

JANUARY 9, 2019

1191 COUNTY RD. 42
MAMTA HOMES DEVELOPMENT
FUNCTIONAL SERVICING & STOMRWATER
MANAGEMENT REPORT

TOWNSHIP OF CLEARVIEW



355310 BLUE MOUNTAINS-EUPHRASIA TOWNLINE
CLARKSBURG, ON N0H 1J0

Table of Contents

Introduction	1
Existing Site Conditions	1
Geotechnical Information	2
Existing Sanitary Sewer	2
Existing Water Servicing	3
Existing Condition Stormwater Modelling	3
Proposed Site Design	5
Sanitary Servicing	5
Water Supply	7
Utilities	7
Stormwater Approval Criteria	7
Proposed Stormwater Management Design	8
Proposed Stormwater Management Model	9
Water Balance and Infiltration Target	12
Phosphorous Budget	13
Erosion and Sediment Controls	14
Construction Phasing	14
Conclusions	15

Drawings

- Drawing C1 – Servicing Plan 1 of 2
- Drawing C2 – Servicing Plan 2 of 2
- Drawing C3 – Grading Plan 1 of 2
- Drawing C4 – Grading Plan 2 of 2
- Drawing C5 – Sanitary Sewer Catchment Areas
- Drawing C6 – Stormwater Management Catchment Areas

Appendices

- Appendix A – Site Plan
- Appendix B – Environmental Review
- Appendix C - Geotechnical Information
- Appendix C – Excerpts from Sept 2017 Aston Meadows SWM Report

Appendix D – Existing Condition Stormwater Information

Appendix E – Sanitary Sewer Design Sheet

Appendix F – Post Development Stormwater Information

Appendix G – Water Balance

Appendix H – Phosphorous Budget

Introduction

CAPES Engineering has been retained by Mamta Homes to prepare a functional servicing and stormwater management report in support of a Site Plan application for the 5.02 ha site located south of Margaret Street and east of County Road 42/Airport Road in Clearview Township.

The existing parcel known as the West Part of Lot 23, Concession 2 is currently vacant and is to be developed as a residential plan of condominium for seniors. A 36.58 m wide Hydro Easement where no development can occur crosses the southern portion of the site and contains a hydro tower which is to remain.

The current proposed Site Plan includes a total of 107 residential units in a mix of semi detached, townhouse and multistory apartment buildings. The site is to be accessed by a temporary connection to Margaret Street and two connections to future development to the east which is also owned by Mamta Homes (East Part of Lot 23, Concession 2).

The additional lands owned by Mamta Homes to the east already have Draft Plan Approval which was granted in 2013 and was extended in 2017. The intent for that parcel is to provide 69 single family and semi-detached residential units serviced by a permanent road connection Margaret Street. For the purposes of the report we have referred to the two sites as Mamta West and Mamta East.

Both the east and west parcels owned by Mamta are dependent on the development of infrastructure (sanitary sewer, sewage pumping station, watermain, intersection upgrades and stormwater management infrastructure) related to the larger Ashton Meadows development which is located to the east to ultimately proceed.

It is anticipated that approvals from Clearview Township and the Nottawasaga Valley Conservation Authority (NVCA) will be required for this development.

Please refer to **Appendix A** for the Site Plan prepared by RPD Studio.

Existing Site Conditions

The existing 5.02 ha site is located at the west end of Margaret Street along the east side of County Road 42 (Airport Road) in Clearview Township and is legally described as the east Part of Lot 23 Concession 2. The site was part of a severance (this site being the retained portion) and Draft Plan approval of the east half of Lot 23 Concession 2 in 2013. An extension to the Draft Plan Approval was granted in 2017 for the severed portion.

The site bound by Margaret Street to the north as well as an existing single-family residence (199 Margaret Street). A 5.0 m road widening is required along the north edge of the site to allow for the expansion of Margaret Street.

To the west of the site is County Road 42/Airport Road (this portion is now owned and maintained by Clearview Township). A 10 m road widening is required along the west side of the site to accommodate an increase in width of County Rd. 42

To the south is an existing Commercial Block which operates as a distribution center for Creemore Springs Brewery. As indicated above the east side of the site is the severed residential development parcel also owned by Mamta Homes. A 36.58 m wide Hydro corridor/easement crosses the southern part of the site and an overhead electrical tower sits within the easement.

East of the two Mamta Homes owned parcels is the Ashton Meadows Development which is currently under the initial stages of construction for Phase 1A.

Please refer to **Drawing C1** which includes a location plan.

The site is largely covered in grass, long weeds and scrub brush/trees which have grown back in the last few years after the site had been cleared.

There is a high point in the SW corner of the site with an elevation of 222.436. The site slopes from this high point in a general north-east direction at approximately 0.8%.

There are several low wet areas along the eastern edge of the property line which have been identified as “isolated wetlands” which have limited environmental function. These wetland areas have been reviewed by Terrastory Environmental Consulting Inc. in consultation with the NVCA. It was determined that these features have little environmental function and can be eliminated from the site. The NVCA requested that best efforts be made to augment infiltration on the site to help compensate for the loss of these features. Measures to help increase infiltration on the site are discussed below. Please refer to Appendix B for the review letter prepared by Terrastory.

Geotechnical Information

A geotechnical study was completed by Orbit Engineering in April 2017 for both the Mamta East and Mamta West with 5 boreholes located on the current development site and an additional 4 boreholes located on the east half. Please refer to **Appendix C** for a copy of the geotechnical report.

Boreholes show an upper layer, to a depth between 0.8 and 1.2 m, of sandy silt to silty sand with a more dense layer below that of the same material to a depth of 6.7 m. Groundwater was found to be between 0.1 m and 2.1 m with the average on the west half of the site at 0.54 m (maximum reading at 0.3 m deep).

The average grain size analysis across the site corresponds to a silty sand or under the USDA Soil Texture classification a Fine Sandy Loam or Loam (Type BC or C soil). The combination of the silty soil material and high groundwater make the potential for infiltration of runoff on the site low. Measures to increase the infiltration on the site are discussed in more detail in the stormwater section below.

Existing Sanitary Sewer

Based on information reviewed from the Ashton Meadows Phase 1B engineering drawing set (Sept. 2017) prepared by Greenland Engineering there is an existing 200 mm dia. sanitary sewer line on Margaret Street at a relatively flat grade of 0.35%. The sanitary sewer extends within 60 m of the intersection of County Rd. 42 and Margaret Street however we do not believe that a service lateral exists to the site. The invert of the existing sanitary sewer at the manhole on Margaret Street is 218.36.

Currently the sanitary sewer flows east along Margaret Street and then north along Clarence to the existing sewage pump station on Dominion Drive. The flow is then pumped via 250 mm dia. forcemain discharging to the Stayner Wastewater Treatment Plant.

We understand from discussions with the Municipal Engineer (RJ Burnside) that there is limited capacity for additional development in the sanitary sewer on Margaret and Clarence Street however it is unclear the precise number of units that could currently be accommodated in the system.

Existing Water Servicing

A 150 mm dia. watermain is located along the northern part of Margaret Street and connects into the trunk 300 mm dia. watermain on the east side of County Rd. 42 providing potable water to the homes on Margaret Street. The trunk watermain connects the water reservoir and chlorination facility (south of the site on County Rd. 42) to the Town of Stayner.

It appears from the information we have received that there is currently no water service connection to the Mamta West site.

Existing Condition Stormwater Modelling

There are no existing stormwater management measures on site, and runoff currently sheet flows toward the east and north-east towards the Mamta east development parcel, the Ashton Meadows development beyond it and Margaret Street. There is an elevation rise along the frontage of the site adjacent to Margaret Street that does not allow runoff to enter the ditch system and forces the water east into the Mamta East site. Some flow is able to reach the Margaret Street ditch moving north-east from the Mamta east property, but the majority of the flow discharges into the Ashton Meadows site.

What little flow from the site that does reach Margaret Street flows east in the ditch on the south side of the road discharging to Macintyre Creek.

We understand that the “Stormwater Management Implementation Report Ashton Meadows Subdivision Phase 1A/B” prepared by Greenland Consulting Engineers (Sept. 2017) included the Mamta Homes development parcel in their existing condition model as part of Catchment B. We have not been provided with copies of any of the Ashton Meadows and were only permitted to review the information at the Township offices and as such we have been unable to include excerpts from their reports and drawings. The 2017 Greenland Report does not specifically break down their catchment areas to delineate the Mamta west (or east) sites and as such we have prepared a site-specific stormwater model.

We have utilized PCSWMM 2017 modelling software (Version 7.1.2480, SWMM version 5.0.013-5.1.012) to undertake the analysis of the existing site.

The contributing drainage area for the site was determined by using a combination of aerial imagery from Simcoe County Mapping (<https://maps.simcoe.ca/public/>), topographic survey of the site completed in 2010 & 2018 and a site visit conducted in September 2018.

The total drainage area for the Mamta West site has been determined to be 5.02 ha in size and largely consists of overland sheet flow east and north-east. There are several low-lying depressions which collect some runoff along the east edge of the site which are believed to be remnant portions of isolated

wetland areas. The overall impervious percentage of the site is low (2.4%) consisting only of the gravel access road to the hydro tower.

We have selected the Green Ampt Method of infiltration for the site for a loam as per the geotechnical study. We have utilized the Green Ampt Values as presented in Rawls et al. (1983) and NVCA guidelines.

$$K_{eff} = 3.4 \text{ mm/hr} \quad \text{Suction Head} = 88.9 \text{ mm} \quad \text{Initial Deficit (fraction)} = 0.231$$

Rainfall data was obtained from the MTO IDF Curve website as per the Town Engineering Standards and has been included in **Appendix D**.

Please also refer to **Appendix D** for a summary of the determination of the other input parameters (n perv, d_{store} Perv etc.) for the existing condition catchment.

The 2-100 year 4-hour Chicago, 24 hr SCS Type II storm events were tested on the site to see which would produce the highest peak flow. In addition, we ran the 25 mm Chicago storm and Timmins Regional Storm. The 24-hour SCS Type II storm generally produced the highest peak flows in the existing condition due to the low level of impervious of the existing site.

Please refer to Table 1 below for a summary of the results from the model.

Table 1 – Existing Condition Modelling Results

Storm Event	Peak Flow Offsite (m ³ /s)
24 Hr SCS Type II	
2-year	0.05
5-year	0.09
10-year	0.13
25-year	0.19
50-year	0.24
100-year	0.29
4 Hr Chicago	
2-year	0.04
5-year	0.06
10-year	0.07
25-year	0.10
50-year	0.12
100-year	0.14
25 mm	0.02
Timmins	0.28

The PCSWMM summary output file for the 100-year SCS Type II storm has been included in **Appendix D** for reference. The remaining output files can be provided upon request in either digital or hardcopy format.

Proposed Site Design

It is proposed to develop the existing site to provide a total of 107 residential units in a mix of semi-detached, townhouse and apartment units. 19 parking spaces will be provided for visitor parking plus 1 accessible space.

Depending on the timing of development between the Mamta east and west sites it has been anticipated that the site may need to be temporarily accessed from Margaret Street. The permanent access will be from the future north-south Municipal road of the Mamta east development site and will consist of two access locations. When this neighbouring street is completed the temporary access from Margaret Street will be eliminated and will become a dead end cul-de-sac. Please refer to **Appendix A** for the Site Plan and to **Drawings C1-C6** for additional details on the proposed site configuration.

We understand that no development can occur within the HONI easement, however access through the easement is permitted to allow development of the southern portion of the site. This is where it is proposed to build the multi-story condominium apartment building and parking area (56 parking spaces + 3 accessible spaces).

An amenity area is proposed on the north side of the HONI easement for resident use and is proposed to contain a party room, gym, movie theatre office space and washrooms. An additional 10 parking spaces including 3 accessible spaces will be provided for the Amenity area.

The internal ROW is to vary between 6.35 m (Condo Apt. Building area, Street C), 7.36 m (Streets B, D, E, F, G) and 8.35 m wide (Street A) depending on the location and whether there is a sidewalk on one side or not. The roadway will include concrete mountable curb with narrow gutter (OPSD 600.100) as per the Town standards with minimum travel width for the road of 6.0 m (face of curb to face of curb).

A 5 m wide walking trail is proposed along the west side of the site adjacent to County Rd. 42 and will extend along SE along the north side of the HONI easement towards the Ashton Meadows site.

Sanitary Servicing

It is proposed to provide a 200 mm dia. gravity sanitary sewer for all 107 units and the amenity area. The pipe will be constructed to connect into the Mamta East development and then into a sanitary stub that is proposed for the Ashton Meadows development. The available sanitary stub for the Mamta development is shown on the Ashton Meadows Phase 1A & 1B Sanitary Drainage Area Plan with an invert of 217.70 on "Street A" (south west corner of Phase 1A).

The Greenland Engineering sanitary calculations assumed a total of 4.6 ha of sanitary drainage area and a serviced population of 550 people from the Mamta development. We believe that the drainage area should have been listed as 7.85 ha to include both the west and east Mamta parcels, as the anticipated number of people (550) and corresponding number of units at 183 (550/3 people per unit) corresponds well with the conceptual plans for both Mamta east and west but the total drainage area does not match.

The Township of Clearview Engineering Standards require the following design flows for the calculation of the sanitary sewer capacity:

- Average Daily per capita Flow- 450 L/cap/day
- Extraneous Flow Allowance - 0.23 L/sec/gross hectare
- 3.0 people per unit

The total anticipated peak sewage flow from the Mamta west site is 6.87 L/s (based on 5.02 ha and 321 people) and it is anticipated that the total from both Mamta east and west would be a total of 10.97 L/s (based on a combined 7.86 ha of development area, 528 people and the Mamta West Amenity Building).

Please refer to **Drawing C1 and C2** for the preliminary sanitary sewer layout, **Drawing C5** for the Sanitary Drainage Areas and to **Appendix E** for the preliminary Sanitary Sewer Design Sheet.

Greenland Engineering has designed for a total of 550 people and 4.6 ha of drainage area equaling 11.32 L/s peak flow at the stub connection point in the SW corner of Phase 1B.

We believe that the lower area value of 4.6 ha used in the Greenland Report is an error, and the 550 people is an overestimate of the population. Due to Greenland assuming an additional 7.3 units (22 people) from the combined Mamta sites our calculated peak sanitary flows to the Ashton Meadows development is lower than their design capacity and therefore no changes are required to the downstream system to accommodate the flow.

The sanitary sewers for the Mamta west site have been designed to ensure the minimum full flow velocity of 0.75 m/s.

Depending on the specific sequencing of development Mamta West may need to install connecting sanitary sewer pipes across Mamta East before Mamta West is constructed. It is anticipated that the coordination and timing of the design will be undertaken at the detailed design stage.

In addition, the construction and connection of Units on Mamta West is dependent on the construction of the infrastructure of Ashton Meadows Phase 1B and presumably the Sewage Pumping Station (SPS). Additional details and coordination will be required from Greenland Engineering and the Municipality to determine the sequence of construction for the proposed infrastructure.

An alternative option for servicing Mamta West for sanitary sewer would be to make a connection to the existing 200 mm dia. sanitary sewer on Margaret Street. The flattest section of existing sanitary sewer runs at 0.4% to the east at approximately 14% full with the existing 29 units connected to it. Both Mamta West and East could be connected to this existing pipe which would then flow at approximately 70% full.

This option could be fully explored at detailed design stage dependent on the timing of construction at Ashton Meadows and the remaining available sewage capacity on Margaret Street. This option would require significant regrading of the site from the functional design presented in this report.

Water Supply

It is proposed to loop a 150 mm diameter watermain from the Mamta east site through the Mamta west site out to Margaret Street. This includes a 150 mm dia. service line to the multi storey apartment building in the south-west corner of the site. A 25 mm dia. water service will be provided to each of the other units.

A minimum of three fire hydrants will be provided on the site with maximum spacing of 150 m between them as per the Town Standards.

Alternative looping options could include connecting from Margaret Street and looping out to the trunk watermain on County Rd. 42. A final option would be to complete a single connection to Margaret Street and provide an internal loop and blow off points. These alternative options could be explored further at detailed design should both Mamta East and Ashton Meadows not proceed to construction in a timely fashion.

Utilities

Currently hydro and telecommunications are provided to the area by overhead service on Margaret Street (north side) and on County Rd. 42 (west side adjacent to the site) while the gas line is located on the south side of Margaret Street and west side of County Rd. 42.

Underground utilities for the site will be provided in a common trench with coordinated design from the Mamta east site.

Utility providers (Bell, Rogers, Hydro One and Union Gas) will be contacted at a later stage of design, however it is not anticipated that there will be any issues in providing service connections from Margaret Street and coordinating design with the Mamta east site and Ashton Meadows.

Canada Post will also be contacted to provide delivery options to the site in keeping with the strategy for Mamta east and Ashton Meadows. It is anticipated that a central delivery box will be required which could be located at the Amenity building.

Stormwater Approval Criteria

As per Section 6.5 and 6.13 of the 2017 (Revised) Township of Clearview Engineering Standards this site is required to provide stormwater quantity control to pre-development levels and to address stormwater quality.

We believe that with respect to stormwater quality that an enhanced level of protection, as defined by the MOECC is appropriate for this site. In practice this level of protection requires the removal of 80% Total Suspended Solids (TSS) over a long-term basis.

Although not specifically required under the Clearview Township Guidelines we believe additional best management practices for this site should also include the safe conveyance of the Regional (Timmins) storm and promotion of infiltration wherever possible.

In addition to the above criteria the Nottawasaga Valley Conservation Authority (NVCA) requires the following related to stormwater management design

- Retention of 5 mm of rainfall on site

- Detention of the 25 mm storm for 48 hrs in Stormwater Management Ponds
- Mitigation for thermal and bacteriological impacts
- Matching of Pre-Development Phosphorous Loading with best efforts towards an additional 20% reduction
- Best efforts toward a water balance where conditions are favourable

Proposed Stormwater Management Design

Based on our review of the Greenland Engineering stormwater management design for Ashton Meadows presented in the Sept. 2017 SWM Implementation report, the runoff for the entire Mamta west site has been accounted for within the proposed Ashton Meadows SWM pond.

Greenland has assumed that a total drainage area of 3.0 ha would direct runoff to a 600 mm dia. storm sewer stub located at the SW corner of their site ("Street A", Phase 1B). An additional 4.4 ha drainage area was assumed to direct runoff to a storm manhole located on their "Street D", Phase 1B.

The remaining portion of the overall Mamta east & west drainage area was assumed to direct runoff to Margaret Street, however Greenland notes that the runoff from the entire Mamta owned parcels has been accommodated into the Ashton Meadows SWM pond design.

It is proposed to install an internal storm sewer network for the Mamta west site to collect the 5-year storm event. The internal storm sewer will connect through the Mamta east site to the locations provided by Greenland Engineering. In addition, the grading of the Mamta west site will be designed to ensure the overland flow route works in conjunction with the Mamta east site and will connect to the Ashton Meadows design for Street A and D.

Greenland chose to utilize the rational method for storm sewer design. The 5-year rational method peak flow from the combined Mamta east and west calculated by Greenland to Street A (or Ashton Meadows S in our modelling) is $0.40 \text{ m}^3/\text{s}$, while the flow to Street D (or Ashton Meadows North in our modelling) is $0.43 \text{ m}^3/\text{s}$.

Greenland chose to design the storm water management pond using Visual OTTHYMO (VO2) and divided the external area including the Mamta sites into catchment B2_1 (7.4 ha) and B2_2 (1.8 ha). Area B2_1 includes 94% of the Mamta east and west sites with the remaining 6% being included in Area B2_2.

As Greenland Engineering has made accommodation for uncontrolled flows from the Mamta sites to enter the Ashton Meadows development and be treated in the proposed stormwater management facility we do not believe that any additional SWM controls (either quantity or quality) are required for the Mamta site.

The Mamta West needs to be raised by between 1-2 m in order to satisfy three main design challenges. The gravity sanitary sewer needs to be connected to the Ashton meadows site (or Margaret Street), a stormwater overland flow route to Ashton Meadows and to overcome the high groundwater condition with finished floor elevations. We propose that the requirement to fill the site is also an opportunity to improve the conditions for infiltration. BY importing fill to the site which has a higher saturated hydraulic conductivity some reduction in peak flows will be achieved, there will be increase in the

groundwater recharge to compensate for the loss of the isolated wetland features, and the NVCA requirement to detain 5 mm of rainfall on site will be achieved. We have incorporated this design feature in our stormwater model below.

Proposed Stormwater Management Model

We have utilized PCSWMM to create a post development stormwater model for the Mamta development site to provide guidance to Greenland Engineering in the accommodation of the uncontrolled peak flows from the site. For ease of reference we have included both the Mamta East and Mamta West properties in our model to be able to more easily compare the proposed peak flows to the anticipated peak flows generated by Greenland Engineering in the Ashton Meadows reporting. We were unable to obtain any of the design reports for the Mamta East site and therefore have generated new peak flows based on this updated PCSWMM model.

Please refer to the Stormwater Area **Drawing C6** which shows the delineation of the post development drainage areas. We have divided the model into 4 sub-catchments; Mamta-West_N, Mamta_West_S, Mamta_East_N, and Mamta_East_S. Please refer to **Appendix F** for the Post Development breakdown for each of these catchments and the PCSWMM input parameters.

As indicated previously we understand that Greenland Engineering anticipated uncontrolled flow from 7.4 ha of external area to be directed through the Ashton Meadows development through a combination of storm sewer (5-year flow) and overland flow (10-100 year & Regional storms) along the proposed Streets. According to Greenland Engineering the Ashton Meadows SWM Pond has been designed to accommodate the full 7.86 ha of the Mamta West and East sites.

In keeping with Greenland Engineering design, as well as NVCA and Town Guidelines we have reviewed the site for the 4-hour Chicago storm and the 24 hr SCS Type II Storm as well as a 25 mm (4hr) Chicago “quality” storm and the Regional (Timmins) Storm event.

Please refer to **Appendix E** for a breakdown of the input parameters used in the post-development model. The information for the Mamta East site was based on the Draft Plan approved design for 69 single family units a full 20 m wide Municipal ROW.

As mentioned above we propose to fill the site with a suitable fill with a higher saturated hydraulic conductivity than the existing. The depth of the fill is deep enough that the imported fill governs for the infiltration rate rather than the poorer soils below. The existing condition soils were modeled as a “Loam” with the following Green Ampt Parameters:

$K_{eff} = 3.4 \text{ mm/hr}$ Suction Head = 88.9 mm Initial Deficit (fraction) = 0.231

It is proposed that the imported fill material would be at a minimum a “Sandy Loam” with the following characteristics:

$K_{eff} = 10.9 \text{ mm/hr}$ Suction Head = 109.982 mm Initial Deficit (fraction) = 0.263

We have provided a breakdown of the site assuming native soils compared to the proposed imported fill material to demonstrate the change in peak flows.

Please refer to Table 3a and 3b below for a break down of the PCSWMM peak flows.

Table 3a – PCSWMM Post Development Peak Runoff Summary – Native “Loam” Soils

Storm Event	Mamta West (North) Peak Flow	Mamta West (South) Peak Flow	Mamta East (North) Peak Flow	Mamta East (South) Peak Flow	Total Peak Flow to Ashton Meadows (North)	Total Peak Flow to Ashton Meadows (South)
	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)
24 Hr - SCS Type II						
2-year	0.25	0.22	0.16	0.15	0.40	0.36
5-year	0.36	0.31	0.23	0.21	0.58	0.52
10-year	0.44	0.38	0.28	0.26	0.70	0.64
25-year	0.55	0.48	0.35	0.32	0.87	0.79
50-year	0.63	0.55	0.40	0.37	1.00	0.90
100-year	0.71	0.62	0.45	0.42	1.13	1.02
4 Hr - Chicago						
2-year	0.26	0.24	0.18	0.16	0.39	0.36
5-year	0.38	0.35	0.26	0.24	0.58	0.54
10-year	0.47	0.43	0.32	0.29	0.70	0.67
25-year	0.59	0.53	0.39	0.36	0.86	0.81
50-year	0.68	0.61	0.45	0.42	1.00	0.92
100-year	0.78	0.70	0.52	0.48	1.14	1.05
25 mm	0.17	0.15	0.11	0.10	0.26	0.23
Timmins	0.29	0.26	0.18	0.17	0.47	0.43

Table 3b – PCSWMM Post Development Peak Runoff Summary – Imported “Sandy Loam” Soils

Storm Event	Mamta West (North) Peak Flow (m ³ /s)	Mamta West (South) Peak Flow (m ³ /s)	Mamta East (North) Peak Flow (m ³ /s)	Mamta East (South) Peak Flow (m ³ /s)	Total Peak Flow to Ashton Meadows (North) (m ³ /s)	Total Peak Flow to Ashton Meadows (South) (m ³ /s)
24 Hr - SCS Type II						
2-year	0.23	0.20	0.14	0.13	0.36	0.33
5-year	0.34	0.30	0.21	0.20	0.54	0.49
10-year	0.41	0.36	0.26	0.24	0.65	0.59
25-year	0.51	0.44	0.32	0.30	0.80	0.73
50-year	0.58	0.51	0.36	0.34	0.92	0.82
100-year	0.66	0.57	0.41	0.38	1.04	0.93
4 Hr - Chicago						
2-year	0.25	0.23	0.17	0.16	0.38	0.35
5-year	0.37	0.34	0.32	0.31	0.56	0.52
10-year	0.46	0.42	0.31	0.28	0.68	0.65
25-year	0.57	0.52	0.38	0.35	0.83	0.79
50-year	0.66	0.59	0.44	0.40	0.96	0.89
100-year	0.75	0.68	0.49	0.46	1.09	1.00
25 mm	0.17	0.15	0.11	0.10	0.26	0.22
Timmins	0.24	0.21	0.15	0.14	0.38	0.34

Please refer to **Appendix F** for a summary of the new 100-year Chicago storm modelling results. Additional storm events and the full digital model can be provided upon request. The proposed imported fill with a minimum Ks of 10.9 mm/hr will result in a 5-10% reduction in the peak flow from the Mamta sites. Fill with higher Ks values are likely only to produce a nominal increase in infiltration due to the high imperviousness of the post development site (average of 52% impervious). In addition, due to the high imperviousness of the proposed condition site, the 4 hr Chicago storm produces higher peak flows than the SCS Type II storm. We recommend that for storm sewer and stormwater management pond design that Greenland Engineering utilize the Chicago Storm peak runoff values presented in Table 3B above.

The internal storm sewer system has been shown on **Drawings C1 and C2** and will be designed based on the 5-year Chicago storm. A full design will be provided at detailed design stage.

Water Balance and Infiltration Target

The NVCA Stormwater Guidelines require areas of sensitive groundwater recharge areas (SGRA), and adjacent to sensitive ecological features (wetlands, woodlands and watercourses) to provide a water balance. In general, the NVCA require best efforts towards a water balance for a site, with the desire to promote a full post to pre-development water balance.

In addition to a best effort toward a water balance the NVCA requests that for erosion control a minimum of 5 mm of runoff is retained on-site. The NVCA determines the required volume by multiplying 5 mm by the entire development area. This volume equates to 251 cu. m of rainfall that should be detained on site ($50,200 \text{ sq. m} \times 0.005 \text{ m} = 251 \text{ cu. m}$).

The NVCA has identified several small isolated pockets of remnant wetland areas near the eastern edge of the site. These pockets have limited environmental function (refer to **Appendix B**) and are believed to be areas which were part of a larger area which exists on the Mamta east site and may have historically existed on the Ashton Meadows site. Their limited function is to provide some groundwater recharge. It is not possible to maintain these features due to the grading and servicing constraints and the NVCA has not requested that they need to be maintained.

Greenland Engineering has approached the overall stormwater design for the Ashton Meadows site to include both of the Mamta sites and we understand they have completed a water balance which also includes the Mamta sites. The pre-development overall water balance for Ashton meadows does not incorporate these remnant wetland pockets, and the post development condition

The conditions on the site are not very favourable for infiltration-based LID as the effective saturated hydraulic conductivity is low at 3.4 mm/hr (50 mm/hr infiltration). In addition, the groundwater condition on the site is high at an average of 0.54 m below existing grade. It is proposed that the site will need to be elevated by 1-2 m to install the gravity sanitary sewer and facilitate the stormwater overland flow route to Ashton Meadows. This could allow the import of soils with a higher infiltration rate to help increase groundwater recharge.

We have completed a Thornthwaite Water Balance as per the MOE (2003) Stormwater Planning and Design Manual and have appended the results (**Appendix G**).

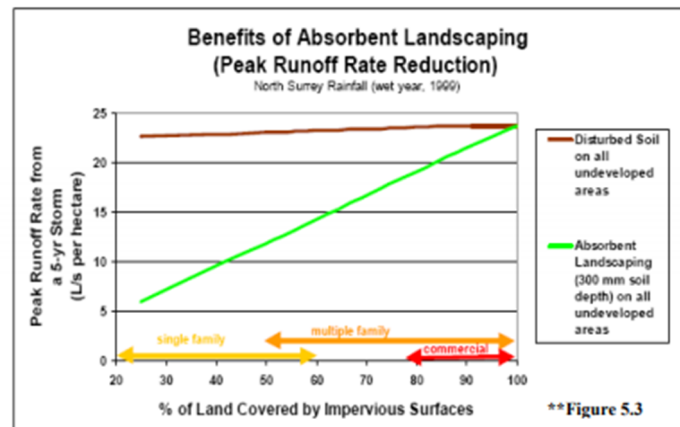
The existing condition total annual runoff volume is 8,013 cu. m while the infiltration is 9,065 cu. m. In the post development condition, it is expected, due to the increase in the impervious area that the runoff would increase by 2.86 times to 22,894 cu. m and the infiltration would decrease by 52% to 4,378 cu. m.

To achieve a full water balance for the site it would be necessary to infiltrate an additional 4,686 cu. m of runoff.

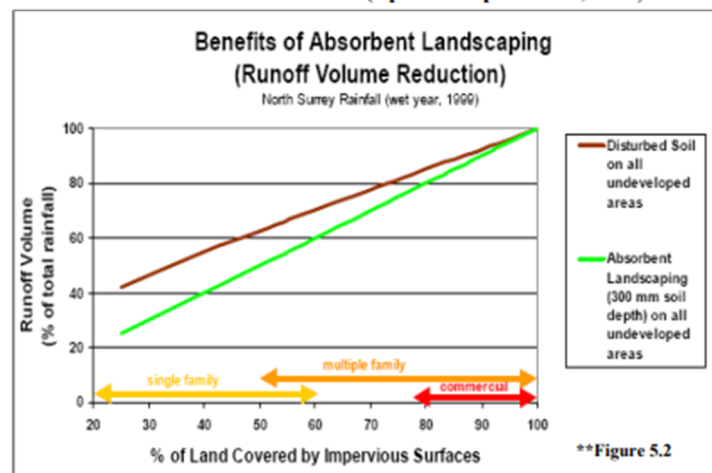
As discussed above it is proposed to import fill which has a higher saturated hydraulic conductivity which will result in a reduction in peak flow runoff of between 5-10%.

An additional option which may be effective for this site is the application of additional topsoil as first outlined in the "Guidebook for British Columbia Stormwater Planner (2005)" and was later included in the City of Toronto Wet Weather Flow Management Guidelines. We have provided an excerpt below

from the study which indicates that for a site of this impervious level (52%) that regardless of the underlying soil type 300 mm of absorbent topsoil could reduce the peak flow and volume of runoff by as much as 10%. This would reduce the volume requirement to achieve a water balance down to approximately 3,890 cu. m and would at a minimum achieve the 5 mm of rainfall retention on site requested by the NVCA.



**** Excerpts from Chapter 7 of Stormwater Planning:
A Guidebook for British Columbia (Updated September 6, 2005)**



We have included this scenario in the Water Balance calculations in Appendix G which demonstrates an overall reduction of runoff by 10% and an increase in the groundwater recharge by 9%. This equates to a retention of 16 mm of rainfall on the site which we believe meets the requirement for best efforts towards a water balance, detention of 5 mm of rainfall and helps replace the function of the isolated wetland pockets that were discussed in **Appendix B**.

Phosphorous Budget

The NVCA promotes the matching of pre-development phosphorus levels in the post development condition, and best efforts to achieve a 20% reduction pre-development levels. The calculation of the phosphorous discharge is based on a 25 mm, 4 hr Chicago storm event.

The primary P removal mechanism for this site is the proposed Ashton Meadows wet stormwater management pond. Based on the NVCA P Budget tool this pond should provide a 63% removal rate.

Additional P removal will come in the form of increased infiltration on pervious areas. The inclusion of 300 mm of topsoil and the proposed filling of the site with soils with a higher saturated hydraulic conductivity will work to detain approximately 16 mm of rainfall on site. This would equate to 64% of the 25 mm quality storm being retained on site which when combined with the Ashton Meadows SWM Pond would increase the P Removal to approximately 87%. Additional options could be explored at detailed design to try and increase the P removal levels including implementation of permeable pavement for select areas (parking areas and driveways) or the implementation of a sorptive media interceptor. In addition, there may be additional P removal efficiency in the Ashton Meadows site as their design includes both quality and quantity controls for the uncontrolled Mamta sites.

Coordination will be required with both Ashton Meadows and the design for Mamta East to ensure P removal levels are as high as possible.

Please refer to **Appendix H** for the preliminary P Budget calculations. We have assumed a 100% P removal efficiency based on the inclusion of increased on-site infiltration (24% removal), 2 sorptive media interceptors (79% P removal rate) such as the Jellyfish filter by Imbrium (or approved equivalent) followed by the Ashton Meadows Wet SWM Pond (63% P removal).

Erosion and Sediment Controls

As per NVCA guidelines we recommend that heavy duty silt fence as per OPSD 219.130 be installed along the perimeter of the site to prevent sediment transport during construction. These controls should be installed prior to the start of any earth moving activities and remain in place and be maintained until the vegetation is established on the site.

A full erosion and sediment control plan will be provided at detailed design stage.

Construction Phasing

The Mamta West site is dependent on infrastructure that is being constructed as part of two other sites as follows:

- The sanitary sewer for Mamta West is currently designed to connect to the system to be constructed on the Mamta East site which in turn connects to the sanitary sewer in Ashton Meadows Phase 1B. Mamta West will need to proceed after Phase 1B of Ashton Meadows has been approved for construction and schedule is available for the installation of the piping. Mamta West could proceed ahead of Mamta East with a temporary pipe through that development connecting to Phase 1B of Ashton Meadows. An alternative would be to connect the sanitary sewer to Margaret Street which has sufficient capacity for Mamta West. It is unknown specifically how many units could be accommodated in the existing sewage pumping station.
- The Ashton Meadows development includes the construction of a new Sewage Pumping Station to accommodate all of the proposed new residential units. The specific timing of the

construction of the SPS and the available capacity in the existing system will have an impact on the timing on the construction of the Mamta West Site

- The Ashton Meadows stormwater management pond has been designed to accommodate the Mamta West uncontrolled runoff. We believe the SWM pond will be constructed as part of Ashton Meadows Phase 1A, but some of the piping to which Mamta West is to connect may not be constructed until Phase 1B. Depending on the timing of construction Mamta West may need to construct temporary piping and/or surface flow routes to the SWM pond if Mamta East or Phase 1B of Ashton meadows is not complete.
- The road network for Mamta West is anticipated to require a temporary connection to Margaret Street if Mamta East is not constructed first. This entrance would be eliminated or possibly not required if Mamta East has been constructed when Mamta West is developed.
- Utilities (hydro, gas, telecommunications) would likely be extended from Mamta East to Mamta West however alternative routing may be required if Mamta West is developed first. Utilities could be extended from either Margaret Street or County Rd. 42

Due to the large number of interrelated infrastructure components coordination with both Mamta East and Ashton Meadows will be a key requirement at detailed design stage.

Conclusions

It is proposed to develop the 5.02 ha Mamta West site south of Margaret Street and east of County Rd. 42/Airport Rd. in the Township of Clearview with 107 residential units. The development is proposed to as a Site Plan and requires approval from the Township and from the NVCA. MOECP approvals are not required.

The site will consist of a mix of semi detached, townhouse and apartment condo units accessed via an internal private road. The existing Hydro One easement and Tower will remain on site and development will not occur within the easement save for an access road through it to the southern part of the site.

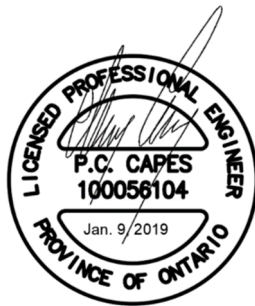
The site can be fully serviced with connections to the Municipal sewer and water system via the Ashton Meadows development site. In addition, the Ashton Meadows design will provide adequate storm water management through the construction of the wet SWM pond on that site. The site has no major technical issues that can not be resolved through the detailed design process in consultation with the Township of Clearview.

Report Prepared By:



Clayton Capes, MSc. P.Eng.

CAPES Engineering Ltd.



Drawings

Drawing C1 – Servicing Plan 1 of 2

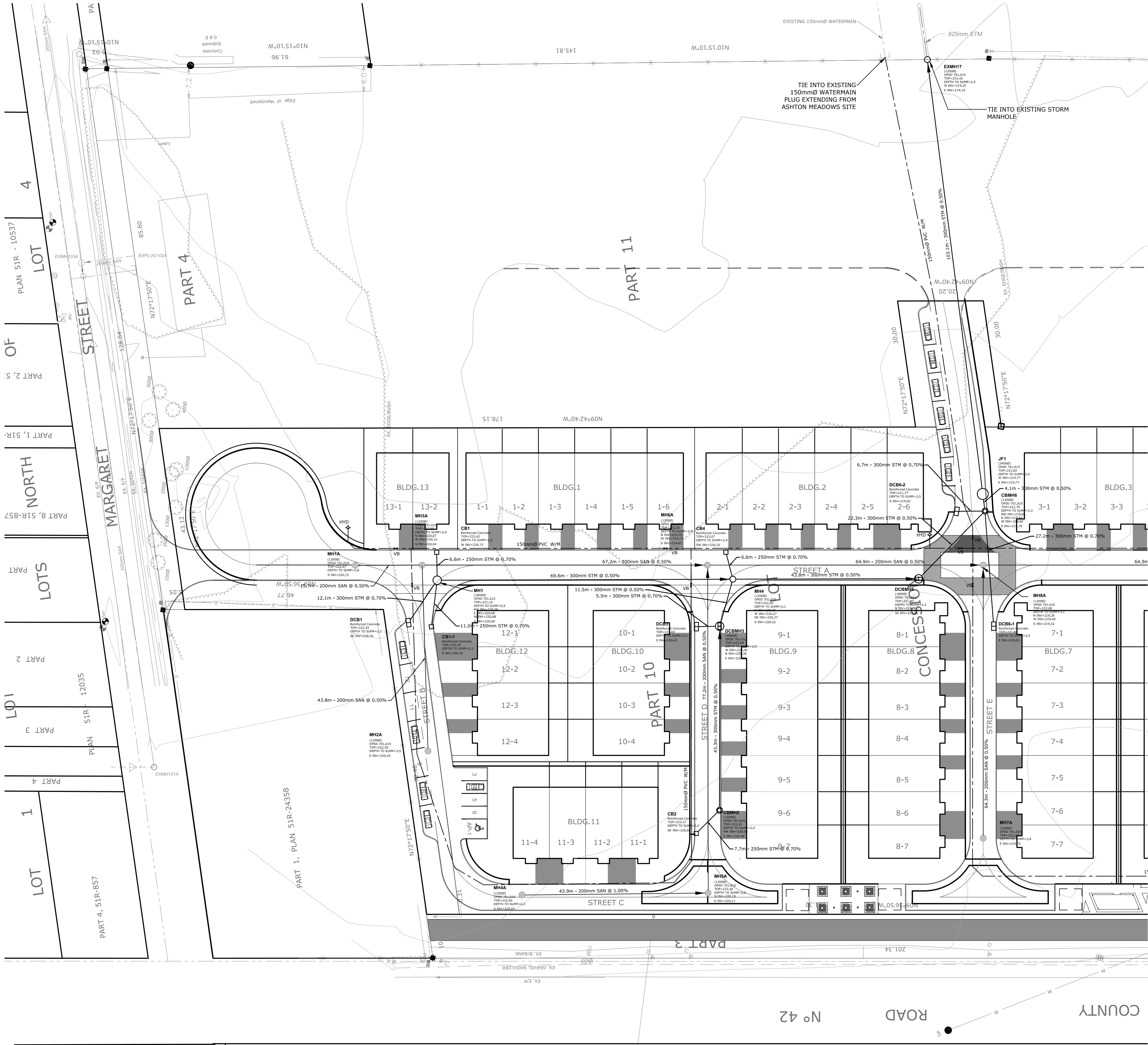
Drawing C2 – Servicing Plan 2 of 2

Drawing C3 – Grading Plan 1 of 2

Drawing C4 – Grading Plan 2 of 2

Drawing C5 – Sanitary Sewer Catchment Areas

Drawing C6 – Stormwater Management Catchment Areas



LEGEND

- STORM SEWER/MANHOLE
- SANITARY SEWER/MANHOLE
- WATERMAIN
- VALVE & BOX
- HYDRANT & VALVE
- EXISTING SANITARY SEWER
- EXISTING STORM SEWER
- EXISTING WATERMAIN

Notes

- This drawing is the exclusive property of CAPES Engineering Ltd. The reproduction of any part without express written consent of this Corporation is strictly prohibited.
- The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to CAPES Engineering Ltd. prior to construction.
- This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

No	Issue / Revision	Date	Auth
1	ISSUED FOR FSR	19/01/14	CC

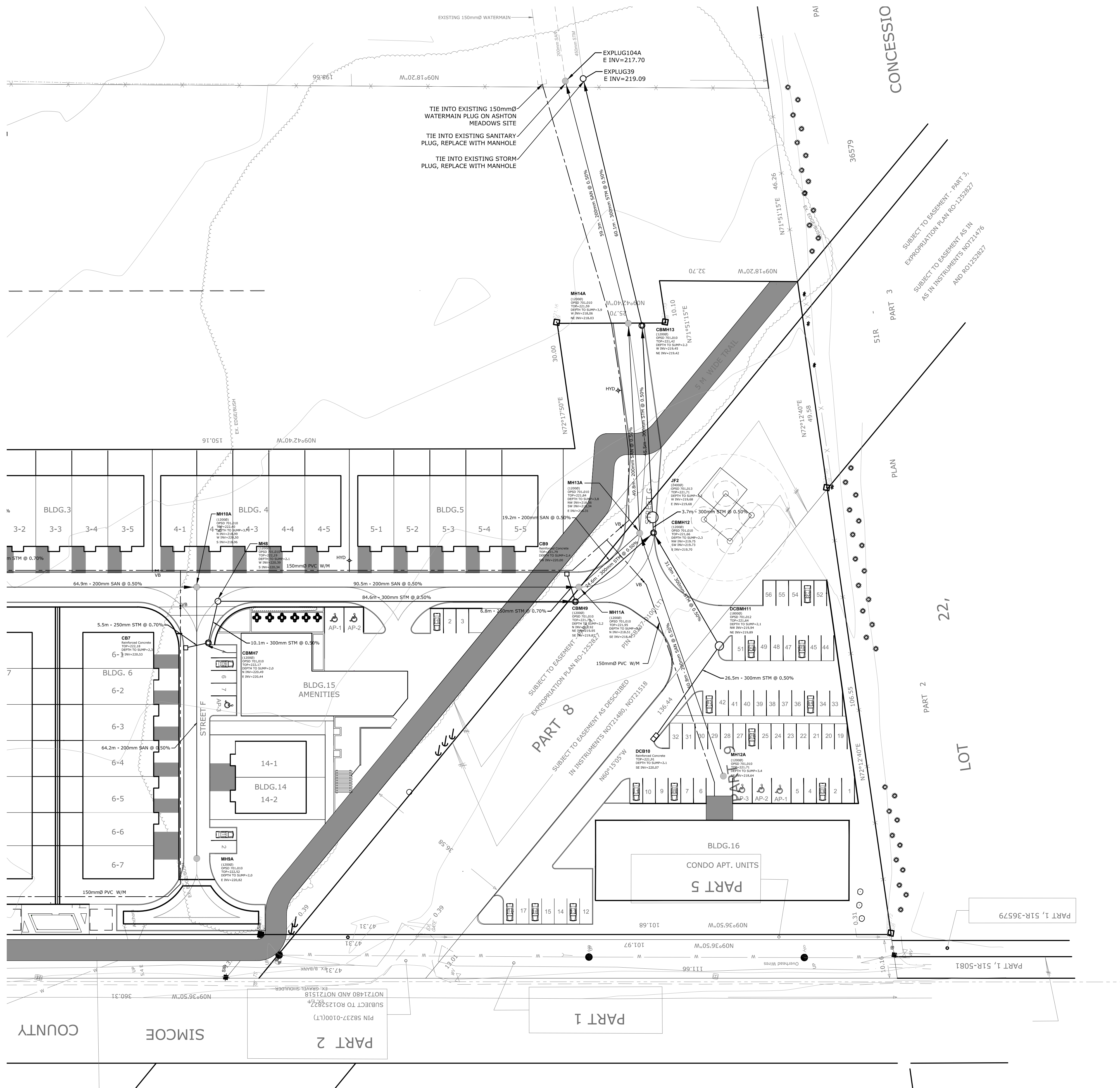
Client
MAMTA HOMES
44 ASTER DRIVE
WASAGA BEACH, ON



355310 BLUE MOUNTAINS - EUPHRASIA TOWNLINE
CLARKSBURG, ON N0W 1J0
TEL: 705-994-4818

MAMTA HOMES STAYNER
SERVICING PLAN (1 OF 2)

Designed K. GRIFFIN	Checked C. CAPES	Date 18/12/07	Drawing No.
Project No. 2018-060	Rev No. 1		C1
Scale 1:500	0 5.0 10.0 20.0 30.0m		



LEGEND

- STORM SEWER/MANHOLE
- SANITARY SEWER/MANHOLE
- WATERMAIN
- VALVE & BOX
- HYDRANT & VALVE
- EXISTING SANITARY SEWER
- EXISTING STORM SEWER
- EXISTING WATERMAIN

Notes

- This drawing is the exclusive property of CAPES Engineering Ltd. The reproduction of any part without express written consent of this Corporation is strictly prohibited.
- The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to CAPES Engineering Ltd. prior to construction.
- This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

No	Issue / Revision	Date	Auth
1	ISSUED FOR FSR	19/01/14	CC

Client
MAMTA HOMES
44 ASTER DRIVE
WASAGA BEACH, ON

CAPES
ENGINEERING

355310 BLUE MOUNTAINS - EUPHRASIA TOWNSHIP
CLARKSBURG, ON N0M 1J0
TEL: 705-994-4818

MAMTA HOMES STAYNER
SERVICING PLAN (2 OF 2)

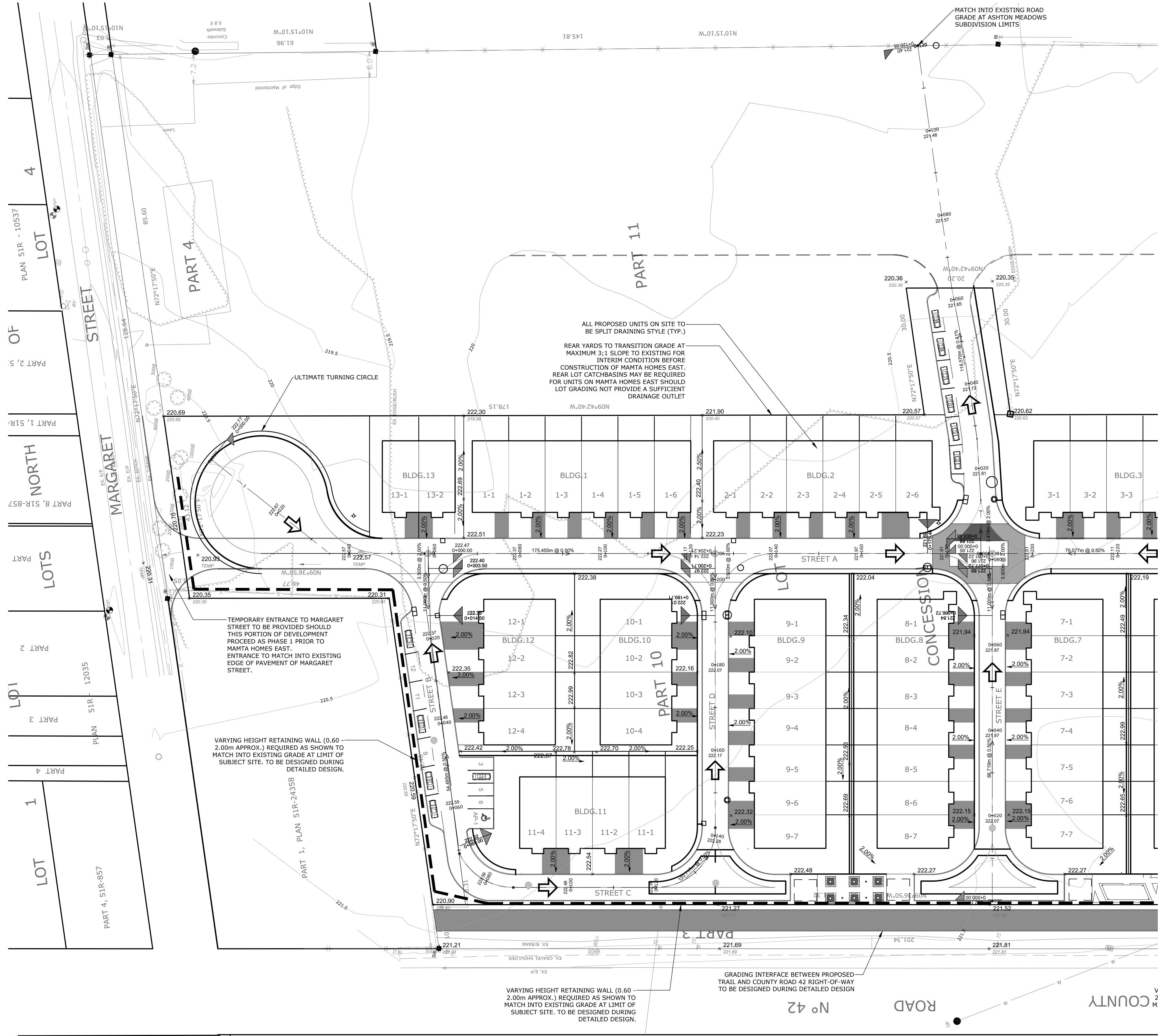
Designed K. GRIFFIN	Checked C. CAPES	Date 18/12/07
Project No. 2018-060	Rev No. 1	

Scale
1:500

0 5.0 10.0 20.0 30.0m

Drawing No.
C2

File: C:\Users\jv\Documents\capes engineering\Projects\2018-060 MAMTA HOMES\Drawings\03 Servicing Plan.dwg Date Plotted: January 9, 2017 9:45 AM



LEGEND

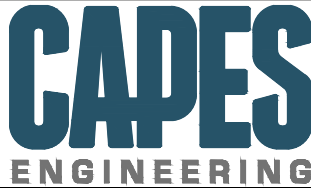
- PROPOSED ELEVATION
- EXISTING ELEVATION
- PROPOSED OVERLAND FLOW DIRECTION
- DIRECTION OF FLOW/PROPOSED SLOPE
- PROPOSED ROAD GRADE AND STATION
- HYDRANT & VALVE

Notes

- This drawing is the exclusive property of CAPES Engineering Ltd. The reproduction of any part without express written consent of this Corporation is strictly prohibited.
- The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to CAPES Engineering Ltd. prior to construction.
- This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

No	Issue / Revision	Date	Auth
1	ISSUED FOR FSR	19/01/14	CC

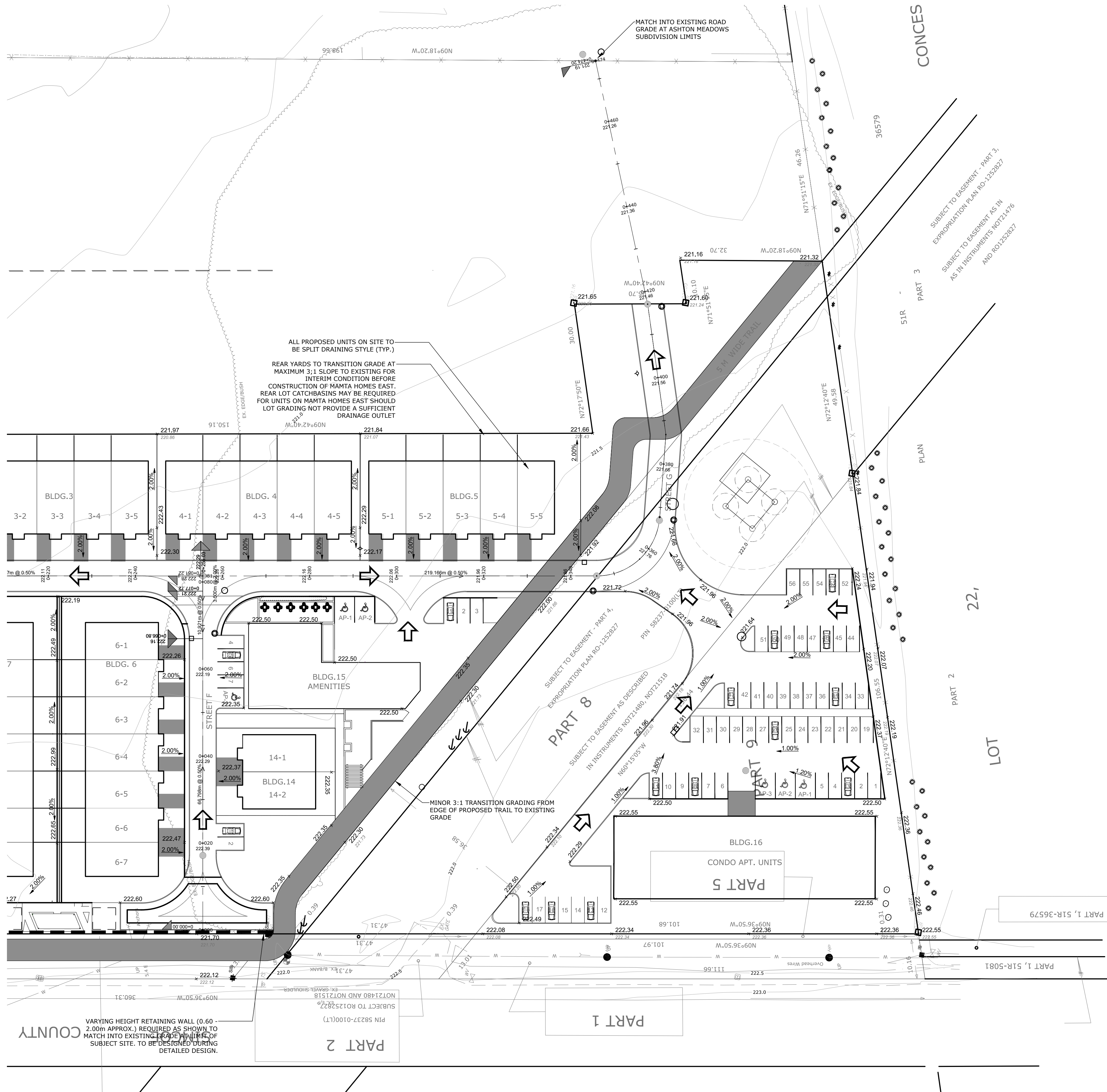
Client
MAMTA HOMES
44 ASTER DRIVE
WASAGA BEACH, ON



355310 BLUE MOUNTAINS - EUPHRASIA TOWNLINE
CLARKSBURG, ON N0W 1J0
TEL: 705-994-4818

MAMTA HOMES STAYNER
GRADING PLAN (1 OF 2)

Designed K. GRIFFIN	Checked C. CAPES	Date 18/12/07	Drawing No.
Project No. 2018-060	Rev No. 1		C3
Scale 1:500	0 5.0 10.0 20.0 30.0m		



LEGEND

- 221.21 PROPOSED ELEVATION
- 221.21 EXISTING ELEVATION
- PROPOSED OVERLAND FLOW DIRECTION
- 44.00m @ 1.84% DIRECTION OF FLOW/PROPOSED SLOPE
- PROPOSED ROAD GRADE AND STATION
- HYDRANT & VALVE
- MAXIMUM 3:1 SLOPE UNLESS OTHERWISE NOTED

- Notes
- This drawing is the exclusive property of CAPES Engineering Ltd. The reproduction of any part without express written consent of this Corporation is strictly prohibited.
 - The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to CAPES Engineering Ltd. prior to construction.
 - This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

No	Issue / Revision	Date	Auth
1	ISSUED FOR FSR	19/01/14	CC

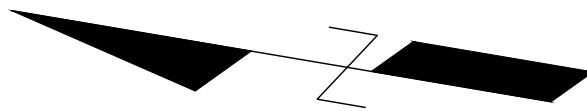
Client
MAMTA HOMES
44 ASTER DRIVE
WASAGA BEACH, ON



355310 BLUE MOUNTAINS - EUPHRASIA TOWNLINE
CLARKSBURG, ON N0W 1J0
TEL: 705-994-4818

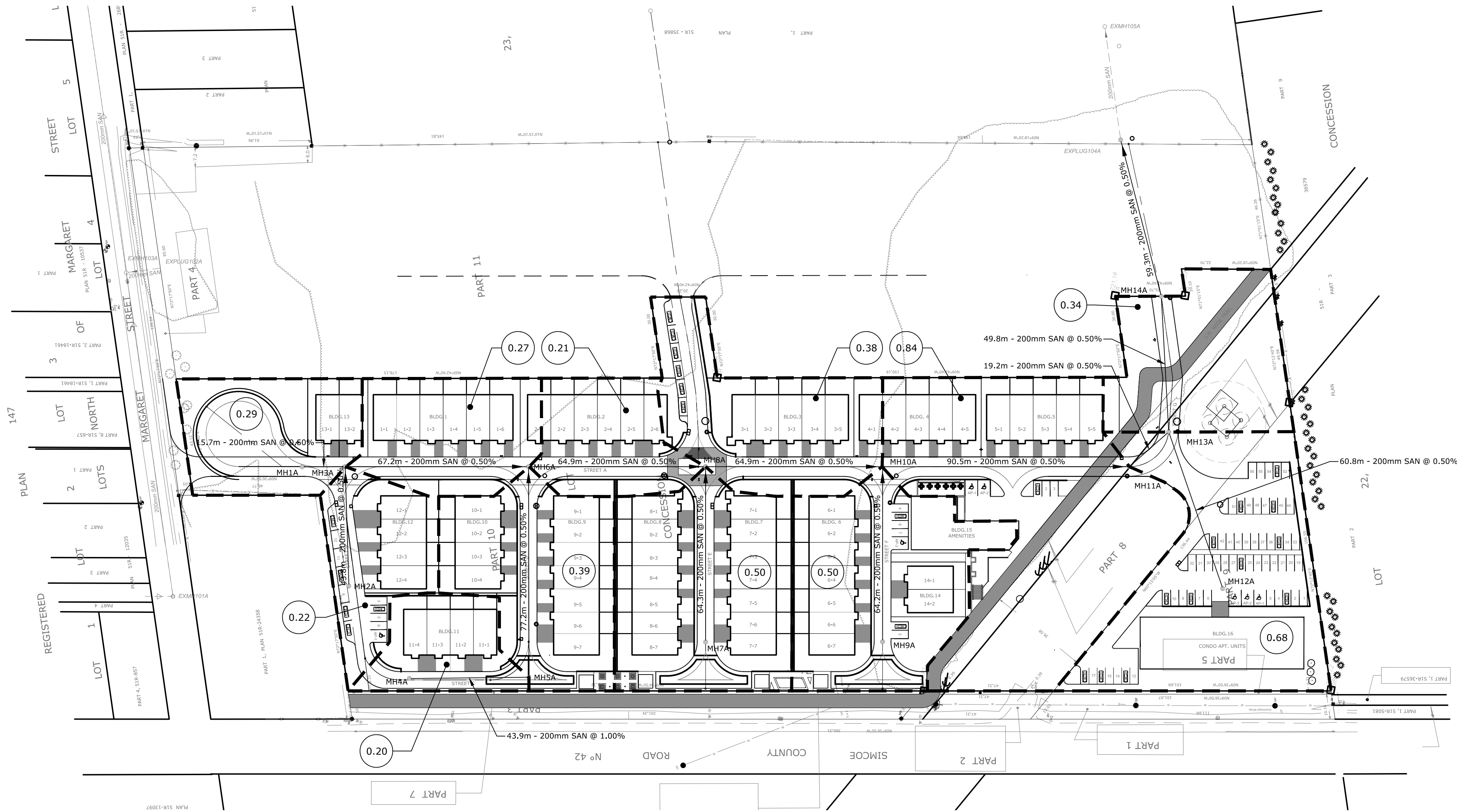
MAMTA HOMES STAYNER
GRADING PLAN (2 OF 2)

Designed K. GRIFFIN	Checked C. CAPES	Date 18/12/07	Drawing No.
Project No. 2018-060	Rev No. 1		C4
Scale 1:500	0 5.0 10.0 20.0 30.0m		



LEGEND

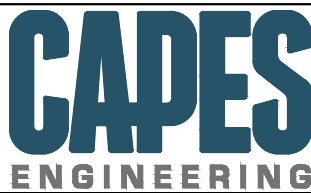
- 0.18 SANITARY DRAINAGE AREA (ha)
- SANITARY SEWER/MANHOLE
- EXISTING SANITARY SEWER
- SANITARY SEWER DRAINAGE BOUNDARY
- VALVE & BOX
- HYDRANT & VALVE



- Notes
- This drawing is the exclusive property of CAPES Engineering Ltd. The reproduction of any part without express written consent of this Corporation is strictly prohibited.
 - The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to CAPES Engineering Ltd. prior to construction.
 - This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

No	Issue / Revision	Date	Auth
1	ISSUED FOR FSR	19/01/14	CC

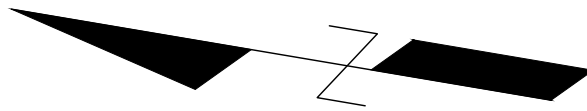
Client
MAMTA HOMES
44 ASTER DRIVE
WASAGA BEACH, ON



355310 BLUE MOUNTAINS - EUPHRASIA TOWNLINE
CLARKSBURG, ON N0W 1J0
TEL: 705-994-4818

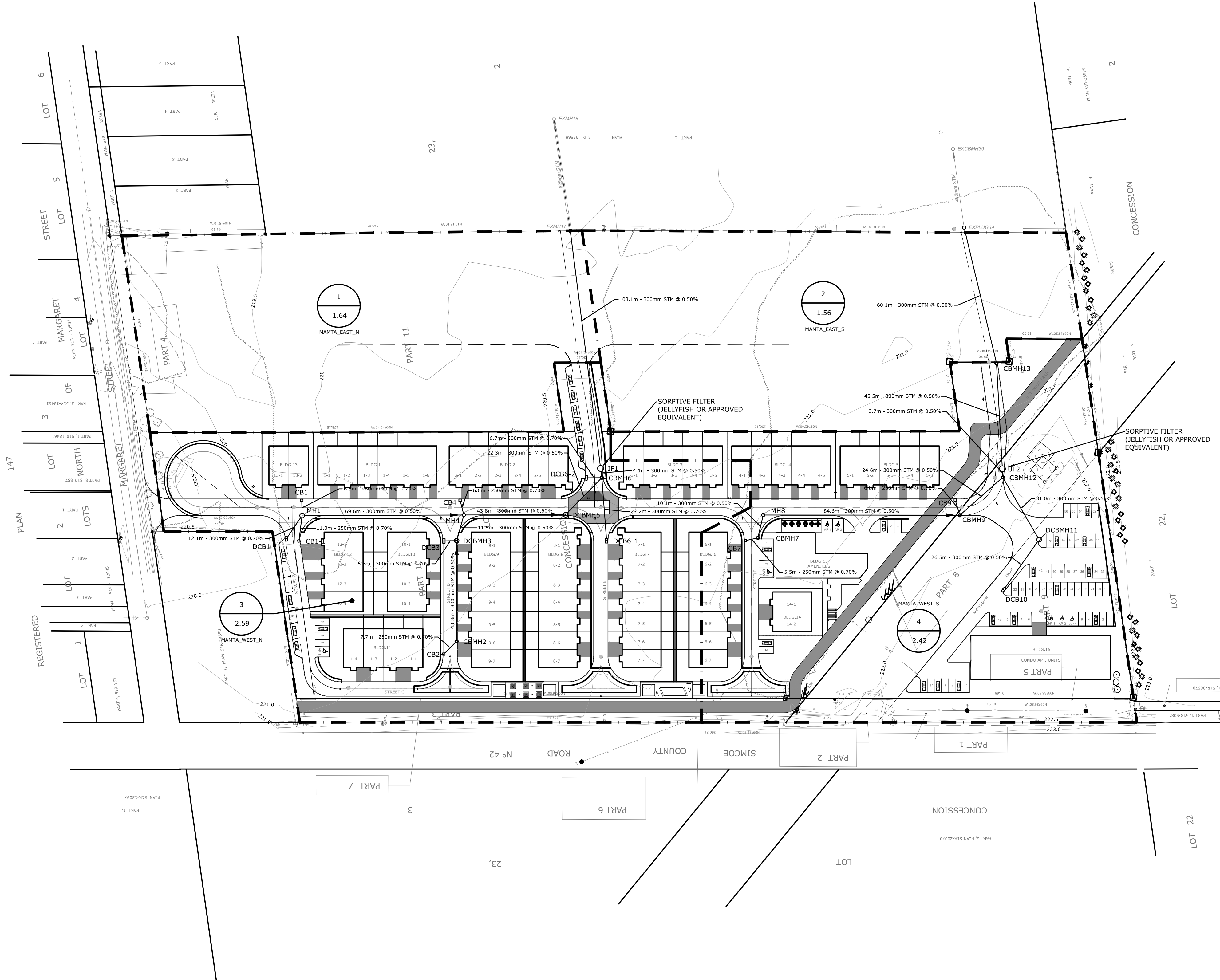
MAMTA HOMES STAYNER
SANITARY DRAINAGE AREA PLAN

Designed K. GRIFFIN	Checked C. CAPES	Date 18/12/07	Drawing No.
Project No. 2018-060	Rev No. 1		C5
Scale 1:1,000	0 20 40 60m		



LEGEND

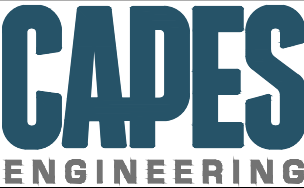
- STORM SEWER/MANHOLE
- DRAINAGE AREA ID
- DRAINAGE AREA (ha)
- POST DEVELOPMENT STORM DRAINAGE AREA PLAN
- VALVE & BOX
- HYDRANT & VALVE
- EXISTING STORM SEWER



- Notes
- This drawing is the exclusive property of CAPES Engineering Ltd. The reproduction of any part without express written consent of this Corporation is strictly prohibited.
 - The contractor shall verify all dimensions, levels, and datums on site and report any discrepancies or omissions to CAPES Engineering Ltd. prior to construction.
 - This drawing is to be read and understood in conjunction with all other plans and documents applicable to this project.

No	Issue / Revision	Date	Auth
1	ISSUED FOR FSR	19/01/14	CC

Client
MAMTA HOMES
44 ASTER DRIVE
WASAGA BEACH, ON



355310 BLUE MOUNTAINS - EUPHRASIA TOWNLINE
CLARKSBURG, ON N0M 1J0
TEL: 705-994-4818

MAMTA HOMES STAYNER
STORMWATER MANAGEMENT CATCHMENT AREAS

Designed K. GRIFFIN	Checked C. CAPES	Date 18/12/07	Drawing No.
Project No. 2018-060	Rev No. 1		C6
Scale 1:1,000			

Appendices

Appendix A – Site Plan

Notes:-

Do not scale drawings.

Contractor to verify all dimensions on site and report any errors and/or omissions to the architect in writing before starting any work.

NO.	DATE	ISSUES & REVISIONS
1.	16 JAN, 2019	TOWN SUBMISSION #1

INTEGRATED DESIGN & PROJECT MANAGEMENT :



RPD STUDIO

SUITE 201B, 7895 TRANMERE DR,
MISSISSAUGA, ON L5S 1V9

CLIENT:

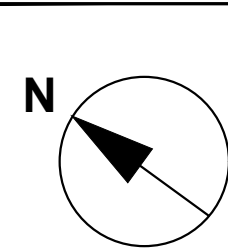


MAMTA DEVELOPMENTS

PROJECT & ADDRESS:

SENIOR HOUSING

1191 COUNTY ROAD 42,
STAYNER, ONTARIO



DRAWING TITLE

SITE PLAN

SCALE: 1:600

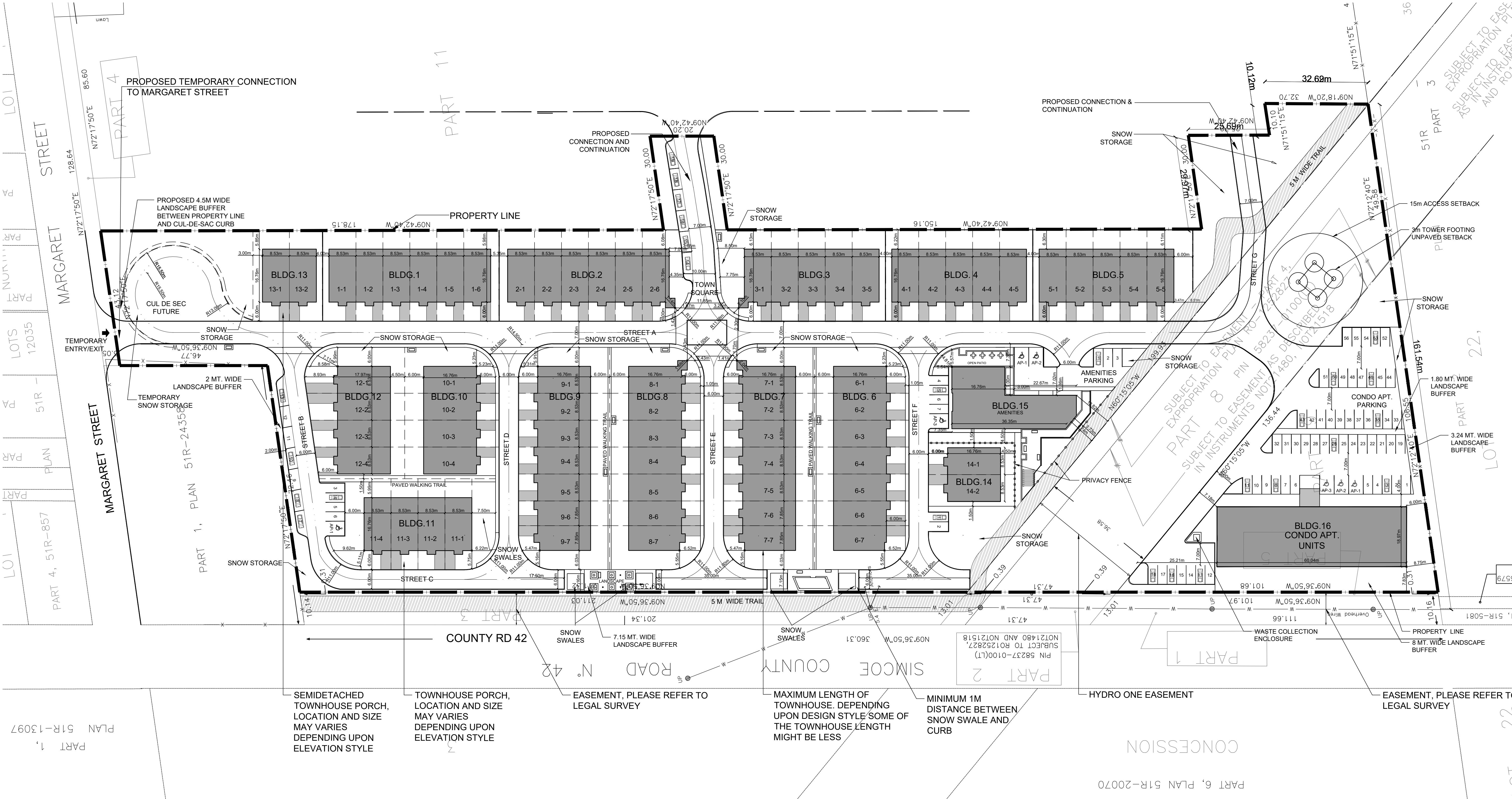
PROJECT: 1822

DATE: 2019-01-16

DRAWN BY: AZ

CHECKED BY: RP

A1

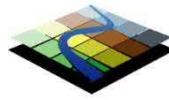


STATISTICS

ZONING	PROGRAMS	PARKING
ZONING: CURRENT ZONE - INR2 PROPOSED ZONE - REFER TO PLANNING JUSTIFICATION REPORT	TOWNHOUSE AND SEMI DETACHED UNITS: 8.53M TOWNHOUSE - 117.52Sq.Mt (1265Sq.Ft.) - 185.80Sq.Mt (2000Sq.Ft.) 7.85M TOWNHOUSE - 102.28Sq.Mt (1101Sq.Ft.) - 157.93Sq.Mt(1700Sq.Ft.) 8.53M SEMI DETACHED - 117.52Sq.Mt (1265Sq.Ft.) -185.80Sq.Mt (2000Sq.Ft.) AMENITY: AREA - 562.15 Sq.Mt. (6051 Sq.Ft.) CONDO APARTMENT: AREA PER FLOOR - 1139.27 Sq.Mt. (12263 Sq.Ft.) NO OF FLOORS - 3 FLOOR NO OF UNITS PER FLOOR (APARTMENT) - 12 UNITS	PARKING: • NO OF PARKING PER CONDO • TOWNHOUSE AND SEMI DETACHED UNIT - 2 (142) • PARKING FOR CONDO APARTMENT - 56+3 AP (59) • VISITOR PARKING - 26+4 AP (30) • (INCLUDING AMENITY AREA) TOTAL PARKING - 231

Appendix B – Environmental Information

January 15, 2019
Project No.: 1861



TERRASTORY
environmental consulting inc.

Mamta Homes
44 Aster Drive
Wasaga Beach, ON
L9Z 2Z8

Sent via email to: harjinder@mamtahomes.com

SUBJECT: Wetland Letter
1191 County Road 42 and adjacent parcel
Community of Stayner, Township of Clearview

Dear Harjinder,

Terrastory Environmental Consulting Inc. (hereinafter “Terrastory”) has prepared this letter in relation to two previously identified wetlands located at 1191 County Road 42 (hereinafter “Mamta West”) and an adjacent parcel with no civic address (hereinafter “Mamta East”) in Stayner. Both parcels are owned by Mamta Homes and are collectively referred to as the “Study Area”. The Study Area is situated on the south side of Stayner and is bounded by County Road 42 (west), Margaret Street (north), lands currently under development (east), and a warehouse facility (south). A 36.58 m wide Hydro easement bisects the southern portion of the Study Area. The Study Area is comprised of small wooded features and former agricultural lands undergoing succession towards shrubland/thicket. The location of the Study Area within its broader landscape setting is shown in **Figure 1**.

Mamta East is subject to a draft plan of subdivision approval granted in 2013 and extended in 2017, which includes a total of 69 single family and semi-detached residential units with a permanent road connection to Margaret Street. A plan of condominium application is being submitted to facilitate the development of Mamta West, which will include a total of 107 residential units associated with semi-detached, townhouse, and multistory apartment buildings. Infrastructure development (e.g., sanitary sewer, sewage pumping station, watermain, stormwater management, etc.) on the Mamta properties will be integrated with a larger development to the east (Ashton Meadows) which is currently under construction.

Two isolated wetlands were identified in the eastern portion of the Study Area during the completion of a previous Environmental Impact Statement (EIS) for Ashton Meadows. These two wetlands principally occur within Mamta East but extend slightly westward onto Mamta West. The location of the wetlands (as mapped in the Ashton Meadows EIS) is shown for reference in **Appendix 1**. Terrastory has not been provided with a copy of the Ashton Meadows EIS for review apart from the wetland map.

Development within the Study Area requires approval by the Nottawasaga Valley Conservation Authority (NVCA) pursuant to O. Reg. 172/06 given the presence of the aforementioned wetlands and wetlands >2 ha on the west side of County Road 42. During initial consultation with the project team, NVCA (A. Knapp) indicated that opportunities to replicate wetland functions on-site should

be considered as part of the stormwater management strategy which supports the development plans. Recent correspondence with NVCA regarding the wetlands is provided for reference in **Appendix 2**.

Overall, the purpose of this letter is to 1) characterize the form and function of the two identified wetlands, 2) anticipate wetland impacts associated with converting the lands to residential uses, and 3) demonstrate how opportunities to replicate lost wetland functions were considered as part of the stormwater management strategy by CAPES Engineering.

SITE ASSESSMENT AND WETLAND COMMUNITIES

A site assessment to review the two wetlands was carried out by Terrastory staff (T. Knight) on October 30, 2018. The site assessment centred on describing the anticipated physical (e.g., topography, drainage, surface water features, etc.) and biological (e.g., vegetation, wildlife, potential habitats, etc.) conditions and functions of the two wetlands to a general level. Representative photographs of the two wetland areas are provided in **Appendix 3**.

The Study Area is relatively flat. The topographic high is situated in the southwest corner of Mamta West along County Road 42 at an approximate elevation of 222.5 masl. The topographic low is situated in the northeast corner of Mamta East along Margaret Street at an approximate elevation of 219 masl. Surface runoff is thereby directed in a north/east direction.

Previous mapping of the “northern” wetland (per **Appendix 1**) is considered inaccurate. Much of this wooded feature contains an overstory of Scots Pine (*Pinus sylvestris*) intermixed with Trembling Aspen (*Populus tremuloides*) and is better described as a fresh-moist mixed deciduous forest. Based on a review of historical aerial photographs it is unclear if the Scots Pine were planted or established naturally; either way, this area lacked woody vegetation in 1954 and may have been used for agricultural purposes at that time. Notwithstanding the above, wetland conditions were observed along the edges (and through the centre) of the “northern” wetland as mapped in **Appendix 1**. Such wetlands are described as Red-osier Dogwood (*Cornus sericea*) dominated thicket swamp. Bebb’s Willow (*Salix bebbiana*) was observed occasionally, with the herbaceous layer dominated by Tall Goldenrod (*Solidago altissima*), Redtop (*Agrostis gigantea*), and other herbaceous species that could not be identified due to frost-related senescence.

Based on the conditions observed, the “northern” wetland is unlikely to contain standing water at a depth or duration sufficient to support breeding activities by wetland-specific wildlife (e.g., anurans, mole salamanders, bird species associated with standing water, etc.). Two boreholes (BH-8 and BH-9) advanced as part of the preliminary geotechnical investigation (Orbit Engineering Ltd. 2017) within or adjacent to the “northern” wetland revealed groundwater elevations at 30 cm below the ground surface in late March 2017 at the time of drilling. While the exact water transfer mechanisms (i.e., groundwater vs. surface water contributions, etc.) responsible for controlling wetland conditions in this area are unknown without more detailed information, it is likely that the northern wetland is predominantly maintained by a perched groundwater table (which is likely below ground for much of the year under average conditions).

Previous mapping of the “southern” wetland (per **Appendix 1**) was found to be reasonably accurate. The “southern” wetland is described herein as a Trembling Aspen dominated deciduous swamp. Red-osier Dogwood and Bebb’s Willow were abundant and attained greater dominance in more open portions of this community. Rough-leaved Goldenrod (*Solidago rugosa*), Late Goldenrod

(*Solidago gigantea*), Tall Goldenrod, and Meadow Horsetail (*Equisetum arvense*) were observed throughout the herbaceous layer. One small woodland pool was observed (see Photograph 8 in **Appendix 3**) with an approximate maximum depth of 5 cm at the time of the site assessment. It is unknown if this pool contains standing water at a depth and duration sufficient to support breeding amphibians, although its very small size would preclude the establishment of an abundant or significant breeding population.

The groundwater table appeared to be close to the surface within the “southern” wetland during the site assessment and was visible beneath the root systems of overturned trees (see Photograph 9 in **Appendix 3**). One borehole (BH-1) advanced as part of the preliminary geotechnical investigation within the “southern” wetland was fitted with a piezometer, with the depth to groundwater recorded at 10 cm in early April 2017.

Based on the information collected and described herein, both the “northern” and “southern” wetlands provide limited and non-significant wetland-specific functions and values within the local landscape. This determination is based on the following rationale:

- **Wetland-specific Wildlife Habitat:** The “northern” wetland is not anticipated to contain standing water at a depth or duration sufficient to support breeding activities by wetland-specific wildlife. A small woodland pool is present in the “southern” wetland, but its size and anticipated depth of standing water likely restricts potential breeding activities by wetland species.
- **Rare/Special Features:** Dogwood thicket swamps and Trembling Aspen deciduous swamps are common wetland communities across Simcoe County (and southern Ontario generally). Although a botanical inventory could not take place due to the timing of the site assessment, both wetlands appear to be quite young (i.e., area was formerly used for agricultural activities) and therefore unlikely to support a diversity of rare flora.
- **Hydrologic Functions:** Both wetlands appear to be hydrologically isolated; therefore, neither wetland provides outflows to adjacent watercourses or supports aquatic habitat. Both wetlands could provide short-term storage of surface water runoff (i.e., flood mitigation), but the effect is likely insignificant due to their small size/depth and the perched groundwater table. Both wetlands provide a groundwater recharge function, but the rate of which would be comparable to adjacent upland areas given the consistency of sandy/silty soil across the Study Area (as described in the preliminary geotechnical investigation by Orbit Engineering). No seeps or springs associated with discrete areas of groundwater discharge were observed.

DEVELOPMENT PLANS

As described above, Mamta East is draft plan approved and will contain 69 residential units. A draft plan of condominium application is being submitted for Mamta West and will contain 107 residential units. NVCA has requested that replication of lost wetland functions during conversion of the lands to residential uses be considered as part of the stormwater management strategy. To address NVCA’s concern, Terrastory has worked closely with the project engineer (CAPES Engineering) and project team to incorporate wetland values into the proposed development plans to the extent achievable.

There are several project- and site-specific constraints which inhibit the protection and/or replication of existing wetland functions (as described above) within the Study Area. First, the

approved draft plan for Mamta East does not recognize the wetlands as development constraints (i.e., via EP zoning, etc.). As a result, significant portions of the “northern” and “southern” wetlands (per **Appendix 1**) will be removed to create residential lots and construct the new road. The remaining portions of these wetlands on Mamta West will be much smaller and disturbed, further limiting their function.

Second, Terrastory understands that the entire Study Area will need to be raised (i.e., filled) by 1-2 m to install servicing (including sanitary). Raising the land profile is required to 1) connect the gravity-controlled sanitary sewer with Ashton Meadows (or Margaret Street), 2) provide stormwater overland flow routes to Ashton Meadows, and 3) ensure sufficient separation between the seasonally high groundwater table and finished floor elevations. Terrastory understands that the fill and earthworks necessary to service the site cannot be implemented in a way that maintains the existing wetlands.

Third, per previous approvals stormwater from the Mamta properties will be directed eastward to a stormwater management pond on Ashton Meadows. The lack of SWM features within the Mamta properties limits opportunities for recreating wetland functions on-site (e.g., through a constructed wetland design for the SWM pond, etc.).

Finally, the stormwater management design and water balance for Ashton Meadows (by Greenland Engineering 2017) was formulated to include the Mamta properties. Terrastory understands that the stormwater design assumes full build-out on the Mamta properties and does not incorporate the retention of any wetland areas. Terrastory also notes that portions of the “southern” wetland extending eastward onto Ashton Meadows were recently removed to facilitate preconstruction grading (see Photograph 12 in **Appendix 3**). This suggests that the overall planning and engineering approvals for the lands were advanced under the assumption that any wetland areas within the Mamta properties would be removed and not replaced.

REPLICATION OPPORTUNITIES AND CONCLUSIONS

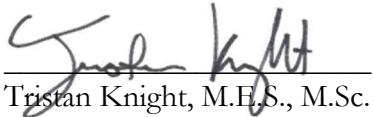
Per the above discussion, the “northern” and “southern” wetland areas were found to exhibit limited wetland values and functions within the local landscape. There are several project- and site-specific challenges that preclude either protecting the wetlands (given previous planning approvals on Mamta East, the need to raise the land profile by 1-2 m, etc.) or recreating their functions elsewhere on-site (given the lack of proposed SWM features within the Study Area).

Notwithstanding the above, it is noted that the proposed fill will have a higher saturated conductivity over existing conditions and will be top-dressed with 300 mm of topsoil. These measures will reduce the peak flow and volume of runoff by as much as 10% (CAPES Engineering 2018, p. 12) and were specifically incorporated into the stormwater design to address the lost groundwater recharge function provided by the existing wetlands. A more complete description of the proposed stormwater strategy is offered within the Functional Servicing & SWM Report (CAPES Engineering 2018).

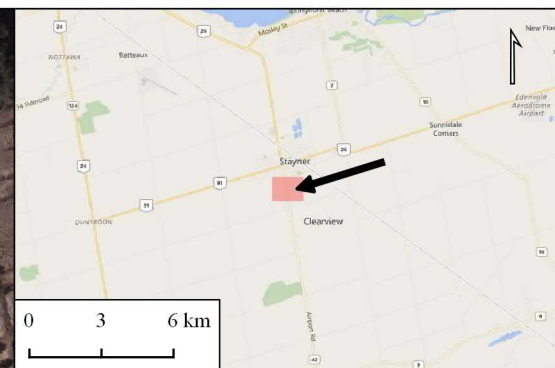
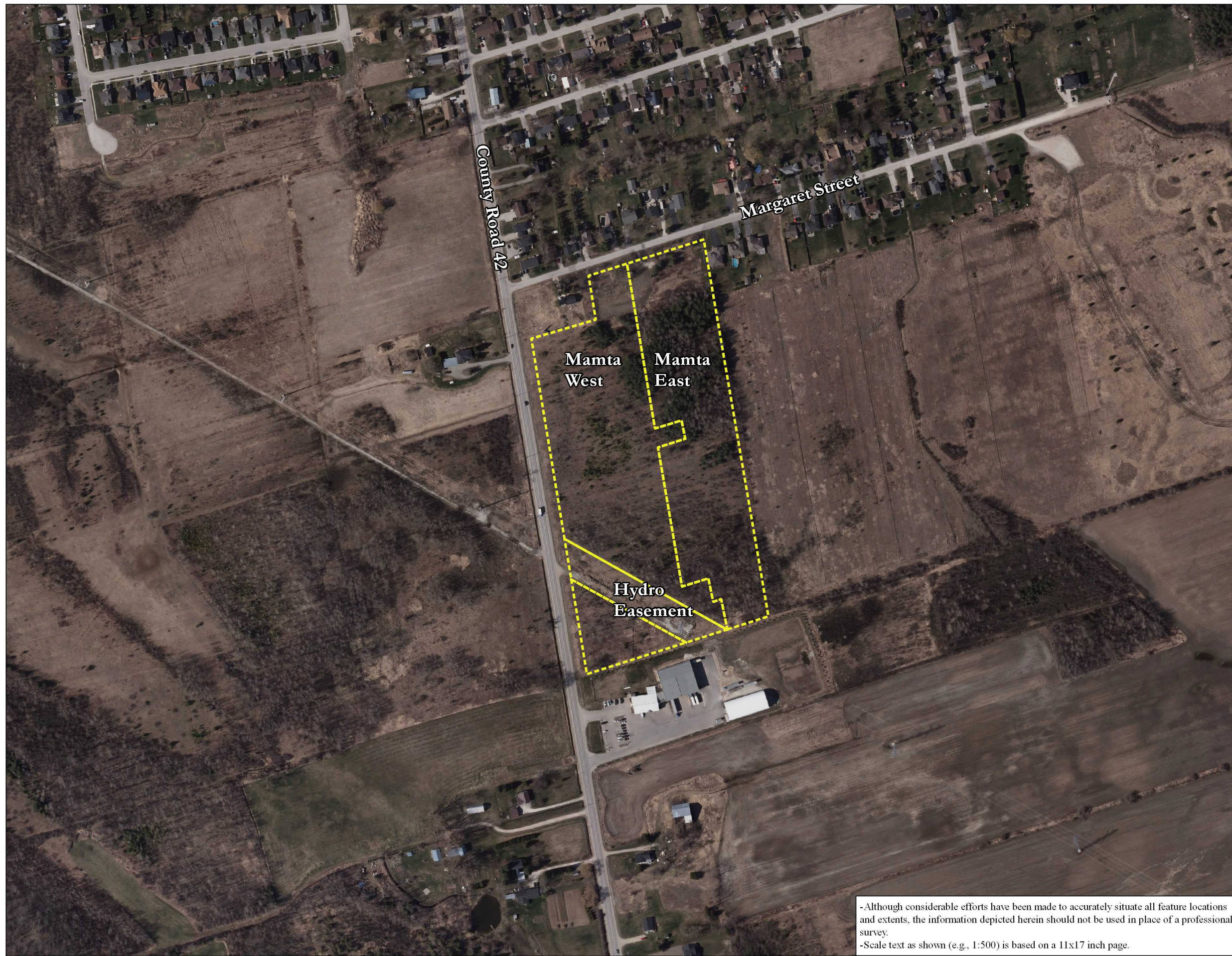
We trust that the information contained in this letter affords a sufficient characterization of the two previously identified wetlands and clarifies the ways in which the project team has considered wetland values as part of this application.

Regards,

Terrastory Environmental Consulting Inc.




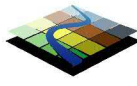
Tristan Knight, M.E.S., M.Sc.
Senior Ecologist / President



Legend

Study Area

 Subject Properties

**TERRASTORY**
environmental consulting inc.


1:4,000

0

50

100

150 m

	Project No.: 1861	By: TK	Date: 2018-12-12
---	----------------------	-----------	---------------------

Orthophotograph Date: 2016 (Simcoe Maps).

Location: 1191 County Road 42, Township of Clearview.

Figure 1. Location of the Study Area

-Although considerable efforts have been made to accurately situate all feature locations and extents, the information depicted herein should not be used in place of a professional survey.
-Scale text as shown (e.g., 1:500) is based on a 11x17 inch page.

Appendix 1. Wetlands as mapped in the Ashton Meadows
EIS



Appendix 2. Correspondence with NVCA

Tristan Knight

From: Amy Knapp <aknapp@nvca.on.ca>
Sent: November 2, 2018 4:18 PM
To: Tristan Knight
Subject: RE: 1191 County Road 42, Stayner

Good Afternoon Tristan,

I would agree with your direction...at the end of the day if the two wetland pockets are not kept due to the site design, we would like consideration to be given in providing 'compensation' (for lack of a better word) to make up for that loss, whether it can be considered through the SWM or enhancements to other existing wetland features that will remain.

If you have any further questions, please let me know.

Sincerely,

Amy Knapp | Planner II

Nottawasaga Valley Conservation Authority

8195 8th Line, Utopia, ON L0M 1T0
T 705-424-1479 ext.233 | **F** 705-424-2115
aknapp@nvca.on.ca | nvca.on.ca

This e-mail message, including any attachments, is for the sole use of the intended recipient(s) and may contain confidential and privileged information. Any unauthorized review, use, disclosure or distribution is prohibited. If you are not the intended recipient, please contact the sender and destroy all copies of the original message.

From: Tristan Knight [mailto:tristan@terrastoryenviro.com]
Sent: Thursday, November 1, 2018 2:33 PM
To: Amy Knapp <aknapp@nvca.on.ca>
Subject: RE: 1191 County Road 42, Stayner

Hi Amy, thanks for the quick response.

I was previously provided with and have reviewed your correspondence on this file dating back to May 1, 2018.

The project team has a strong interest in submitting the necessary development plans in a way that addresses NVCA requirements upfront. In this regard, I was retained to review the "two isolated wetland pockets" previously identified in the EIS for Ashton Meadows in order to characterize their anticipated functions to support the SWM strategy. I visited the area earlier this week.

The project team would greatly appreciate a bit more clarity in relation to what "replicate" means in the context of the SWM strategy. There is a suite of wetland conditions that could to a greater or lesser extent be replicated or even enhanced (e.g., water balance, water transfer mechanisms, substrate, wildlife habitat), but as you know SWM ponds and wetlands are somewhat different systems. I take your previous direction to mean that the SWM strategy should *consider* how the isolated wetland features functioned to the extent possible, and to articulate how such functions will (or cannot) be replicated (or enhanced) via the SWM design. Again our intent here is to minimize the need for major changes via resubmission by having a better understanding of the standards of review upfront.

Thanks again, and please feel free to reach out by phone or email.
T.

Tristan Knight M.E.S., M.Sc.
Senior Ecologist | President
Terrastory Environmental Consulting Inc.
(c) 905-745-5398
www.terrastoryenv.com

From: Amy Knapp <aknapp@nvca.on.ca>
Sent: November 1, 2018 2:08 PM
To: Tristan Knight <tristan@terrastoryenviro.com>
Subject: RE: 1191 County Road 42, Stayner

Good Afternoon Tristan,

In reply to your recent voicemail message and email, I was wondering if you could provide me some further detail via email in regards to your questions regarding the wetland features etc. so I may have the appropriate staff provide you with a response.

With respect to general questions on the review process, I have attached my response which I provided to Raj from RPD Studio.

If you have any further questions for me, please feel free to email me.

Sincerely,

Amy Knapp | Planner II

Nottawasaga Valley Conservation Authority
8195 8th Line, Utopia, ON L0M 1T0
T 705-424-1479 ext.233 | F 705-424-2115
aknapp@nvca.on.ca | nvca.on.ca

This e-mail message, including any attachments, is for the sole use of the intended recipient(s) and may contain confidential and privileged information. Any unauthorized review, use, disclosure or distribution is prohibited. If you are not the intended recipient, please contact the sender and destroy all copies of the original message.

From: Tristan Knight [<mailto:tristan@terrastoryenviro.com>]
Sent: Wednesday, October 31, 2018 1:14 PM
To: Amy Knapp <aknapp@nvca.on.ca>
Subject: 1191 County Road 42, Stayner

Good Afternoon Amy,

I just left you a message regarding this application. If you could give me a call back (number below) at your earliest convenience that would be greatly appreciated.

Thanks so much,
Tristan.

Tristan Knight M.E.S., M.Sc.
Senior Ecologist | President
Terrastory Environmental Consulting Inc.

Appendix 3. Representative Photographs



Photo 1. Eastern parcel within an area previously identified as wetland (northern) looking northeast towards a parcel under development (Ashton Meadows) (October 10, 2018).



Photo 2. Eastern parcel within an area previously identified as wetland (northern) looking east towards a parcel under development (Ashton Meadows) (October 10, 2018).



Photo 3. Red-osier Dogwood thicket swamp on the eastern parcel looking east (October 10, 2018).



Photo 4. Red-osier Dogwood thicket swamp on the eastern parcel looking east (October 10, 2018).



Photo 5. Red-osier Dogwood thicket swamp on the eastern parcel (October 10, 2018).



Photo 6. Red-osier Dogwood thicket swamp on the eastern parcel looking northeast (October 10, 2018).



Photo 7. Trembling Aspen deciduous swamp on the eastern parcel (October 10, 2018).



Photo 8. Small pool of standing water in the Trembling Aspen deciduous swamp (October 10, 2018).



Photo 9. Apparent groundwater table elevation in the Trembling Aspen deciduous swamp as exposed beneath an overturned tree (October 10, 2018).



Photo 10. Portions of the previously identified southern wetland removed to support development on lands to the east (Ashton Meadows) (October 10, 2018).

Appendix C – Geotechnical Report

DRAFT

**PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
209 Margaret Street, Stayner, Ontario**



Prepared for:

Mamta Developments Inc.

By:

Orbit Engineering Limited

Project No. OE17386AG

April 10, 2017



Orbit Engineering Limited
1900 Clark Boulevard, Unit 9, Brampton, ON, L6T 0E9
Tel: 1-905-494-0074 , Fax: 1-855-666-3355
www.orbitengineering.ca , info@orbitengineering.ca

Mamta Developemnts Inc.
54 Howell St,
Brampton, ON
L6Y 3H7
Email: harjinder@ojalabankers.com

Attention: Harjinder Kang

Dear Mr. Kang,

**RE: Preliminary Geotechnical Investigation
Proposed Residential Development
209 Margaret Street, Stayner, Ontario**

Enclosed please find the preliminary geotechnical investigation report related to the above noted site.

For and on behalf of Orbit Engineering Limited,

Hafiz Muneeb Ahmad, M.Sc., M.Eng., P.Eng. QP_{ESA}
Senior Principal Engineer

EXECUTIVE SUMMARY

A preliminary geotechnical investigation was carried out for the proposed residential development located at 209 Margaret Street, Stayner, Ontario. The project will entail a residential subdivision consisting of single family houses, roads and sewers.

The topsoil thickness generally ranged from 200mm to 400mm at the borehole locations. Thickness of topsoil may vary between and beyond the boreholes. The surficial topsoil was underlain by the following layers of native soils:

- The *Upper* native soil zone to depths ranging from 0.8 to 1.2m below the existing grade consisted of weathered/disturbed sandy silt to silty sand with some topsoil inclusions and rootlets, and trace gravel and clay.
- The *Lower* portion of native soil consisted of generally compact to very dense sandy silt to silty sand and/or till with trace to some gravel and clay, extending to maximum explored depth of 6.7m. The till deposit can contain occasional layers of sandy silt to silty sand as observed at the location of boreholes BH1 and BH2 located on the south portion of the property.

The groundwater levels in piezometer wells installed at the location of boreholes (BH1, BH3, BH7 and BH9) were measured on April 7, 2017 (after about 1 week of installation) at approximate depths of 0.1m to 2.1m below the existing grade (corresponding to geodetic elevations of 220.8m and 217.7m respectively). It should be noted that groundwater levels can vary and are subjected to seasonal fluctuations and can respond to major precipitation events. The depth of groundwater table can also be influenced by the presence of underground features such as utility trenches.

In light of borehole information, the proposed house foundations can be supported on undisturbed native soils at 0.8m to 1.2m below the existing levels for a geotechnical reaction of 100kPa (2000psf) at the Serviceability Limit States (SLS) and a factored geotechnical resistance of 150kPa at the Ultimate Limit States (ULS). These values would be suitable for the use of normal spread footing foundations to support normal single family dwellings.

Alternatively, the proposed structures can be supported by conventional spread and strip footings founded on engineered fill for a geotechnical reaction of 100kPa (2000psf) at the Serviceability Limit States (SLS) and a factored geotechnical resistance of 150kPa ULS. The engineered fill supporting footings should be constructed in accordance with the guidelines presented in Appendix C. Other requirements of engineered fill are given in Section 4.4.

The floor slab can be supported on grade, provided all topsoil, existing weathered/disturbed and surficially softened or loose materials are removed, and the subgrade thoroughly proof rolled. Any loose spots or areas revealed from proof rolling must be sub-excavated, backfilled and compacted.

Prior to the placement of the engineered fill, all of the existing weathered/disturbed and softened or loose native soils must be removed and the exposed surface proof rolled. The depths of sub-excavation required for the construction of engineered fill at the borehole locations ranged from 0.8m to 1.2m, as listed in **Table 4.2**.

Where the excavation base for engineered fill consists of sandy soils (sand or sandy silt to silty sand) below the groundwater level, dewatering will be required to lower the water table below the excavation base. It is possible to lower the groundwater table for about 0.6m to 1.0m by pumping from perimeter sumps and trenches.

Standard borings may not assess dewatering requirements for layered granular soils below the groundwater table. Prior to excavation, we strongly recommend that test pits be carried to further explore the groundwater and seepage conditions and to confirm the need for positive dewatering. A contractor specializing in dewatering should be retained to design the dewatering systems in the area where the excavations extend well into the sandy soils below the groundwater level, such as for the deep service trenches (if required).

Discussion and recommendations for the construction of roads, sewers, excavations and backfill, and stormwater management pond are presented in Section 4.

Based on the borehole information, the subject site for the proposed building structures can be classified as “Class D” for seismic site response.

TABLE OF CONTENTS

1	INTRODUCTION	1
2	FIELD AND LABORATORY WORKS	2
3	SITE AND SUBSURFACE CONDITIONS	2
3.1	Topsoil.....	3
3.2	Native Soils	3
3.2.1	Weathered/Disturbed Sandy silt to Silty sand.....	3
3.2.2	Sandy Silt to Silty Sand Till	3
3.3	Groundwater Conditions	4
4	DISCUSSION & RECOMMENDATIONS	5
4.1	Frost Susceptibility of Soils	5
4.2	Roads	5
4.2.1	Stripping, Sub-excavation and Grading	6
4.2.2	Construction	6
4.2.3	Drainage.....	7
4.3	Sewers	7
4.3.1	Trenching	7
4.3.2	Bedding	7
4.3.3	Backfilling of Trenches	8
4.3.4	Thrust Blocks	8
4.4	Engineered Fill and Sub-Excavation.....	10
4.5	House Foundation Conditions.....	12
4.6	Floor Slab and Permanent Drainage	13
4.7	Earth Pressures	15
4.8	Earthquake Considerations.....	15
4.9	Corrosivity Evaluation	15
4.10	Stormwater Infiltration	16
4.11	Stormwater Management Pond (SWM).....	17
4.11.1	General	17
4.11.2	Construction of SWM Pond.....	17
4.11.3	Liner Consideration.....	18
4.11.4	Slope Protection and Erosion Control.....	19
5	GENERAL COMMENTS	20
6	CLOSURE	21

TABLES

Table 3.1	Groundwater Levels Observed In Boreholes.....	4
Table 4.1	Reduction Factors For Thrust Blocks.....	9
Table 4.2	Depths Of Sub-Excavation For Engineered Fill Construction.....	10
Table 4.3	Estimated Permeability Coefficient For Native Soil Samples	16

DRAWINGS

Drawing 1	Approximate Site Location Plan
Drawing 1A	Approximate Borehole Location Plan
Drawing 1B	Notes on Sample Descriptions
Drawing 2-10	Borehole Logs
Drawing 11	Generalized Subsurface Profile at Borehole Locations
Drawing 12	Thrust Blocks
Drawing 13	Drainage and Backfill Recommendations without Underfloor Drainage
Drawing 14	Drainage and Backfill Recommendations with Underfloor Drainage

APPENDICES

Appendix A	Limitations of Report
Appendix B	Geotechnical Laboratory Test Results
Appendix C	General Requirements for Engineered Fill

1 INTRODUCTION

Orbit Engineering Limited (Orbit) was retained by Mamta Developments Inc. to undertake a preliminary geotechnical investigation for the proposed residential development located at 209 Margaret Street, Stayner, Ontario. The site plan and approximate location of the proposed development are shown on **Drawings 1** and **1A** respectively.

In light of the information provided to us by the client, it is our understanding that the project will entail a residential subdivision consisting of single family houses, roads and sewers.

The purpose of this preliminary geotechnical investigation was to obtain information about the subsurface conditions by means of a limited number of boreholes (nine boreholes BH1 to BH9) and from the findings in the boreholes to make recommendations pertaining to the geotechnical design of underground utilities and subdivision roads and to comment on the foundation conditions for general house construction.

This report contains the findings of the investigation, together with our recommendations and comments. The anticipated construction conditions are also discussed but only to the extent that they may affect the geotechnical design. The construction methods discussed express our opinion only and are not intended to direct contractors how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all factors that may have an effect upon construction.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with the applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for Mamta Developments Inc. and its designers. Third party use of this report without Orbit consent is prohibited. The limitation conditions presented in **Appendix A** form an integral part of the report and they must be considered in conjunction with this report.

2 FIELD AND LABORATORY WORKS

Prior to drilling operations, underground utilities were cleared at the borehole locations by representatives of the public utilities company working with personnel from Orbit.

A total of nine (9) boreholes (BH1 to BH9, see **Drawing 1A** for locations) were drilled on March 30-31, 2017 to a maximum depth of 6.7m with solid stem continuous flight augers by a drilling sub-contractor under the direction and supervision of Orbit personnel. Samples were retrieved with a 50mm O.D. split-barrel sampler driven with a hammer weighing 63.5kg and dropping 760mm in accordance with the Standard Penetration Test (SPT) method (ASTM D1586). The samples were logged in the field and returned to the Orbit's laboratory for detailed examination by the project engineer and for index laboratory testing.

As well as visual examination in the laboratory, all the soil samples were tested for moisture content and selected samples for grain size analyses.

Water level observations were made during drilling and in the open boreholes at the completion of the drilling operations. Piezometers (20mm) were installed in four (4) boreholes (BH1, BH3, BH7 and BH9) for an extended period of groundwater level monitoring.

The ground surface elevations at the borehole locations were interpreted from the topographical survey provided by the owner to Orbit. Note these elevations are approximate only for the purpose of relating borehole soil stratigraphy and should not be used or relied on for other purposes.

3 SITE AND SUBSURFACE CONDITIONS

The project site is at 209 Margaret Street, Stayner, Ontario. The site consists of a rectangle shaped vacant land (approximately 20 acres) located at the south-east corner of County Road 42 and Margaret Street (**Drawing 1**). The majority of the land is covered with vegetation including shrubs, grass and tall trees. A total of nine (9) boreholes (BH1 to BH9) were advanced at this site. The approximate borehole locations are shown on **Drawing 1A**. Notes on sample descriptions are presented on **Drawing 1B**. Detailed subsurface conditions are presented on the Borehole Logs, **Drawings 2 to 10**. The generalized subsurface profile is presented on **Drawing 11**.

The borehole logs indicate the subsurface conditions only at the borehole locations. Note the material boundaries indicated on the borehole logs are approximate and based on visual observations. These boundaries typically represent a transition from one material type to another and should not be regarded as an exact plane of geological change. It should be pointed out that the subsurface conditions will vary across this site. The subsurface soil conditions are summarized as follows.

3.1 Topsoil

The thickness of the topsoil explored in the boreholes generally ranged from 200 to 400mm. The data provided here pertaining to the topsoil thickness is confirmed at the borehole locations only, and may vary between and beyond the boreholes. This information is not considered to be sufficient for estimating topsoil quantities and associated costs.

3.2 Native Soils

The surficial topsoil layer was underlain by the following layers of native soils.

3.2.1 Weathered/Disturbed Sandy silt to Silty sand

The *Upper* native soil zone to depths ranging from 0.8 to 1.2m below the existing grade consisted of weathered/disturbed sandy silt to silty sand with some topsoil inclusions and rootlets, and trace gravel and clay. The weathered/disturbed soil layer was generally in very moist to wet, greyish brown to dark brown or grey color and in very loose to loose state.

3.2.2 Sandy Silt to Silty Sand Till

The *Lower* native soil below the weathered and/or disturbed zone generally consisted of sandy silt to silty sand till with trace to some gravel and clay, extending to maximum explored depth of 6.7m (excluding boreholes BH1 and BH2). The till deposit at the site can contain occasional layers of sandy silt to silty sand as observed at the location of boreholes BH1 and BH2. The deposit was generally very moist to wet, greyish brown to brownish grey or grey in color and in compact to very dense state. The measured moisture contents of these native deposits are shown on the borehole logs, which are generally less than twenty (20) but typically about ten (10) percent by weight.

Typical grain size distribution curves of seven (7) sandy deposit samples from different depths in boreholes BH1 through BH9 are given on **Figure B1** in **Appendix B** and show the following gradation:

Gravel:	0 – 23	Average:	11	%
Sand:	8 – 54	Average:	40	%
Silt:	21 – 86	Average:	43	%
Clay	2 – 13	Average:	6	%

The results indicate that the native deposit at the site can generally be classified as “sandy silt to silty sand with trace some gravel and clay”. Based on the Unified Soil Classification, the native deposit is called as “silty sand (SM)”.

3.3 Groundwater Conditions

During drilling and at the completion, the short term (not stabilized) groundwater was found in boreholes at shallow depths varying from 0.1 to 0.9m below the existing ground surface. The groundwater levels in piezometers installed at the location of boreholes (BH1, BH3, BH7 and BH9) were measured on April 7, 2017 (after about 1 week of installation) at approximate depths of 0.1m to 2.1m below the existing grade (corresponding to geodetic elevations of 220.8m and 217.7m respectively). This indicates that the stabilized water level measurements are consistent in general with our observations during the drilling and at the completion. The results are summarized in **Table 3.1** and shown on the borehole logs in **Drawings 2, 4, 8 and 10** in details.

Table 3.1 Groundwater Levels Observed in Boreholes

BH No.	Date of Drilling	Date of Water Measurement	Depth/Elevation of the Tip of Piezometer (m)	Depth/Elevation of Groundwater (m)	Piezometer
BH1	Mar 30, 2017	During drilling	6.7/214.2	0.1*/220.8	Yes
		Apr 7, 2017		0.1*/220.8	
BH2	Mar 30, 2017	During drilling	–	0.3*/221.2	–
BH3	Mar 30, 2017	During drilling	6.3/215.2	0.9*/220.6	Yes
		Apr 7, 2017		0.3/221.2	
BH4	Mar 30, 2017	During drilling	–	0.4*/220.1	–
BH5	Mar 30, 2017	During drilling	–	0.6*/220.2	–
BH6	Mar 30, 2017	During drilling	–	0.6*/219.9	–
BH7	Mar 31, 2017	During drilling	6.3/215.3	0.9*/220.7	Yes
		Apr 7, 2017		1.8*/219.8	
BH8	Mar 31, 2017	During drilling	–	0.3*/219.2	–
BH9	Mar 31, 2017	During drilling	6.7/213.1	0.3*/219.5	Yes
		Apr 7, 2017		2.1/217.7	

*groundwater table not stabilized

Perched water may be encountered in excavated areas during wet seasons. A perched water condition can develop within and above fine-grained materials especially during and following periods of sustained precipitation.

Note that the groundwater level can vary and is subject to seasonal fluctuations and in response to major weather events. The depth of groundwater table can also be influenced by the presence of underground features such as utility trenches.

4 DISCUSSION & RECOMMENDATIONS

It is proposed to develop the site as a residential subdivision. The lots therefore will be serviced by a network of roads, storm and sanitary sewers and watermain.

4.1 Frost Susceptibility of Soils

The frost depth penetration in this area is considered to be 1.5m. Based on the grain size analysis and using the Ministry of Transportation (MTO) category for frost susceptibility soils, the on-site native soils can be classified as low to moderate susceptible to frost heaving.

4.2 Roads

The investigation has shown that the predominant subgrade soil, after stripping the topsoil, very loose to loose weathered/disturbed silty and sandy soil and otherwise unsuitable subsoil, will generally consist of cohesionless soils. The stabilized groundwater table was found at depths varying from 0.1m to 2.1m below the existing grade.

Based on the above and assuming that traffic usage will be residential minor local or local, the following minimum pavement thickness is recommended:

- 40mm HL3 Asphaltic Concrete
- 65mm HL8 Asphaltic Concrete
- 150mm Granular 'A'
- 300mm Granular 'B'

For bus routes and collector roads, the following minimum pavement thickness is recommended:

- 40mm HL3 Asphaltic Concrete
- 80mm HL8 Asphaltic Concrete
- 200mm Granular 'A'
- 250mm Granular 'B'

These values may need to be adjusted according to the Town of Stayner Standards. The site subgrade and weather conditions (i.e. if wet) at the time of construction may necessitate the placement of thicker granular sub-base layer in order to facilitate the construction. Furthermore, heavy construction equipment may have

to be kept off the newly constructed roads before the placement of asphalt and/or immediately thereafter, to avoid damaging the weak subgrade by heavy truck traffic.

4.2.1 Stripping, Sub-excavation and Grading

The site should be stripped of all topsoil, weathered/disturbed native and any topsoil or otherwise unsuitable soils to the full depth of the roads, both in cut and fill areas.

Following stripping, the site should be graded to the subgrade level and approved. The subgrade should then be proof-rolled, in the presence of the Geotechnical Engineer, by at least several passes of a heavy compactor having a rated capacity of at least 8 tonnes. Any soft spots thus exposed should be removed and replaced by select fill material, similar to the existing subgrade soil and approved by the Geotechnical Engineer. The subgrade should then be re-compacted from the surface to at least 98% of its Standard Proctor Maximum Dry Density (SPMDD). The final subgrade should be cambered or otherwise shaped properly to facilitate rapid drainage and to prevent the formation of local depressions in which water could accumulate.

In view of the low to medium permeability of the subsoil, proper cambering and allowing the water to escape towards the sides (where it can be removed by means of subdrains) is considered to be beneficial for this project. Otherwise, any water collected in the granular sub-base materials could be trapped thus causing problems due to softened subgrade, differential frost heave, etc. For the same reason damaging the subgrade during and after placement of the granular materials by heavy construction traffic should be avoided. If the moisture content of the local material cannot be maintained at $\pm 2\%$ of the optimum moisture content, imported granular material may need to be used.

Any fill required for regarding the site or backfill should be select, clean material, free of topsoil, organic or other foreign and unsuitable matter. The fill should be placed in thin layers and compacted to at least 95% of its Standard Proctor Maximum Dry Density (SPMDD). The degree of compaction should be increased to 98% within the top 1.0m of the subgrade, or as per City Standards. The compaction of the new fill should be checked by frequent field density tests.

4.2.2 Construction

Once the subgrade has been inspected and approved, the granular base and sub-base course materials should be placed in layers not exceeding 200mm (uncompacted thickness) and should be compacted to at least 100% of their respective SPMDD. The grading of the material should conform to current OPS (Ontario Provincial Standards) Specifications.

The placing, spreading and rolling of the asphalt should be in accordance with OPS Specifications or, as required by the local authorities.

Frequent field density tests should be carried out on both the asphalt and granular base and sub-base materials to ensure that the required degree of compaction is achieved.

4.2.3 Drainage

All paved surfaces should be sloped to provide satisfactory drainage towards catch basins. Installation of full-length subdrains on all roads is recommended. The subdrains should be properly filtered to prevent the loss of (and clogging by) soil fines.

4.3 Sewers

As a part of the site development, a network of new storm and sanitary sewers is to be constructed.

4.3.1 Trenching

As indicated in the boreholes, the trenches will be generally dug through cohesionless soils (sand and silt or sandy silt to silty sand till) with trace of clayey materials.

The groundwater levels observed in the piezometer wells were at depths ranging from 0.1-2.1m below the existing grade. Where the anticipated trench base is below the groundwater level, positive dewatering such as well points will be required to lower the water table to at least 1.0m below the excavation base. Otherwise, it will result in an unstable base and flowing sides. Standard borings may not assess dewatering requirements for layered granular soils below the groundwater table. Prior to excavation, we strongly recommend that test pits be carried to further explore the groundwater and seepage conditions and to confirm the need for positive dewatering. A contractor specializing in dewatering should be retained to design the dewatering systems in the area where the excavations extend well into the sandy soils below the groundwater level, such as for the deep service trenches (if required).

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the compact to dense cohesionless deposits above the water table can be classified as Type 3 soil and very loose to loose deposits as type 4. Sandy soils below the water table can also be classified as Type 4.

As a general rule, the excavations in Type 3 soil can be carried out using minimum side slopes of (1 to 1.5)H : 1V. The excavations in Type 4 soils will require at a minimum, flatter side slopes of 3H to 1V. These slopes should be visually monitored for any movement especially if workers are present within the excavation. These temporary slopes should only be utilized for a short duration.

4.3.2 Bedding

The undisturbed compact cohesionless soils (sandy silt to silty sand till) can provide adequate support for the sewer pipes and allow the use of normal Class B type bedding. The recommended minimum thickness

of granular bedding below the invert of the pipes is 150mm. The thickness of the bedding may, however, have to be increased depending on the pipe diameter or in accordance with local standards or if wet or weak subgrade conditions are encountered, especially when the soil at the trench base level consists of wet, dilatant silts and sandy silts to clayey silt. The bedding material should consist of well graded granular material such as Granular 'A' or equivalent. After installing the pipe on the bedding, a granular surround of approved bedding material, which extends at least 300mm above the obvert of the pipe, or as set out by the local Authority, should be placed.

To avoid the loss of soil fines from the subgrade, uniformly graded clear stone should not be used below the granular bedding material, unless a suitable approved filter fabric (geotextile) is placed. The geotextile should extend along the sides of the trench and should be wrapped all around the poorly graded bedding material.

4.3.3 Backfilling of Trenches

Based on visual and tactile examination, the on-site excavated sandy deposits without topsoils and rootlets are generally considered to be suitable for re-use as backfill in the service trenches provided their moisture contents at the time of construction are at or near optimum. However, the silts are poorly graded soils and are very sensitive to their moisture contents. As such, they will be very difficult to handle and to compact, especially when excavated below the water table. Under unfavourable conditions, they may not be suitable for trench backfill.

The backfill should be placed in maximum 200mm thick layers at or near ($\pm 2\%$) their optimum moisture content and each layer should be compacted to at least 95% SPMDD. Unsuitable materials such as organic soils, boulders, cobbles, frozen soils, etc. should not be used for backfilling.

The on-site excavated soils may not be used in confined areas (e.g. around catch basins and laterals under roadways) where heavy compaction equipment cannot be operated. The use of imported granular fill together with an appropriate frost taper would be preferable in confined areas and around structures, such as catch basins.

In light of borehole information, it is recommended that underground services should be kept as high as possible to avoid penetrating the excavation below the wet cohesionless deposits.

4.3.4 Thrust Blocks

Pressurized fluids in buried pipelines generate unbalanced, thrust forces at bends, junctions, valves pump starts or stops, valve closures, air vents and all restrictions to, and changes in direction of flows. Generally, the thrust forces depend on the internal pressure, the cross sectional area of the pipe and the deflection angle. For pipes which are not anchored, the unbalanced thrust forces must be resisted either by thrust blocks and collars or by thrust restraint systems or a combination of both.

Thrust blocks are passive systems which prevent the pipe joint leaking by blocking the pipe movements and the separation of unrestrained joints. Depending on the source of the thrust force, their resistance comes either from the mobilization of soil bearing capacity or dead weight: the bearing type thrust blocks resist thrust forces corresponding to concave vertical and horizontal bends, while the gravity ones secure the convex vertical bends. Because they need to immobilize the pipes, the allowable soil stresses must be considerably smaller than those required to cause ultimate failure of the thrust block itself. The thrust block design is satisfactory if the design force, F_d , is less than the ultimate resistance R_{ult} , reduced by a suitable reduction (safety) factor which will ensure that the displacements will be relatively small.

Values for thrust reduction factors for thrust blocks are given in **Table 4.1** for different soil and rock types. If these lead to unacceptably large thrust blocks, the reduction factor may be re-assessed by determining the actual relationship between thrust reduction factor and displacement under defined load and ground conditions.

Table 4.1 Reduction Factors for Thrust Blocks

Soil or Rock Type	Reduction Factor (T_r)
Dense sandy silt to silty sand deposit	2 to 3
Compact sandy silt to silty sand deposit	3 to 4
Very loose to loose sandy silt to silty sand deposit	4 to 5

Thrust blocks normally consist of a volume of concrete, usually of nominal strength (20-40 MPa), which may be lightly reinforced. The size and shape of the block is decided on the basis of the forces to be restrained, the size and style of the pipe fitting or component, and local ground conditions. The effectiveness of any thrust block is determined by its mass, shape, position relative to the pipeline, the soil reactions on the block, and friction between the pipeline and the surrounding ground.

Thrust blocks for the underground services under pressure may be constructed in native soils in areas where there is no risk of future excavations. The back of the thrust blocks should be vertical and should be cast directly against undisturbed natural soils. The ultimate lateral resistance of thrust blocks can be calculated in accordance with **Drawing 12**.

Thrust restraint systems are alternative to thrust blocks. They are active systems in the sense that they rely on the mobilization of pipe/soil friction and/or passive resistance in the soil for a sufficient length away from the junction. The length of pipeline required to develop the resisting force crucially depends on the type of junction, pipeline material, type and compaction/consistency of the backfill, etc.

4.4 Engineered Fill and Sub-Excavation

The elevation of the existing grade varies across the site. Detailed site grading plans for the proposed development were not available to us at the time of preparation of this report. However based on the existing topography at the site, cut and fill operations are expected to require as part of the proposed development.

In the areas where earth fill is required for site grading purposes, engineered fill can be constructed below house foundations, roads, boulevards, etc. Prior to the placement of the engineered fill, all of the existing topsoil and surficially weathered/disturbed native soils must be removed and the exposed surface proof rolled. Any soft spots revealed during proof rolling must be sub-excavated and re-engineered. The depths of sub-excavation required for the construction of engineered fill at the borehole locations approximately ranged from 0.8m to 1.2m, as listed in **Table 4.2**.

The stabilized groundwater levels observed in boreholes were at depths ranging from 0.1m to 2.1m below the existing ground surface (refer to Table 3.1). Where the excavation base for engineered fill consists of cohesionless soils (sand or sandy silt to silty sand) below the groundwater level, dewatering will be required to lower the water table below the excavation base. It is possible to lower the groundwater table for about 0.6m to 1.0m by pumping from perimeter sumps and trenches.

Where the excavations extend well into the cohesionless soils (sandy silt to silty sand) below the groundwater level, such as for the deep service trenches, a positive dewatering system such as well points will be required to lower the water table below the excavation base.

Table 4.2 Depths of Sub-Excavation for Engineered Fill Construction

Borehole No.	Depth of Sub-Excavation of weathered/disturbed Materials (m)	Depth of Observed Groundwater (m)
BH1/Piezo.	1.2	0.1 (after about 1 week)
BH2	0.8	0.3 (at the completion of BH)
BH3/Piezo.	0.8	0.3 (after about 1 week)
BH4	0.8	0.4 (at the completion of BH)
BH5	0.8	0.6 (at the completion of BH)
BH6	0.7	0.6 (at the completion of BH)
BH7/Piezo.	0.8	1.8 (after about 1 week)
BH8	0.8	0.3 (at the completion of BH)
BH9/Piezo.	0.8	2.1 (after about 1 week)

It is however highly prudent that all footings and underground utilities be placed at elevations as high as possible to avoid penetration into wet native sandy deposit and required dewatering systems.

General guidelines for the placement and preparation of engineered fill are presented on **Appendix C**. A geotechnical reaction 100kPa at the Serviceability Limit States (SLS) and factored geotechnical resistance 150kPa at the Ultimate Limit States (ULS) can be used on engineered fill, provided that all requirements on **Appendix C** are adhered to. To reduce the risk of improperly placed engineered compacted fill, full-time supervision of the contractor is essential. Despite full time supervision, it has been found that contractors frequently bulldoze loose fill into areas and compact only the surface. The inspector, either busy on other portions of the site or absent during “off hours” will be unaware of this condition. For this reason, we cannot guarantee the performance of the engineered fill, and this guarantee must be the responsibility of the contractor. The owner and his representatives must accept the risk involved in the use of engineered fill and offset this risk with the monetary savings of avoiding deep foundations. This potential problem must be recognized and discussed at a pre-construction meeting. Procedures can then be instigated to reduce the risk of settlement resulting from un-compacted fill.

In the areas where earth fill is required for site grading purposes, an engineered fill may be constructed below house foundations, roads, boulevards, etc.

The following is a recommended procedure for engineered fill:

1. Prior to site work involving engineered fill, a site meeting to discuss all aspects must be convened. The surveyor, contractor, design engineer and geotechnical engineer must attend the meeting. At this meeting, the limits of the engineered fill will be defined. The contractor must make known where all fill material will be obtained and samples must be provided to the geotechnical engineer for review, and approval before filling begins.
2. Detailed drawings indicating the lower boundaries as well as the upper boundaries of the engineered fill must be available at the site meeting and be approved by the geotechnical engineer.
3. The building footprint and base of the pad, including basements, garages, etc. must be defined by offset stakes that remain in place until the footings and service connections are all constructed. Confirmation that the footings are within the pad, service lines are in place, and that the grade conforms to drawings, must be obtained by the owner in writing from the surveyor and Orbit Engineering Limited. Without this confirmation no responsibility for the performance of the structure can be accepted by Orbit Engineering Limited. Survey drawing of the pre and post fill location and elevations will also be required.
4. The area must be stripped of all topsoil and weathered/disturbed materials. Subgrade must be proof rolled. Soft spots must be dug out. The stripped native subgrade must be examined and approved by Orbit Engineering Limited engineer prior to placement of fill.

5. The approved engineered fill must be compacted to 100% Standard SPMDD throughout. Granular Fill preferred. Engineered fill should not be placed (where it will support footings) during the winter months. Engineered fill compacted to 100% SPMDD will settle under its own weight approximately 0.5% of the fill height and the structural engineer must be aware of this settlement. In addition to the settlement of the fill, additional settlement due to consolidation of the underlying soils from the structural and fill loads will occur.
6. Full-time geotechnical inspection by Orbit Engineering Limited during placement of engineered fill is required. Work cannot commence or continue without the presence of the Orbit representative.
7. The fill must be placed such that the specified geometry is achieved. Refer to sketches for minimum requirements. Take careful note that the projection of the compacted pad beyond the footing at footing level is a minimum of 2m (**Appendix C**). The base of the compacted pad extends 2m plus the depth of excavation beyond the edge of the footing.
8. A geotechnical reaction of 100kPa (2000psf) may be used provided that all conditions outlined above are adhered to. A minimum footing width of 500mm (20 inches) is suggested and footings should be provided with nominal steel reinforcement.
9. All excavations must be done in accordance with the Occupational Health and Safety Regulations of Ontario.
10. After completion of the pad, a second contractor may be selected to install footings. All excavations must be backfilled under full time supervision by Orbit to the same degree as the engineered fill pad. Surface water cannot be allowed to pond in excavations or to be trapped in clear stone backfill. Clear stone backfill can only be used with the approval of Orbit.
11. After completion of compaction, the surface of the pad must be protected from disturbance from traffic, rain and frost.
12. If there is a delay in construction, the engineered fill pad must be inspected and accepted by the geotechnical engineer. The location of the structure must be reconfirmed that it remains within the pad.

The native soils are considered suitable for use as engineered fill, provided that they comprise no topsoils and rootlets and their moisture contents at the time of construction are at or near optimum. The silts are poorly graded soils and are very sensitive to their moisture contents. As such, they will be very difficult to handle and to compact, especially at wet conditions. Under unfavourable conditions, they may not be suitable for engineered fill as mentioned in Section 4.3.3.

4.5 House Foundation Conditions

The proposed house foundations can be supported on undisturbed native soils at 0.8m to 1.2m below the existing levels for a geotechnical reaction of 100kPa (2000psf) at the Serviceability Limit States (SLS) and a

factored geotechnical resistance of 150kPa at the Ultimate Limit States (ULS). These values would be suitable for the use of normal spread footing foundations to support normal single family dwellings. Footings should be kept as high as possible to avoid penetrating into wet sandy soils.

Alternatively, the proposed structures can be supported by conventional spread and strip footings founded on engineered fill for a geotechnical reaction of 100kPa (2000psf) at the Serviceability Limit States (SLS) and a factored geotechnical resistance of 150kPa ULS. The engineered fill supporting footings should be constructed in accordance with the guidelines presented in **Appendix C**. Other requirements of engineered fill are given in Section 4.4.

Variations in the soil conditions are expected in between the borehole locations, and during construction, the soil bearing pressures should be confirmed by the Geotechnical Engineer.

The base of all footings must be inspected by this office to ensure of their placement on the competent native soil.

Foundations designed to the specified bearing values are expected to settle less than 25mm total and 20mm differential.

All footings exposed to seasonal freezing conditions must have at least 1.5m of soil cover for frost protection.

Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper footing.

It should be noted that the recommended bearing capacities have been calculated by Orbit Engineering Limited from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to conditions between test pits and boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by Orbit Engineering Limited to validate the information for use during the construction stage.

4.6 Floor Slab and Permanent Drainage

The floor slab can be supported by engineered fill, if engineered fill is used to support the foundations.

The weathered/disturbed sand and silt layer present on the site is not suitable for supporting the slab-on-grade. The floor slab can be supported on grade, provided all topsoil, existing weathered/disturbed and surficially softened or loose materials are removed, and the subgrade thoroughly proof rolled. Any loose

spots or areas revealed from proof rolling must further be sub-excavated and replaced with imported Granular A and/or Granular B Type 2.

The imported granular material must meet the specifications defined in OPSS-1010-13. The existing weathered/disturbed soil free from topsoil and rootlets may be used to raise the grade, provided it is confirmed by a qualified geotechnical professional from Orbit at the time construction. The fill required to raise the grade must be placed in shallow lifts (each lift not more than 200mm) and compacted to at least 98 percent of Standard Proctor Maximum Dry Density (SPMDD).

A moisture barrier consisting of at least 200mm thick layer of well compacted 19mm clear crushed stone is recommended to place directly under the floor slab. The stone bed would act as a barrier and prevent capillary rise of moisture from the subgrade to the floor slab. This moisture barrier has been proven to be effective for conventional floor surfaces such as carpet, vinyl tile and ceramic tile. However, if special floor coverings such as sheet P.V.C. with heat sealed seams, as is used in gymnasiums, is considered, either a high efficiency vapour barrier or venting may be required to prevent moisture accumulating between the concrete floor and the P.V.C. flooring.

The estimated modulus of subgrade reaction (k_s) equal to 25 MN/m³ may be used for the design of slab-on-grade supported on native or structural fill soils, provided that the construction is in accordance with the recommendations provided herein. If structural fill (Granular A or B Type II) having minimum thickness of 300mm, this value can be increased to 30 MN/m³. The estimated value provided above may need to be adjusted based on the structure size and locations of detail design.

The base floor slabs should not be tied to any load-bearing walls or columns unless they have been designed accordingly. Contraction/expansion joints should be provided for the slabs as required by the structural engineer.

If the base floor slab is more than about 200mm higher than the exterior grade, then perimeter drainage is not considered to be necessary. If the floor is lower, then use of a perimeter drainage system (**Drawing 13**) is recommended.

The perimeter and under floor drainage system shown on **Drawing 14** is recommended for the basement area along the entire perimeter. The first row of the underfloor weeper must be placed close to the perimeter wall. From there-on, the underfloor weepers should be placed in parallel rows not more than 8m centres one way. The placement of perimeter and underfloor drainage systems will be subjected to approval of concerned authorities having jurisdiction over the project site.

Where the exposed subgrade in the basements consists of cohesionless soil below the water table, all openings including the subgrade and permanent drainage systems must be covered or wrapped with filter

fabric, typically a Class II non-woven textile with a filtration opening size (F.O.S.) of 0.1mm to 0.3mm. The design of permanent drainage systems should be reviewed by this office prior to the construction.

4.7 Earth Pressures

The lateral earth pressures acting on retaining walls (if any) may be calculated from the following expression:

$$p = K (\gamma h + q)$$

where:

- p : Lateral earth pressure in kPa acting at depth z
- K : Earth pressure coefficient equal to 0.4 for vertical walls and horizontal backfill used for permanent construction. Water pressure must be considered, if continuous wall drains are not used.
- γ : Unit weight of backfill, a value of 20.5 kN/m³ may be assumed
- z : Depth to point of interest in meters
- q : Equivalent value of surcharge on the ground surface in kPa

The above expression assumes that the perimeter drainage system prevents the buildup of any hydrostatic pressure behind the wall.

4.8 Earthquake Considerations

Based on boreholes information and according to the 2012 Ontario Building Code (OBC 2012), the subject site seismic response for the proposed building structures can be classified as “Class D” (Table 4.1.8.4.A of OBC 2012). Accordingly, the foundation factors F_a can be obtained from Table 4.1.8.4.B and F_v from Table 4.1.8.4.C of the OBC for the design of the buildings.

Consideration can be given to conduct an earthquake site assessment with the use of in-situ testing of the seismic characteristics (i.e. Geophysical testing – Multi-channel Analysis of Surface Waves “MASW”), which can lead to an improved site classification (i.e., from Class D to Class C).

4.9 Corrosivity Evaluation

Three (3) selected samples were submitted for corrosivity analysis to assess the aggressiveness of soil. The test results will be presented in the final report.

4.10 Stormwater Infiltration

Grain size analysis were carried out on selected seven (7) soil samples at specified locations (as explained earlier in section 3.2 and presented in **Appendix B, Figure B1**).

For preliminary estimation purpose, the permeability of the native sandy silt till samples can be estimated using the following expression (Puckett et al, 1985, 1992) which is related to the typical clay size particle content from the results of grain size analyses:

$$k = 4.36 \times 10^{-3} \times e^{(-0.1975 \times \% \text{clay})}$$

where:

k = permeability (cm/s);

% clay = percentage of clay size particles

The results are shown in **Table 4.3**, which show the range of permeability as 3.3E-04 to 2.9E-03 cm/sec.

Table 4.3 Estimated Permeability Coefficient for Native Soil Samples

Sample No.	Soil Type	Depth (m)	Permeability Coefficient, k (cm/sec)
BH1 – SS2	Native Sandy silt to silty sand	0.8 – 1.4	2.0E-03
BH1 – SS4	Native Sandy silt to silty sand	2.3 – 2.9	2.9E-03
BH1 – SS6	Native Sandy silt to silty sand	4.6 – 5.2	1.3E-03
BH3 – SS2	Native Sandy silt to silty sand till	0.8 – 1.4	1.3E-03
BH7 – SS3	Native Sandy silt to silty sand till	1.5 – 2.1	1.1E-03
BH9 – SS2	Native Sandy silt to silty sand till	0.8 – 1.4	3.3E-04
BH9 – SS3	Native Sandy silt to silty sand till	1.5 – 2.1	1.6E-03

The grain size gradations were also compared to published MOEE grain size curves for hydraulic conductivity's (Manual of policy, Procedures and Guidelines for Onsite Sewage Systems). Based on these criteria, the percolation time of the native soil samples may be estimated as 8 to 20 minutes per centimetre for materials with permeability range of 10^{-3} to 10^{-5} cm/sec and may be considered as medium to low permeability. Therefore, the subsurface native soil consisting of sandy silt to silty sand and/or till materials is not considered as free draining soil.

It should be noted that since the groundwater level at this site is near the surface. Therefore it is recommended that if any stormwater infiltration system is implemented, the base of infiltration trench at least 0.6m above the groundwater elevation at the site to provide sufficient water flow gradient.

4.11 Stormwater Management Pond (SWM)

4.11.1 General

If a SWM facility is planned for this site, it may be in cut, with excavated wall slopes of 3 horizontal to 1 vertical or flatter. The borehole log results indicate that beneath the topsoil, weathered/disturbed sand and silt was encountered to depth 1.2m, and sandy silt to silty sand and/ or till deposit to explored depth 6.7m.

The groundwater levels in the piezometers were measured on Apr 7, 2017 (after 1 week of installation) at approximate depth range of 0.1 to 2.1m below the existing grade.

It is recommended that the base of management pond (if applicable) should be at least 0.6m above the stabilized groundwater level at the site.

4.11.2 Construction of SWM Pond

The results of the borehole investigation indicate that the pond base is likely to comprise the native till deposit. Based on the sieve and hydrometer analysis results, the permeability of the native soils is generally estimated to be on the order of 10^{-3} to 10^{-5} cm/sec (**Table 4.3**). Therefore, due to the presence of relatively permeable soils which are likely to comprise the pond bottom, a pond liner is recommended to retain a permanent pool level in the pond.

The base of pond and side slopes should be inspected during construction and water seepage conditions (e.g., presence of water bearing and relatively pervious layers / zones) should be assessed by a qualified geotechnical engineer. These areas should be protected by a special filter blanket, such as well graded gravel or sand and gravel layer, or synthetic filter fabric manufactured for this purpose, as directed by the geotechnical engineer during construction.

Based on the discussion in Section 4.4, no major problems with groundwater are anticipated for excavation to approximate depth of 0.5m below the existing ground surface. Seepage from wet sandy and silty layers should be expected but in all likelihood water seepage should be controllable by the use of conventional pumping from collection sumps and ditches for most excavations. Contractors should be prepared to employ more elaborate dewatering procedures if the flow becomes a problem. We recommend that test pit should be dug prior to excavation activity to confirm the groundwater levels.

The final slope surface and all bare or exposed areas (where applicable) should be provided with suitable ground cover or erosion protection. The slope surface should be provided with a thin layer of topsoil

(minimum 100mm thick) and should be hydro-seeded with a grass mixture and mulch. If seeded, during the first 2 to 3 years, the surface cover of topsoil and seeding may require periodic maintenance on slopes, due to surface erosion until the vegetation becomes well established.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the compact to dense cohesionless deposits above the water table can be classified as Type 3 soil and very loose to loose deposits as type 4. Cohesionless soils below the water table can also be classified as Type 4.

As a general rule, the excavations in Type 3 soil can be carried out using minimum side slopes of (1 to 1.5)H : 1V. The excavations in Type 4 soils will require at a minimum, flatter side slopes of 3H to 1V.

4.11.3 Liner Consideration

The liner may consist of a natural soil material (such as clay or clayey silt) or a synthetic membrane liner (such as a High Density Polyethylene, Geo-synthetic Clay Liner, or PVC). A natural soil liner may be preferable based on the following considerations:

- Low permeability clayey silt materials may be available locally for the construction of the liner.
- A clay liner is readily constructed using locally available construction equipment and manpower.
- A synthetic liner requires more elaborate design and construction considerations with respect to fabrication and protection of the completed liner.

However, a synthetic liner would perform satisfactorily and could be considered if a suitable and sufficient clay source were not available.

It is recommended that the minimum liner thickness be 0.5m, and that the liner be inspected on an annual basis, to deal with these considerations.

The liner must be constructed of low permeability materials (clayey silt or silty clay) in order to perform adequately and to provide a liner bulk permeability on the order of 1×10^{-7} cm/s. The liner material should consist of clean mineral soil. The grain size distribution of the liner material must conform to the following:

- No particle greater than 100mm dimension
- Not greater than 15 percent of the material larger than 4.8mm (No. 4 sieve)
- Minimum of 35 percent of the material finer than 0.08mm (i.e., passing No. 200 sieve)
- Minimum 15 percent finer than 0.002mm (clay size)
- Not greater than 5% organic content, with no visible roots or topsoil.

Also for the clay liner material, the plasticity index should not be less than 15, liquid limit should not be less than 30 and not greater than 60, and plastic limit should not be less than 11 and not greater than 30.

A strict control and monitoring of the liner material must be maintained to collect samples to verify its composition based on laboratory test results and to identify any variation in the material. The liner material must be placed at water contents 2 to 4 percent wet of the optimum moisture content. This is required to ensure that the material is compacted to a homogenous mass, and does not remain as distinct "clods" or "clumps". The liner should be constructed in thin lifts (not exceeding 150mm thick) and be heavily compacted to a minimum of 95 percent SPMDD. Liner materials should not contain any frozen soil should the construction proceeds under winter conditions. Also, adequate protection against frost penetration must be provided if required (e.g. straw bales, tarping, heating). The clay liners may be subject to developing desiccation cracks during and after installation if left exposed to dry environment. This can be prevented by placing 0.6m to 0.9m of drainage blanket, protective cover or synthetic cover on top of the liner.

It is recognized that a broad range of soil materials will be suitable for a clay liner (i.e., will meet the specifications noted above). It is recommended that contractors bidding on the project provide the results of testing, to indicate the following:

- The location (source) of the clay material
- Verification of the uniformity of the material
- Demonstration that sufficient material is available for the project
- Laboratory testing to demonstrate that the material meets the minimum specifications noted above.

The liner construction must be conducted under the full time supervision of a qualified geotechnical engineer.

Alternatively, as noted before, a synthetic liner (such as HDPE, Geosynthetic Clay Liner or PVC) may be used. Manufacturer's specifications and recommendations must be referred for the design and construction of a synthetic liner.

4.11.4 Slope Protection and Erosion Control

The following slope protection measures should be considered in the design of the stormwater management pond:

- Site development and construction activities should be conducted in a manner which does not result in overloading or surface erosion on the slope. Final site grading and drainage (including surface drainage) should be designed to prevent direct concentrated or channelized runoff from flowing directly over the slopes.

- The slope layout and slope angle should not be altered without prior consultation with Orbit. Any stockpile of materials, construction equipment, temporary and permanent structures should not be placed on the slope or within 5m of the slope crest.
- As a good slope protection practice, the pond slopes should be inspected by a qualified geotechnical engineer each season for including but not limited to the following. Any slope defective area that may affect the slope safety should be repaired immediately by appropriate techniques.
 - i. Any slope movement, slope surface erosion or leakage of water-carrying services, and to ensure good slope maintenance conditions.
 - ii. Inspection of liner surface for discontinuities or holes as a result of burrowing animals, vandalism, settlement or the like.
 - iii. Removal of unwanted vegetation (tree seedlings and the like) from the pond base.

5 GENERAL COMMENTS

The recommended bearing capacities and the corresponding founding elevations would need to be confirmed by the representative of Orbit during construction. It should be noted that the recommended bearing capacities have been calculated by Orbit from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions becomes available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by Orbit to validate the information for use during the construction.

In this regard, Orbit should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, Orbit will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

6 CLOSURE

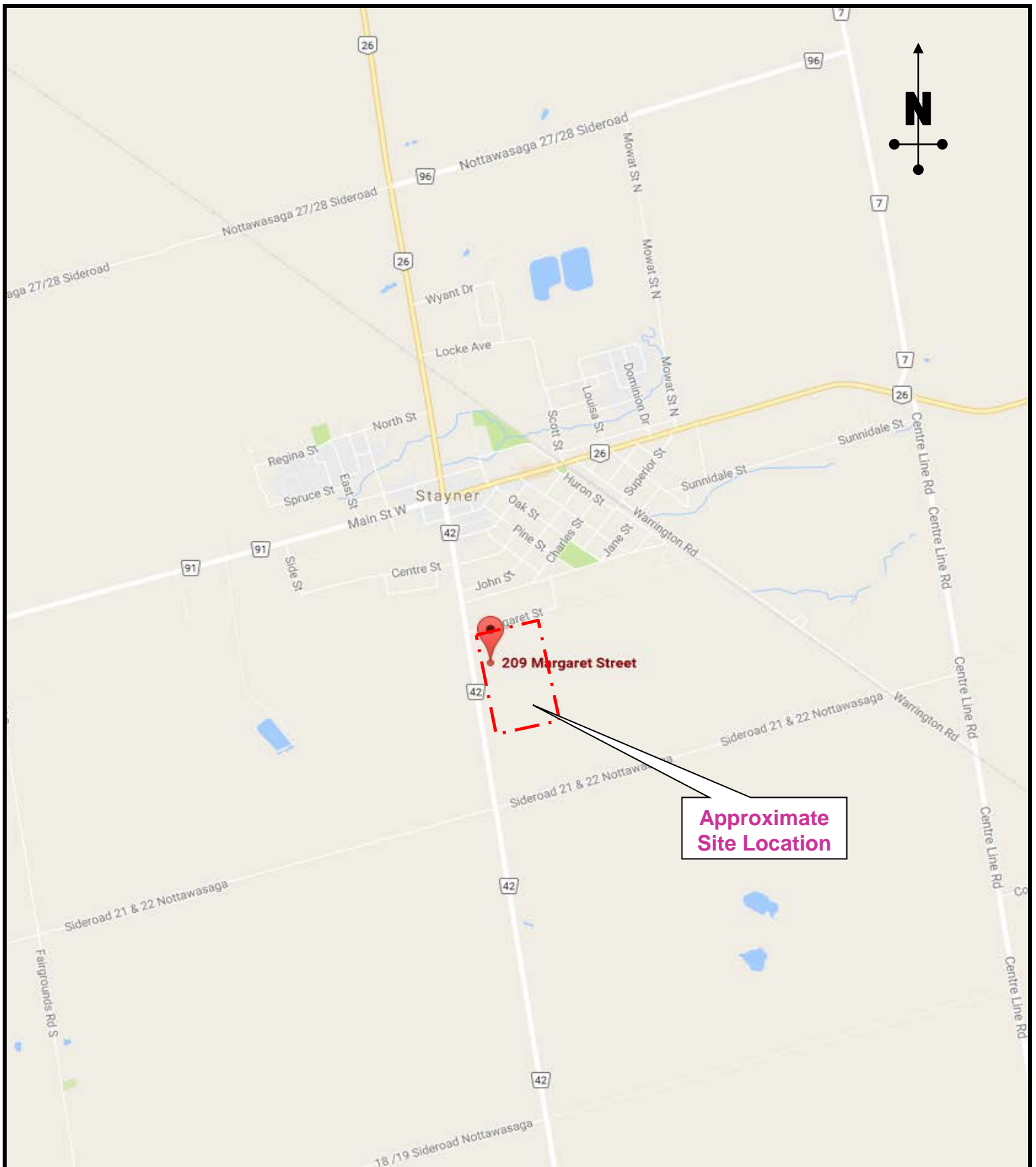
We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

For and on behalf of Orbit,

Hossein Ghiassian, Ph.D.

Hafiz Muneeb Ahmad, M.Eng., M.Sc., P.Eng., QP_{ESA}
Senior Principal Engineer

Drawings



APPROXIMATE SITE LOCATION PLAN



Date: **April, 2017**

Project: **OE17386AG**

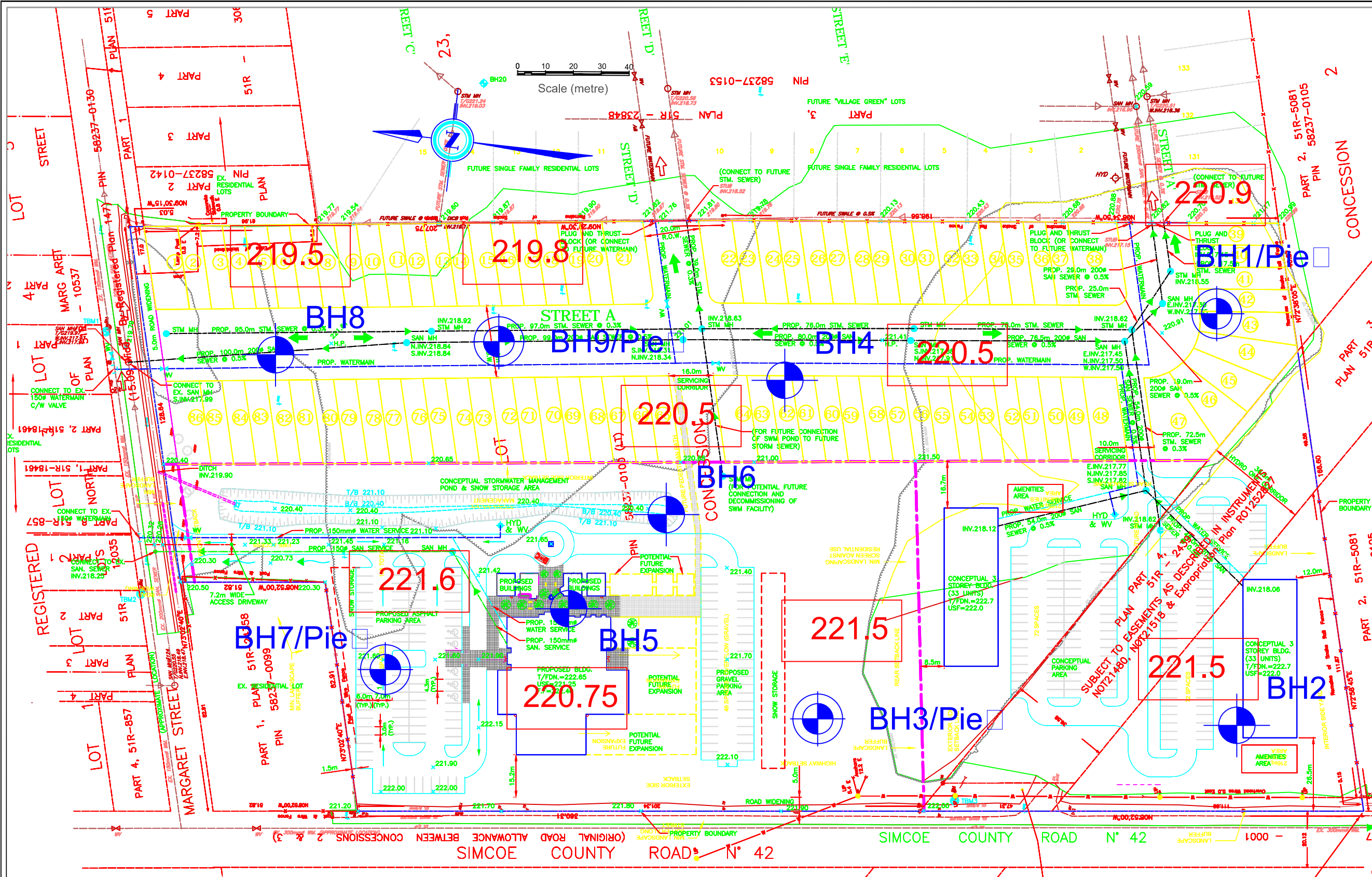
**Geotechnical Investigation
Proposed Residential Development
209 Margaret Street, Stayner, On**

Prepared for: **Mamta Developments Inc.**

Prepared By: **A.T**

Reviewed By: **H.A.**

Drawing No **1**



NOTES:

1. The boundaries and soil types have been established only at borehole locations. Between boreholes they are assumed and may be subject to considerable error.
2. Soil samples will be retained in storage for three months then discarded unless the client advises an extended time period is required.
3. Topsoil, Asphalt, Concrete and Granular materials quantities should not be established from the information provided at the borehole locations.
4. Borehole elevations should not be used to design building grades.
5. This drawing forms part of the report (project number as referenced) and should only be used in conjunction with this report.

LEGEND

- Approximate Borehole Location
- Approximate Borehole Location (Piezometre Installed)
- Existing Ground Surface Field Elevation at the time of Field Investigation



Drawn	A.T.
Approved	I.K.
Date	April, 2017
Scale	As shown
Original Site	Tabloid



Client:	Mamta Developments Inc.
Project:	Geotechnical Investigation Proposed Residential Development 209 Margaret Street, Stayner, On
Title:	Approximate Borehole Location Plan
Project No:	OE17386AG
Drawing No:	1A

Drawing 1B: Notes on Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by Orbit Engineering Limited also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

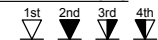
ISSMFE SOIL CLASSIFICATION													
CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS		
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE				
	0.002	0.006	0.02	0.06	0.2	0.6	2.0	6.0	20	60	200		
EQUIVALENT GRAIN DIAMETER IN MILLIMETRES													
CLAY (PLASTIC) TO				FINE		MEDIUM		CRS.		FINE		COARSE	
SILT (NONPLASTIC)				SAND				GRAVEL					
UNIFIED SOIL CLASSIFICATION													

UNIFIED SOIL CLASSIFICATION

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc.; none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advice of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.
3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

PROJECT: Geotechnical Investigation CLIENT: Mamta Developments Inc. PROJECT LOCATION: 209 Margaret St, Stayner, Ontario DATUM: Geodatic BH LOCATION: See Borehole Location Plan								DRILLING DATA Method: Solid Stem Auger Diameter: 0.2 Date: Mar/30/2017								REF. NO.: OE17386AG DRAWING NO.: 2			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)		
(m)	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)				WATER CONTENT (%)							
ELEV DEPTH								○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE				W _p	W	W _L					
220.9								20	40	60	80	100	10	20	30		GR SA SI CL		
0.0	Topsoil: 300mm						W. L. 220.8 m Apr 07, 2017												
220.6	Sand and Silt: weathered / disturbed with some topsoil and rootlets, trace gravel and clay, greyish brown, wet, very loose to loose		1	SS	2														
0.3																			
219.7	Sandy Silt to Silty Sand: trace to some gravel and clay, grey, wet, compact		2	SS	10												6 48 42 4		
1.2																			
			3	SS	15														
			4	SS	19												23 54 21 2		
			5	SS	21														
			6	SS	14												0 8 86 6		
			7	SS	29														
214.2	End of Borehole																		
6.7	Notes: (i) 20mm piezometer installed at completion (ii) Water level readings: Date Depth Mar 30, 2017 0.1m (at completion) Apr 07, 2017 0.1m (after one week)																		

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th


GROUND
NOTES

+ 3 , × 3 : Numbers refer to Sensitivity

○ s=3% Strain at Failure

PROJECT: Geotechnical Investigation
CLIENT: Mamta Developments Inc.
PROJECT LOCATION: 209 Margaret St, Stayner, Ontario
DATUM: Geodatic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Solid Stem Auger
Diameter: 0.2
Date: Mar/30/2017

REF. NO.: OE17386AG
DRAWING NO.: 3

[illegible]

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH
NOTES

$+^3, \times^3$: Numbers refer to Sensitivity

○ **$\epsilon=3\%$** Strain at Failure

PROJECT: Geotechnical Investigation										DRILLING DATA									
CLIENT: Mamta Developments Inc.										Method: Solid Stem Auger									
PROJECT LOCATION: 209 Margaret St, Stayner, Ontario										Diameter: 0.2									
DATUM: Geodatic										Date: Mar/30/2017									
BH LOCATION: See Borehole Location Plan										REF. NO.: OE17386AG									
										DRAWING NO.: 4									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kNm ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)	
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)					WATER CONTENT (%)						
												20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100				W _p
221.5 0.0	Topsoil: 300m																GR SA SI CL		
221.2 0.3	Sandy Silt to Silty Sand: weathered/disturbed with some topsoil and rootlets, trace gravel and clay, greyish brown to grey, wet, very loose		1	SS	1		W. L. 221.2 m Apr 07, 2017												
220.7 0.8	Sandy Silt to Silty Sand Till: trace to some gravel and clay, grey, wet, compact		2	SS	11												14 48 32 6		
219.5 2.0	frequently cobble/boulder encountered, very dense below 2m		3	SS	27		220												
			4	SS	75/ 275mm		219												
			5	SS	50/ 125mm		218												
			6	SS	87/ 125mm		217												
							216												
215.2 6.3	End of Borehole		7	SS	50/ 125mm														
Notes:																			
(i) 20mm piezometer installed at completion																			
(ii) Water level readings:																			
Date Depth																			
Mar 30, 2017 (at completion) 0.9m																			
Apr 07, 2017 (after one week) 0.3m																			

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GROUND
NOTES

+ 3 , × 3 : Numbers refer to Sensitivity

○ s=3% Strain at Failure

PROJECT: Geotechnical Investigation CLIENT: Mamta Developments Inc. PROJECT LOCATION: 209 Margaret St, Stayner, Ontario DATUM: Geodatic BH LOCATION: See Borehole Location Plan								DRILLING DATA Method: Solid Stem Auger Diameter: 0.2 Date: Mar/30/2017 REF. NO.: OE17386AG DRAWING NO.: 5										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)					W _p W W _L					
220.5								20 40 60 80 100					20 40 60 80 100					
0.0	Topsoil: 400mm		1	SS	1		220											
220.1																		
0.4	Sandy Silt to Silty Sand: weathered / disturbed with some topsoil and rootlets, trace gravel and clay, greyish brown, wet, very loose																	
219.7																		
0.8	Sandy Silt to Silty Sand Till: trace to some gravel and clay, grey, wet, compact		2	SS	24		219											
			3	SS	23													
			4	SS	21		218											
			5	SS	20		217											
							216											
215.9																		
4.6	very dense below 4.6m		6	SS	50/ 125mm													
215.6																		
4.9	End of Borehole																	
	Notes:																	
	At Completion:																	
	(i) Depth to Cave 4.3m																	
	(ii) Water Level 0.4m																	

GROUNDWATER ELEVATIONS

1st 2nd 3rd 4th
Measurement

GRAPH NOTES

+ 3 , × 3 : Numbers refer to Sensitivity

○ s=3% Strain at Failure





PROJECT: Geotechnical Investigation
CLIENT: Mamta Developments Inc.
PROJECT LOCATION: 209 Margaret St, Stayner, Ontario
DATUM: Geodatic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Solid Stem Auger
Diameter: 0.2
Date: Mar/30/2017

REF. NO.: OE17386AG
DRAWING NO.: 6

[illegible]

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

GRAPH
NOTES

$+^3, \times^3$: Numbers refer to Sensitivity

○ **$\epsilon=3\%$** Strain at Failure





PROJECT: Geotechnical Investigation
CLIENT: Mamta Developments Inc.
PROJECT LOCATION: 209 Margaret St, Stayner, Ontario
DATUM: Geodatic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Solid Stem Auger
Diameter: 0.2
Date: Mar/30/2017

REF. NO.: OE17386AG
DRAWING NO.: 7

[illegible]

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

GRAPH
NOTES

+ 3, × 3: Numbers refer to Sensitivity

○ **$\epsilon = 3\%$** Strain at Failure





PROJECT: Geotechnical Investigation
CLIENT: Mamta Developments Inc.
PROJECT LOCATION: 209 Margaret St, Stayner, Ontario
DATUM: Geodatic
BH LOCATION: See Borehole Location Plan

DRILLING DATA
Method: Solid Stem Auger
Diameter: 0.2
Date: Mar/31/2017

REF. NO.: OE17386AG
DRAWING NO.: 8

[illegible]

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

GRAPH
NOTES

$+^3, \times^3$: Numbers refer to Sensitivity

○ **$\epsilon=3\%$** Strain at Failure

PROJECT: Geotechnical Investigation							DRILLING DATA													
CLIENT: Mamta Developments Inc.							Method: Solid Stem Auger													
PROJECT LOCATION: 209 Margaret St, Stayner, Ontario							Diameter: 0.2							REF. NO.: OE17386AG						
DATUM: Geodatic							Date: Mar/31/2017							DRAWING NO.: 9						
BH LOCATION: See Borehole Location Plan																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kNm ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)			
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)				WATER CONTENT (%)								
								20 40 60 80 100				W _p W W _L								
219.5																				
0.0	Topsoil: 300mm		1	SS	1		219													
219.2	Sandy Silt to Silty Sand: weathered/disturbed with some topsoil and rootlets, trace gravel and clay, dark brown, wet, very loose																			
0.3																				
218.7	Sandy Silt to Silty Sand Till: trace to some gravel and clay, grey, wet, compact to very dense		2	SS	22		218													
0.8																				
			3	SS	31															
			4	SS	27		217													
			5	SS	51		216													
							215													
			6	SS	31															
214.3																				
5.2	End of Borehole																			
	Notes:																			
	At Completion:																			
	(i) Depth to Cave 2.1m																			
	(ii) Water Level 0.3m																			

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH NOTES

+ 3 , × 3 : Numbers refer to Sensitivity

○ s=3% Strain at Failure

PROJECT: Geotechnical Investigation CLIENT: Mamta Developments Inc. PROJECT LOCATION: 209 Margaret St, Stayner, Ontario DATUM: Geodatic BH LOCATION: See Borehole Location Plan							DRILLING DATA Method: Solid Stem Auger Diameter: 0.2 Date: Mar/31/2017							REF. NO.: OE17386AG DRAWING NO.: 10			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	POCKET PEN. (C _u) (kPa)	NATURAL UNIT WT (kN/m ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)									
219.8 0.0	Topsoil: 300m																GR SA SI CL
219.5 0.3	Sandy Silt to Silty Sand: weathered/disturbed with some clay, topsoil and rootlets, trace gravel and clay, greyish brown, very moist to wet, very loose		1	SS	1		W. L. 219.5 m Mar 31, 2017							63.7			
219.0 0.8	Sandy Silt to Silty Sand Till: trace to some gravel and clay, greyish brown to grey, very moist to wet, loose		2	SS	10		219										5 30 52 13
218.3 1.5	compact below 1.5m		3	SS	18		218										18 46 31 5
			4	SS	19		217										
			5	SS	23		216										
215.2 4.6	dense below 4.6m		6	SS	45		215										
			7	SS	42		214										
213.1 6.7	End of Borehole Notes: (i) 20mm piezometer installed at completion (ii) Water level readings: Date Depth Mar 31, 2017 0.3m (at completion) Apr 07, 2017 2.1m (after one week)																

GROUNDWATER ELEVATIONS

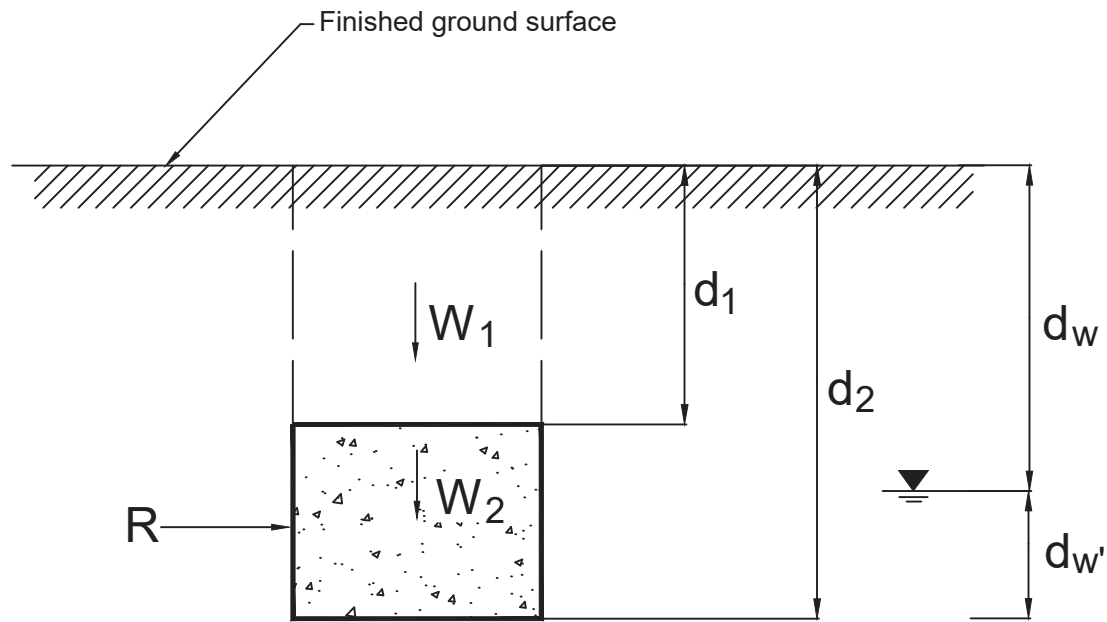
Measurement 1st 2nd 3rd 4th

GROUND NOTES

+ 3 , × 3 : Numbers refer to Sensitivity

○ s=3% Strain at Failure





R = ultimate lateral resistance of thrust block

Case 1 $d_w < d_1$

$$R = B[1/2 K_p \gamma' (d_2^2 - d_1^2) + K_p \gamma_w d_w (d_2 - d_1)] + (W_1 + W_2)f$$

Case 2 $d_1 < d_w < d_2$

$$R = B[1/2 K_p \gamma (d_2^2 - d_1^2) - 1/2 K_p \gamma_w (d_w')^2] + (W_1 + W_2)f$$

Case 3 $d_2 < d_w$

$$R = B[1/2 K_p \gamma (d_2^2 - d_1^2)] + (W_1 + W_2)f$$

R = Ultimate earth resistance, kN.

B = width of block, m.

K_p = coefficient of passive earth pressure = 2.5

γ = total unit weight of soil = 19 kN/m³

γ' = submerged unit weight of soil = 9 kN/m³

γ_w = unit weight of water = 10 kN/m³

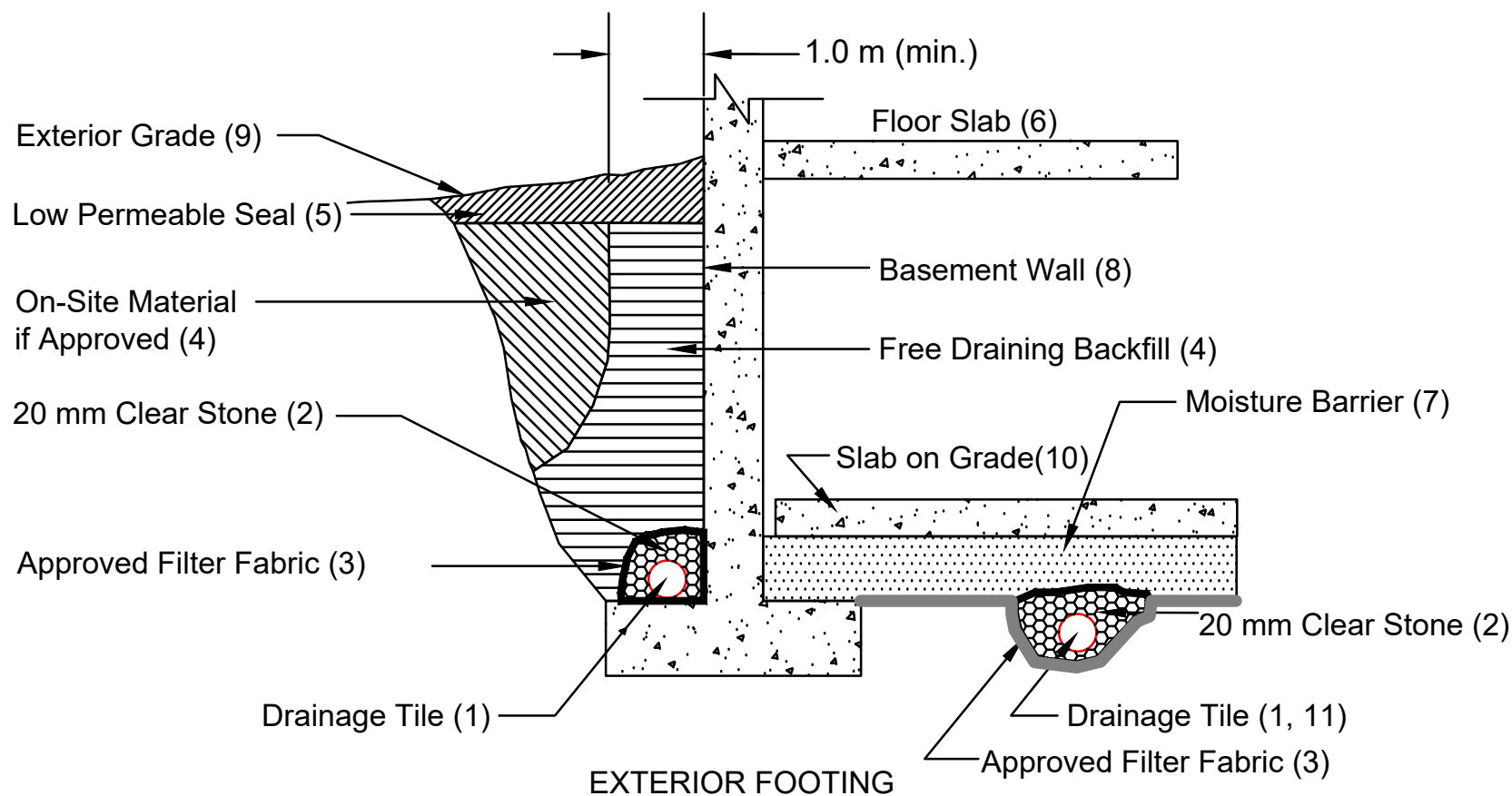
W_1 = weight of soil above thrust block

W_2 = weight of thrust block

f = coefficient of friction between block and soil = 0.3

Thrust Block Analysis

Drawing No. 12



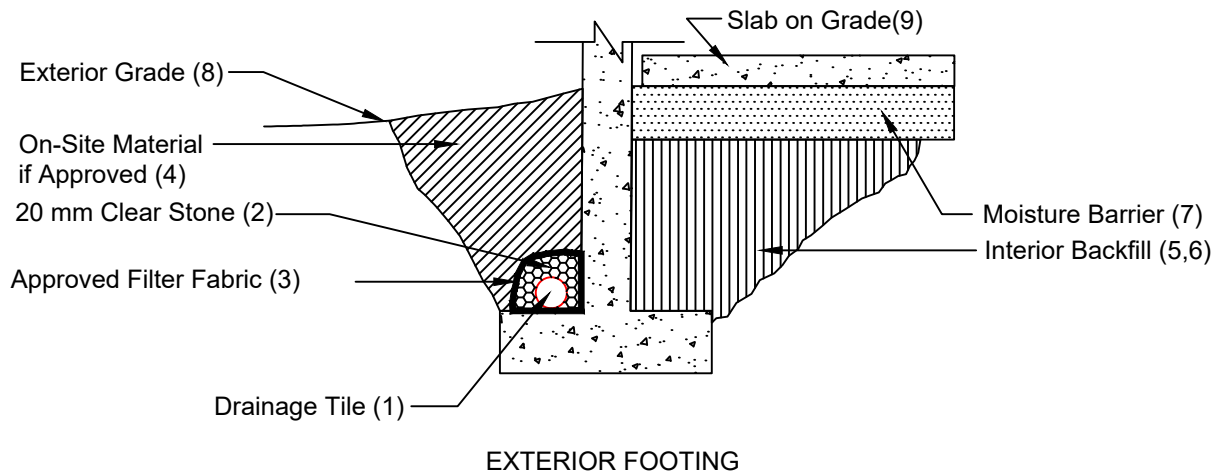
Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place 100 mm (4 inches) of stone below drain.
3. Wrap the clear stone with an approved filter fabric (Terrafix 400R or equivalent).
4. Free Draining backfill - OPSS Granular B or equivalent compacted to the specified density. Do not use heavy compaction equipment within 450 mm (18") of the wall. Use hand controlled light compaction equipment within 1.8 m (6') of wall. The minimum width of the Granular 'B' backfill must be 1.0 m.
5. Low permeable backfill seal - compacted clay, clayey silt or paved with concrete/asphalt or equivalent. If original soil is free-draining, seal may be omitted. Maximum thickness of seal to be 0.5 m.
6. Do not backfill until wall is supported by basement and floor slabs or adequate bracing.
7. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
8. Basement wall to be water proofed.
9. Exterior grade to slope away from building.
10. Typically slab on grade is not structurally connected to the wall or footing. However, if it is connected to the wall, it should be designed accordingly.
11. Underfloor drain invert to be at least 300 mm (12") below underside of floor slab.
12. Drainage tile placed in parallel rows 4 to 6 m (15 to 20') centers one way. Place drain on 100 mm (4") clear stone with 150 mm (6") of clear stone on top and sides. Enclose stone with filter fabric as noted in (3).
13. The entire subgrade to be sealed with approved filter fabric (Terrafix 400R or equivalent).
14. Do not connect the underfloor drains to perimeter drains.
15. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

DRAINAGE AND BACKFILL RECOMMENDATIONS

Basement with Underfloor Drainage

(not to scale)



Notes

1. Drainage tile to consist of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet.
2. 20 mm (3/4") clear stone - 150 mm (6") top and side of drain. If drain is not on footing, place 100 mm (4 inches) of stone below drain.
3. Wrap the clear stone with an approved filter fabric (Terrafix 270R or equivalent).
4. The on-site material, if approved, can be used as backfill.
5. The interior fill may be any clean non-organic soil which can be compacted to the specified density in this confined space.
6. Do not use heavy compaction equipment within 450 mm (18") of the wall. Do not fill or compact within 1.8 m (6') of the wall unless fill is placed on both sides simultaneously.
7. Moisture barrier to be at least 200 mm (8") of compacted clear 20 mm (3/4") stone or equivalent free draining material. A vapour barrier may be required for specialty floors.
8. Exterior grade to slope away from building.
9. Typically, slab on grade is not structurally connected to the wall or footing. However, if it is connected to the wall, it should be designed accordingly.
10. Review the geotechnical report for specific details. Final detail must be approved before system is considered acceptable.

DRAINAGE AND BACKFILL RECOMMENDATIONS

Slab on Grade Construction Without Underfloor Drainage

(not to scale)

Appendices

Appendix A

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Orbit Engineering Limited. at the time of preparation. Unless otherwise agreed in writing by Orbit Engineering Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Orbit Engineering Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

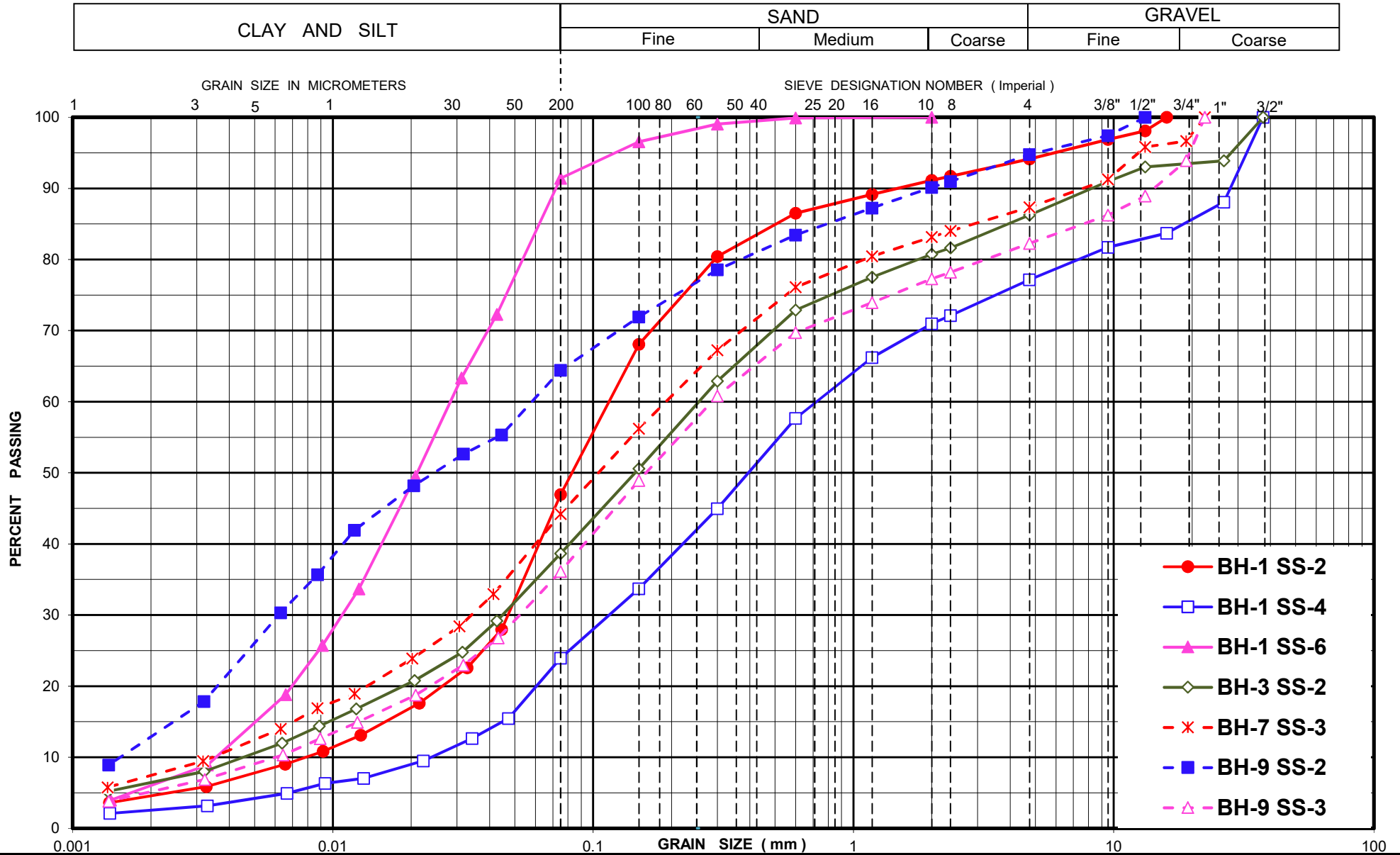
We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time. Any user of this report specifically denies any right to claims against the Consultant, Sub-Consultants, their officers, agents and employees in excess of the fee paid for professional services.

Appendix B

Geotechnical Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

LS 702/D 422



Appendix C

General Requirements for Engineered Fill

GENERAL REQUIREMENTS FOR ENGINEERED FILL

Compacted imported soil that meets specific engineering requirements and is free of organics and debris and that has been continually monitored on a full-time basis by a qualified geotechnical representative is classified as engineered fill. Engineered fill that meets these requirements and is bearing on suitable native subsoil can be used for the support of foundations.

Imported soil used as engineered fill can be removed from other portions of a site or can be brought in from other sites if suitable. In general, most of Ontario soils are too wet to achieve the 100% Standard Proctor Maximum Dry Density (SPMDD) and will require drying and careful site management if they are to be considered for engineered fill. Imported non-cohesive granular soil is preferred for all engineered fill. For engineered fill, Orbit Engineering Limited (Orbit) recommends use of OPSS Granular 'B' sand and gravel fill material only.

Adverse weather conditions such as rain make the placement of engineered fill to the required degree of density difficult or impossible; engineered fill should not be placed during freezing conditions, i.e. normally not between December 15 and April 1 of each year. If the project demands placement of engineered fill in winter (December 15- April 1) it can be placed only under the following conditions:

- All frozen material and or snow must be removed before placement of engineered fill on a daily basis
- Only Granular B Type 2 or Granular A (including crushed concrete or crushed limestone)
- The fill placement must be supervised on a full time basis by a geotechnical consultant

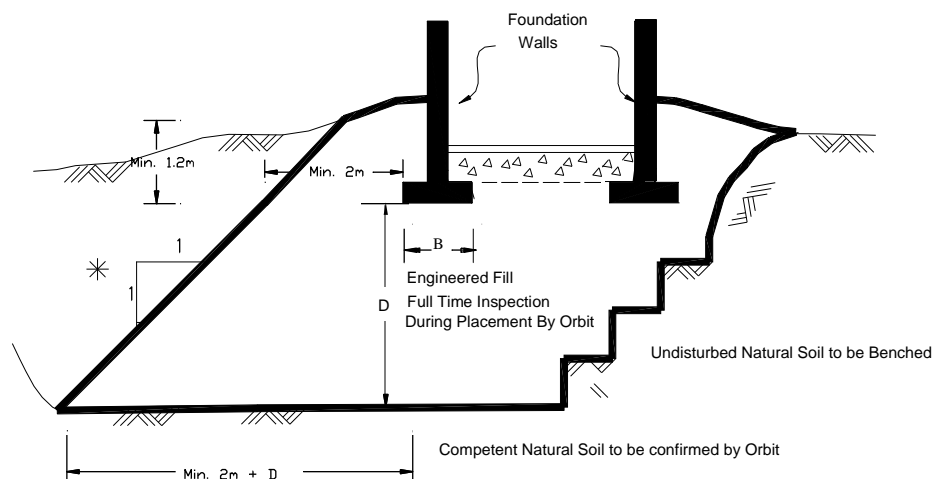
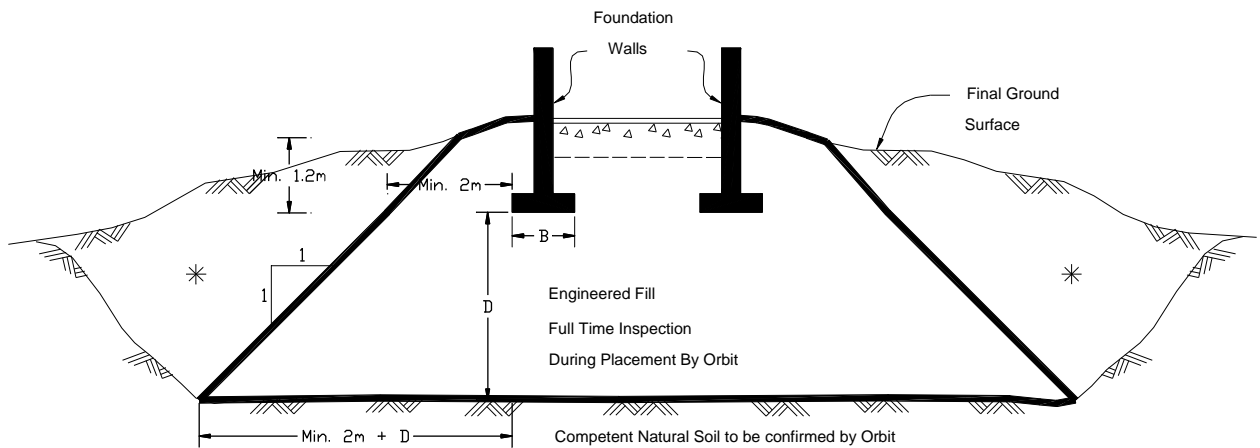
The location of the foundations on the engineered soil pad is critical and certification by a qualified surveyor that the foundations are within the stipulated boundaries is mandatory. Since layout stakes are often damaged or removed during fill placement, offset stakes must be installed and maintained by the surveyors during the course of fill placement so that the contractor and engineering staff are continually aware of where the engineered fill limits lie. Foundations placed within the engineered soil pad must be backfilled with the same conditions and quality control as the original pad.

To perform satisfactorily, engineered fill requires the cooperation of the designers, engineers, contractors and all parties must be aware of the requirements. The minimum requirements are as follows, however, the geotechnical report must be reviewed for specific information and requirements.

1. Prior to site work involving engineered fill, a site meeting to discuss all aspects must be convened. The surveyor, contractor, design engineer and geotechnical engineer must attend the meeting. At this meeting, the limits of the engineered fill will be defined. The contractor must make known where all fill material will be obtained and samples must be provided to the geotechnical engineer for review, and approval before filling begins.
2. Detailed drawings indicating the lower boundaries as well as the upper boundaries of the engineered fill must be available at the site meeting and be approved by the geotechnical engineer.

3. The building footprint and base of the pad, including basements, garages, etc. must be defined by offset stakes that remain in place until the footings and service connections are all constructed. Confirmation that the footings are within the pad, service lines are in place, and that the grade conforms to drawings, must be obtained by the owner in writing from the surveyor and Orbit Engineering Limited. Without this confirmation no responsibility for the performance of the structure can be accepted by Orbit Engineering Limited. Survey drawing of the pre and post fill location and elevations will also be required.
4. The area must be stripped of all topsoil and fill materials. Subgrade must be proofrolled. Soft spots must be dug out. The stripped native subgrade must be examined and approved by an Orbit engineer prior to placement of fill.
5. The approved engineered fill must be compacted to 100% Standard Proctor Maximum Dry Density throughout. Granular Fill preferred. Engineered fill should not be placed (where it will support footings) during the winter months. Engineered fill compacted to 100% SPMDD will settle under its own weight approximately 0.5% of the fill height and the structural engineer must be aware of this settlement. In addition to the settlement of the fill, additional settlement due to consolidation of the underlying soils from the structural and fill loads will occur and should be evaluated prior to placing the fill.
6. Full-time geotechnical inspection by Orbit during placement of engineered fill is required. Work cannot commence or continue without the presence of the Orbit representative.
7. The fill must be placed such that the specified geometry is achieved. Refer to sketches for minimum requirements. Take careful note that the projection of the compacted pad beyond the footing at footing level is a minimum of 2 m. The base of the compacted pad extends 2 m plus the depth of excavation beyond the edge of the footing.
8. The allowable bearing pressure provided in the accompanying report may be used provided that all conditions outlined above are adhered to. A minimum footing width of 500 mm (20 inches) is suggested and footings must be provided with nominal steel reinforcement.
9. All excavations must be done in accordance with the Occupational Health and Safety Regulations of Ontario.
10. After completion of the pad a second contractor may be selected to install footings. The prepared footing bases must be evaluated by engineering staff from Orbit Engineering Limited prior to footing concrete placements. All excavations must be backfilled under full time Orbit Engineering Limited supervision by Orbit to the same degree as the engineered fill pad. Surface water cannot be allowed to pond in excavations or to be trapped in clear stone backfill. Clear stone backfill can only be used with the approval of Orbit Engineering Limited.
11. After completion of compaction, the surface of the pad must be protected from disturbance from traffic, rain and frost. During the course of fill placement, the engineered fill must be smooth-graded, proof rolled and sloped/crowned at the end of each day, prior to weekends and any stoppage in work in order to promote rapid runoff of rainwater and to avoid any ponding surface water. Any stockpiles of fill intended for use as engineered fill must also be smooth-bladed to promote runoff and/or protected from excessive moisture take up.

12. If there is a delay in construction, the engineered fill pad must be inspected and accepted by the geotechnical engineer. The location of the structure must be reconfirmed that it remains within the pad.
13. The geometry of the engineered fill as illustrated in these General Requirements is general in nature. Each project will have its own unique requirements. For example, if perimeter sidewalks are to be constructed around the building, then the projection of the engineered fill beyond the foundation wall may need to be greater.
14. These guidelines are to be read in conjunction with Orbit Engineering Limited report attached.

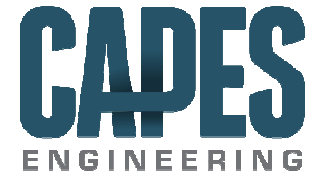


Appendix D – Existing Condition Stormwater Information



PCSWMM Hydrologic Parameters Summary

Project: 1191 County Rd. 42 - Mamta Homes
Project No.: 2018-060
Designed by: CC
Date: 6-Dec-18



Existing Condition

Drainage Area Mamta_West

Catchment Name	Mamta_West
Catchment Area (ha)	5.03
Length (m)	362
h ₁ (m)	222.44
h ₂ (m)	219.74
Dh (m)	2.7
Slope (%)	0.75%

Land Cover	N Perv	Dstor Perv	Area(ha)	% by Area	Nperv x Area	DSPerv x Area
Woodland	0.40	10	0.98	19.48%	0.392	9.8
Pasture	0.17	8	3.92	77.93%	0.6664	31.36
Lawn	0.15	5	0	0.00%	0	0
Field (Row Crop)	0.05	7	0	0.00%	0	0
Weighted Nperv and Dstor Perv			4.90	97.42%	0.22	8.40
Land Cover	N Imp	Dstor Imp	Area(ha)	% by Area		
Impervious	0.013	2	0.12	2.39%		
Total			5.02	99.80%		

Mamta Homes - Existing Condition - 100 Year 24 Hr SCS Type II

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.012)

WARNING 04: minimum elevation drop used for Conduit C1

Element Count

Number of rain gages 14

Number of subcatchments ... 1

Number of nodes 2

Number of links 1

Number of pollutants 0

Number of land uses 0

Raingage Summary

Name	Data Source	Data Type	Recording Interval
25mmStorm	25mm	INTENSITY	5 min.
Chicago_4h_100YR	Chicago_4h_100YR	INTENSITY	5 min.
Chicago_4h_10YR	Chicago_4h_10YR	INTENSITY	5 min.
Chicago_4h_25YR	Chicago_4h_25YR	INTENSITY	5 min.
Chicago_4h_2YR	Chicago_4h_2YR	INTENSITY	5 min.
Chicago_4h_50YR	Chicago_4h_50YR	INTENSITY	5 min.
Chicago_4h_5YR	Chicago_4h_5YR	INTENSITY	5 min.
SCS_Type_II_110.1mm_50YR	SCS_Type_II_110.1mm_50YR	INTENSITY	6 min.
SCS_Type_II_121.0mm_100YR	SCS_Type_II_121.0mm_100YR	INTENSITY	6 min.
SCS_Type_II_54.9mm_2YR	SCS_Type_II_54.9mm_2YR	INTENSITY	6 min.
SCS_Type_II_72.6mm_5YR	SCS_Type_II_72.6mm_5YR	INTENSITY	6 min.
SCS_Type_II_84.3mm_10YR	SCS_Type_II_84.3mm_10YR	INTENSITY	6 min.
SCS_Type_II_99.2mm_25YR	SCS_Type_II_99.2mm_25YR	INTENSITY	6 min.
Timmins_Storm_(0-25)	Timmins_Storm_(0-25)	INTENSITY	60 min.

Subcatchment Summary

Name	Area	Width	%Imperv	%Slope	Rain Gage	Outlet
Mamta_West	5.03	138.00	2.40	0.7400	SCS_Type_II_121.0mm_100YR	J1

Node Summary

Name	Type	Invert Elev.	Max. Depth	Ponded Area	External Inflow
J1	JUNCTION	0.00	0.00	0.0	
OF1	OUTFALL	0.00	0.00	0.0	

Link Summary

Name	From Node	To Node	Type	Length	%Slope	Roughness
C1	J1	OF1	CONDUIT	6.5	0.0047	0.0100

Cross Section Summary

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full Flow
C1	DUMMY	0.00	0.00	0.00	0.00	1	0.00

NOTE: The summary statistics displayed in this report are based on results found at every computational time step, not just on results from each reporting time step.

Analysis Options

Flow Units CMS

Process Models:

Rainfall/Runoff YES
RDII NO
Snowmelt NO
Groundwater NO
Flow Routing YES
Ponding Allowed YES

Water Quality NO
 Infiltration Method GREEN_AMPT
 Flow Routing Method DYNWAVE
 Starting Date 11/13/2018 00:00:00
 Ending Date 11/15/2018 00:00:00
 Antecedent Dry Days 0.0
 Report Time Step 00:01:00
 Wet Time Step 00:05:00
 Dry Time Step 00:05:00
 Routing Time Step 5.00 sec
 Variable Time Step YES
 Maximum Trials 8
 Number of Threads 1
 Head Tolerance 0.001500 m

	Volume hectare-m	Depth mm
*****	-----	-----
Runoff Quantity Continuity		

Total Precipitation	0.609	121.000
Evaporation Loss	0.000	0.000
Infiltration Loss	0.357	70.933
Surface Runoff	0.252	50.043
Final Storage	0.000	0.048
Continuity Error (%)	-0.020	

	Volume hectare-m	Volume 10^6 ltr
*****	-----	-----
Flow Routing Continuity		

Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.252	2.518
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	0.252	2.518
Flooding Loss	0.000	0.000
Evaporation Loss	0.000	0.000
Exfiltration Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.000	

 Time-Step Critical Elements

None

Highest Flow Instability Indexes

All links are stable.

Routing Time Step Summary

Minimum Time Step	:	4.50 sec
Average Time Step	:	5.00 sec
Maximum Time Step	:	5.00 sec
Percent in Steady State	:	0.00
Average Iterations per Step	:	2.00
Percent Not Converging	:	0.00

Subcatchment Runoff Summary

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
Mamta_West	121.00	0.00	0.00	70.93	50.04	2.52	0.30	0.414

Node Depth Summary

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
J1	JUNCTION	0.00	0.00	0.00	0 00:00	0.00
OF1	OUTFALL	0.00	0.00	0.00	0 00:00	0.00

Node Inflow Summary

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
J1	JUNCTION	0.305	0.305	0 12:00	2.52	2.52	0.000
OF1	OUTFALL	0.000	0.305	0 12:00	0	2.52	0.000

Node Surge Summary

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
J1	JUNCTION	48.00	0.000	0.000

Node Flooding Summary

No nodes were flooded.

Outfall Loading Summary

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
OF1	47.60	0.031	0.305	2.518
System	47.60	0.031	0.305	2.518

Link Flow Summary

Link	Type	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Veloc m/sec	Max/ Full Flow	Max/ Full Depth
C1	DUMMY	0.305	0 12:00			

Flow Classification Summary

----- Fraction of Time in Flow Class -----										
	Adjusted	Up		Down	Sub	Sup	Up	Down	Norm	Inlet
Conduit	/Actual	Dry	Dry	Dry	Crit	Crit	Crit	Crit	Ltd	Ctrl
	Length									

Conduit Surcharge Summary

No conduits were surcharged.

Analysis begun on: Tue Dec 11 17:08:02 2018
Analysis ended on: Tue Dec 11 17:08:02 2018
Total elapsed time: < 1 sec

Appendix E – Sanitary Sewer Design Sheet

Project:	Mamta Homes
Municipality:	Township of Clearview
Project No.:	2018-060
Analyzed by:	PCC
Date:	January 9, 2019
Manning n Value	0.013

Sanitary Sewer Design Sheet

Township of Clearview



DESIGN SEWAGE FLOWS												SANITARY SEWER CAPACITY					
Location of Section	Cumulative Sanitary Catchment Area (ha)	From Upstream MH #	To Downstream MH #	Cumulative No. Units	Cumulative Serviced Population Cap.	Peaking Factor	Average Flow Residential L/s	Peak Flow Residential L/s	Peak Flow Infiltration: L/s	Total Peak Flow Infiltration L/s	Total Peak Flow L/s	Pipe Length m	Pipe Diameter mm	Pipe Grade %	Full Flow. Cap. L/s	Full Flow Velocity m/s	Percentage Full
Mamta West - Street A	0.29	MH1A	MH3A	2	6	4.43	0.03	0.14	0.040	0.040	0.18	15.7	200	0.5	23.4	0.75	0.76%
Mamta West - Street B	0.22	MH2A	MH3A	4	12	4.41	0.06	0.28	0.031	0.031	0.31	43.8	200	0.5	23.4	0.75	1.31%
Mamta West - Street A	0.78	MH3A	MH6A	12	36	4.34	0.19	0.81	0.109	0.109	0.92	67.2	200	0.5	23.4	0.75	3.94%
Mamta West - Street C	0.20	MH4A	MH5A	4	12	4.41	0.06	0.28	0.029	0.029	0.30	43.9	200	1.0	32.8	1.04	0.93%
Mamta West - Street D	0.60	MH5A	MH6A	15	45	4.32	0.23	1.01	0.084	0.084	1.10	77.2	200	0.5	23.4	0.75	4.68%
Mamta West - Street A	1.59	MH6A	MH8A	33	99	4.24	0.52	2.19	0.222	0.222	2.41	64.9	200	0.5	23.4	0.75	10.29%
Mamta West - Street E	0.50	MH7A	MH8A	14	42	4.33	0.22	0.95	0.070	0.070	1.02	64.3	200	0.5	23.4	0.75	4.34%
Mamta West - Street A	2.48	MH8A	MH10A	53	159	4.18	0.83	3.46	0.347	0.347	3.81	64.9	200	0.5	23.4	0.75	16.27%
Mamta West - Street F*	0.50	MH9A	MH10A	9	27	4.36	0.14	0.68	0.069	0.069	0.75	64.2	200	0.5	23.4	0.75	3.21%
Mamta West - Street A	3.81	MH10A	MH11A	71	213	4.14	1.11	4.66	0.534	0.534	5.19	90.5	200	0.5	23.4	0.75	22.18%
Mamta West - Street A	4.05	MH11A	MH13A	71	213	4.14	1.11	4.66	0.567	0.567	5.23	19.2	200	0.5	23.4	0.75	22.32%
CONDO AREA	0.68	MH12A	MH13A	36	108	4.23	0.56	2.38	0.095	0.095	2.48	60.8	200	0.5	23.4	0.75	10.58%
Mamta West - Street G	5.02	MH13A	MH14A	107	321	4.07	1.67	6.87	0.702	0.702	7.57	49.8	200	0.5	23.4	0.75	32.32%
Mamta East Street A	7.86	MH14A	EXPLUG104A	176	528	3.96	2.75	10.97	1.100	1.100	12.07	59.3	200	0.5	23.4	0.75	51.52%

NOTE:	BASED ON 3.0 PEOPLE PER UNIT
	AVERAGE DAILY PER CAPITA FLOW = 450 L/cap/day
	EXTRANEIOUS FLOW ALLOWANCE = 0.23 L/sec/gross hectare
	PEAKING FACTOR : HARMON
	*AMENITY AREA ASSUMES AN ADDITIONAL: 4 EMPLOYEES @50L/day/employee + 60 people @ 15L/day in the Theatre and 130 people @ 36 L/day for the Party Area as PER OBC TABLE 8.2.1.3.B = 5750 L/day
	MANNING "n" = 0.013

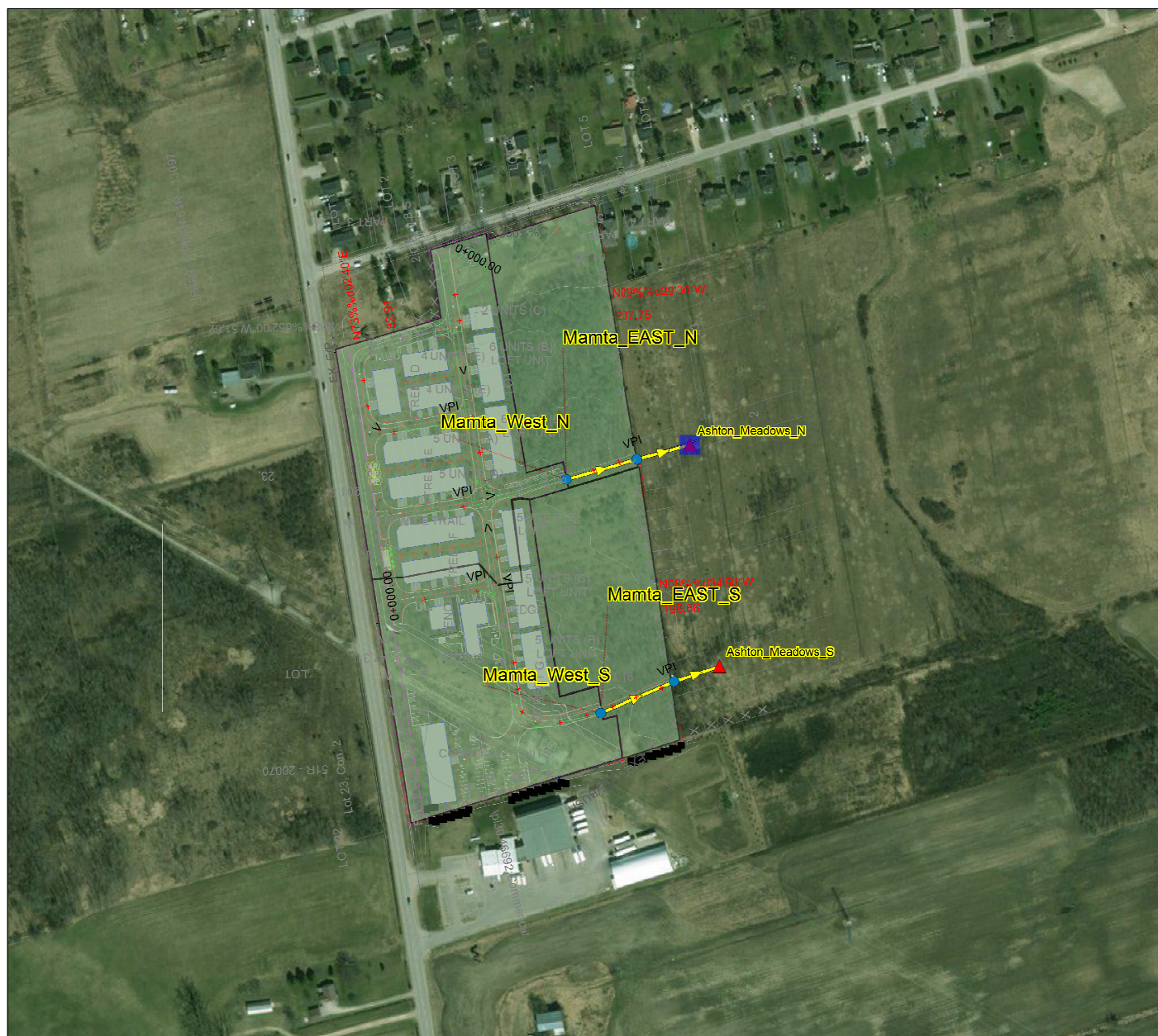
Appendix F – Post Development Stormwater Information

Legend

- Junctions
- ▲ Outfalls
- Conduits
- ▭ Subcatchments
- ACAD-2018-060 Grading and Servicing Concept



150 m



PCSWMM Hydrologic Parameters Summary

Project: 1191 County Rd. 42 - Mamta Homes
 Project No.: 2018-060
 Designed by: CC
 Date: 6-Dec-18



Drainage Area Mamta_West_N

Catchment Name	Mamta_West_N
Catchment Area (ha)	2.5965
Length (m)	250
h ₁ (m)	222.65
h ₂ (m)	221.61
Dh (m)	1.04
Slope (%)	0.42%

Land Cover	N Perv	Dstor Perv	Area(ha)	% by Area	Nperv x Area	DSPerv x Area
Woodland	0.40	10	0	0.00%	0	0
Pasture	0.17	8	0	0.00%	0	0
Lawn	0.15	5	1.1595	44.66%	0.173925	5.7975
Field (Row Crop)	0.05	7	0	0.00%	0	0
Weighted Nperv and Dstor Perv			1.16	44.66%	0.15	5.00
Land Cover	N Imp	Dstor Imp	Area(ha)	% by Area		
Impervious	0.013	2	1.437	55.34%		
Total			2.60	100.00%		

Drainage Area Mamta_West_S

Catchment Name	Mamta_West_S
Catchment Area (ha)	2.4297
Length (m)	245
h ₁ (m)	222.72
h ₂ (m)	221.45
Dh (m)	1.27
Slope (%)	0.52%

Land Cover	N Perv	Dstor Perv	Area(ha)	% by Area	Nperv x Area	DSPerv x Area
Woodland	0.40	10	0	0.00%	0	0
Pasture	0.17	8	0.44	18.11%	0.0748	3.52
Lawn	0.15	5	0.7884	32.45%	0.11826	3.942
Field (Row Crop)	0.05	7	0	0.00%	0	0
Weighted Nperv and Dstor Perv			1.23	50.56%	0.16	6.07
Land Cover	N Imp	Dstor Imp	Area(ha)	% by Area		
Impervious	0.013	2	1.2013	49.44%		
Total			2.43	100.00%		

Drainage Area Mamta_East_N

Catchment Name	Mamta_East_N
Catchment Area (ha)	1.6239
Length (m)	183
h ₁ (m)	222.34
h ₂ (m)	221.42
Dh (m)	0.92
Slope (%)	0.50%

Land Cover	N Perv	Dstor Perv	Area(ha)	% by Area	Nperv x Area	DSPerv x Area
Woodland	0.40	10	0	0.00%	0	0
Pasture	0.17	8	0	0.00%	0	0
Lawn	0.15	5	0.844428	52.00%	0.1266642	4.22214
Field (Row Crop)	0.05	7	0	0.00%	0	0
Weighted Nperv and Dstor Perv			0.84	52.00%	0.15	5.00
Land Cover	N Imp	Dstor Imp	Area(ha)	% by Area		
Impervious	0.013	2	0.7795	48.00%		
Total			1.62	100.00%		

Drainage Area Mamta_East_S

Catchment Name	Mamta_East_S
Catchment Area (ha)	1.5598
Length (m)	156
h ₁ (m)	221.5
h ₂ (m)	221.05
Dh (m)	0.45
Slope (%)	0.29%

Land Cover	N Perv	Dstor Perv	Area(ha)	% by Area	Nperv x Area	DSPerv x Area
Woodland	0.40	10	0	0.00%	0	0
Pasture	0.17	8	0	0.00%	0	0
Lawn	0.15	5	0.811096	52.00%	0.1216644	4.05548
Field (Row Crop)	0.05	7	0	0.00%	0	0
Weighted Nperv and Dstor Perv			0.81	52.00%	0.15	5.00
Land Cover	N Imp	Dstor Imp	Area(ha)	% by Area		
Impervious	0.013	2	0.748704	48.00%		
Total			1.56	100.00%		

Mamta Homes - Post Development 100 Year 4 Hr Chicago - With Imported Fill

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.012)

WARNING 02: maximum depth increased for Node ExMH17
WARNING 02: maximum depth increased for Node EXPLUG39

Element Count

Number of rain gages 14
Number of subcatchments ... 4
Number of nodes 6
Number of links 4
Number of pollutants 0
Number of land uses 0

Raingage Summary

Name	Data Source	Data Type	Recording Interval
25mmStorm	25mm	INTENSITY	5 min.
Chicago_4h_100YR	Chicago_4h_100YR	INTENSITY	5 min.
Chicago_4h_10YR	Chicago_4h_10YR	INTENSITY	5 min.
Chicago_4h_25YR	Chicago_4h_25YR	INTENSITY	5 min.
Chicago_4h_2YR	Chicago_4h_2YR	INTENSITY	5 min.
Chicago_4h_50YR	Chicago_4h_50YR	INTENSITY	5 min.
Chicago_4h_5YR	Chicago_4h_5YR	INTENSITY	5 min.
SCS_Type_II_110.1mm_50YR	SCS_Type_II_110.1mm_50YR	INTENSITY	6 min.
SCS_Type_II_121.0mm_100YR	SCS_Type_II_121.0mm_100YR	INTENSITY	6 min.
SCS_Type_II_54.9mm_2YR	SCS_Type_II_54.9mm_2YR	INTENSITY	6 min.
SCS_Type_II_72.6mm_5YR	SCS_Type_II_72.6mm_5YR	INTENSITY	6 min.
SCS_Type_II_84.3mm_10YR	SCS_Type_II_84.3mm_10YR	INTENSITY	6 min.
SCS_Type_II_99.2mm_25YR	SCS_Type_II_99.2mm_25YR	INTENSITY	6 min.
Timmins_Storm_(0-25)	Timmins_Storm_(0-25)	INTENSITY	60 min.

Subcatchment Summary

Name	Area	Width	%Imperv	%Slope	Rain Gage	Outlet
Mamta_EAST_N	1.62	88.70	48.00	0.5000	Chicago_4h_100YR	J1-S

Mamta_EAST_S	1.56	99.99	48.00	0.2900	Chicago_4h_100YR	J2-S
Mamta_West_N	2.60	110.00	55.00	0.4200	Chicago_4h_100YR	J1-S
Mamta_West_S	2.43	99.17	49.44	0.5200	Chicago_4h_100YR	J2-S

Node Summary

Name	Type	Invert Elev.	Max. Depth	Ponded Area	External Inflow
ExMH17	JUNCTION	219.19	2.46	0.0	
EXPLUG39	JUNCTION	219.04	2.24	0.0	
J1-S	JUNCTION	221.61	0.23	0.0	
J2-S	JUNCTION	221.40	0.23	0.0	
Ashton_Meadows_N	OUTFALL	218.00	0.00	0.0	
Ashton_Meadows_S	OUTFALL	218.00	0.00	0.0	

Link Summary

Name	From Node	To Node	Type	Length	%Slope	Roughness
C1-S	J1-S	ExMH17	CONDUIT	55.5	0.3425	0.0140
C2-S	J2-S	EXPLUG39	CONDUIT	60.3	0.5805	0.0140
C3	ExMH17	Ashton_Meadows_N	CONDUIT	41.1	2.8968	0.0100
C4	EXPLUG39	Ashton_Meadows_S	CONDUIT	36.0	3.0293	0.0100

Cross Section Summary

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full Flow
C1-S	Street1	0.23	2.11	0.06	20.00	1	1.36
C2-S	Street1	0.23	2.11	0.06	20.00	1	1.77
C3	DUMMY	0.00	0.00	0.00	0.00	1	0.00
C4	DUMMY	0.00	0.00	0.00	0.00	1	0.00

Transect Summary

Transect Street1

Area:

0.0005	0.0020	0.0045	0.0080	0.0126
0.0181	0.0246	0.0322	0.0407	0.0503
0.0608	0.0724	0.0849	0.0985	0.1131
0.1287	0.1453	0.1628	0.1814	0.2010
0.2206	0.2403	0.2600	0.2796	0.2993
0.3190	0.3386	0.3583	0.3780	0.3976
0.4173	0.4370	0.4567	0.4776	0.4999
0.5236	0.5486	0.5751	0.6029	0.6321
0.6626	0.6946	0.7279	0.7626	0.7987
0.8362	0.8751	0.9154	0.9570	1.0000

Hrad:

0.0295	0.0591	0.0886	0.1181	0.1477
0.1772	0.2067	0.2362	0.2658	0.2953
0.3248	0.3544	0.3839	0.4134	0.4430
0.4725	0.5020	0.5315	0.5611	0.4442
0.4871	0.5300	0.5728	0.6156	0.6582
0.7008	0.7432	0.7856	0.8279	0.8701
0.9123	0.9543	0.9711	0.9524	0.9385
0.9286	0.9221	0.9184	0.9172	0.9181
0.9208	0.9252	0.9310	0.9380	0.9461
0.9553	0.9653	0.9762	0.9878	1.0000

Width:

0.0230	0.0460	0.0690	0.0920	0.1150
0.1380	0.1610	0.1840	0.2070	0.2300
0.2530	0.2760	0.2990	0.3220	0.3450
0.3680	0.3910	0.4140	0.4370	0.4500
0.4500	0.4500	0.4500	0.4500	0.4500
0.4500	0.4500	0.4500	0.4500	0.4500
0.4500	0.4500	0.4624	0.4940	0.5256
0.5573	0.5889	0.6205	0.6521	0.6838
0.7154	0.7470	0.7786	0.8103	0.8419
0.8735	0.9051	0.9368	0.9684	1.0000

NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.

Analysis Options

Flow Units CMS
 Process Models:
 Rainfall/Runoff YES
 RDII NO
 Snowmelt NO
 Groundwater NO
 Flow Routing YES
 Ponding Allowed YES
 Water Quality NO
 Infiltration Method GREEN_AMPT
 Flow Routing Method DYNWAVE
 Starting Date 11/13/2018 00:00:00
 Ending Date 11/15/2018 00:00:00
 Antecedent Dry Days 0.0
 Report Time Step 00:01:00
 Wet Time Step 00:05:00
 Dry Time Step 00:05:00
 Routing Time Step 5.00 sec
 Variable Time Step YES
 Maximum Trials 8
 Number of Threads 1
 Head Tolerance 0.001500 m

*****	Volume	Depth
Runoff Quantity Continuity	hectare-m	mm
*****	-----	-----
Total Precipitation	0.580	70.593
Evaporation Loss	0.000	0.000
Infiltration Loss	0.247	30.054
Surface Runoff	0.330	40.181
Final Storage	0.008	1.014
Continuity Error (%)	-0.931	

*****	Volume	Volume
Flow Routing Continuity	hectare-m	10^6 ltr
*****	-----	-----
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.330	3.299
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	0.330	3.299
Flooding Loss	0.000	0.000
Evaporation Loss	0.000	0.000

Exfiltration Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.001	

Time-Step Critical Elements

None

Highest Flow Instability Indexes

All links are stable.

Routing Time Step Summary

Minimum Time Step : 4.50 sec
Average Time Step : 5.00 sec
Maximum Time Step : 5.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00
Percent Not Converging : 0.00

Subcatchment Runoff Summary

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
Mamta_EAST_N	70.59	0.00	0.00	31.16	39.18	0.64	0.49	0.555
Mamta_EAST_S	70.59	0.00	0.00	31.40	38.91	0.61	0.46	0.551
Mamta_West_N	70.59	0.00	0.00	27.33	42.81	1.11	0.75	0.606
Mamta_West_S	70.59	0.00	0.00	31.37	38.86	0.94	0.68	0.551

Node Depth Summary

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
ExMH17	JUNCTION	0.04	0.79	219.98	0 01:26	0.78
EXPLUG39	JUNCTION	0.00	0.45	219.49	0 01:26	0.45
J1-S	JUNCTION	0.01	0.22	221.83	0 01:26	0.22
J2-S	JUNCTION	0.01	0.18	221.58	0 01:26	0.18
Ashton_Meadows_N	OUTFALL	0.00	0.00	218.00	0 00:00	0.00
Ashton_Meadows_S	OUTFALL	0.00	0.00	218.00	0 00:00	0.00

Node Inflow Summary

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
ExMH17	JUNCTION	0.000	1.093	0 01:26	0	1.75	-0.008
EXPLUG39	JUNCTION	0.000	1.003	0 01:26	0	1.55	-0.009
J1-S	JUNCTION	1.248	1.248	0 01:25	1.75	1.75	0.014
J2-S	JUNCTION	1.132	1.132	0 01:25	1.55	1.55	0.008
Ashton_Meadows_N	OUTFALL	0.000	1.093	0 01:26	0	1.75	0.000
Ashton_Meadows_S	OUTFALL	0.000	1.004	0 01:26	0	1.55	0.000

Node Surcharge Summary

No nodes were surcharged.

Node Flooding Summary

No nodes were flooded.

 Outfall Loading Summary

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
Ashton_Meadows_N	27.74	0.036	1.093	1.748
Ashton_Meadows_S	25.29	0.035	1.004	1.551
System	26.51	0.072	2.096	3.299

 Link Flow Summary

Link	Type	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Veloc m/sec	Max/ Full Flow	Max/ Full Depth
C1-S	CHANNEL	1.093	0 01:26	0.75	0.81	0.84
C2-S	CHANNEL	1.003	0 01:26	0.87	0.57	0.74
C3	DUMMY	1.093	0 01:26			
C4	DUMMY	1.004	0 01:26			

 Flow Classification Summary

Conduit	Adjusted /Actual Length	Fraction of Time in Flow Class								
		Up Dry	Down Dry	Sub Dry	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl	
C1-S	1.00	0.01	0.00	0.00	0.00	0.00	0.99	0.00	0.00	
C2-S	1.00	0.01	0.00	0.00	0.00	0.00	0.99	0.00	0.00	

 Conduit Surcharge Summary

No conduits were surcharged.

Analysis begun on: Tue Dec 11 12:01:02 2018
Analysis ended on: Tue Dec 11 12:01:02 2018
Total elapsed time: < 1 sec

Appendix G – Water Balance

THORNTHWAITE WATER BALANCE CALCULATIONS

PROJECT No. 2018-060
1191 County Rd. 42 - Mamta Homes
Clearview Township



Thorntwaite Water Balance												
Land Use Description	Approx. Land Area* (m ²)	Estimated Impervious Fraction for Land Use	Estimated Impervious Area (m ²)	Runoff from Impervious Area (m/a)	Runoff Volume from Impervious Area (m ³ /a)	Estimated Pervious Area (m ²)	Runoff from Pervious Area (m/a)	Runoff Volume from Pervious Area (m ³ /a)	Recharge from Pervious Area (m/a)	Recharge Volume from Pervious Area (m ³ /a)	Total Runoff (Direct and Indirect) Volume (m ³ /a)	Total Recharge Volume (m ³ /a)
Pre Development Site	50,300	0.024	1,207	0.738	891	49,093	0.145	7,122	0.185	9,065	8,013	9,065
TOTAL PRE-DEVELOPMENT	50,300		1,207		891	49,093		7,122		9,065	8,013	9,065
Post Development Site	50,300	0.52	26,156	0.738	19,311	24,144	0.148	3,582	0.181	4,378	22,894	4,378
TOTAL POST-DEVELOPMENT	50,300		26,156		19,311	24,144		3,582		4,378	22,894	4,378
Post Development Site with LID	50,300	0.52	26,156	0.738	19,311	24,144	0.115	2,786	0.214	5,174	22,098	5,174
TOTAL POST-DEVELOPMENT	50,300		26,156		19,311	24,144		2,786		5,174	22,098	5,174
% Change from Pre to Post											286	52
% Change from Pre to Post with LID											276	43
Effect of development (<u>with no mitigation</u>)											2.86 times increase in runoff	52% reduction of recharge
Effect of development (<u>with mitigation</u>)											2.76 times increase in runoff	43% reduction of recharge

To balance pre- to post-, the recharge target (m³/a)= **4,686**
To balance pre- to post-with LID, the recharge target (m³/a)= **3,890**

THORNTHWAITE WATER BALANCE CALCULATIONS

PROJECT No. 2018-060
1191 County Rd. 42 - Mamta Homes
Clearview Township



TABLE 1

Pre- and Post-Development Monthly Water Balance Components													
Potential Evapotranspiration Calculation	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Average Temperature (Degree C) ¹	-7.7	-6.6	-2.1	5.6	12.3	17.9	20.8	19.7	15.3	8.7	2.7	-3.5	6.9
Heat index: i = (t/5) ^{1.514}	0.00	0.00	0.00	1.19	3.91	6.90	8.66	7.97	5.44	2.31	0.39	0.00	36.8
Unadjusted Daily Potential Evapotranspiration U (mm)	0.00	0.00	0.00	25.18	58.76	88.02	103.48	97.59	74.33	40.47	11.47	0.00	499
Adjusting Factor for U (Latitude 44° 22' N) ²	0.81	0.82	1.02	1.13	1.27	1.29	1.3	1.2	1.04	0.95	0.8	0.76	
Adjusted Potential Evapotranspiration PET (mm)	0	0	0	28	75	114	135	117	77	38	9	0	593
PRE-DEVELOPMENT WATER BALANCE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Precipitation (P) ³	83	62	58	62	82	85	77	90	84	78	89	74	923
Potential Evapotranspiration (PET)	0	0	0	28	75	114	135	117	77	38	9	0	593
P - PET	83	62	58	34	8	-29	-57	-27	7	39	80	74	330
Change in Soil Moisture Storage	0	0	0	0	0	-29	-57	-27	7	39	68	0	0
Soil Moisture Storage max 250 mm	250	250	250	250	250	221	164	137	143	182	250	250	
Actual Evapotranspiration (AET)	0	0	0	28	75	114	135	117	77	38	9	0	593
Soil Moisture Deficit max 250 mm	0	0	0	0	0	29	86	113	107	68	0	0	
Water Surplus - available for infiltration or runoff	83	62	58	34	8	0	0	0	0	0	12	74	330
Potential Infiltration (based on MOE methodology*; independent of temperature)	46	35	33	19	4	0	0	0	0	0	7	41	185
Potential Direct Surface Water Runoff (independent of temperature)	36	27	26	15	3	0	0	0	0	0	5	32	145
POST-DEVELOPMENT WATER BALANCE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Precipitation (P)	83	62	58	62	82	85	77	90	84	78	89	74	923
Potential Evapotranspiration (PET)	0	0	0	28	75	114	135	117	77	38	9	0	593
P - PET	83	62	58	34	8	-29	-57	-27	7	39	80	74	330
Change in Soil Moisture Storage	0	0	0	0	0	-29	-57	-27	7	39	68	0	0
Soil Moisture Storage max 125 mm	125	125	125	125	125	96	39	12	18	57	125	125	
Actual Evapotranspiration (AET)	0	0	0	28	75	114	135	117	77	38	9	0	593
Soil Moisture Deficit max 125 mm	0	0	0	0	0	29	86	113	107	68	0	0	
Water Surplus - available for infiltration or runoff	83	62	58	34	8	0	0	0	0	0	12	74	330
Potential Infiltration (based on MOE methodology*; independent of temperature)	45	34	32	19	4	0	0	0	0	0	7	40	181
Potential Direct Surface Water Runoff (independent of temperature)	37	28	26	15	3	0	0	0	0	0	5	33	148
Potential Evaporation (PE) from impervious areas (assume 20%)	17	12	12	12	16	17	15	18	17	16	18	15	185
P-PE (surplus available for runoff from impervious areas)	66	49	46	50	66	68	62	72	67	62	71	59	738
Water surplus change compared to pre-condition (for areas that change from vegetated open areas to impervious areas)	-17	-12	-12	16	58	68	62	72	67	62	59	-15	738
POST-DEVELOPMENT WATER BALANCE WITH LID	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Precipitation (P)	83	62	58	62	82	85	77	90	84	78	89	74	923
Potential Evapotranspiration (PET)	0	0	0	28	75	114	135	117	77	38	9	0	593
P - PET	83	62	58	34	8	-29	-57	-27	7	39	80	74	330
Change in Soil Moisture Storage	0	0	0	0	0	-29	-57	-27	7	39	68	0	0
Soil Moisture Storage max 300 mm	300	300	300	300	300	271	214	187	193	232	300	300	
Actual Evapotranspiration (AET)	0	0	0	28	75	114	135	117	77	38	9	0	593
Soil Moisture Deficit max 300 mm	0	0	0	0	0	29	86	113	107	68	0	0	
Water Surplus - available for infiltration or runoff	83	62	58	34	8	0	0	0	0	0	12	74	330
Potential Infiltration (based on MOE methodology*; independent of temperature)	54	40	38	22	5	0	0	0	0	0	8	48	214
Potential Direct Surface Water Runoff (independent of temperature)	29	22	20	12	3	0	0	0	0	0	4	26	115
Potential Evaporation (PE) from impervious areas (assume 20%)	17	12	12	12	16	17	15	18	17	16	18	15	185
P-PE (surplus available for runoff from impervious areas)	66	49	46	50	66	68	62	72	67	62	71	59	738
Water surplus change compared to pre-condition (for areas that change from vegetated open areas to impervious areas)	-17	-12	-12	16	58	68	62	72	67	62	59	-15	409

Existing Condition		
Soil Moisture Storage - Silt Loam/Pasture & Shrubs	250 mm	<-- See "Water Holding Capacity" values in Table 3.1, MOE SWMPDM, 2003
		Forest 19.52%
		Urban Lawn 0.00%
		Pasture 78.09%
		Crops 0.00%
		Impervious 2.39%
		Total 100.00%
*MOE SWM infiltration calculations		<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
topography - Hilly land	0.15	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
soils - silt loam	0.3	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
cover - pasture/woodland	0.11	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
Infiltration factor	0.56	

Proposed Condition		
Soil Moisture Storage - Silt Loam/Urban Lawn	125 mm	<-- See "Water Holding Capacity" values in Table 3.1, MOE SWMPDM, 2003
		Forest 0.00%
		Urban Lawn 48.00%
		Pasture 0.00%
		Crops 0.00%
		Impervious 52.00%
		Total 100.00%
*MOE SWM infiltration calculations		<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
topography - Hilly land	0.15	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
soils - silt loam	0.3	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
cover - urban lawn	0.1	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
Infiltration factor	0.55	

Proposed Condition with LID		
Soil Moisture Storage - Sandy Loam/Urban Lawn/300 mm topsoil	300 mm	<-- See "Water Holding Capacity" values in Table 3.1, MOE SWMPDM, 2003
		Forest 0.00%
		Urban Lawn 48.00%
		Pasture 0.00%
		Crops 0.00%
		Impervious 52.00%
		Total 100.00%
*MOE SWM infiltration calculations		<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
topography - Hilly land	0.15	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
soils -sandy loam	0.4	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
cover - urban lawn	0.1	<-- Infiltration Factors from Table 3.1, MOE SWMPDM, 2003
Infiltration factor	0.65	

Latitude of site (or climate station) 44 ° N.

USER INPUTS

Notes

- 1 From Environment Canada
2 From J. M. Lorente (1961). pp. 206
3 Precipitation data from Barrie WPCC Climate Station (1981 - 2010), Environment Canada
4 Assume January storage is 100% of Soil Moisture Storage

Appendix H – Phosphorous Budget

Development Export Summary

Development : Mamta Homes - Stayner

Updated : Sept 2014

Pre-Development Phosphorus Export

DEVELOPMENT : Mamta Homes - Stayner			
Landuse	Area (ha)	P coeff (kg/ha)	Pload (kg/yr)
Natural Heritage			
Forest	0.98	0.06	0.06
Hay/Pasture	3.92	0.08	0.31
Unpaved Roads	0.12	0.83	0.10
Natural Heritage Land use Class Total :		5.02	0.47
Development Total :		5.02	0.47

12/11/2018

Page 1 of 1

Updated : Sept 2014

Cropland Site Sediment & Phosphorus Pre-Development Export

DEVELOPMENT : Mamta Homes - Stayner			
COLOUR KEY :	Site Specific Input	Constant / Lookup	Calculation
SubArea :			

Slope Area (ha)	R (rainfall / runoff for Lake Simcoe)
Surface Slope Gradient (%)	K (soil erodability factor)
Length of Slope (m)	NN (determined by slope)
Cropt Type Factor)	LS (slope length gradient factor)
Tillage Type Factor	C (crop management factor)
	P (prevention + capture)
	Soil Loss (kg/year)
	Phosphorus export (kg/ha/yr)
	Phosphorus load (kg/yr)

PRE Developed Area (ha) :
 Phosphorus export (kg/ha/yr) :
 Phosphorus load (kg/yr) :

12/11/2018

Page 1 of 1

Post-Development Phosphorus Export

DEVELOPMENT : Mamta Homes - Stayner				
Landuse	Area (ha)	P coeff (kg/ha)	Pload (kg/yr)	
Urban				
Residential	5.02	0.41	2.06	
Urban Land use Class Total :		5.02	2.06	
Development Total :		5.02	2.06	

12/11/2018

Page 1 of 1

Updated : Sept 2014

Cropland Site Sediment & Phosphorus Post-Development Export

DEVELOPMENT : Mamta Homes - Stayner			
COLOUR KEY :	Site Specific Input	Constant / Lookup	Calculation
SubArea :			
Slope Area (ha)	R (rainfall / runoff for Lake Simcoe)		
Surface Slope Gradient (%)	K (soil erodability factor)		
Length of Slope (m)	NN (determined by slope)		
Cropt Type Factor	LS (slope length gradient factor)		
Tillage Type Factor	C (crop management factor)		
	P (prevention + capture)		
	Soil Loss (kg/year)		
	Phosphorus export (kg/ha/yr)		
	Phosphorus load (kg/yr)		

PRE Developed Area (ha) :

Phosphorus export (kg/ha/yr) :

Phosphorus load (kg/yr) :

12/11/2018

Page 1 of 1

Post Dev BMP

Area (ha)	Treated Area %	P coefficient	P coefficient	P Load Reduction (kg/yr)	Rationale
Best Management Practices (BMP) Applied (and Rationale)					
Residential					
5.02	100	0.41	100 %	2.06	
User Entry					

12/11/2018

Page 1 of 1

Updated : Sept 2014

Development Area P and BMP Summary

Total PreDevelopment Area (ha):	5.02
PreDevelopment Area excluding Wetlands (ha):	5.02
Total PostDevelopment Area (ha):	5.02
Total Area treated by BMP's (ha):	5.02
Treated Area total:	5.02
Total PreDevelopment Load (kg/yr):	0.47
Total PostDevelopment Load (kg/yr):	2.06
Total P Load Reduction with BMP's (kg/yr):	2.06
Minimum P Load Reduction Required:	1.59
Total PostDevelopment Load with BMP's (kg/yr)	0.00
Conclusion :	No Net Increase in P Load.

12/11/2018

Page 1 of 1

Updated : Sept 2014

Post Dev Construction

12/11/2018

Page 1 of 1

12/11/2018

Page 3 of 3