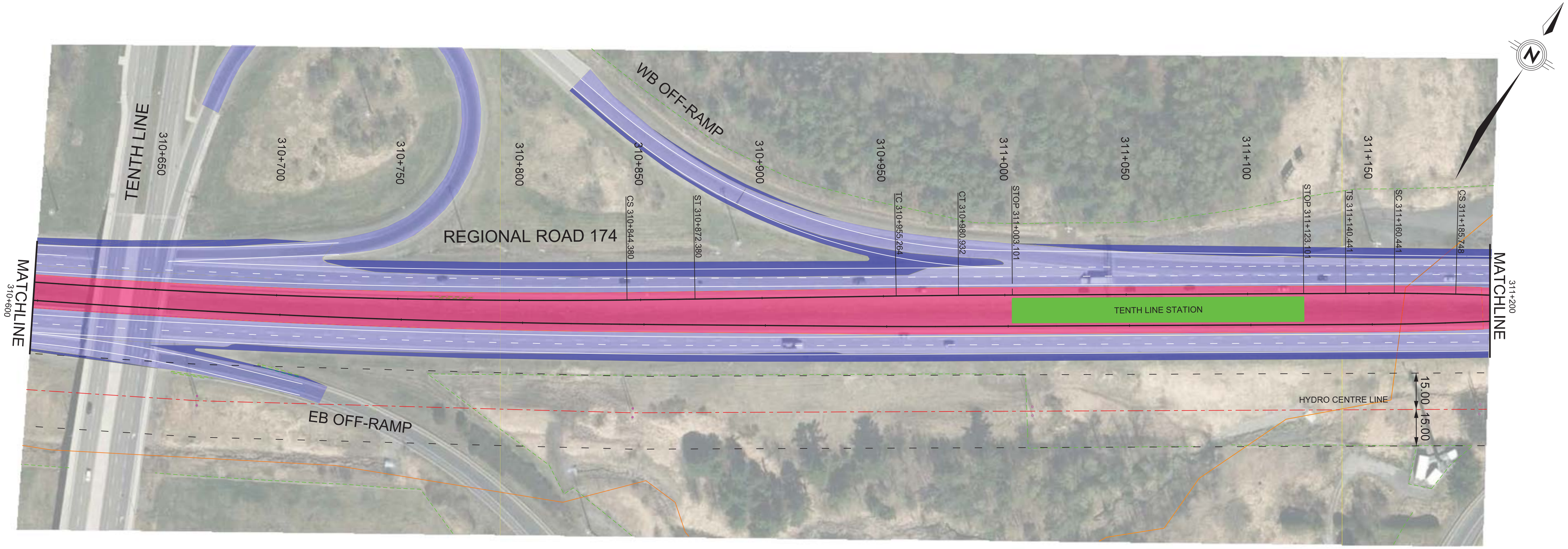
Ottawa

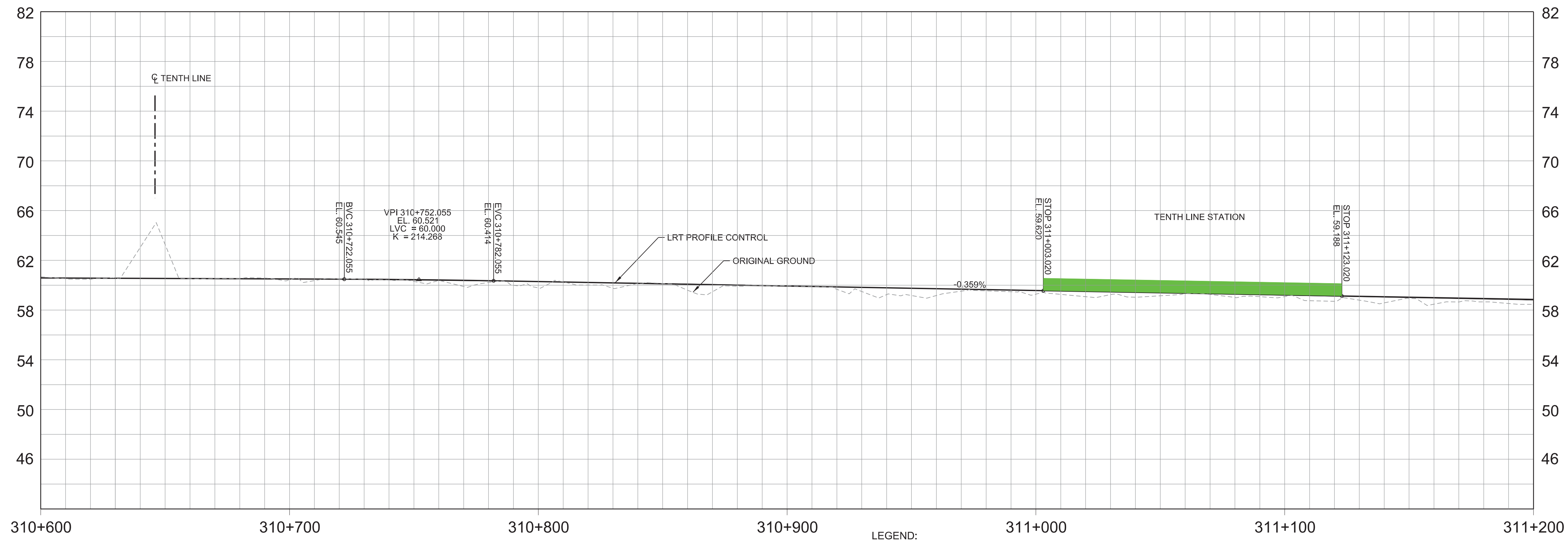
EASTERN LRT
STA. 310+000 to STA 310+600

Drawings No.:

Revision 00

Sheet No. 18





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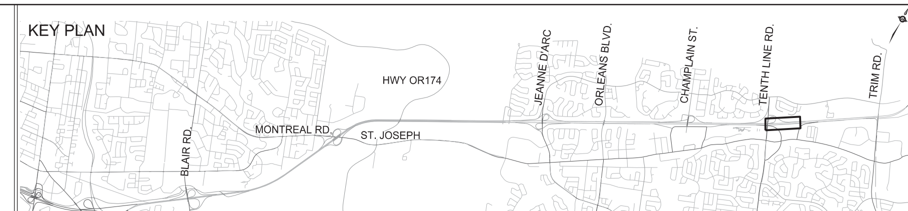

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-19.DGN

Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 310+000 to STA 310+600

Drawings No.:

Revision 00

Sheet No. 19

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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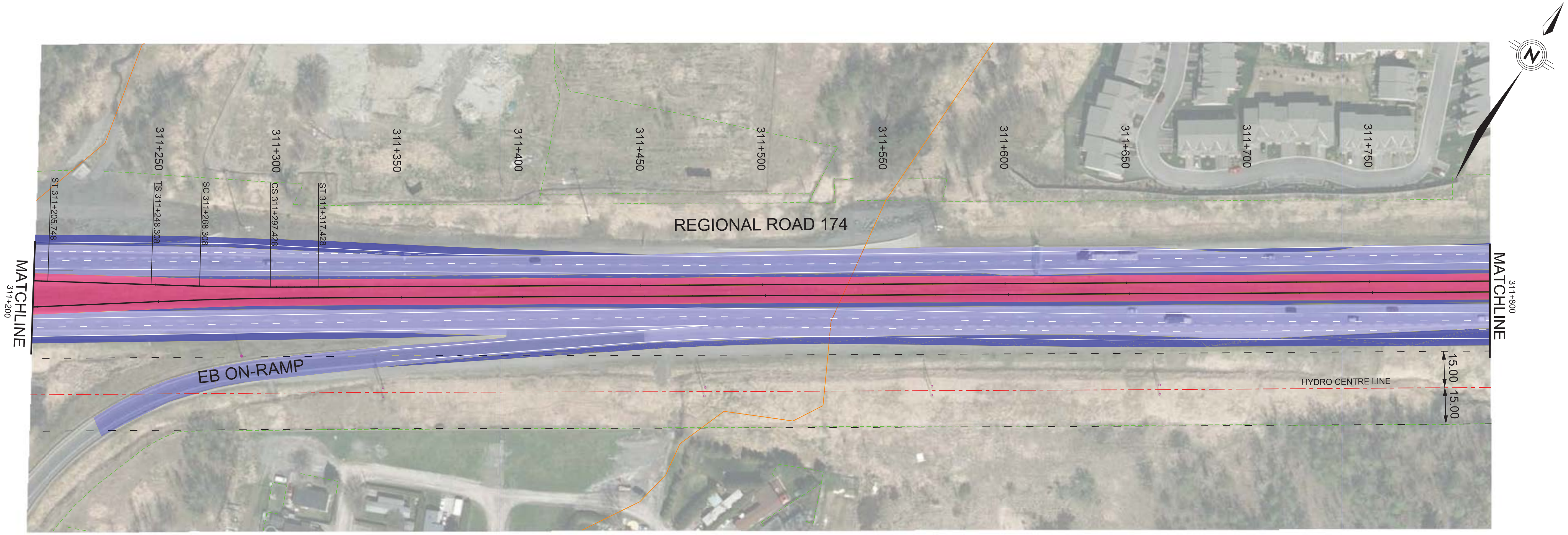

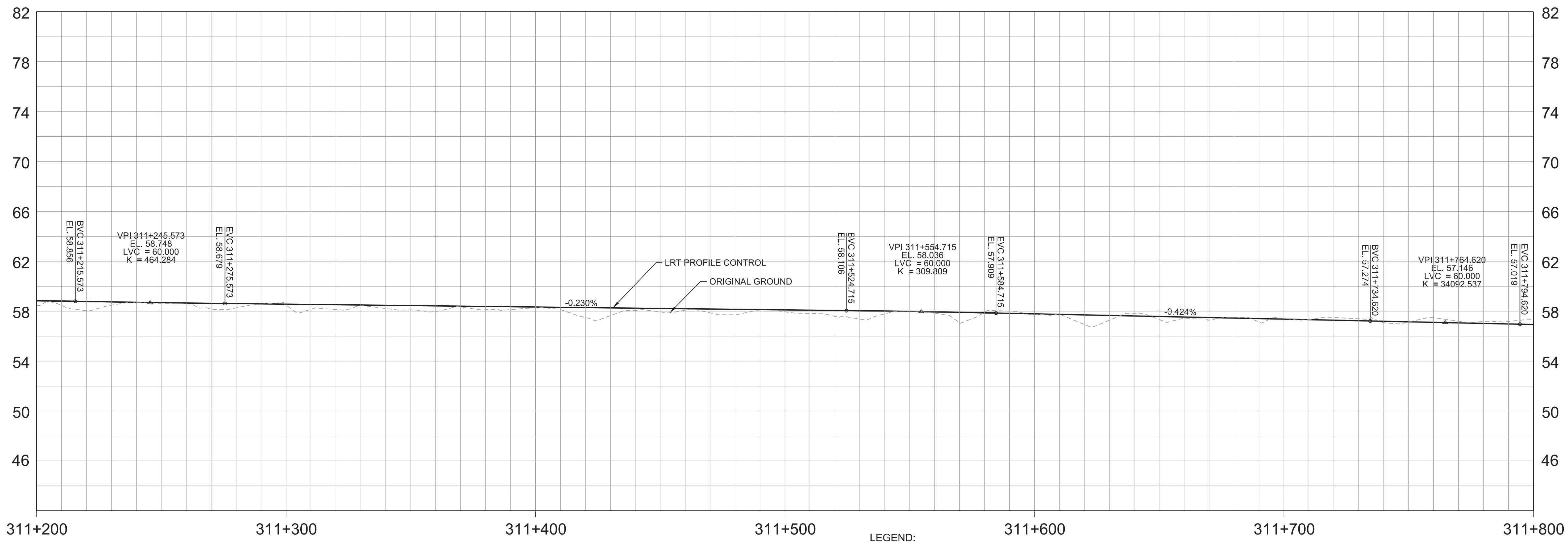

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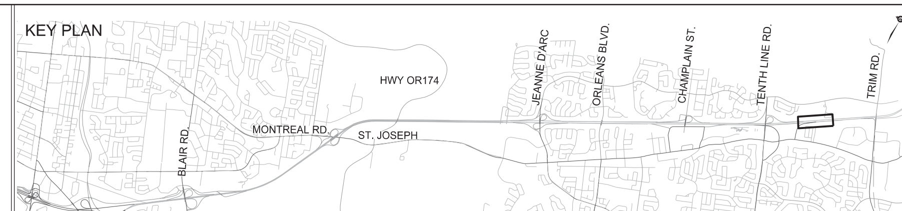

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Ottawa

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-20.DGN		Plot Date: XX/XX/XXXX

EASTERN LRT
STA. 311+200 to STA 311+800

Drawings No.: | Revision 00 | Sheet No. 20

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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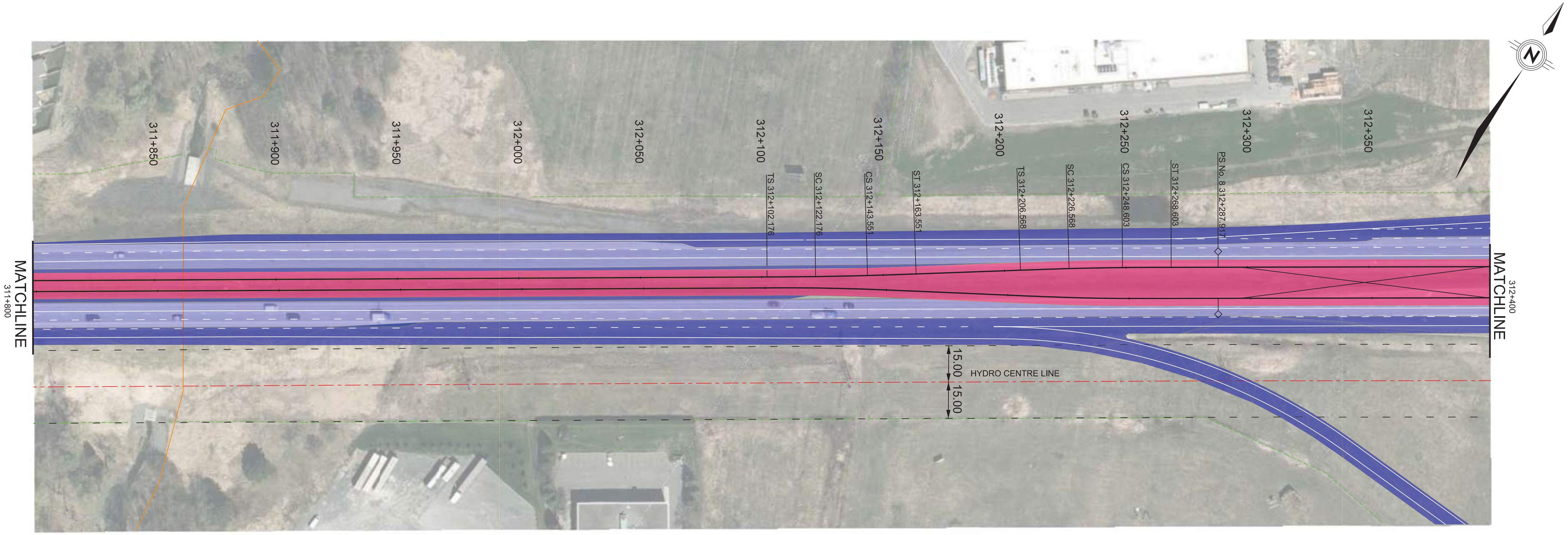

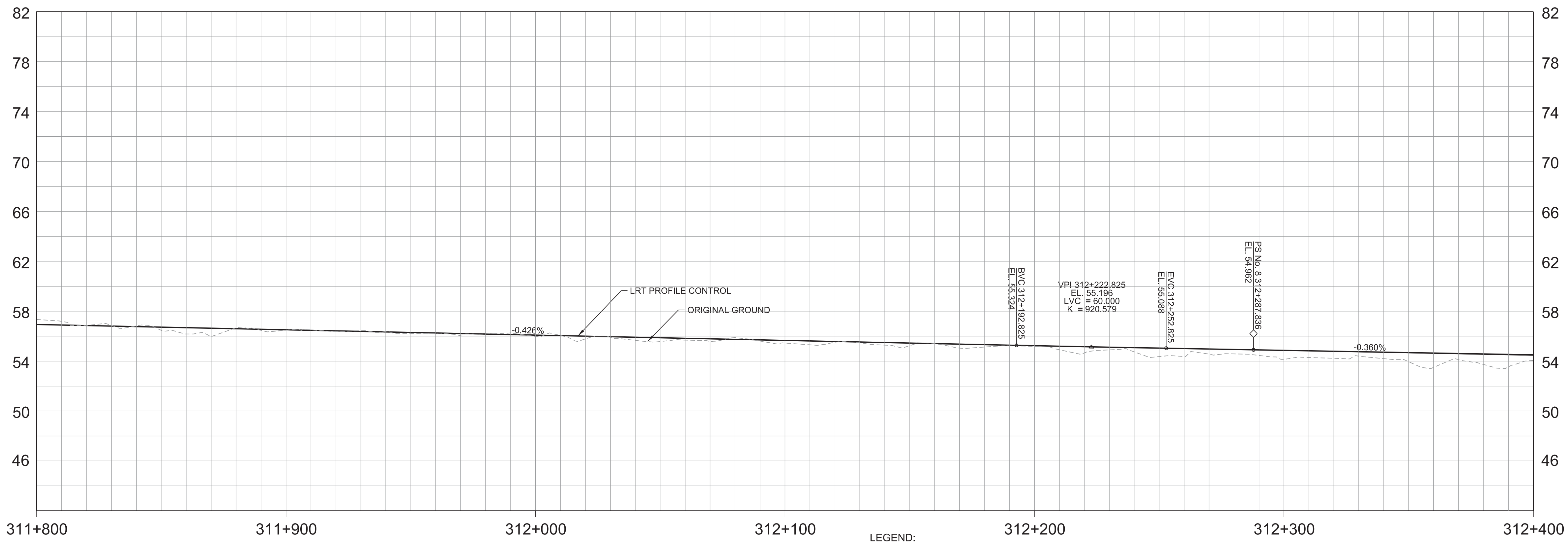

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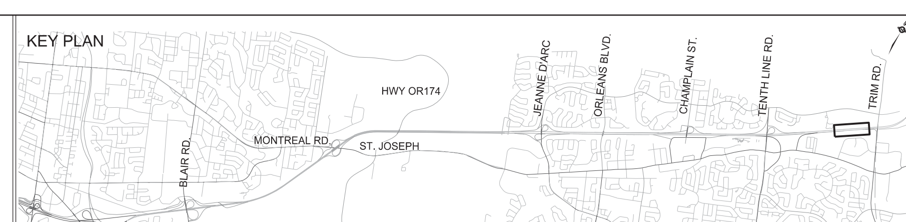

LEGEND:

- PROPOSED LRT CORRIDOR
- PROPOSED LRT
- PROPOSED LRT STATIONS
- PROPOSED WIDENING
- PROPOSED LANE ALIGNMENT
- EXISTING ROADWAY
- WATERCOURSES
- PROPERTY LIMITS
- WATERMAIN
- HYDRO CENTRELINE
- HYDRO CORRIDOR
- EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:

Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20

CAD File Name: TO3175TOC- 01-ELRT-21.DGN
Plot Date: XX/XX/XXXX

Ottawa

EASTERN LRT
STA. 311+800 to STA 312+400

Drawings No.:
Revision 00
Sheet No. 21

TITLE FRAME: 790mm x 534mm City of Ottawa 2008

Plot Date: $DATE$ $TIME$

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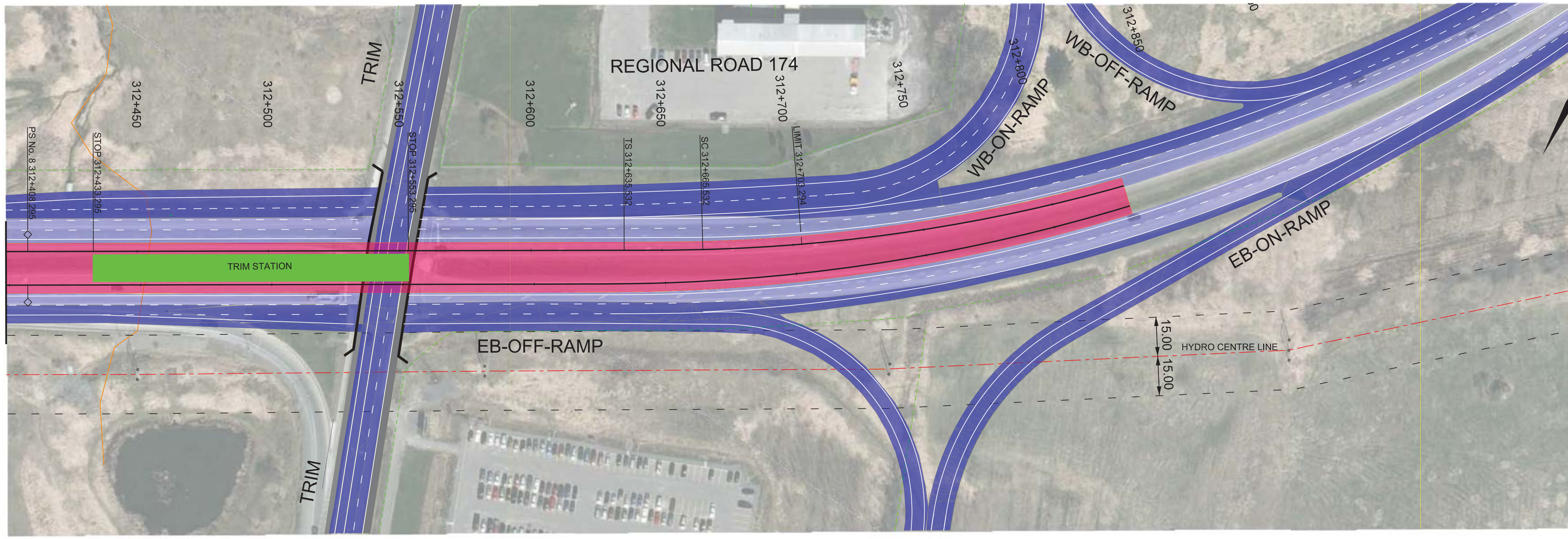

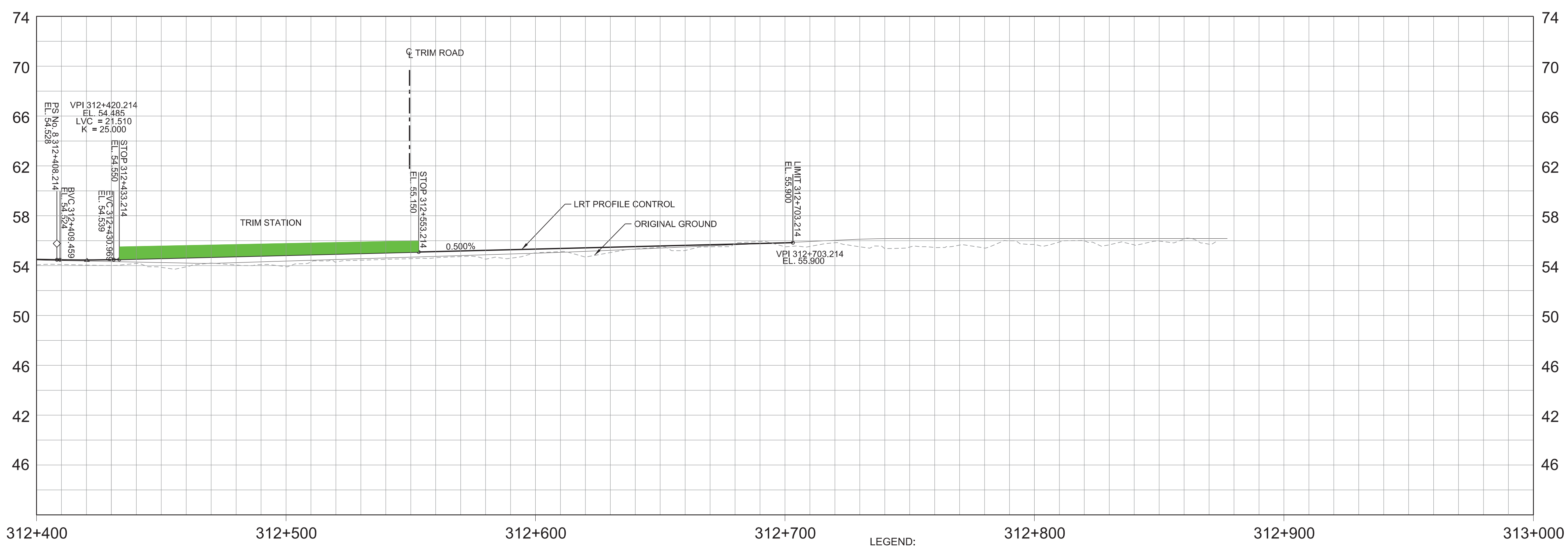

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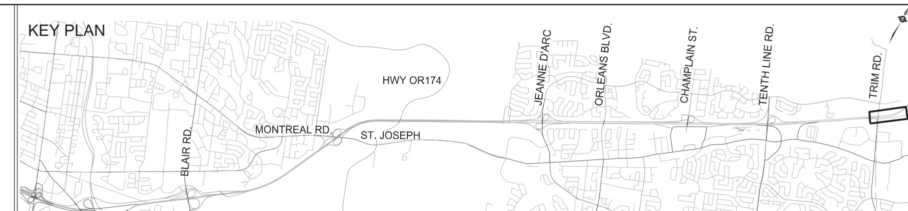


LEGEND:

PROPOSED LRT CORRIDOR	EXISTING ROADWAY
PROPOSED LRT	WATERCOURSES
PROPOSED LRT STATIONS	PROPERTY LIMITS
PROPOSED WIDENING	WATERMAIN
PROPOSED LANE ALIGNMENT	HYDRO CENTRELINE
	HYDRO CORRIDOR
	EXISTING HYDRO POLES

PARSONS

Date: JULY 2015	Designed By:	Drawn By:
Project Manager:	Dicipline Engineer:	Checked By:
Scale: HORIZONTAL 0m 10 20 50 100; VERTICAL 0m 2.5 5 10 20		
CAD File Name: TO3175TOC- 01-ELRT-22.DGN	Plot Date: XX/XX/XXXX	

Ottawa

EASTERN LRT
STA. 312+400 to STA 313+000

Drawings No.:	Revision	Sheet No.
	00	22

APPENDIX B

Blair Road Interchange
Relevant Borehole Logs by Others

Houle Chevrier Engineering

Report to: AECOM Canada Ltd.
Project: 14-275 (September 10, 2015)

PROJECT 871-2120

RECORD OF BOREHOLE 23

SHEET 1 of 1

LOCATION: See Figure 2
BORING DATE: April 24, 1987
DATUM: GEODETIC
SAMPLER HAMMER, 63.6kg, DROP, 760mm
PENETRATION TEST HAMMER, 63.6kg, DROP, 760mm

STA. 7+118, 4.5m Lt of CL

Boring method: Power Auger, 200mm Diam. (Hollow Stem)

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3M	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	77.51 / 0.00					Bentonite Seal
	TOPSOIL	77.08 / 0.43					Backfill
1	Very loose brown sandy silt and clayey silt, some gravel, and clay, trace organic matter (FILL)	76.32 / 1.19	1	50 DO	2	/MH Org = 2.6 %	
2	Compact to dense brown to grey brown sandy silt, some gravel, trace clay, occasional boulder (GLACIAL TILL)	74.34	2	50 DO	33		
			3	50 DO	16		
3			4	50 DO	12/150 mm 100/150 mm		
	Weathered SHALE BEDROCK	3.20					
	End of Hole Auger Refusal						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH Cu, kPa: nat.V.- + Q.- ● rem.V.- ⊕ U.- O
HYDRAULIC CONDUCTIVITY, k, CM/SEC
WATER CONTENT, PERCENT: Wp — W — Wl; 20 40 60 80

W.L in Standpipe at Elev.76.78 May.5,1987

16-◆-5 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25

Golder Associates

LOGGED S.L.
CHECKED AC

RECORD OF BOREHOLE 27

SHEET 1 of 1

LOCATION See Figure 2 | BORING DATE April 24, 1987 | DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

BORING METHOD: Power Auger, 200mm Diam. (Hollow Stem)

STA. 7+281, on CL

DESCRIPTION	ELEV. / DEPTH (M)
Ground Surface	75.48 / 0.00
TOPSOIL	75.33 / 0.15
Compact to very dense brown to grey sandy silt, some gravel and shale fragments, trace clay (GLACIAL TILL)	72.74 / 2.74
Moderately weathered SHALE BEDROCK	72.58 / 2.90
End of Hole Auger Refusal	

SAMPLE NUMBER	TYPE	BLOWS/0.3M
1	50 DO	12
2	50 DO	33
3	50 DO	59

PIEZOMETER OR STANDPIPE INSTALLATION: Bentonite Seal; Backfill

W.L in Standpipe at Elev.74.29 May.5,1987

DEPTH SCALE 1 : 25

Golder Associates

LOGGED S.L

CHECKED AC

RECORD OF BOREHOLE 28

SHEET 1 of 1

LOCATION See Figure 2 | BORING DATE April 27, 1987 | DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

BORING METHOD: Power Auger, 200mm Diam. (Hollow Stem)

STA. 14+000 on CL

DESCRIPTION	ELEV. / DEPTH (M)
Ground Surface	75.74 / 0.00
TOPSOIL	75.59 / 0.15
Brown sandy silt, some gravel and clay (FILL)	74.12 / 1.62
Dark brown TOPSOIL	73.76 / 1.98
Compact to very dense brown to grey sandy silt, some gravel and shale fragments, trace clay (GLACIAL TILL)	71.84 / 3.90
End of Hole Auger Refusal	

SAMPLE NUMBER	TYPE	BLOWS/0.3M	WATER CONTENT / ADDITIONAL LAB. TESTING
1	50 DO	7	114 %; Org = 67.3 %
2	50 DO	22	
3	50 DO	79	

W.L in Open Hole at Elev.73.91 May.5,1987

DEPTH SCALE 1 : 25

Golder Associates

LOGGED S.L

CHECKED AC

RECORD OF BOREHOLE 29

SHEET 1 of 1

LOCATION See Figure 2 — BORING DATE April 27, 1987 — DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm — PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

BORING METHOD: Power Auger 200mm Diam. (Hollow Stem)

STA. 8+300, on CL

DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3M
Ground Surface	74.67			
TOPSOIL	0.00			
Brown sandy silt, some gravel and clay (FILL)	0.09 / 74.15			
TOPSOIL	0.52 / 73.94			
Loose to very dense brown to grey sandy silt, some gravel and shale fragments, trace clay (GLACIAL TILL)	0.73 / 71.32	1	50 DO	9
		2	50 DO	55
		3	50 DO	25
		4	50 DO	25
Weathered SHALE BEDROCK				
End of Hole Auger Refusal	3.41			

PIEZOMETER OR STANDPIPE INSTALLATION: W.L in Open Hole at Elev.73.91 May.5,1987

DEPTH SCALE 1 : 25 — Golder Associates — LOGGED S.L — CHECKED AC

RECORD OF BOREHOLE 32

SHEET 1 of 1

LOCATION See Figure 2 — BORING DATE April 30, 1987 — DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm — PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

BORING METHOD: Power Auger 200mm Diam. (Hollow Stem)

STA. 13+628, on CL

DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3M
Ground Surface	76.33			
TOPSOIL	76.2[illegible]			
Very loose brown sandy silt, some gravel and clay (FILL)	0.09 / 75.26	1	50 DO	4
Compact to dense brown sandy silt, some gravel, trace clay occasional silty sand pocket (GLACIAL TILL)	1.07 / 74.10	2	50 DO	3[illegible]
Weathered SHALE BEDROCK	2.23			
End of Hole Auger Refusal	2.28			

PIEZOMETER OR STANDPIPE INSTALLATION: W.L in Open Hole at Elev.75.80 May.5,1987

DEPTH SCALE 1 : 25 — Golder Associates — LOGGED S.L — CHECKED AC

RECORD OF BOREHOLE 34

SHEET 1 of 2

LOCATION See Figure 2 — BORING DATE July.28.1987 — DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm — PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 871-2120

STA. 5 + 009, 14.5M LT OF CL

Boring method: Power Auger 200mm Diam (Hollow Stem)

Depth scale (metres)	Description	Elev. / Depth (m)	Sample number	Type	Blows/0.3m	Piezometer or standpipe installation
0	Ground Surface	79.57				
	TOPSOIL	0.00				
	Loose brown silty sand, some gravel, trace clay, occasional cobble (FILL)	0.09				Backfill
1			1	50 DO	4	Bentonite Seal
	Dark brown silty clay, some organic matter (FILL)	78.35 / 1.22				
	Compact to very dense brown and grey brown silty sand, trace clay, some gravel, cobbles and boulders (FILL)	78.02 / 1.55	2	50 DO	10	Backfill
2–3			3	50 DO	55	
			4	50 DO	39	
4	Dense to very dense brown to grey sandy silt some gravel, trace clay, numerous shale fragments, occasional cobbles and boulders (GLACIAL TILL)	76.03 / 3.54	5	50 DO	57	Bentonite Seal
			6	50 DO	64	Backfill
5	Hole Continued	74.57 / 5.00				

DEPTH SCALE 1 : 25

Golder Associates

LOGGED S.Leighton

CHECKED AC

RECORD OF BOREHOLE 34

SHEET 2 of 2

LOCATION See Figure 2 — BORING DATE July 28, 1987 — DATUM Geodetic

SAMPLER HAMMER, 63.5kg, DROP, 760mm — PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 871-2120

Boring method: Power Auger 200 mm Diam (Hollow Stem) to 5.82 m; Rotary Drilling BXL Core below

Depth scale (metres)	Description	Elev. / Depth (m)	Sample number	Type	Blows/0.3m	Core Recovery (%)	R.Q.D. (%)	Piezometer or standpipe installation
5	Hole Continued	74.57 / 5.00						Backfill
	Dense to very dense brown to grey sandy silt some gravel, trace clay, numerous shale fragments, occasional cobbles and boulders (GLACIAL TILL)		6	50 DO	64			
			7	50 DO	>100			
	Highly weathered SHALE BEDROCK	74.11 / 5.46						
6	Fresh to faintly weathered laminated dark grey SHALE BEDROCK, occasional limey shale seam, some zones of fractured and moderately weathered rock composed of slivers of shale. Bedding near horizontal (BILLINGS FORMATION)	73.75 / 5.82	8	BXL RC	-	95	0	
			9	BXL RC	-	100	0	
7–8			10	BXL RC	-	100	0	
9			11	BXL RC	-	100	0	
10	End of Hole	69.69 / 9.88						W.L in Standpipe at Elev. 77.59 Aug.18,1987

DEPTH SCALE 1 : 25

Golder Associates

LOGGED J.COBISA

CHECKED AC

RECORD OF BOREHOLE 41

SHEET 1 of 1

LOCATION See Figure 2 — BORING DATE Dec.10,1987 — DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm — PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

Boring method: Power Auger, 200mm Diam (Hollow Stem)

STA. 14 + 100 on CL

Description	Elev. / Depth (m)
Ground Surface	75.28
TOPSOIL	0.00
Loose grey brown sandy silt, trace gravel and clay, trace organic matter (FILL)	0.06
Compact grey brown silty sand, some gravel and shale fragments, trace clay (GLACIAL TILL)	73.88 / 1.40
Dense grey silty sand, some gravel and shale fragments, occasional cobble and boulder (GLACIAL TILL)	72.69 / 2.59
Weathered dark grey SHALE BEDROCK	72.08 / 3.20
End of Hole	71.93 / 3.35

Sample Number	Type	Blows/0.3m
1	50 DO	7
2	50 DO	7
3	50 DO	12
4	50 DO	39
5	50 DO	> 50

Column headings: DEPTH SCALE METRES; BORING METHOD; SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (M)); SAMPLES (NUMBER, TYPE, BLOWS/0.3M); DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○; HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80; ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

16 ◆ 6 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25 — Golder Associates — LOGGED S.Leighton — CHECKED AC

RECORD OF BOREHOLE 42

SHEET 1 of 1

LOCATION See Figure 2 — BORING DATE Dec.10,1987 — DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm — PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

Boring method: Power Auger, 200mm Diam (Hollow Stem)

STA. 14 + 200 on CL

Description	Elev. / Depth (m)
Ground Surface	75.08
Loose to compact brown sandy silt, some gravel, trace clay rootlets and organic matter (FILL)	0.00
Compact grey brown silty sand, some gravel, trace clay, occasional cobble and boulder (GLACIAL TILL)	73.71 / 1.37
Compact grey sandy silt, some gravel and shale fragments, trace clay (GLACIAL TILL)	72.18 / 2.90
End of Hole	71.42 / 3.66

Sample Number	Type	Blows/0.3m	Additional Lab. Testing
1	50 DO	4	MH, Org = 3.6
2	50 DO	11	
3	50 DO	10	
4	50 DO	20	
5	50 DO	25	

16 ◆ 6 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25 — Golder Associates — LOGGED S.Leighton — CHECKED AC

RECORD OF BOREHOLE 43

SHEET 1 of 1

LOCATION: See Figure 2 | BORING DATE: Dec.10,1987 | DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

STA. 14 + 300 EAST BOUND
12.0m RT. of CL

Boring method: Power Auger, 200mm Diam (Hollow Stem)

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3M	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	73.52				
	Dark brown sandy silt TOPSOIL	0.00	1	50 DO	4	Backfill
	Grey brown SILTY SAND	73.00 / 0.37 → 0.52				
1	Very stiff grey brown SILTY CLAY, occasional sand seam (Weathered Crust)	71.66 / 1.86	2	50 DO	8	Bentonite Seal
2			3	50 DO	5	
	Loose grey brown SILT, occasional silty clay seam	70.78 / 2.74	4	50 DO	7	
3	Compact grey brown to grey silty sand, some gravel, trace clay (GLACIAL TILL)	69.86 / 3.66	5	50 DO	12	
	End of Hole					
4						W.L in Standpipe at Elev. 72.90 Jan.5,1988
5						

DEPTH SCALE 1 : 25

LOGGED S.Leighton
CHECKED

Golder Associates

RECORD OF BOREHOLE 44

SHEET 1 of 1

LOCATION: See Figure 2 | BORING DATE: Dec.10,1987 | DATUM: GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

STA. 14 + 430 WEST BOUND
5.0m RT. of CL

Boring method: Power Auger, 200mm Diam (Hollow Stem)

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3M
0	Ground Surface	72.67			
	TOPSOIL	0.00			
	Loose brown fine to coarse SAND, trace silt	0.09 → 72.15	1	50 DO	4
1	Very stiff grey brown SILTY CLAY, (Weathered Crust)	0.52 → 70.51	2	50 DO	6
2			3	50 DO	4
	Very dense grey brown to grey silty sand, some gravel, trace clay (GLACIAL TILL)	2.16 → 69.93	4	50 DO	62
	End of Hole Auger Refusal	2.74			
3					
4					
5					

Column headings (both sheets): SOIL PROFILE; SAMPLES; DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa — nat.V.- + Q.- ● rem.V.- ⊕ U.- ○; HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80; ADDITIONAL LAB. TESTING; STRATA PLOT; BORING METHOD; PIEZOMETER OR STANDPIPE INSTALLATION; 16 - 6 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25

LOGGED S.Leighton
CHECKED

Golder Associates

RECORD OF BOREHOLE 45

SHEET 1 of 1

LOCATION: See Figure 2
BORING DATE: Dec.10,1987
DATUM: GEODETIC
SAMPLER HAMMER, 63.5kg, DROP, 760mm
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

STA. 14 + 520 WEST BOUND on CL

Boring method: Power Auger, 200mm Diam (Hollow Stem)

DEPTH (M)	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	71.64					
	TOPSOIL	0.00					Backfill
	Loose brown SILTY SAND, trace organic matter and rootlets	0.09	1	50 DO	5	Org=3.4	
	Very stiff grey brown SILTY CLAY (Weathered Crust)	71.05 / 0.49	2	50 DO	7		Bentonite Seal
1–2			3	50 DO	6		
	Compact grey SILT, some silty clay and silty sand seams	69.38 / 2.16	4	50 DO	10		
3	Loose grey silty sand, some gravel and clay (GLACIAL TILL)	68.49 / 3.05	5	50 DO	7		
	End of Hole	67.88 / 3.66					

W.L in Standpipe at ELev.71.11 Jan.5,1988

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat.V.- + Q.- ● rem.V.- ⊕ U.- ○); HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT (Wp, W, Wl: 20 40 60 80)

16 ◆ 6 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25

LOGGED S.Leighton
CHECKED AC

Golder Associates

RECORD OF BOREHOLE 46

SHEET 1 of 2

LOCATION: See Figure 2
BORING DATE: Dec.10,1987
DATUM: GEODETIC
SAMPLER HAMMER, 63.5kg, DROP, 760mm
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

STA. 14 + 850 WEST BOUND 4.8m RT. of CL

Boring method: Power Auger, 200mm Diam (Hollow Stem)

DEPTH (M)	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	72.89 / 0.00					Backfill
	Compact grey sand and gravel, some silt (FILL)		1	50 DO	18	M	
1	Loose dark grey silty sand, trace gravel and organic matter (FILL)	72.25 / 0.64	2	50 DO	9	Org=3.6	
2	Very stiff grey brown SILTY CLAY, occasional silty sand seam (Weathered Crust)	71.03 / 1.86	3	50 DO	9		Bentonite Seal
			4	50 DO	8		
3			5	50 DO	4		
4			6	50 DO	4		
	Stiff grey SILTY CLAY, occasional thin sand seam	68.32 / 4.57	7	50 DO	2		
5	Hole Continued	67.89 / 5.00					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat.V.- + Q.- ● rem.V.- ⊕ U.- ○); HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT (Wp, W, Wl: 20 40 60 80)

16 ◆ 6 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25

LOGGED S.Leighton
CHECKED AC

Golder Associates

RECORD OF BOREHOLE 46

SHEET 2 of 2

LOCATION See Figure 2 | BORING DATE Dec.10,1987 | DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm | PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa (nat.V.- + Q.- ● rem.V.- ⊕ U.- ○) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
5		Hole Continued		67.89							
	Power Auger 200mm Diam (Hollow Stem)	Stiff grey SILTY CLAY, occasional thin sand seam		5.00	7	50 DO	2				Backfill
		End of Hole		67.10 5.79							
6											W.L in Standpipe at Elev. 70.66 Jan.8,1988
7											
8											
9											
10											

16-⊕-6 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 25

Golder Associates

LOGGED S.Leighton

CHECKED AC

APPENDIX C

Montreal Road Interchange
Relevant Borehole Logs by Others
Record of Boreholes 15-1 and 15-2
Figure C1

FORM S-10

G. C. McROSTIE
CONSULTING CIVIL ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

Trans Canada - East of Ottawa
Montreal Road crossing

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 183.6
REMARKS Geodetic datum

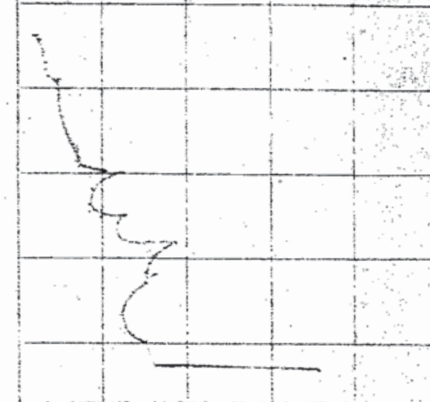
HOLE No. 1

BORINGS BY Mc Rostie TESTING BY Mc Rostie DATE 9 Jan 56

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
			GROUND SURFACE	0	183.6
3.17, 2.99		1, 2	Top soil; Firm slightly fissured brownish grey clay 7.5		
1.75, 1.75, 1.8, 2.16, 1.94, 1.73, 1.8, 1.59, 1.73, 2.16, 1.8, 1.97, 1.73, 1.00, 2.59, 2.59, 2.44, 2.27		3–20	Medium Soft grey clay 83.0	10, 20, 30, 40, 50, 60, 70, 80	133.6 (at 50)
			Soft soil - no samples - see penetration tests →	90, 100, 110	83.6 (at 100)
			Bottom of hole 123.0	120, 130, 140, 150	33.6 (at 150)

PENETRATION TEST
......LB. HAMMER ......INCH DROP NO CASING ......INCH DIA. ROD
BLOWS PER FOOT

Groundwater at surface.



10 20 30 40 50 60 70 80 90 100
blows / foot no casing
140 lb hammer 30 in drop 1/4" E rod

% WATER CONTENT

PLATE 2

L.-M. CO—A14294

FORM S-10

G. C. McROSTIE
CONSULTING CIVIL ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

Trans-Canada - East of Ottawa -
Montreal Road crossing

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 186.04
REMARKS Geodetic survey

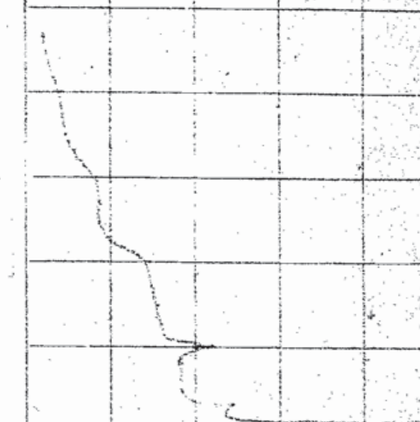
HOLE No. 2

BORINGS BY Mc Rostie TESTING BY Mc Rostie DATE 12 Jan 56

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
			GROUND SURFACE	0	186.04
3.18, 2.6, 2.16		1, 2, 3	Top soil; Hard slightly fissured brownish grey clay; Firm slightly fissured brownish grey clay		
2.45, 2.29, 2.16, 1.8, 1.94, 2.59, 2.59, 2.59, 2.59, 1.37, 2.09, 1.75, 2.16, 2.16, 1.94, 2.94		4–19	Medium, soft grey clay 84.0	10, 20, 30, 40, 50, 60, 70, 80	136.04 (at 50)
			Soft soil no samples see penetration tests →	90, 100, 110, 120	86.04 (at 100)
			Bottom of hole 127.0	130, 140, 150	

PENETRATION TEST
......LB. HAMMER ......INCH DROP NO CASING ......INCH DIA. ROD
BLOWS PER FOOT

Groundwater at surface.



20 40 60 80
blows/ft. no casing 140 lb hammer
30 in drop E rod

% WATER CONTENT

PLATE 3

L.-M. CO—A14294

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION – ENGINEERING MATERIALS OFFICE – SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1

W P 911-73-00 LOCATION Sta. 16+276.0; O/S 21.00 m Lt. ¢ Hwy 17 W.B.L. ORIGINATED BY M.M.
DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Augers & Cone Test & Washboring COMPILED BY M.M.
DATUM Geodetic DATE April 17-19, 1978 CHECKED BY R S

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa / SENSITIVITY	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
44.33	Ground Level								
0.00	Silty Clay Rust Mottled and Oxidized Very Stiff	1	SS	4			95 KPa		
		2	TW	PH	▽ Head		95 KPa 95KPa	19.43	e_o=0.6 C_c=0.11
41.33 3.00	Stiff	3	TW	PH			5, 4		
	Stiff to Very Stiff	4	TW	PH		40.00	6, 5	15.80	0 0 26 74
	Sensitive Clay Grey	5	TW	PH			6, 6		
		6	TW	PH	Artesian Water		11		
							95 KPa		
		7	TW	PH		35.00	95 KPa		
		8	SS	3			95 KPa		
		9	TW	PH			95 KPa	16.19	e_o=1.74 σ_p' =335 kPa C_c=1.19
		10	SS	10		30.00			
		11	TW	PH			6		
	With Inclusions or Pockets of Organics						95 KPa		
		12	SS	7			95 KPa		
		13	TW	PH		25.00		17.27	
						20.00			
		14	SS	10					
14.33 30.00						15.00			

Continued

$+^3$, $\times^5$: Numbers refer to Sensitivity   15 ⟡ 5 (%) STRAIN AT FAILURE (20 / 10)

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION – ENGINEERING MATERIALS OFFICE – SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1 Continued

W P 911-73-00 LOCATION Sta. 16+276.0, O/S 21.00 m Lt. ¢ Hwy. 17 W.B.L. ORIGINATED BY M.M.
DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Augers, Cone Test & Washboring COMPILED BY J.A.
DATUM Geodetic DATE April 17-19, 1978 CHECKED BY R S

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14.33	continued							
30.00	Sensitive Clay							Method of Boring 0-27.5 m Auger 27.5-39.0 m Tri-cone Ahead 39-39.5 m BX Casing
11.72 32.61	Heterogeneous Mixture Clayey Silt, Sand and Gravel, Hard	15	SS	17	▽ Encountered	10.00		
10.19 34.14	Heterogeneous Mixture Silt, Sand & Gravel With Cobbles, Diameter 50-150 mm, Every 450 mm, Very Dense (Glacial Till)	16	SS	70/0.15 m				
5.32 39.01	Black Shale Bedrock Sound	17	RC BXL	REC 100% RQD 90 %		5.00		
3.30 41.03								

$+^3$, $\times^5$: Numbers refer to Sensitivity   15 ⟡ 5 (%) STRAIN AT FAILURE (20 / 10)

RECORD OF BOREHOLE 15-1

PROJECT: 14-275
LOCATION: See Site Plan, Figure 2
BORING DATE: May 29, 2015

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 m

Boring method: Power Auger, 200 mm Diameter Hollow Stem (0 to 16.76 m); Dynamic Penetration Cone Test, Uncased (below 16.76 m)

Soil Profile – Description	Elev. / Depth (m)
Ground Surface	60.21
Grey crushed sand and gravel, trace to some silt (BASE / SUBBASE MATERIAL)	
Stiff, grey brown SILTY CLAY (Weathered Crust)	58.15 / 2.06
Stiff, grey SILTY CLAY	55.64 / 4.57
Possible silty clay deposits	43.45 / 16.76
End of borehole	29.73 / 30.48

Sample Number	Type	Blows/0.3m
1	D.O.	33
2	D.O.	23
3	D.O.	8
4	D.O.	7
5	D.O.	2
6	D.O.	1
7	D.O.	1
8	D.O.	1
9	D.O.	1
10	D.O.	1
11	D.O.	1

Piezometer or Standpipe Installation: Bentonite; Backfilled with auger cuttings; Groundwater conditions not observed

Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat. V - +, Q - ●, rem. V - ⊕, U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}); WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80); ADDITIONAL LAB. TESTING

DEPTH SCALE 1 to 160

Houle Chevrier Engineering

LOGGED: A.N.
CHECKED: J.C.

RECORD OF BOREHOLE 15-2

PROJECT: 14-275
LOCATION: See Site Plan, Figure 2
BORING DATE: May 28, 2015

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 m

Boring method: Power Auger, 200 mm Diameter Hollow Stem (0 to 16.76 m); Dynamic Penetration Cone Test, Uncased (below 16.76 m)

Soil Profile – Description	Elev. / Depth (m)
Ground Surface	50.44
Grey crushed sand and gravel, trace silt (BASE / SUBBASE MATERIAL)	
Stiff, fine grey brown SILTY CLAY (Weathered Crust)	49.43 / 1.01
Very stiff, grey SILTY CLAY	47.39 / 3.05
Possible silty clay deposits	33.68 / 16.76
End of borehole	19.96 / 30.48

Sample Number	Type	Blows/0.3m
1	C.S.	
2	D.O.	6
3	D.O.	4
4	D.O.	2
5	D.O.	2
6	D.O.	1
7	D.O.	1
8	D.O.	2
9	D.O.	4
10	D.O.	2

Piezometer or Standpipe Installation: Bentonite; Backfilled with auger cuttings; Groundwater conditions not observed

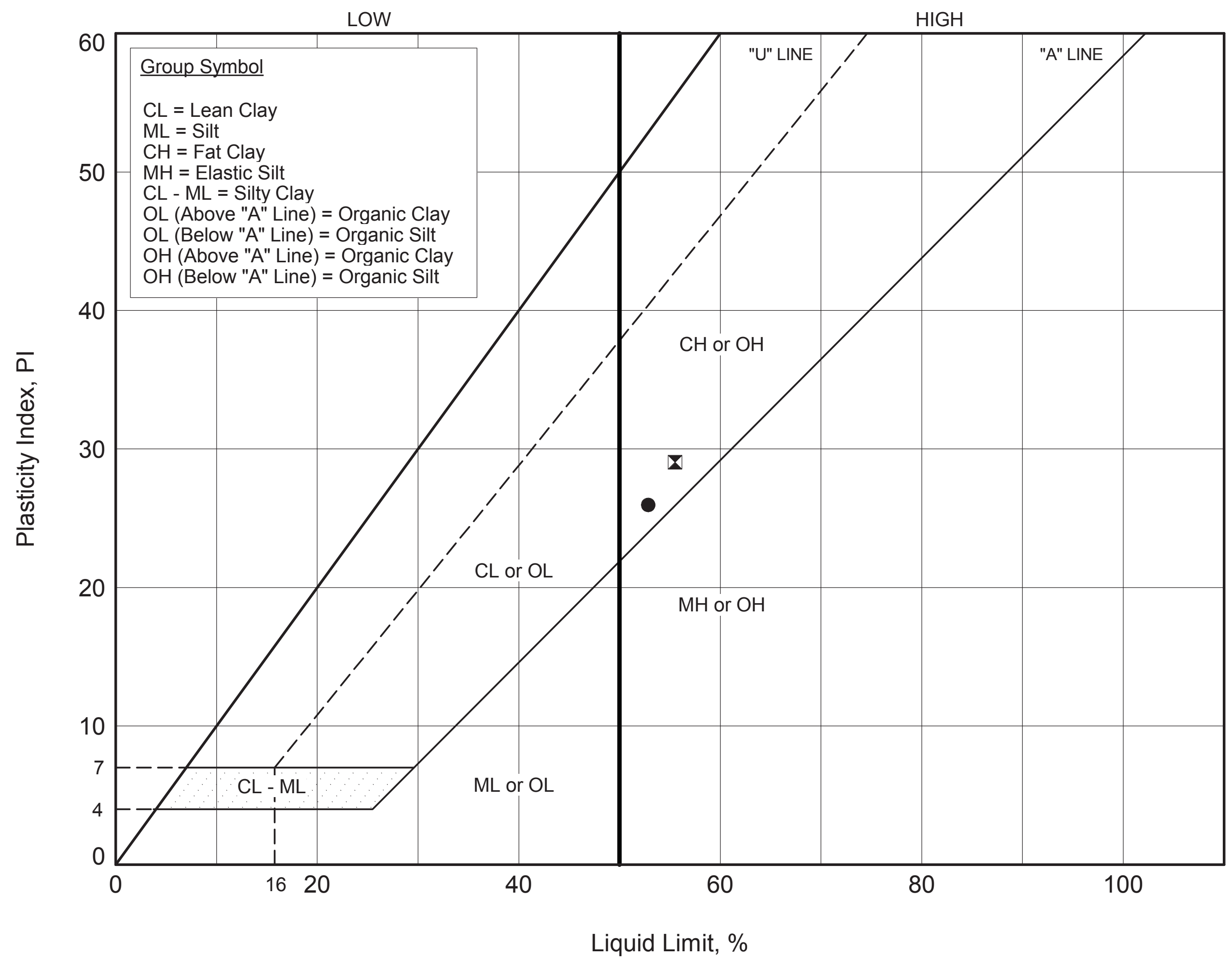
Chart columns: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat. V - +, Q - ●, rem. V - ⊕, U - ○); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}); WATER CONTENT, PERCENT (Wp, W, Wl; 20, 40, 60, 80); ADDITIONAL LAB. TESTING

DEPTH SCALE 1 to 160

Houle Chevrier Engineering

LOGGED: A.N.
CHECKED: J.C.

PLASTICITY CHART	FIGURE C1



Legend	Borehole	Sample	Depth (m)	LL %	PL %	PI %
●	15-1	9	9.1 - 9.8	52.8	26.9	26.0
⧗	15-2	5	4.6 - 5.2	55.5	26.5	29.1

HCE ATTERBERG LIMITS 14275 BHS.GPJ HOULE CHEVRIER FEB 9 2011.GDT 8-5-15

Houle Chevrier Engineering

a GEMTEC company

Date: July 2015

Project: 14-275

geotechnical

environmental

hydrogeology

materials testing & inspection

experience • knowledge • reliability