Stewart Weir

Town of Wainwright

Enstrom Subdvision

Phase 1

Prepared for: Town of Wainwright

Prepared by: Stewart Weir

Our file no.: ED60 33962

Date: November, 2010

Preliminary Design Report

BEST SMALL & MEDIUM EMPLOYERS IN CANADA

SYSTEM CERTIFICATION ISO 9001:2008 SGS

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1.0 INTRODUCTION

1.1 BACKGROUND

Stewart, Weir & Co. Ltd. was retained by the Town of Wainwright to prepare a detailed design for Phase 1 of the Enstrom Subdivision. This preliminary design report identifies and appraises the existing infrastructure, presents feasible servicing options for the subdivision, discusses project challenges and limitations and outlines the anticipated project costs. This report is a valuable tool in the design process allowing review and discussion with key project stakeholders at a critical point in the design process.

1.2 OBJECTIVES

The objectives of this study are as follows:

1) To identify the capacity of the existing infrastructure to support the proposed development and suggest appropriate off-site upgrades.
2) To identify the most suitable servicing options allowing for future development.
3) To estimate the capital cost associated with the proposed options.
4) To make a recommendation as to the most feasible overall system for the subdivision and present a preliminary layout.

1.3 SCOPE OF WORK

The scope of work for this design report includes the following features:

- Gather and review all relevant information including as-built plans, maintenance records, previous studies, population growth and future projections.
- Discussion and recommendation of required upgrades to existing off-site infrastructure.
- Discussion of design parameters used to prepare the preliminary layouts.
- Present findings of geotechnical investigation.
- Presentation of the conceptual design for the recommended servicing options.
- Prepare a preliminary construction cost estimate.

- Provide cost-effective recommendations for decision-making.

1.4 PROJECT LOCATION

The Enstrom subdivision is within SE – 32 – 44 – 06 - W4M in the Town of Wainwright, Alberta, bounded on the east by 23rd Street, on the west by Highway 41 (Buffalo Trail) and on the north and south by 8th Avenue and 1st Avenue respectively. Phase 1 of the subdivision encompasses approximately the west half of the quarter section. The project location is shown on *Figure 1 – Location Plan*.

1.5 LIMITATIONS

This study presents the preliminary design concepts and the design parameters used in their development. This study should not be interpreted as a final detailed design. Detailed design drawings and specifications will be prepared as the next stage of the project.

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION, PHASE 1

LOCATION PLAN

S.E.1/4 SEC.32-44-6-W4

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NOT TO SCALE

FIGURE 1.1

October 26, 2010

\\Swg.ca\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 8x11 LOCATION PLAN.dwg

2.0 INFORMATION GATHERING AND REVIEW

2.1 PREVIOUS STUDIES

The following plans and studies were supplied by the Town and reviewed in preparation of the preliminary design:

- Town of Wainwright Design Standards
- Town of Wainwright Water Supply System Study, July 2003 by Morrison Hershfield
- Baier Subdivision Phase 1 Residential, 8th Ave Water and Sewer Plan and Profile, June 2001
- Town of Wainwright 23rd Street Storm Drainage System, 1994 by Maxim Engineering Inc.
- East Wainwright Subdivision – Phase 1, Dry Pond/Lift Station, 1991
- East Wainwright Area Structure Plan, July 1990
- Town of Wainwright Servicing Study, East Wainwright Subdivision, July 1990 by Wardrop Alberta Ltd.
- Town of Wainwright 1st Ave Sanitary Trunk Collector Sewer, Plan and Profile 1st Ave, 1978 by W.J. Francl & Associates
- Water Extension Plan, Profile 1st Ave, East Wainwright Subdivision, 1977

2.2 SITE INSPECTION

A review of the existing site conditions was undertaken by the Stewart Weir team during a site visit on June 23, 2010. Town administration and operations personnel were consulted during this visit and project start-up session.

Information gathered during the site inspection and discussions with Town personnel include the following:

- operation and maintenance information for the existing systems
- design drawings of the sanitary / water / storm systems
- existing studies and planning documents
- proposed lot and road layout and land use

- road cross sections for the various road classifications
- preferred utility alignments
- future growth areas

Visual inspection and operational experience of the Town Public Works personnel provided the primary tool for capacity assessment of the existing infrastructure.

2.3 SITE SURVEY

The site survey was conducted by others under a separate contract to the Town and provided to Stewart Weir for the design component. The Stewart Weir design staff identified additional features requiring survey. The Town f Wainwright provided the identified survey requirements at a later date.

3.0 EXISTING UTILITIES

3.1 TOPOGRAPHY AND EXISTING FEATURES

The topography in the area is lightly rolling with general slopes towards the southwest. Most of the quarter section is currently farmed with the exception of a stand of trees in the west-central portion of the land. The trees surround a low area that has been noted to pond intermittently during wet weather.

The land is bordered on the north and west by existing residential and light industrial development. Farmland is located to the east and south of the development.

3.2 WATER DISTRIBUTION SYSTEM

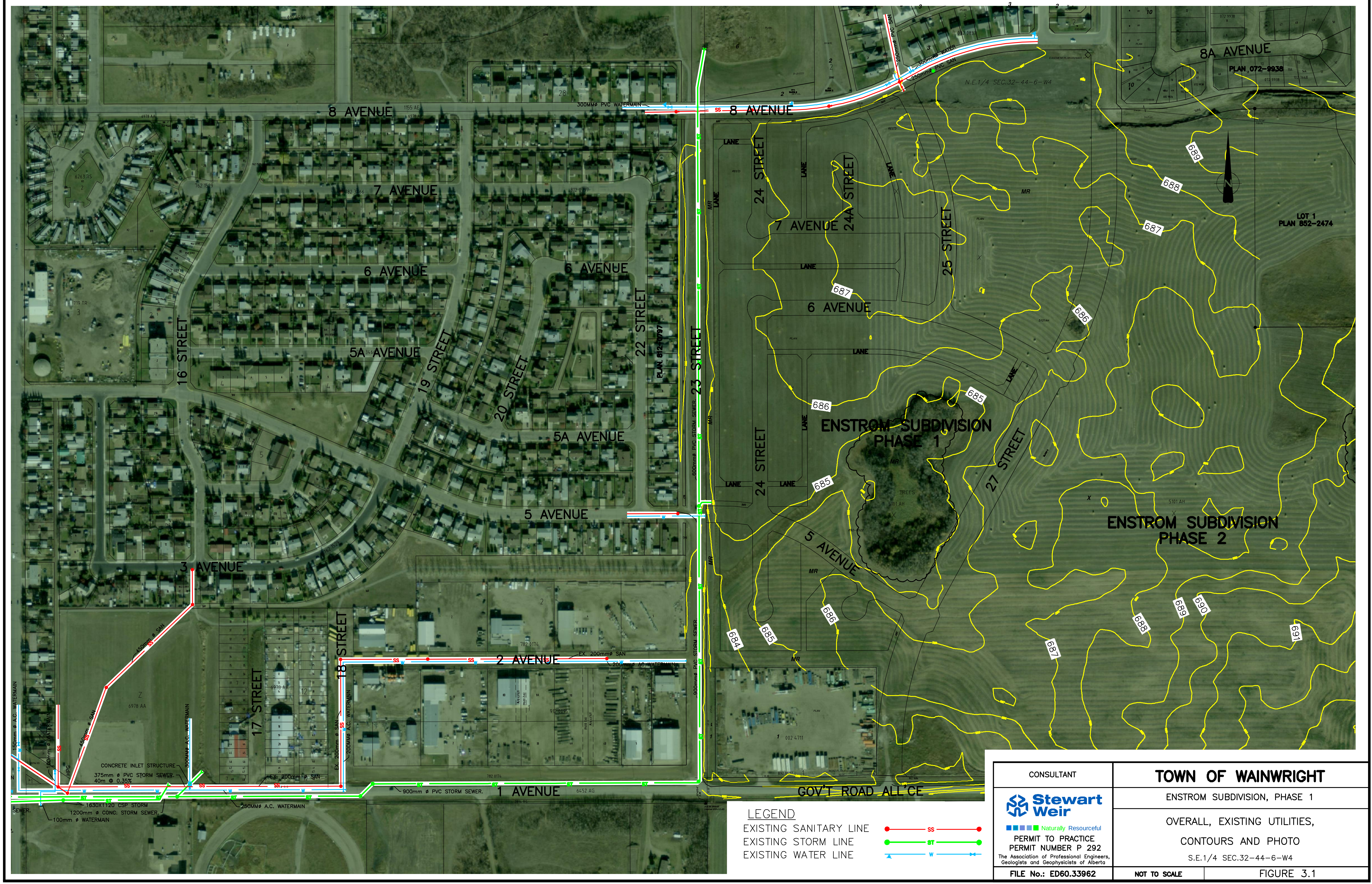
The water distribution system consists of a piped network adjacent to the subdivision to the north and west. The overall water network has been studied in previous reports and upgrades have been made by the Town over the past several years. The existing system is capable of supporting the proposed subdivision and the additional demand has been factored into the overall modeling. A small portion of off-site trunk main extension will be required along 1st Ave.

3.3 SANITARY SEWER SYSTEM

The town sanitary sewer system consists of a network of gravity sewer lines flowing directly to the lagoon with only one lift station servicing a small area of town. A trunk sewer line is located west of the proposed subdivision on 1st Avenue that has been identified as the logical connection point. Portions of the existing lines in the downtown area are approaching their capacity and Town personnel have expressed their desire to divert flow from the Baier subdivision, south through the proposed subdivision to the trunk line on 1st Avenue in order to alleviate potential bottle necking.

3.4 STORM SEWER SYSTEM

The existing storm sewer system consists of a network of ditching, storm ponds and piped storm sewer draining generally to the southwest. Some concerns have been noted with the system capacity in various portions of the Town including the storm outfall to the intermittent water body south of the railway tracks. Detailed study of the overall town stormwater management system is required but does not form part of this design report.



LEGEND

EXISTING SANITARY LINE	●——SS——●
EXISTING STORM LINE	●——ST——●
EXISTING WATER LINE	——W——

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION, PHASE 1

OVERALL, EXISTING UTILITIES,
CONTOURS AND PHOTO

S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 3.1

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DATE: September 1, 2010

4.0 DESIGN PARAMETERS

4.1 STANDARDS AND GUIDELINES

The design of the water and wastewater systems will meet the Standards and Guidelines for Municipal Wastewater Systems published by Alberta Environment and the Town of Wainwright Design Standards.

4.2 POPULATION PROJECTION

The population design criteria used for analysis is as follows:

• Proposed number of lots	139
• Population density	3.5 persons/dwelling
• Total population	487

4.3 SEWAGE COLLECTION

The following design criteria will be used in the preliminary design of the sanitary sewer system:

• Wastewater generation	400 litres/capita/day
• Peaking Factor	Harmon’s formula
• Infiltration Allowance	0.28 litres/second/ha
• Maximum velocity	3.0 m/sec
• Minimum velocity	0.6 m/sec
• Minimum pipe cover	2.75m

Gravity System

• Minimum slope (gravity system)	0.4%
• Maximum manhole spacing	120m
• Minimum pipe size	200 mm diameter
• Minimum service size	100 mm diameter

4.4 WATER SUPPLY

The following design criteria will be used in preparing the preliminary design for the water distribution system:

Average daily consumption	450 litres/capita/day
Peak Hour Demand	1,100 litres/capita/day
Maximum Day Demand	800 litres/capita/day
Minimum residual pressure (fire flow)	140 kPa (20 psi)
Minimum residual pressure (peak flow)	280 kPa (40 psi)
Maximum velocity	3.0 m/sec
Minimum velocity	0.6 m/sec
Maximum Hydrant Spacing	150m
Minimum Distribution Main Size	150mm diameter
Minimum service size	20mm diameter
Minimum cover for frost protection	3.0 m

4.5 ROADWAYS

The geometric design standards used in the development of the preliminary design include:

Minimum grade around curves	0.60%
Minimum gutter grades around curb returns	1.00%
Minimum grade along tangents	0.50%
Maximum gutter grade	6.0%
Design speed	50km/hr
Minimum curb return radius	8m

5.0 GEOTECHNICAL INVESTIGATION

5.1 REPORT SUMMARY

On August 4th, 2010, fifteen boreholes (BH 10-1 to BH 10-15) were drilled within the Phase 1 boundaries. Six boreholes were drilled to a depth of 8.7 m and nine boreholes were drilled to a depth of 5.7 m below ground surface. Supervision of drilling, soil sampling, and logging of the various soil strata was performed by Mr. Dale Johnston, a senior geotechnical technician at Stewart Weir. The soil conditions encountered during drilling were described in accordance with the Modified Unified Soil Classification System described in the report. Approximate borehole locations are presented on the site plan included with the report.

Standpipe peizometers were installed in 10 of the boreholes to measure the groundwater elevations. Groundwater was measured 11 days after installation and ranged from a depth of 2.0 to 4.10 meters.

Topsoil was discovered at a depth ranging from 0.1 to 0.2m deep across the site. These depths will be used when calculating stripping volumes and roadway cut/fill volumes.

The soils conditions discovered on site are considered generally favourable for the proposed building foundations, utility installation and road construction. No extraordinary conditions were noted that will significantly impact the construction. Detailed recommendations for building foundations, trenching, pipe bedding, backfilling, road structure and storm pond construction are included in the full Geotechnical Investigation included as Appendix A to this report.

6.0 SANITARY SEWER SYSTEM

6.1 SYSTEM CONCEPT

The existing subdivisions to the north and west of the Enstrom subdivision have gravity sewer systems that were investigated as possible connection points for the development. As a result of discussions with Town public works personnel, it was discovered that the existing systems drain generally to the southwest. The sewer along 8th Avenue on the north boundary of Phase 1 collects wastewater from the Baier Subdivision along with anticipated future development to the northeast. The design capacity of this line has been reached and the Town has expressed the desire to divert flow from this area through the Enstrom subdivision to relieve possible bottlenecking further west in the existing Town system.

The subdivision immediately west of Enstrom is also serviced by a gravity system flowing to the west. A possible connection to the existing line on 5th Avenue has been ruled out as a feasible option due to capacity and depth restrictions.

An existing 600mm diameter trunk sewer main is located along 1st Avenue at 14th Street. A smaller diameter line services the existing businesses and residences along 1st Ave to the east of 14th Street. In order to provide adequate service to the proposed Enstrom subdivision, the trunk main should be extended along 1st Avenue to 23 Street as shown on Figure 6.1 Sanitary Sewer System Concept. This trunk main extension should be sized to intercept flow from the existing lots on 1st Avenue as well as the anticipated flow from Enstrom Phase 1 and 2 and the diverted flow from the Baier subdivision to the north.

As part of the preliminary design process, it was discovered that there is not sufficient grade to divert the flow by gravity from the existing sewer on 8th Avenue, through the Enstrom subdivision to the extended trunk main on 1st Avenue while meeting minimum pipe grade requirements. As a result, we are proposing a possible future lift station site at the intersection of 8th Avenue and 25th Street that would intercept flow and pump it into the Enstrom system. The trunk main through Phase 1 of the Enstrom subdivision has been sized to accommodate this additional future flow.

6.2 PROPOSED LAYOUT AND PHASING

Figure 6.1 Sanitary Sewer System Concept outlines the proposed system layout for Phase 1 and the off-site upgrades. The off-site trunk sewer extension will be located along the north side of 1st Avenue. The Town is in the process of acquiring the lots adjacent to 1st Avenue at 17th Street in order to construct the trunk main outside of the current road right-of-way. This option is being pursued in order to minimize conflict with existing utilities.

East of 18th Street, the trunk main alignment will fall within the existing gravel service road to 23rd Street and north along 23rd Street to 5th Avenue. This alignment was selected over a 27th Street alignment in order to minimize the length of oversized main to the proposed future lift station location.

As shown on Figure 6.1, the internal main from 23rd Street to the future lift station location has been oversized in order to accommodate the future flow. Connection points have also been planned for Phase 2 of the development at 6th Avenue and 5th Avenue. The 200mm minimum pipe diameter has sufficient capacity at these points to accommodate this future flow.

6.3 CATCHMENT AREAS AND FLOW CALCULATIONS

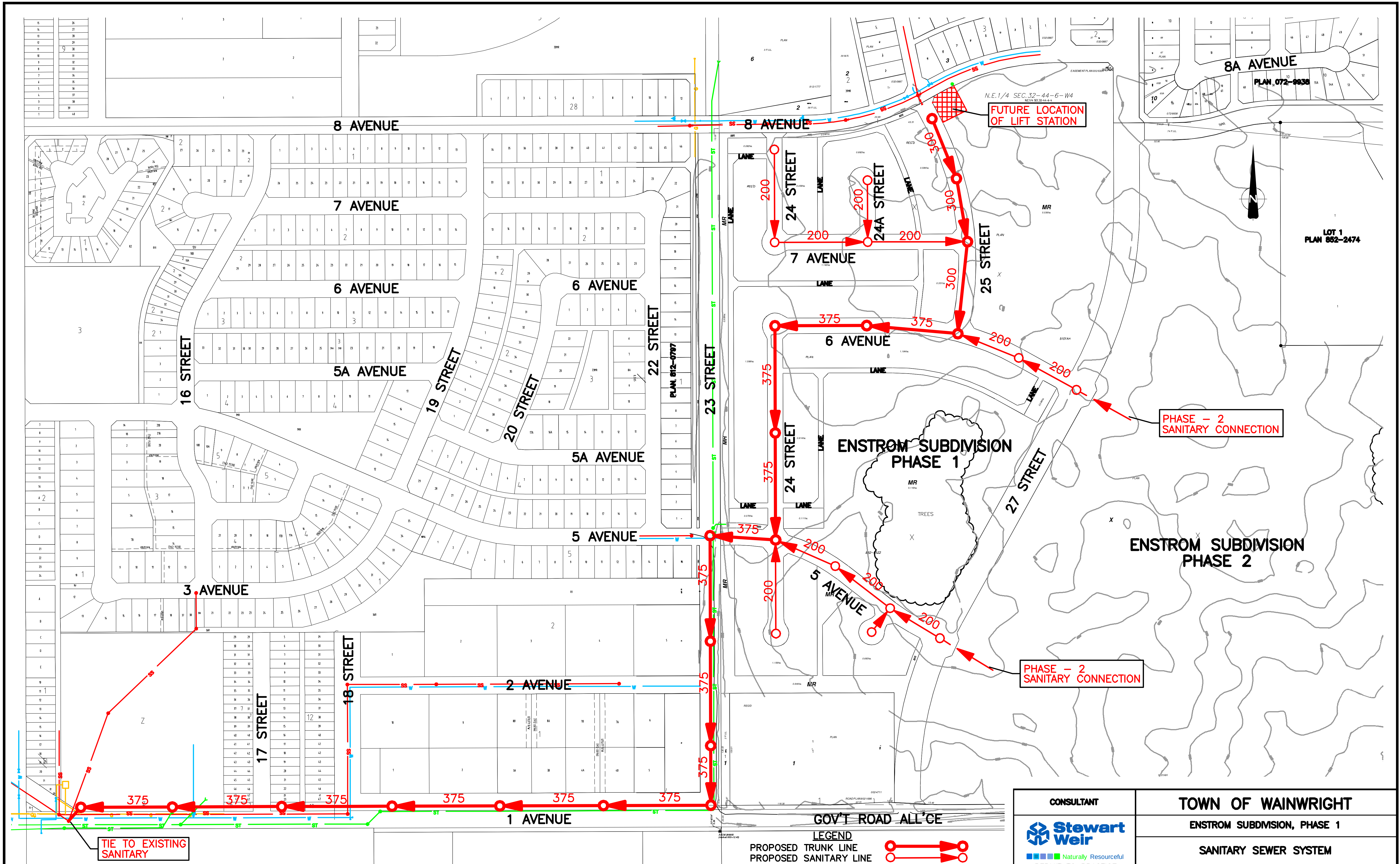
The catchment area for the sanitary sewer system includes Phase 1 and 2 of the Enstrom Subdivision as well as Phases 1, 2 and 3 of the existing Baier Subdivision.

The contributing area from Phase 2 of the Enstrom subdivision has been divided into two parts. The north section will contribute to the proposed stub at 6th Avenue and 27th Street. Using the concept plan provided by the Town, we have estimated a total of 127 lots for this north portion. The south portion will contribute to the proposed stub at 5th Avenue and 27th Street. The total number of lots for the south contributing area has been estimated at 96 for a total of 223 lots in Phase 2 of the development.

The main trunk line through the subdivision from the future proposed lift station location to the existing 600mm dia. trunk main has been sized to accommodate flow from Phases 1, 2 and 3 of the existing Baier Subdivision. This has been estimated at a total of 558 lots. Since a significant portion of this contributing area is already being directed west along 8th Avenue, the Town will have the option of allocating this additional 558 lot capacity for future development east of the Baier subdivision. This will require further study as amendments to the current Area Structure Plan are made.

Future contributions from developments east of Highway 41 are limited by the grade on the sewer in Phase 2 of the Enstrom Subdivision. Although the natural grade is generally towards the south west for lands east of the Highway, it is unlikely that a gravity sewer system could be extended while maintaining minimum depth requirements.

Undeveloped lands south of 1st Avenue have similar limitations. The trunk sewer extension along 1st Ave will be able to accommodate direct connections from land immediately adjacent to the avenue, although significant subdivision development would likely have to be supported by a new lift station to the south and west.



LEGEND

PROPOSED TRUNK LINE
PROPOSED SANITARY LINE
EXISTING SANITARY LINE — SS
EXISTING STORM LINE — ST
EXISTING WATER LINE — W
EXISTING GAS LINE — G

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION, PHASE 1

SANITARY SEWER SYSTEM
CONCEPT

S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 6.1

\\Swp.ca\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 11x17 SANITARY FIGURES.dwg

DATE November 8, 2010

7.0 WATER DISTRIBUTION SYSTEM

7.1 SYSTEM CONCEPT

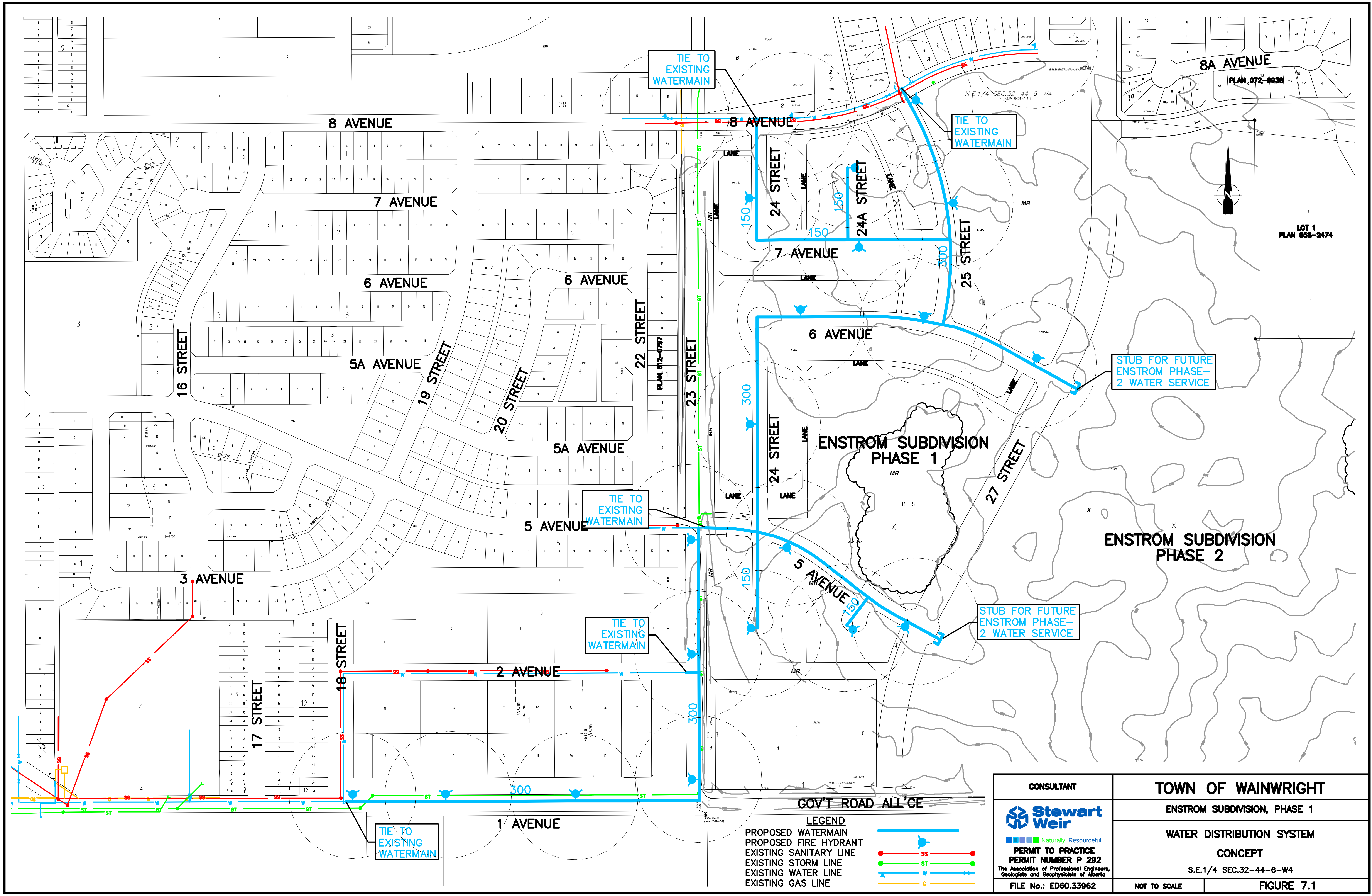
Upgrades to the town water supply system have been implemented in recent years. A Water Supply System Study was prepared by Morrison Hershfield in 2003 assessing the complete water network for the town. The study area includes the proposed Enstrom Subdivision with a mix of residential, mobile home and commercial uses.

No significant concerns were noted in the study or by public works personnel regarding the proposed subdivision. The study recommends the installation of a trunk 300mm diameter watermain with connection at the north on 8th Avenue in the Baier subdivision and connection on the south (off-site) at 18th Street and 1st Avenue. Peak hour pressure ranges are expected to be between 45 and 62 psi.

7.2 PROPOSED LAYOUT AND PHASING

Smaller diameter looping connections will be made along 23rd Street at 2nd and 5th Avenues as well as 24th Street and 8th Avenue. Stub connections will be installed for Phase 2 at 5th and 6th Avenue.

Figure 7.1 shows the proposed layout, pipe sizing and stubs for future connection.



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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION, PHASE 1

WATER DISTRIBUTION SYSTEM CONCEPT

S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 7.1

DATE November 8, 2010

\\Swg.ca\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 11x17 WATERLINE FIGURES.dwg

8.0 STORMWATER MANAGEMENT

8.1 SYSTEM CONCEPT

Drainage for the residential lots within the proposed subdivision will all be overland flow, with the majority of the lots having split drainage. Runoff from the lots will either drain towards the street, a back lane, a drainage ditch or the storm pond. Runoff that does not drain directly into the storm pond, will be collected by catch basins, and will enter the storm pond via the storm sewer. Runoff will be detained within the storm pond and will be released at the pre-development rate of release.

8.2 OUTLET CAPACITY

The outlet of the storm pond will connect into the existing storm trunk line that runs north to south along the west edge of the development. The tie-in point for the storm pond outlet will be at the existing storm manhole located at 5th avenue and 23rd street. At this manhole the trunk line transitions from a 600mm diameter pipe to a 900mm diameter pipe. Up stream of the manhole, the 600mm diameter line drains the existing storm detention pond north of 8th avenue.

The available capacity of the storm trunk line at the tie-in manhole for the proposed storm pond outlet is restricted by difference of capacity between the 600mm diameter pipe and the 900mm diameter pipe. The calculated difference in capacity of these two pipes is 0.349 m3/sec. This is assuming that during a major storm event the 600mm diameter line is running at full capacity.

The calculated 1:100 year, pre-development flow rate calculated for the area of Enstrom Phase 1 and 2 is 2.22 m3/sec. Ideally, this is the recommended rate of release for the proposed storm pond. However, based on the above calculations, it is evident that the existing trunk line does not have the capacity to accommodate this output.

8.3 STORMWATER MANAGEMENT POND

The MR for the proposed storm pond for Enstrom Phase 1 and 2 is to be located between 5th and 6th avenue, and 24th and 27th street. Preferably, the pond will occupy the west half of the MR, keeping intact the majority of the treed area located to the east. In this scenario, the pond is sized for a 1:100yr event with an allowable release rate equal to the pre-development flow rate of the development. However, as mentioned in section 8.2 the pre-development rate is greater than the supplementary capacity of the storm trunk sewer. This proves to be a crucial issue as reducing the discharge rate of the pond to only the supplementary capacity of the trunk line, the pond would need to be approximately 2.5 times in size. This would mean that the pond would need to be at a deeper depth, and occupy the majority of the MR.

8.4 CATCHMENT AREA AND FLOW CALCULATIONS

Phase 1 of the proposed Enstrom Subdivision has been divided into 6 catchment areas. Catchment area 1 is located in the north-west corner, and consists of the area between 6th and 8th avenue, west of 25th street. Catchment area 2 is comprised of the proposed school site, and the adjacent properties along 25th street, in the north-east corner of the development. Catchment area 3 covers the north half of 27th street, and the future north half of Enstrom Phase 2. The south half of 27th street and the south half of the future Enstrom phase 2 development are covered by catchment area 4. Catchment area 5 includes the properties south of 5th avenue up to 27th street, and approximately half the bordering lots, along 24th street, between 5th and 6th avenue. The storm water pond itself and the bordering properties along 24th street and 6th avenue comprise catchment area 6.

Each catchment area is defined as a separate area that contributes runoff to the storm pond. The catchment areas are separated by separate storm sewer systems which drain certain areas of the subdivision directly into the storm pond.

Due to grading constraints there are certain areas that will not drain into the storm pond. These areas are relatively small and isolated and will not affect the overall drainage pattern of the existing surrounding areas. The south portion of 27th street, up to approximately the north boundary of the adjacent private property will need to drain onto 1st avenue. Additionally, the back of lots, west of 24th street will drain into the existing drainage ditch, which runs north to south along the west boundary of the site.

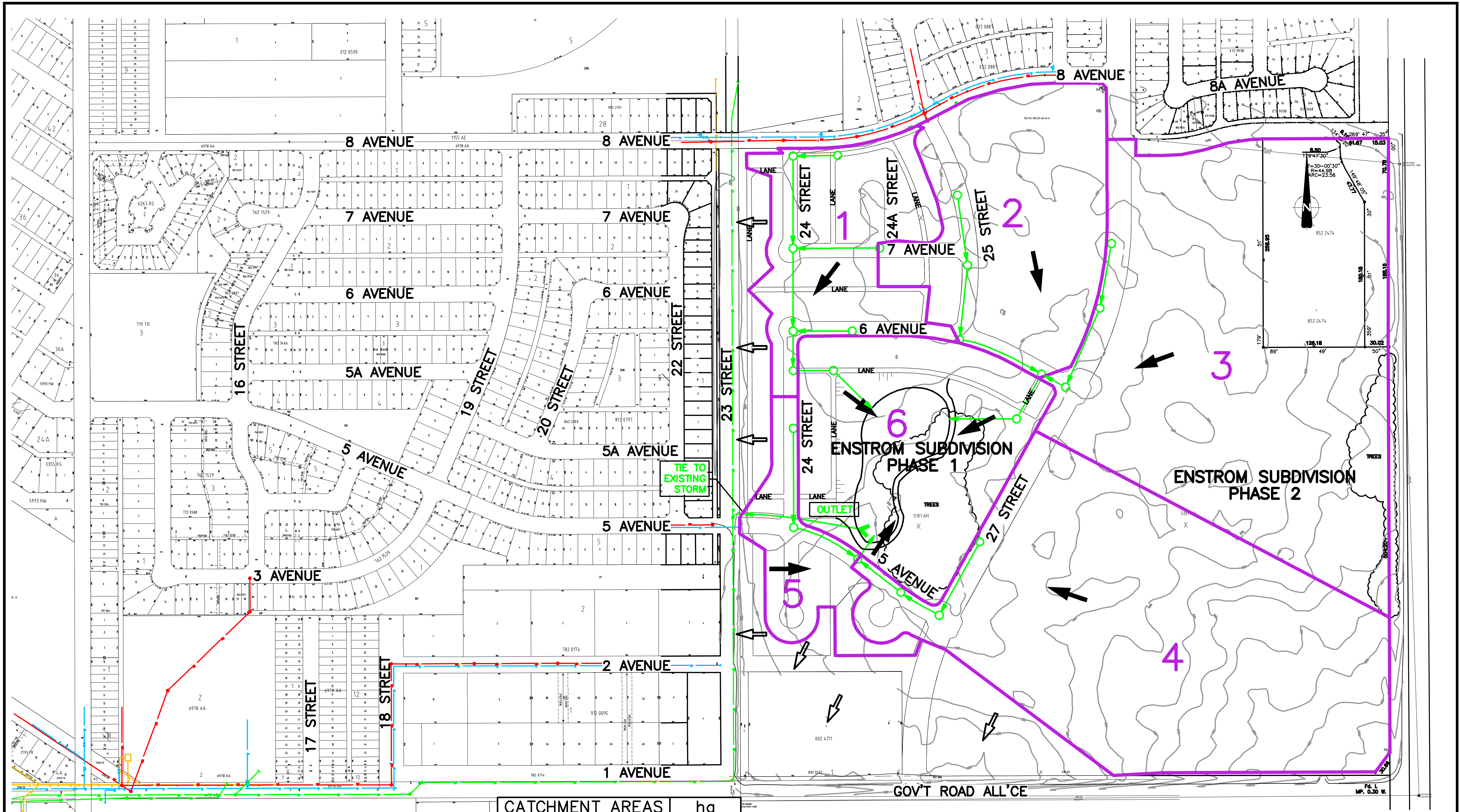
8.5 PROPOSED LAYOUTAND PHASING

The proposed storm sewer system has been laid out to drain the separate catchment areas to the storm water management pond. The storm sewer is to be set at minimum grades and burial depths wherever practical. Straight line sewers will be set at a minimum of 0.2% with curved sewers at 0.3%. As a means of frost protection, the main lines will be buried at a minimum depth of 1.2 from finished grade to the crown of the pipe. Where the proper depth for frost protection cannot be attained, insulation around the pipe will be used. Catch basin leads will be set at a minimum of 1%, and will also be set at an adequate depth to prevent freezing.

In general, the storm sewer will be within the road right-of-way, offset from the center line of the road. A few instances have been identified where the storm sewer or catch basins are required to be within the gravel lane ways. These instances however have been kept to a minimum, as the majority of the lane ways are to be graded so that they drain either towards the road, ditch or the storm pond. At this point, only one PUL has been indentified for the storm sewer. The location of the PUL is along 24th street, between 7th and 6th avenue.

As a means of keeping the costs down, individual storm services have not been proposed for the lots as this point. Storm services typically will tie into a foundation's subsurface drainage system. While the requirement for a subsurface drainage system is as specified in the Alberta Building Code, the outlet of the subsurface drainage system should divert the ground water above land, away from the foundation of the house. Should the requirement for individual storm services arise, the additional costs would involve the installation of additional storm services per each lot, and the extension of the storm sewer main line in certain areas for the services to tie into.

To accommodate phase 2 construction, main line stubs for future storm sewer development will be installed at the intersections of 27th street and 5th avenue, and at 27th street and 6th avenue. The downstream portions of the storm sewer from these locations to their outlets at the storm pond will be sized to also accommodate the post development runoff of phase 2.



CATCHMENT AREAS	ha
1	4.86
2	7.38
3	18.41
4	16.38
5	2.41
6	7.43

LEGEND

OUTSIDE CATCHMENT AREA OVERLAND FLOW DIRECTION	⇨
WITHIN CATCHMENT AREA OVERLAND FLOW DIRECTION	➡

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION

STORMWATER MANAGEMENT CONCEPT

S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 8.1

DATE November 8, 2010

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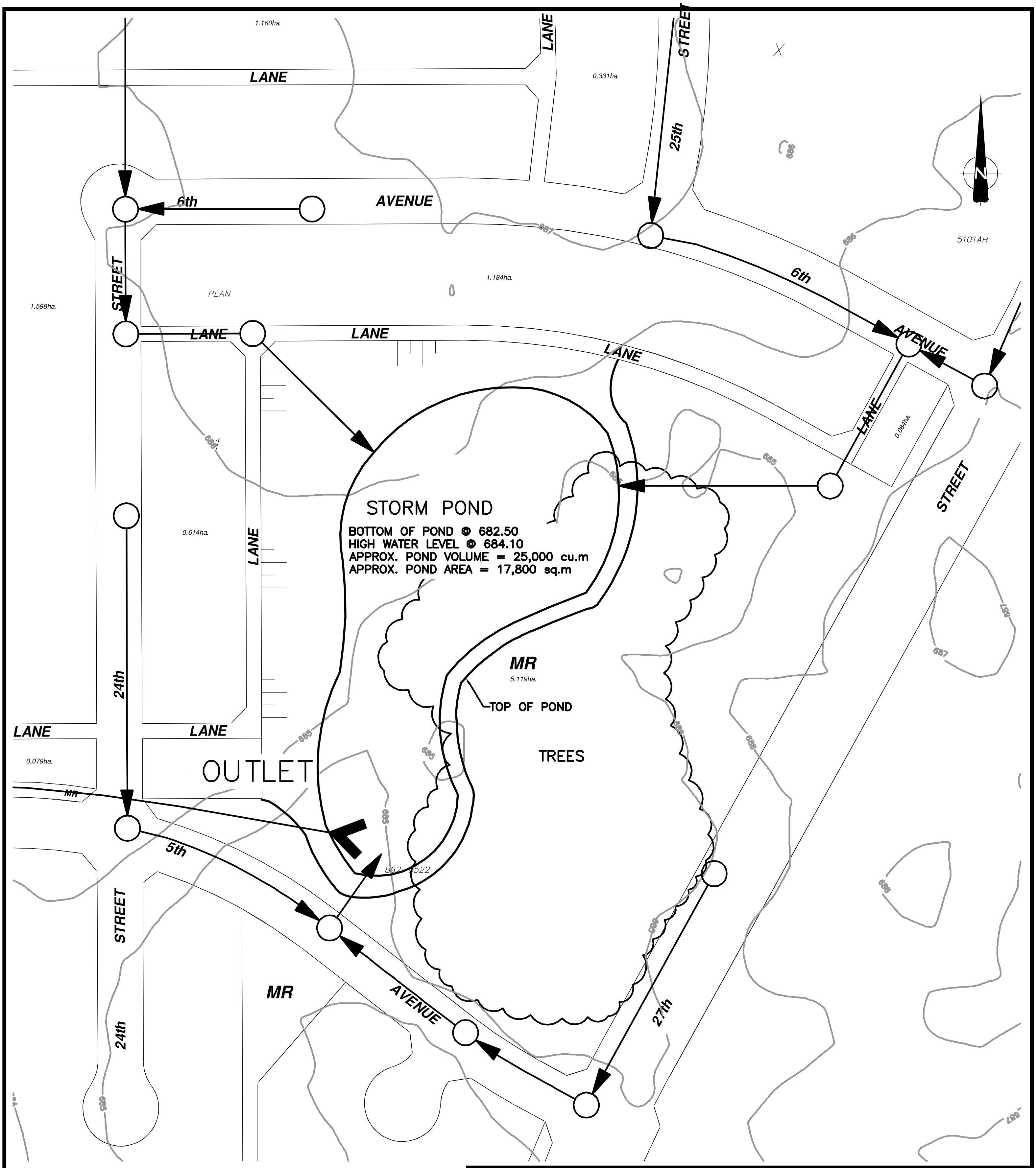

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ENSTROM SUBDIVISION, PHASE 1

PROPOSED STORMWATER MANAGEMENT POND

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FIGURE 8.2

October 26, 2010

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9.0 ROADWAYS

9.1 PROPOSED ROAD CROSS SECTIONS

Four road cross sections have been developed for use in the Enstrom subdivision. Figure 9.1 outlines the proposed road layout. The cross sections are detailed on Figures 9.2 through 9.6. Figure 9.7 shows the proposed location of the deep utilities and street furniture as it relates to the road centreline.

11.5m Arterial (27th Street)

- 30m wide right-of-way
- 11.5m wide finished surface from face of curb
- Rolled face curb and gutter
- Separate 2.5m asphalt multi-use trail
- No parking

12.5m Collector (5th Avenue)

- 20m wide right-of-way
- 12.5m wide finished surface from face of curb
- Rolled face curb and gutter
- Monolithic curb, gutter and sidewalk on one side
- Parking permitted on one side

11.5m Local

- 18m wide right-of-way
- Rolled face curb and gutter
- Monolithic curb, gutter and sidewalk on one side
- Parking permitted on one side
- Parking permitted on both sides for cul-de-sacs.

11.5m Local (School Zone)

- 20m wide right-of-way
- Straight face curb and gutter bordering school property
- Rolled face curb and gutter facing residential property
- Monolithic curb, gutter and sidewalk on school side
- Parking permitted on residential side

4.0m Lane

- 6m wide right-of-way
- 4m wide gravel surface
- Reverse cross-fall allowing drainage down centre of lane or cross-fall across entire lane allowing drainage to one side (bordering park or storm pond)

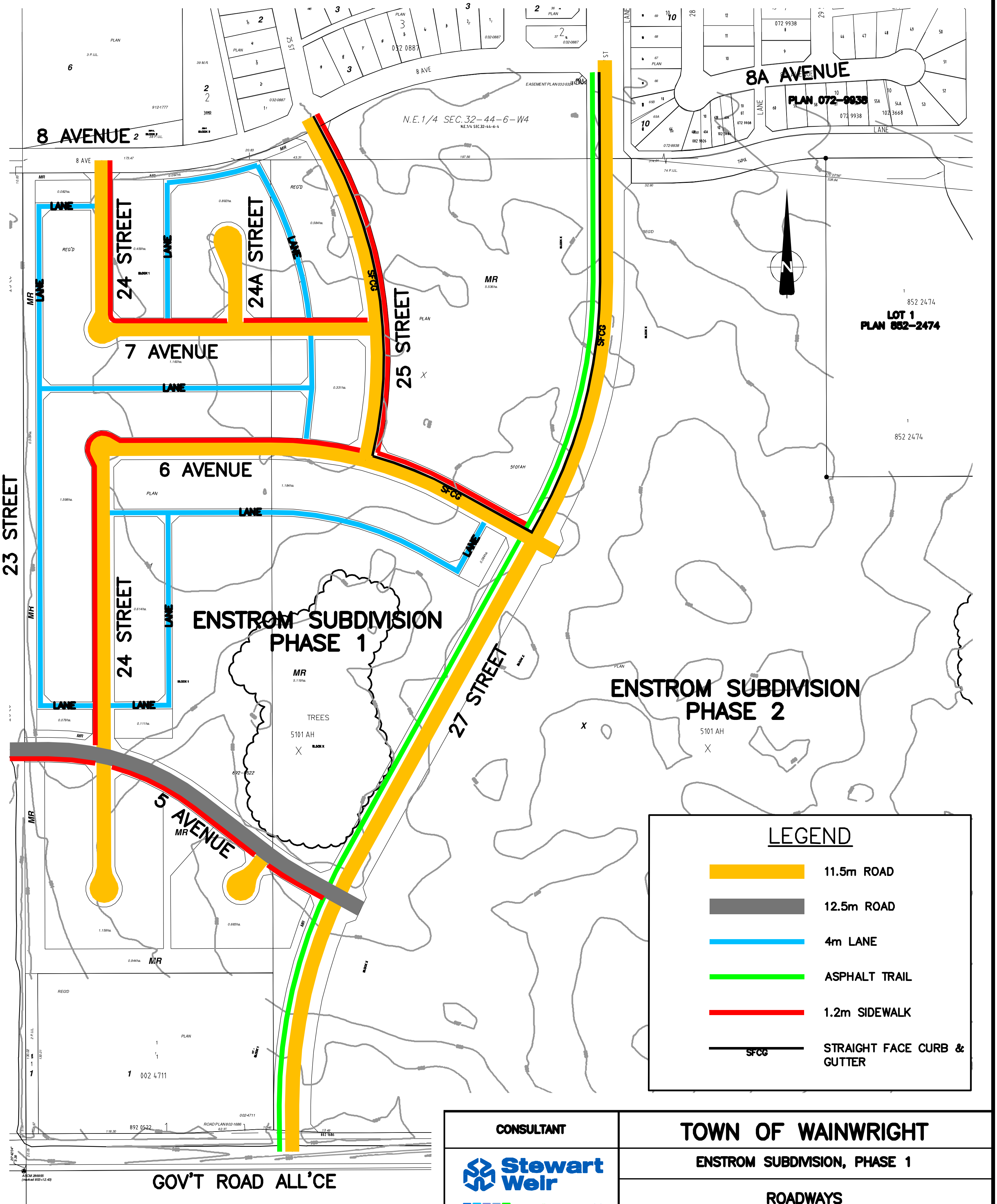
9.2 SIDEWALKS AND MULTI-USE TRAILS

Monolithic curb, gutter and sidewalk is proposed for one side of the street throughout most of the subdivision. No sidewalk is proposed for the three cul-de-sacs. A separate asphalt multi-use trail is proposed along the west side of 27th Street according to the overall trail plan for the Town of Wainwright. Some initial discussion was had regarding the possibility of extending this multi-use trail west along 5th Avenue, however, the proposed 20m right-of-way and wide road proposed to allow for parking limits the available space for a separate walkway.

9.3 PROPOSED ROAD STRUCTURES

The geotechnical investigation included as Appendix A outlines proposed road and laneway structure based on the existing soil conditions. The table is replicated below:

Road Classification	Proposed Structure
Local Residential	35 mm asphalt concrete @ FAC 65 mm asphalt concrete
	250 mm crushed gravel (20 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade
Minor Collector (5th Avenue)	35 mm asphalt concrete @ FAC 65 mm asphalt concrete
	275 mm crushed gravel (20 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade
Major Collector (27th Street)	35 mm asphalt concrete @ FAC 90 mm asphalt concrete
	300 mm crushed gravel (20 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade
Residential Lane	100 mm crushed gravel (20 mm) 250mm crushed gravel (50mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade



LEGEND

🟨	11.5m ROAD
▬	12.5m ROAD
🟦	4m LANE
🟩	ASPHALT TRAIL
🟥	1.2m SIDEWALK
— SFCG —	STRAIGHT FACE CURB & GUTTER

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION, PHASE 1

ROADWAYS
CONCEPT

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NOT TO SCALE

FIGURE 9.1

DATE November 8, 2010

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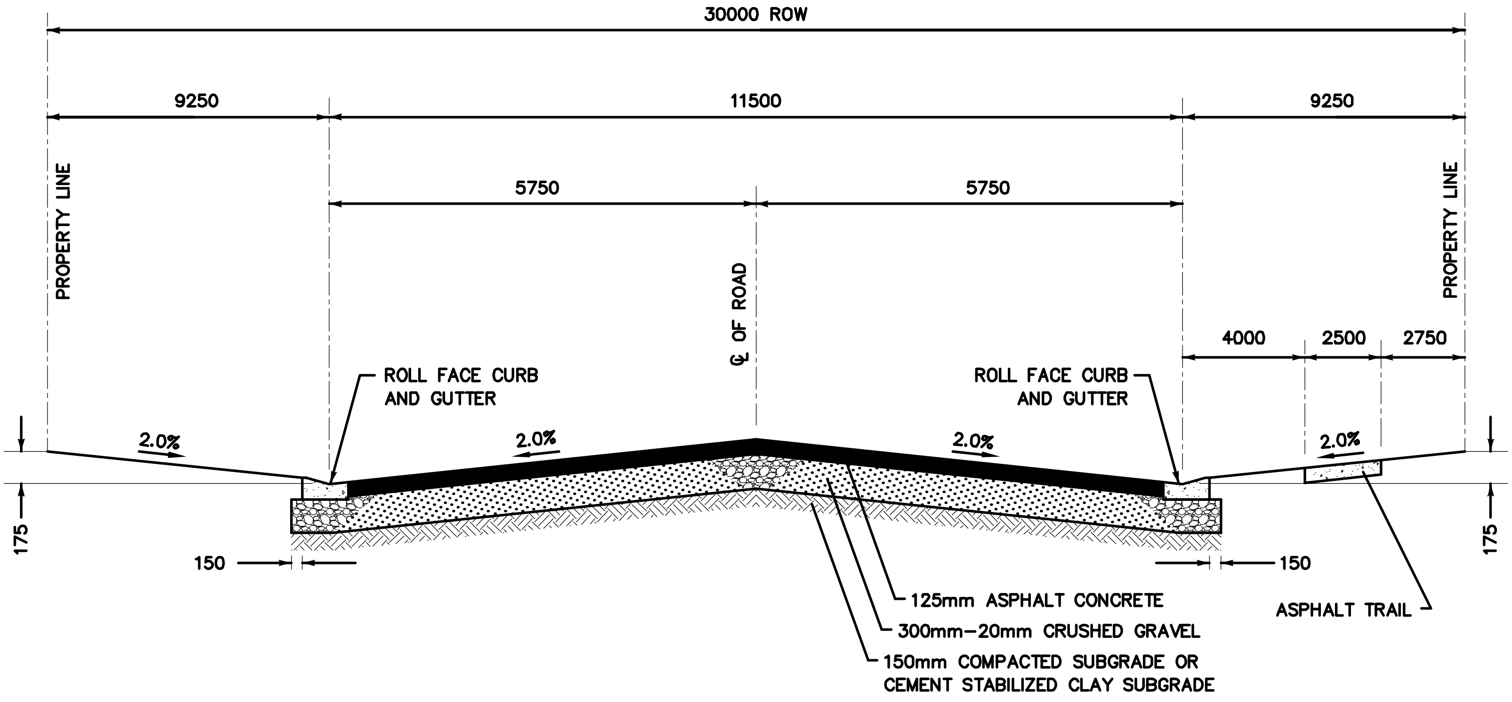

11.5m ARTERIAL without PARKING
(27th STREET)

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FILE No.: ED60.33962

TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION

11.5m ARTERIAL without PARKING

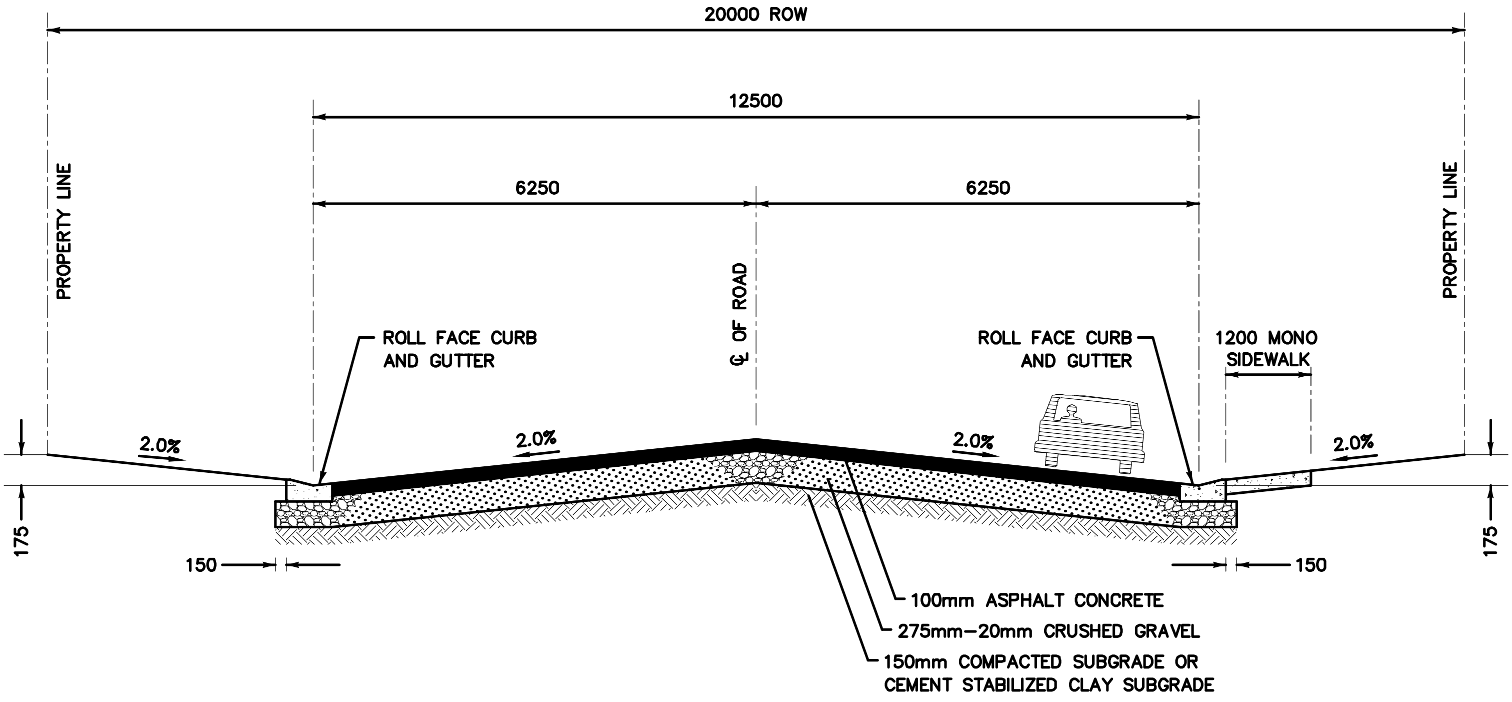
S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 9.2

\\Swg.ca\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 8x11 X-SEC FIGURES.dwg

DATE: NOVEMBER 8, 2010



12.5m COLLECTOR with PARKING
(5th AVENUE)

CONSULTANT

TOWN OF WAINWRIGHT

Stewart Weir
Naturally Resourceful

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ENSTROM SUBDIVISION

12.5m COLLECTOR with PARKING

S.E.1/4 SEC.32–44–6–W4

FILE No.: ED60.33962

NOT TO SCALE

FIGURE 9.3

DATE: NOVEMBER 8, 2010

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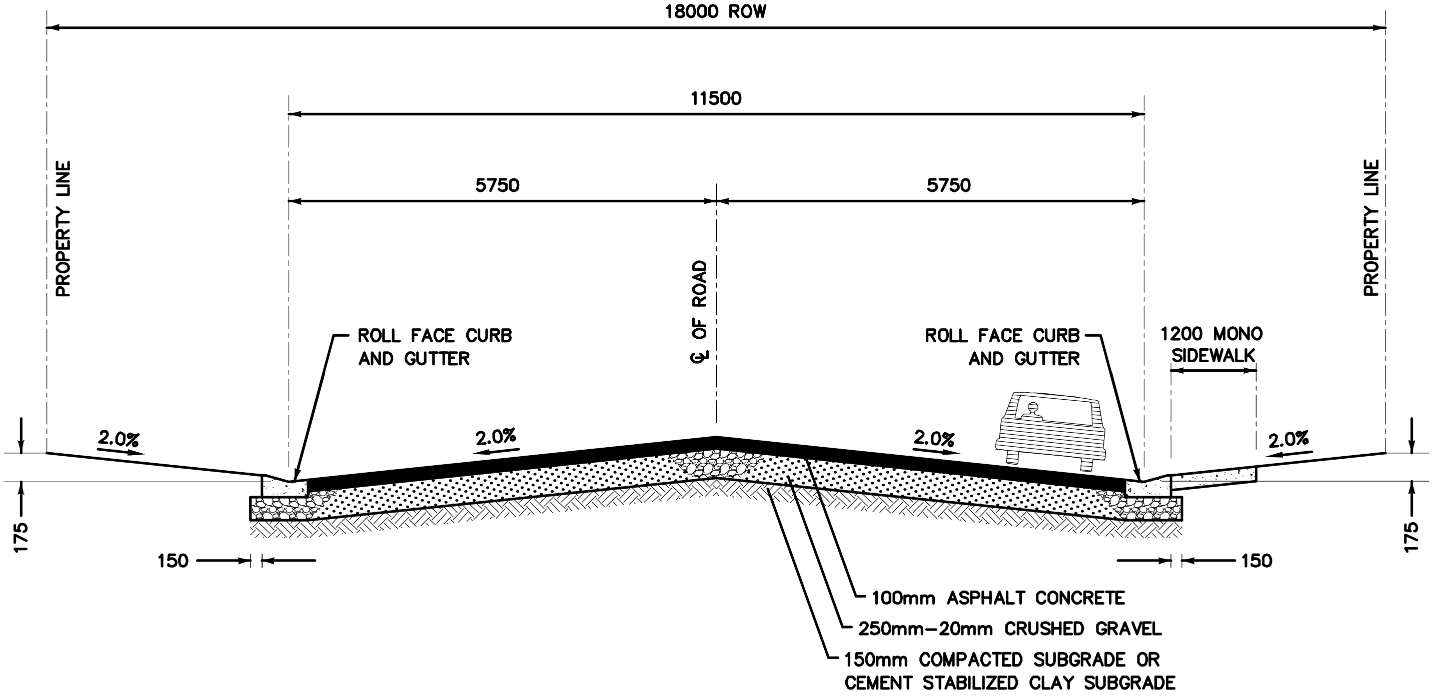

11.5m LOCAL with PARKING

CONSULTANT

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION

11.5m LOCAL with PARKING

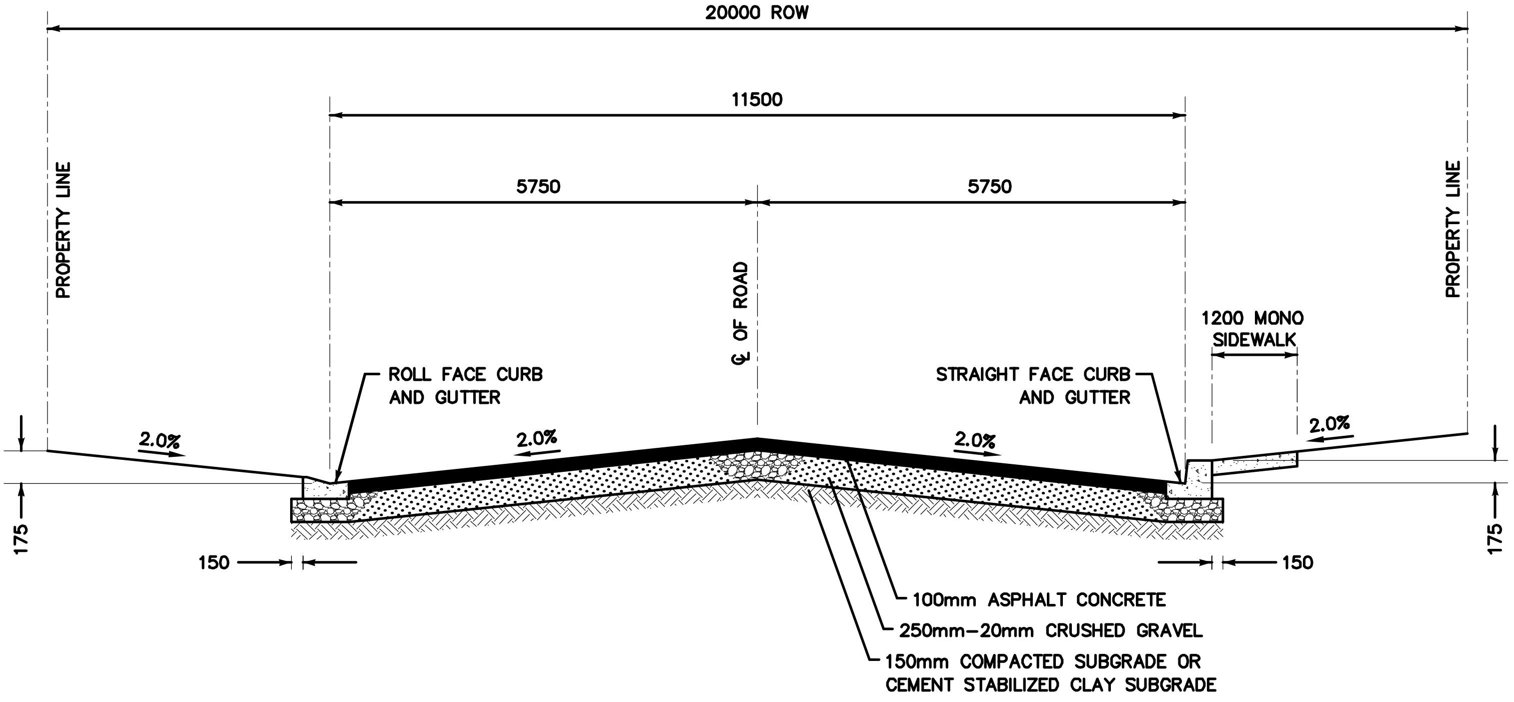
S.E.1/4 SEC.32–44–6–W4

NOT TO SCALE

FIGURE 9.4

\\Swg.co\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 8x11 X-SEC FIGURES.dwg

DATE: NOVEMBER 8, 2010



11.5m LOCAL with STRAIGHT FACE CURB AND GUTTER
(6th AVENUE & 25th STREET)

CONSULTANT

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION

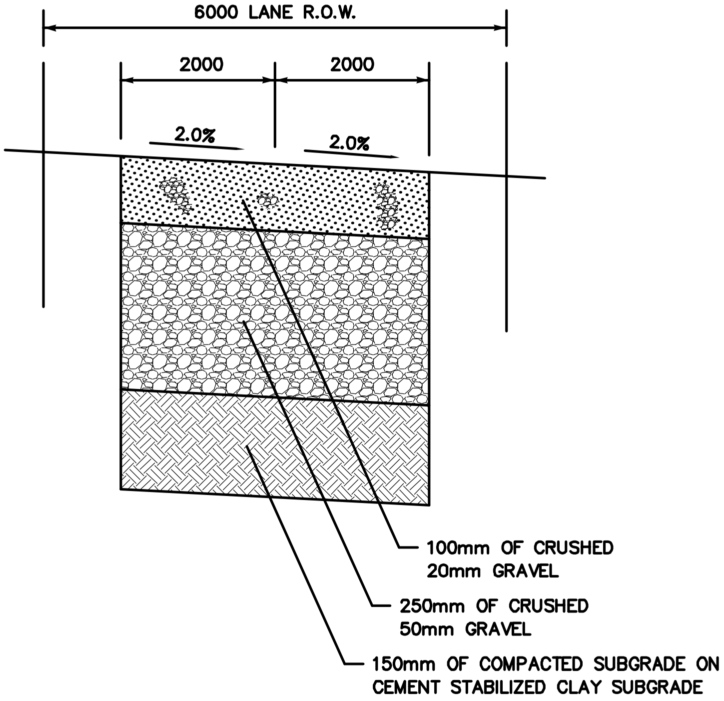
11.5m LOCAL
with STRAIGHT FACE CURB AND GUTTER
S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

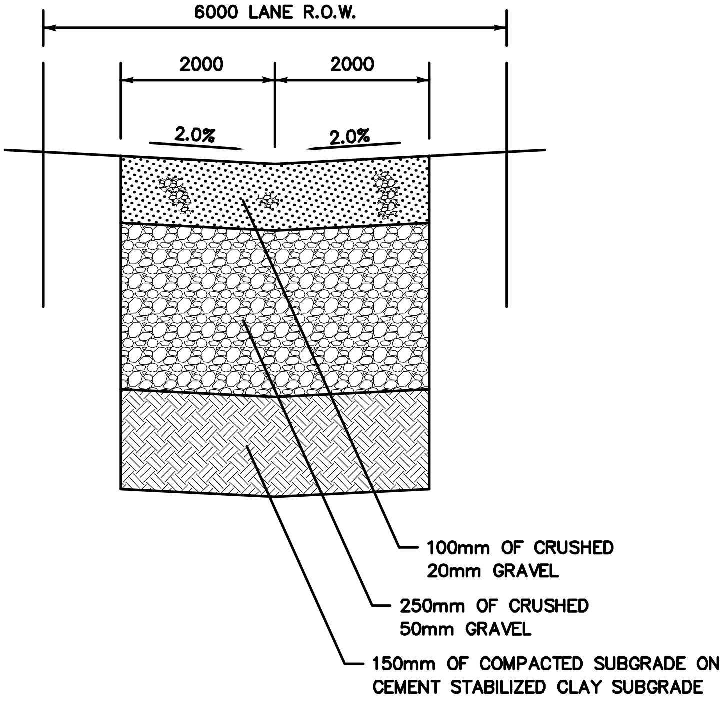
FIGURE 9.5

DATE: NOVEMBER 8, 2010

\\Swg.ca\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 8x11 X-SEC FIGURES.dwg



LANE 1 CROSS SECTION



LANE 2 CROSS SECTION (SWALED)

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION

LANE 1 CROSS SECTION &
LANE 2 CROSS SECTION (SWALED)

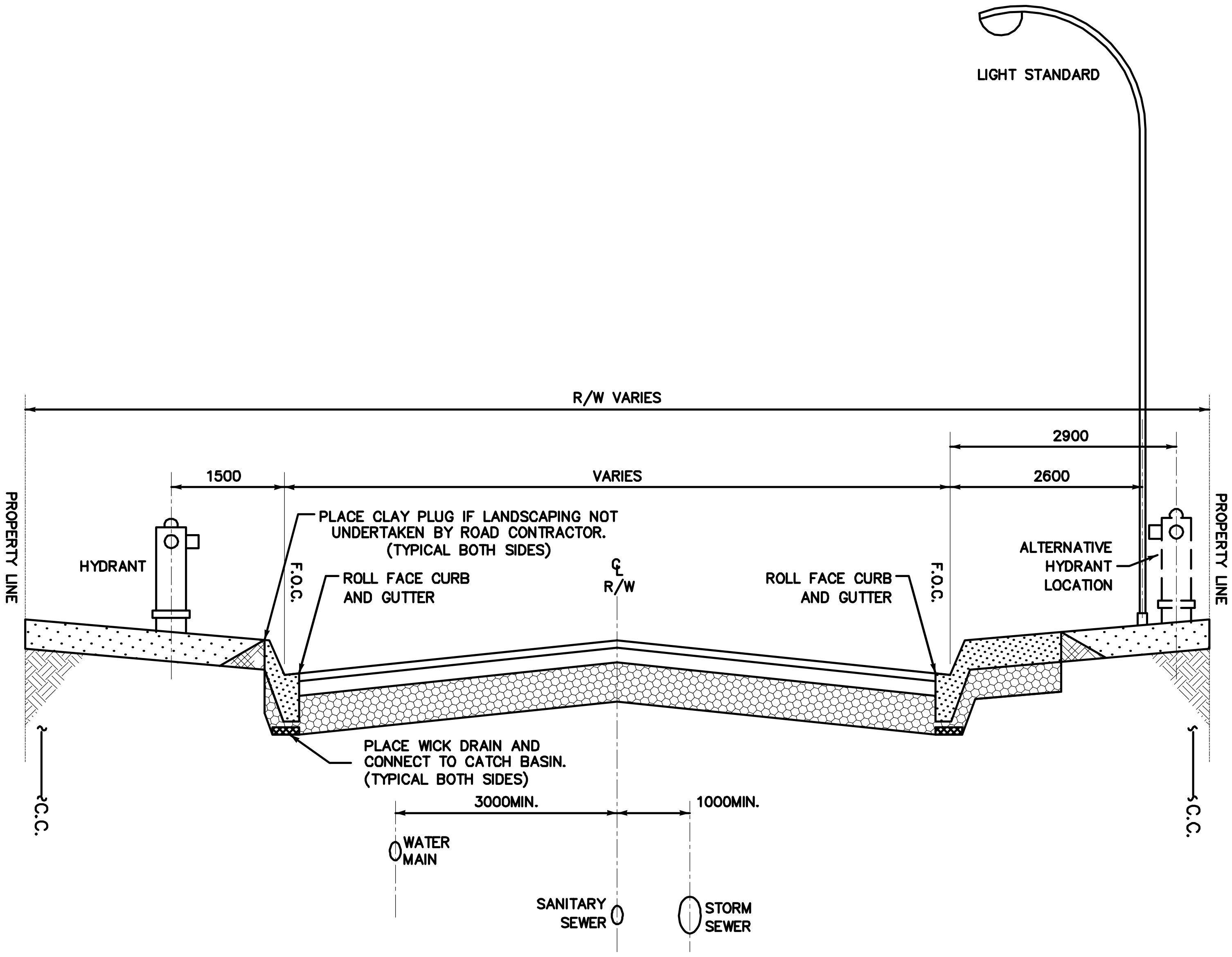
S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 9.6

DATE: NOVEMBER 8, 2010

\\Swg.ca\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 8x11 X-SEC FIGURES.dwg



TYPICAL UTILITY LOCATION PLAN

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TOWN OF WAINWRIGHT

ENSTROM SUBDIVISION

TYPICAL UTILITY LOCATION PLAN

S.E.1/4 SEC.32-44-6-W4

NOT TO SCALE

FIGURE 9.7

\\Swg.co\Files\Jobs\33000\33962 ED60 Town of Wainwright\CAD\ENSTROM CONCEPT FIGURES SEPT 2010\33962 8x11 X-SEC FIGURES.dwg

DATE: NOVEMBER 8, 2010

10.0 COST ESTIMATE

The following table provides a preliminary capital cost estimate for the construction of Phase 1 of the Enstrom Subdivision. The estimate has been prepared based on preliminary quantity take-off from the concept plans. It is intended for budgeting purposes only and will be updated at the detailed design stage.

Table 10.1 - Preliminary Construction Cost Estimate

Site Works & Misc.

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	Topsoil Stripping	Strip and stockpile topsoil from entire construction area including lots.	47,000	m^3	$ 5.00	$ 235,000.00
2	Lot Grading	Rough lot grading including excavation, placement, spreading and compaction to finished lot corner elevations prior to topsoil placement.	1	ls.	$ 225,000.00	$ 225,000.00
3	Clearing and Grubbing	Clearing and grubbing treed area in MR for storm pond construction	0.5	ha	$ 3,000.00	$ 1,500.00

Subtotal (Site Work)= $ 461,500.00

Sanitary Sewer

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	100mm dia PVC DR35 Sanitary Sewer Service	Supply and install 100mm dia sanitary sewer services from main line to the property line. Includes in-line tee, connections, piping, bedding, excavation, backfill and marker post.	139	each	$ 1,500.00	$ 208,500.00
2	200mm dia PVC DR35 sanitary sewer	Supply and install 200mm dia PVC DR35 sanitary sewer main line including pipe, connections, excavation, bedding and backfill.	950	lin.m.	$ 120.00	$ 114,000.00
3	300mm dia PVC DR35 sanitary sewer	Supply and install 300mm dia PVC DR35 sanitary sewer main line including pipe, connections, excavation, bedding and backfill.	300	lin.m.	$ 140.00	$ 42,000.00
4	375mm dia PVC DR35 sanitary sewer	Supply and install 375mm dia PVC DR35 sanitary sewer main line including pipe, connections, excavation, bedding and backfill.	1600	lin.m.	$ 160.00	$ 256,000.00
5	1200mm dia concrete manhole	Supply and install 1200mm dia concerete manhole barrels including pipe connections, excavation, bedding and backfill.	120	vert. m	$ 1,000.00	$ 120,000.00
6	1200mm dia. Pre benched concrete manhole base	Supply and install 1200mm dia. pre-benched manhole base including excavation and backfill.	29	each	$ 700.00	$ 20,300.00
7	NF 80 Frame and Cover	Supply and install NF 80 manhole frame and cover	29	each	$ 500.00	$ 14,500.00
8	Connection to existing manhole	Connect 150mm dia. forcemain to existing manhole.	1	each	$ 2,000.00	$ 2,000.00

Subtotal (Sanitary)= $ 777,300.00

Water

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	AWWA C900 PVC Water main 150mm dia	Supply and install 150mm dia. PVC water main in Class ‘B’ bedding including all trenching, backfilling, surface grading and thrust blocking. 3m minimum depth of bury.	620	lin m	$ 100.00	$ 62,000.00
2	AWWA C900 PVC Water main 200mm dia	Supply and install 200mm dia. PVC water main in Class ‘B’ bedding including all trenching, backfilling, surface grading and thrust blocking. 3m minimum depth of bury.	430	lin m	$ 120.00	$ 51,600.00
3	AWWA PVC C900 Water main 300mm dia	Supply and install 300mm dia. PVC water main in Class ‘B’ bedding including all trenching, backfilling, surface grading and thrust blocking. 3m minimum depth of bury.	1600	m	$ 130.00	$ 208,000.00
4	AWWA C509 150mm dia. Gate Valve	Supply and install 150mm dia Gate Valve, including all necessary fittings, trenching, Class ‘B’ bedding, backfilling, thrust blocking, casing, operating rod and cathodic protection.	10	each	$ 1,500.00	$ 15,000.00
5	AWWA C509 200mm dia. Gate Valve	Supply and install 200mm dia Gate Valve, including all necessary fittings, trenching, Class ‘B’ bedding, backfilling, thrust blocking, casing, operating rod and cathodic protection.	4	each	$ 1,800.00	$ 7,200.00
6	AWWA C509 300mm dia Gate Valve	Supply and install 300mm dia Gate Valve, including all necessary fittings, trenching, Class ‘B’ bedding, backfilling, thrust blocking, casing, operating rod and cathodic protection.	18	each	$ 2,000.00	$ 36,000.00
7	Fire Hydrant Assembly AWWA C502.80	Supply and install compression type fire hydrant including drain pit, hydrant tee, 150mm dia. lead and connection, cathodic protection, concrete thrust blocking, 150mm dia. gate valve with operating rod and cathodic protection.	19	each	$ 7,000.00	$ 133,000.00
8	25mm Copper Water Service	Supply and install 25mm type K annealed copper tubing water service from main to property line.	139	each	$ 1,500.00	$ 208,500.00
9	Fittings	Supply and install plugs, reducers, tees, crosses and elbows. (Lump sum estimate)	1	Lump Sum	$ 10,000.00	$ 10,000.00

Subtotal (Water) = $ 731,300.00

Storm System

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	Control Structures	Supply and install storm pond control structures.	1	Lump Sum	$ 8,000.00	$ 8,000.00
2	Storm Pond	Excavate and shape storm pond to design elevations. Excavated material to be hauled and placed as required for lot grading.	1	Lump Sum	$ 350,000.00	$ 350,000.00
4	Storm sewer main	Supply and install PVC DR 35 Ultra-Rib storm sewer	1,800	lin.m.	$ 250.00	$ 450,000.00
6	Catch basin leads	Supply and install PVC DR 35 ultra-rib catch basin leads.	320	lin.m.	$ 150.00	$ 48,000.00
7	Concrete catch basins	Supply and install concrete pre-cast catch basin	35	each	$ 1,500.00	$ 52,500.00
8	1200mm dia concrete manhole	Supply and install 1200mm dia concerete manhole barrels including pipe connections, excavation, bedding and backfill.	75	vert. m	$ 1,000.00	$ 75,000.00
9	1200mm dia. Pre benched concrete manhole base	Supply and install 1200mm dia. pre-benched manhole base including excavation and backfill.	24	each	$ 700.00	$ 16,800.00
10	NF 80 Frame and Cover	Supply and install NF 80 manhole frame and cover	24	each	$ 500.00	$ 12,000.00

Subtotal (Storm) = $ 1,012,300.00

Roadways

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	Common Excavation	Excavate, load, haul, place, spread and compact approved material in fill areas to 98% SPD including shaping and grading.	18,000	m^3	$ 7.00	$ 126,000.00
2	Subgrade Preparation	Scarify, windrow and compact subgrade to 100 % SPD	43,000	m^2	$ 10.00	$ 430,000.00
3	Granular Base Course	Supply and place 20mm crushed granular base course to specified depth.	25,000	tonnes	$ 20.00	$ 500,000.00
4	ACP	Supply and place ACP to specified depth (first and second lifts)	8,000	tonnes	$ 75.00	$ 600,000.00
5	Rolled face curb and gutter	Supply and place rolled face curb and gutter	3,000	lin m	$ 120.00	$ 360,000.00
6	Monolithic curb, gutter and sidewalk (straight and rolled face)	Supply and place monolithic curb, gutter and sidewalk	1,500	lin m	$ 220.00	$ 330,000.00
7	Straight face curb and gutter	Supply and place straight face curb and gutter	820	lin m	$ 140.00	$ 114,800.00
8	Culverts	Supply and install centreline culverts	40	lin m	$ 250.00	$ 10,000.00
9	Signage	Supply and install traffic control signage as required.	1	l.s.	$ 6,000.00	$ 6,000.00

Subtotal (Roadways) = $ 2,476,800.00

Landscaping

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	Topsoil Replacement	Replace topsoil into all ditches, P.U.L., and boulevards at a minimum compacted depth of 150mm	2600	m^3	$ 5.00	$ 13,000.00
2	Seeding, Fertilizing, and Harrowing	Supply and install required seeding, fertilizing and harrowing of disturbed areas including ditch side slopes	2	ha	$ 1,000.00	$ 2,000.00

Subtotal (Landscaping) = $ 15,000.00

Franchise Utilities

No.	Item	Description	Approx Qty.	Unit	Unit Price	Extension
1	Natural Gas		1	Lump Sum	$ 400,000.00	$ 400,000.00
2	Streetlighting, Power and Communications		1	Lump Sum	$ 550,000.00	$ 550,000.00

Subtotal (Franchise Utilities) = $ 950,000.00

Subtotal Site Works =	**$**	**461,500**
Subtotal Sanitary=	**$**	**777,300**
Subtotal Water=	**$**	**731,300**
Subtotal Storm=	**$**	**1,012,300**
Subtotal Roadways=	**$**	**2,476,800**
Subtotal Landscaping=	**$**	**15,000**
Subtotal Franchise Utilities=	**$**	**950,000**
Sub-Total =	**$**	**6,424,200**
Contingency (10%) =	**$**	**640,000**
Total Estimated Cost =	**$**	**7,064,200**

11.0 SUMMARY AND CLOSURE

11.1 SUMMARY

This report presents preliminary design concepts for the servicing of Phase 1 of the proposed Enstrom Subdivision in the Town of Wainwright. Servicing concepts have been prepared and presented along with design criteria and assumptions. The intent of the report is to communicate the level of service that will be provided and ensure that the proposed work does not impact the existing infrastructure or restrict any future development to the east and south.

11.2 CLOSURE

This report was prepared by Stewart, Weir & Co. Ltd for the Town of Wainwright. The material in this report reflects Stewart Weir & Co.'s best judgement in light of the information available at the time of preparation. 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. Stewart, Weir & Co. Ltd. accept no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

Stewart Weir & Co. Ltd.

____

Kirsten Davis, P.Eng.

Design Project Director, Community Infrastructure

____

Shakeeb Bashir, C.E.T.

Design Technologist, Community Infrastructure

PERMIT TO PRACTICE
STEWART, WEIR & CO. LTD.

Signature: ____________________________

Date: ____________________________

PERMIT NUMBER: P 292

The Association of Professional Engineers, Geologists and Geophysicists of Alberta

KIRSTEN J. DAVIS

APPENDIX A

GEOTECHNICAL INVESTIGATION

Stewart Weir

Naturally Resourceful

Town of Wainwright
1018 - 2nd Avenue
Wainwright, Alberta
T9W 1R1

Our File: ED60 33962
Date: October 21, 2010

Attention: Wes Kroening, Director of Planning and Development

Dear Sir:

Re: **Geotechnical Site Investigation**
Proposed Urban Residential Subdivision – (Enstrom Subdivision)
SE- 32-44-06-W4M
Wainwright, Alberta

Please find enclosed a copy of our geotechnical site investigation report for the above referenced urban residential subdivision. The investigation included drilling fifteen boreholes, groundwater monitoring, and laboratory testing on representative soil samples; which together with an engineering assessment are presented in this report.

The ground profile at the borehole locations predominantly consisted of topsoil over clay till which extended to the termination depths of thirteen out of fifteen boreholes. Boreholes 10-4 and 10-7 encountered sand below the clay till and extended to the termination depths of both boreholes. Groundwater levels were monitored during drilling, at drilling completion and 11 days later and are detailed in this report.

The geotechnical conditions for foundation support and access roads are considered generally favourable for the proposed residential subdivision and are provided in Section 6.0 of the enclosed report.

Type 50 (HS), Sulphate resistant cement, possessing a minimum 56-day compressive strength of 32 MPa, should be used for all concrete in contact with the native soils.

Thank you for allowing Stewart Weir to serve you on this project. Should you have any questions or require additional information, please contact our office at (780) 410 – 2580.

Yours truly,
Stewart, Weir & Co. Ltd.
APEGGA Permit No.: P 292

Sajid Mahmood

GURPREET GILL

OCT 21/2010

Sajid Mahmood, M. Eng., E.I.T.
Geotechnical Engineer

Gurpreet Gill, M. Eng., P. Eng
Geotechnical Engineering Manager

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APPENDIX

1.0 INTRODUCTION

This report presents results of a geotechnical site investigation undertaken at the site of a proposed urban residential subdivision located within SE – 32 – 44 – 06 - W4M, Wainwright, Alberta. Stewart, Weir & Co. Ltd. was retained by the **Town of Wainwright** on June 21, 2010 to undertake the above noted investigation. The objectives of this investigation were to determine the nature and condition of the existing surface and subsurface soil and groundwater conditions for design and construction of the proposed residential development. In addition, design recommendations are provided for concrete type, pavement structures, storm water pond and lateral earth pressures.

2.0 SITE AND PROJECT DESCRIPTION

The site is approximately within the west ½ of SE – 32 – 44 – 06 - W4M in the Town of Wainwright, Alberta, bounded on the east by 23rd street and on the north by 8th Avenue. Access to the property was available through an existing approach along 23rd street. Topography of the site is classified as very gently undulating, draining towards south and currently in agricultural use. The neighbouring land is in agricultural and residential subdivision use. There is a treed area near the south middle third of the subject site, comprising of poplar and willow trees and was dry at the time of drilling. It is understood that the development of the subdivision includes single and multi-family dwellings, mobile homes, parks, school, storm water pond and access roads. The total area of development is approximately 28.6 hectare, (70.7 acres).

3.0 INVESTIGATION PROCEDURES

On August 4th, 2010, fifteen boreholes (BH 10-1 to BH 10-15) were drilled at the above mentioned site. Boreholes 10-1, 10-3, 10-7, 10-10, 10-12 and 10-14 were drilled to a depth of 8.7 m and boreholes 10-2, 10-4, 10-5, 10-6, 10-8, 10-9, 10-11, 10-12 and 10-13 were drilled to a depth of 5.7 m below ground surface. Field drilling was carried using a truck mounted M-45 drill rig owned and operated by Mobile Augers and Research Ltd., equipped with continuous flight, 150 mm diameter, solid-stem augers and SPT capability. Supervision of drilling, soil sampling, and logging of the various soil strata was performed by Mr. Dale Johnston, a senior geotechnical technician at Stewart Weir, Sherwood Park office. The soil conditions encountered during drilling were described in accordance with the Modified Unified Soil Classification System described in the Appendix. Approximate borehole locations are presented on the site plan in the Appendix.

Sampling for the boreholes generally consisted of disturbed auger samples at 0.75 m intervals in all boreholes. In addition, to obtain an indication soil consistency and unconfined compressive strength of cohesive soils, pocket penetrometer (PP) readings were taken on intact cohesive soil samples. Standard Penetration Tests, (SPT), were carried out at various depths in all boreholes.

4.0 LABORATORY TESTING

Laboratory testing included a visual classification and determination of the natural moisture content of all soil samples obtained from borehole logs. In addition, fifteen selected soil samples were analyzed for Atterberg limits and six selected soil samples were analyzed for water-soluble sulphates. Furthermore, sieve analysis tests were performed on selected soil samples. The laboratory test results are summarized on the borehole logs in the Appendix. An explanation of the symbols and terms used to describe the borehole logs is also presented in the Appendix.

5.0 SUBSURFACE CONDITIONS

No fill was encountered at the borehole locations, but some might be present at the site. The ground profile at the borehole locations mainly consisted of topsoil over clay till which extended to the termination depths of thirteen out of fifteen boreholes. The soil strata are shown on the borehole logs in the Appendix and are described in the following sections.

5.1 TOPSOIL

Topsoil was encountered at ground surface in all boreholes and found extending to depths ranging from 0.10 to 0.20 m below ground surface. The topsoil was generally described as damp to moist and black in colour.

5.2 CLAY TILL

Clay till was encountered below the topsoil in all boreholes at depths ranging from 0.10 to 0.20 m and extended to the termination depths of all boreholes, except for boreholes 10-4 and 10-7, in which the clay till was found extending to depths of 5.2 and 6.2 m respectively.

The clay till was generally described as sandy with some silt, brown to dark brown in colour, moist, medium plastic and firm to very stiff in consistency with occasional gravel size rock pieces, trace to some white salts, occasional rust specks and trace coal specks. The clay till was further noted to contain 5 to 35 cm thick, fine to medium grained, medium dense and wet sand layers within boreholes 10-1, 10-2, 10-4 and 10-14 at depths ranging from 4.0 to 8.45 m below ground surface.

Pocket penetrometer (PP) readings taken on intact auger samples of clay till revealed approximate unconfined compressive strengths, Q_u, ranging from 50 to 300 kPa. A more typical range would be 100 to 225 kPa. The un-factored, standard penetration test (N) values in clay till ranged from 7 to 26, confirming to firm to very stiff consistency. It should be noted that clay till encountered within the borehole 10-14 was moist to wet and showed lower PP and N values.

In-situ moisture contents within the clay till ranged from 9 to 22 percent with an average of 15 percent. Fifteen Atterberg limit tests conducted on selected samples of clay till at depths ranging from 0.75 to 6.75 m below ground surface yielded Liquid Limits ranging from 23 to 46 percent and Plastic Limits ranging from 10 to 17 percent, indicative of low to medium plastic clay. The low to medium plastic clay has low to moderate potential for swelling with changes in moisture content. Based on the Plastic Limits, as determined by the Atterberg Limit tests, the existing moisture contents in clay till are slightly above Optimum Moisture, on average.

The clay till encountered at the site is generally a competent material and exhibits low compressibility under moderate to heavy loads. Sand layers and cobble/boulder size rocks are not uncommon within the clay till.

5.3 SAND

Sand was encountered below the clay till deposit within boreholes 10-4, 10-7 and 10-10 at depths ranging from 5.2 to 6.2 m and extended to a depth of 7.7 m below ground surface in borehole 10-10, while extended to the termination depths of boreholes 10-4 and 10-7. The sand was generally described as fine to medium grained, brown to rusty brown in colour, wet and medium dense in relative density with trace gravel size rocks. The un-factored, standard penetration test (N) values in sand ranged from 23 to 38, confirming medium dense relative density.

In-situ moisture contents within the sand ranged from 13 to 17 percent with an average of 15 percent. It should be noted that the moisture content measurement of natural sand samples is very difficult and values change with the percentage of fine content and the amount of water that would drain during sampling. Nominal moisture content of about 17 percent might be used for design purposes. Saturated sand, hole sloughing and water flow during pile installation is expected.

5.4 FROST PENETRATION

The expected maximum depth of frost penetration for various soil types is given in table 1. The penetration is based on a freezing index for a 25-year return period of 2480 degrees-days Celsius. The depth of frost penetration assumes a uniform soil type without topsoil or snow cover.

TABLE 1: ESTIMATED DEPTH OF FROST PENETRATION

Soil Type		Depth of Frost Penetration (m)
In-situ	Clay and Clay Till	2.8
	Silt and Sand	3.3
Compacted Backfill (95 percent SPMDD*)	Clay and Clay Till	2.6
	Silt and Sand	3.1

*SPMDD- Standard Proctor Maximum Dry Density

The clay till encountered at this site is considered frost susceptible and has expected potential frost action of medium to high, and with an adequate supply of moisture near the ground surface, significant frost heave may occur. Therefore, it is considered necessary to provide mitigation measures for frost damage prior to construction. Remedial measures would typically include the use of rigid insulation and/or removal of frost-susceptible material and replacement with gravel with less than 10 percent silt and clay.

5.5 GROUNDWATER CONDITIONS

Standpipe piezometres were installed in boreholes 10-1, 10-2, 10-3, 10-5, 10-7, 10-10, 10-11, 10-12, 10-14 and 10-15 to allow for long term monitoring of the groundwater table. The groundwater conditions in the boreholes were observed during drilling, at drilling completion and 11 days following drilling. A summary of the groundwater observations is shown in table 2.

TABLE 2: SUMMARY OF GROUNDWATER OBSERVATIONS

Borehole Number	Depth Drilled (m)	Depth of Water Seepage (m)	Depth of Slough (m)	Depth of Water (m)	
				At completion	After 11 days
10-1	8.71	5.25	8.00	7.3	2.80
10-2	5.71	5.25	5.00	4.0	3.45
10-3	8.71	-	-	Dry	3.80
10-4	5.71	5.20	4.80	3.3	-
10-5	5.71	-	-	Dry	3.30
10-6	5.71	-	-	Dry	-
10-7	8.71	6.20	5.80	3.7	3.20
10-8	5.71	-	-	Dry	-
10-9	5.71	-	-	Dry	-
10-10	8.71	5.30	4.10	2.1	2.90
10-11	5.71	-	-	Dry	3.00
10-12	5.71	-	-	Dry	4.10
10-13	5.71	-	-	Dry	-
10-14	8.71	4.00	8.30	5.6	2.00
10-15	8.71	-	8.60	Dry	2.25

It should be recognized that the level of the groundwater table is dependent on meteorological cycles and surface drainage on a regional scale. Higher groundwater levels than those observed in this investigation may be encountered following spring thaw and periods of prolonged precipitation. Seasonal fluctuations under normal conditions are expected to be ± 1.0 m.

6.0 GEOTECHNICAL EVALUATION

It is understood that the development of the subdivision includes single and multi-family dwellings, mobile homes, parks, school, storm water pond and access roads. Based on the information obtained in this investigation, the soil conditions at the site are considered generally favourable for foundation support for the proposed subdivision. Shallow footings bearing in native, undisturbed soil or engineered fill should be suitable as a foundation system for single and multi family residences. Alternatively, cast-in-place concrete friction piles are considered suitable for this project.

It should be noted that proposed school area covered under boreholes 10-14 and 10-15 indicated the presence of moist to wet and soft clay till and resulted in lower “PP” and “N” values. Further geotechnical investigation is recommended in the subject area, when location and size of the proposed school is finalized.

6.1 SITE PREPARATION

Site preparation will include removing all topsoil from the single and multi family houses and roads foot print areas and the created excavations should be backfilled with engineered fill such as pitrun gravel or low to medium plastic clay compacted to at least 98 percent of Standard Proctor maximum dry density within ±2 percent of its optimum moisture content. All fill should be free of frozen soil, ice, snow and organics. Estimates of topsoil soil thickness within borehole locations may be obtained from borehole logs. However, variation in topsoil thickness should be expected between borehole locations. Site grading should not be carried out under freezing conditions due to problems with stripping frozen soils.

A qualified geotechnical engineer or technologist should be on site during excavations over 1.5 m deep and placement of engineering backfill. Excavation of side-slopes more than 1.5 m deep should be cut back to no steeper than 1 horizontal to 1 vertical for short-term excavations. For excavations that are left open for long-term or are deeper than 1.5 m, additional geotechnical engineering analysis is required. The analysis can be carried out by Stewart Weir upon request.

The native clay till encountered at this site is considered suitable for general site grading. Uniformity and compactive effort of the engineered fill are important factors in minimizing the potential for differential settlement. The engineered fill should be compacted to the following standards.

- All site-raising fill under building areas should be placed in 150 mm lifts compacted thickness and compacted to at least 98 percent of standard Proctor maximum dry density within ±2 percent of its optimum moisture content.

- Site raising fill under the roadway area should be placed in 150 mm lifts compacted thickness and compacted to at least 95 percent of standard Proctor maximum dry density within ±2 percent of its optimum moisture content.

- General site grading fills outside the building footprints should also be placed in 150 mm lifts compacted thickness and compacted to at least 95 percent of standard Proctor maximum dry density within ±2 percent of its optimum moisture content.

6.2 STRIP AND SQUARE FOOTINGS

It is recommended that footing excavations be inspected by a qualified geotechnical engineer prior to pouring concrete to confirm foundation soil conditions and bearing pressures.

Due to high groundwater table encountered at site, dewatering of footing trenches might be necessary. All footings must be founded on undisturbed soil. Footings should not be placed in any topsoil, uncontrolled fill, organic soils, loose, disturbed, or frozen soils. Footing excavations must be protected from frost, desiccation, or the ingress of water. Bearing soils, which become frozen, dried or softened, should be removed and replaced with concrete or the footings should be extended to reach soil in an unaffected condition. It is essential that the foundation soils not be allowed to freeze at any time before or after pouring of concrete.

No loose, disturbed, remoulded or slough material should be allowed to remain in the open footing excavations. Hand cleaning is recommended if an acceptable surface cannot be prepared by mechanical equipment. In order to avoid disturbing the bearing surface, all basement excavations should be excavated by a backhoe operating remote from the bearing surface. Care should be taken to ensure that all exposed soils are protected from excessive drying.

A non-deteriorating vapour barrier should be placed immediately below the floor slab in order to prevent the desiccation of the subgrade material.

It is recommended that floor joists and basement slabs be placed prior to backfilling the excavation in order to minimize any detrimental effects on the foundation walls caused by backfilling operations.

The time span between start of the excavation to the installation of basement footings, walls, peripheral weeping tile and backfilling operations should be minimized in order to prevent any problems developing with the excavation due to ingression of ground or surface waters, or desiccation of the soil. For excavations deeper than 1.5 m, side slopes should be cut back to no steeper than 1 horizontal to 1 vertical or a slope stability analysis should be performed.

During winter construction, it is essential that all interior fill and load bearing materials remain frost-free. Recommended winter construction practices, with respect to hoarding and heating of the wood forms and the fresh concrete, should be followed. In order to minimize potential frost heave problems, the interior of the building must be heated as soon as the walls have been poured. The period in which the excavation is left open to freezing conditions should be as short as possible. If doubt remains as to the suitability of the foundation during construction, the builder should consult a qualified geotechnical engineer.

6.2.1 Single & Multi Family House Foundations

Shallow footings founded in the native clay till at depths ranging from 1.5 to 2.0 m below ground surface may be designed using recommended allowable bearing pressure values, (SLS) of 120 kPa and 140 kPa for strip and square footings, respectively. The allowable bearing pressures may be increased by a factor of 1.5 to obtain factored ultimate bearing resistance, (ULS). For adequate frost protection, the exterior footings must be at least 1.5 m below the final grade, assuming that the building is heated. For unheated parts of the houses, footings should be at least 2.5 m below final grade for adequate frost protection. In case of basements, footings may be founded immediately below basement level provided the minimum depth of 1.5 m below grade is maintained.

A 150 mm layer of sand or sand-gravel mixture should be placed immediately below all floor slabs for new homes. This material should be uniformly compacted to 100 percent of the corresponding SPMDD at OMC. In case of ingression of groundwater or a soft subgrade, a drain rock layer may be required.

Water dispersed on the property from roof drains must not be allowed to reduce the integrity of the foundations. To ensure this, provisions must be made to ensure that runoff is not allowed to accumulate around the basements.

6.2.2 Proposed Future School

Shallow footings founded in the native clay till at depths ranging from 1.5 to 2.0 m below ground surface may be designed using recommended allowable bearing pressure values, (SLS) of 90 kPa and 105 kPa for strip and square footings, respectively. The allowable bearing pressures may be increased by a factor of 1.5 to obtain factored ultimate bearing resistance, (ULS). It is recommended that additional geotechnical investigation should be done to confirm allowable bearing pressure values once location in finalized.

For adequate frost protection, the exterior footings must be at least 1.5 m below the final grade, assuming that the building is heated. For unheated parts of the proposed school, footings should be at least 2.5 m below final grade for adequate frost protection. In case of basements, footings may be founded immediately below basement level provided the minimum depth of 1.5 m below grade is maintained.

6.2.3 Footing Uplift Resistance

The uplift resistance of a footing to transient uplift loads can be calculated as the effective weight of a prism of soil formed by lines rising at an angle of 30 degrees to the vertical, from the top of the footing, provided all backfill is compacted to a minimum field density of 95 percent of standard proctor maximum dry density (SPMDD). In addition, the buoyant weight of the footing should be considered. For permanent uplift loads, it is recommended that the prism be assumed as rising vertically from the edge of the footing. A unit weight of 19.5 kN/m^3 may be used in calculations for re-compacted clay. A minimum factor of safety of 1.5 should be applied.

6.3 CAST-IN-PLACE CONCRETE PILES

6.3.1 Friction Piles

A foundation system of cast-in-place concrete friction piles could be considered for the proposed single and multi family houses and future school. Allowable skin friction may be taken as given in table 3. Friction piles should be designed solely based on skin friction, and no extra capacity from end bearing should be added to the pile capacity.

TABLE 3: ALLOWABLE SKIN FRICTION FOR CAST-IN-PLACE CONCRETE PILES

Depth Below Existing Grade (m)	Soil Type	Allowable Skin Friction (SLS), (kPa)*	Factored Ultimate Skin Friction (ULS), (kPa)**
0 to 1.5	-	0	0
1.5 to 5.0	Clay till	18	23
Below 5.0	Clay till/Sand	19	25

*Allowable Skin Friction calculated using F.S. = 2.0
**Using Geotechnical resistance factor of 0.65 based on AASHTO

Due to the shrinkage effects that would otherwise not provide intimate contact between the soil and concrete pile, the skin friction for that portion of the pile shaft within the upper 1.5 m should be discounted as zero. The load end bearing capacity that could be derived from end bearing should be ignored for two reasons: firstly, the base of the bored straight-shaft pile usually contains disturbed soil created by the pile auger; and secondly, the pile displacements required to attain end-bearing are much greater than those required to mobilize shaft friction.

Piles supporting grade beams should be embedded at least 6.0 m below the finished exterior grade. For unheated parts of the building, the piles should be embedded to a minimum of 8.0 m. A minimum pile-shaft diameter of 400 mm is recommended to minimize void formation during pouring of the concrete. The minimum centre–to-centre spacing of straight-shaft piles should be 3 pile diameters. A nominal percentage of longitudinal steel reinforcement (0.50 percent of the cross sectional area of the pile) should be provided in the upper 4.5 m of the pile to prevent potential uplift forces of the pile due to frost action and seasonal moisture variations. If the piles are designated as tension elements, longitudinal reinforcing steel should extend into the bottom of the piles, and the piles should be designed to resist the anticipated uplift stresses using 67 percent of design values provided above.

Casing will be necessary where water seepage is excessive or where saturated sand is encountered. If saturated sand is encountered at or near the base of the pile, the pile may need to be extended lower in order for the casing to adequately seal off water seepage and sand sloughing. Alternatively, driven steel piles may be used.

A competent and experienced inspector should be on site during the entire duration of the pile installation. The inspector should keep complete and accurate records of the pile installation operation. Concrete should be poured immediately after drilling of the pile hole to reduce the risk of groundwater seepage and sloughing of the soil.

6.3.2 Pile Uplift Resistance

The uplift resistance for straight shaft piles could be calculated as follows. For sustained uplift loads (other than those due to frost action), the allowable skin friction should be taken as 75 percent of the skin friction values given for compressive loading above. For short-term uplift loads, the allowable skin friction against uplift loads may be taken as 95 percent of the value presented for downward compressive loads.

6.3.3 Pile Caps

Precautions should be taken to minimize the potential of heaving of the pile caps due to frost penetration or swelling of the underlying soil. The potential for frost heaving forces can be greatly reduced by the placement of a compressible material or by providing a void between the underside of the pile cap and the soil. A product such as *Voidform* (or equivalent) is recommended. The minimum thickness of the void should be 100 mm. Should a compressible material be used as an alternative to *Voidform*, the uplift pressure acting on the underside of the pile caps may be taken as the crushing strength of the compressible medium. The finished grade adjacent to each pile cap should be capped with clay and sloped away so the surface runoff is not allowed to accumulate in the void space or in the compressible medium. If water is allowed to accumulate in the void spaces, the beneficial effect of the void space will be negated and frost-heaving pressures acting on the underside of the pile caps will occur.

6.3.4 Concrete Grade Beams

If piles are used to support attached garage structures, etc., a concrete grade beam is required along the top of the piles. Precautions should be taken to prevent heaving of the grade beams, due to frost penetration, where the grade beams lie less than 1.5 m below the ground surface.

The recommended construction procedure for preventing heave under the grade beam is to use a crushable, non-degradable void filter that is incorporated at the base of the grade beam. In this method, the grade beam must be designed in accordance with the crushing strength of the void filler used and the piles must be constructed to take the resulting uplift.

6.4 SEISMIC SITE CLASSIFICATION

Site classification for seismic site response for the above referenced site is “D” according to National Building Code of Canada (Table 4.1.8.4.A).

6.5 BASEMENT EXCAVATIONS

Basement excavations, if used, should be carried out in accordance with Alberta Occupational Health and Safety Regulations with respect to shoring and cutback. The sides of the basement excavation should be sloped to comply with Occupation Health and Safety (OH&S) regulation with respect to shoring and cutback. Detailed recommendations for cutback procedures and temporary shoring can be provided after the depth of excavation and construction details are provided. For this project, the main excavation will be approximately 3.0 m deep for one level underground basement. In this case, the soil conditions will consist predominately of clay till. Temporary cut or fill embankment slopes should be constructed no steeper than 1.0 (H) to 1.0 (V) in clay. If wet conditions are noted due to seasonal rains or spring melts, it may become necessary to cut back the slopes to a shallower slope in some areas. Alternatively, temporary retaining systems might be used. Specialty contractors generally provide the design for temporary shoring systems. The foundation soils beneath the proposed footings should not be allowed to access free water during construction.

The native clay till may be used for backfilling around the basement walls provided it is free of organic soils. The soils should be carefully placed and hand tamped in lifts of 300 millimeters or less to obtain uniform compaction. If compacted backfill is used, the foundation walls should be designed using an equivalent fluid pressure of 10 kN/m^3.

6.6 INTERIOR AND EXTERIOR WEEPING TILES (SUBDRAINAGE)

Due to anticipated high groundwater table, it is recommended that a sub-floor drainage system (interior weeping tiles) be installed below the basement floor slab. The purpose of the permanent (interior) subdrainage system is to intercept any water that may percolate through the soil near base of reservoir, to reduce uplift pressure and to promote a dry space.

The interior weeping tiles should consist of perforated rigid plastic pipes surrounded by a filter of free draining gravel and enveloped in a non-woven geotextile. Weeping tiles should be connected to lateral drains spaced at 4.5 m interval. These pipes would then run to a header pipe, which in turn, would drain to an internal sump system storm sewer system from which it should be pumped well away from and down gradient of the basement floor level.

An exterior weeping tile system should also be provided around the perimeter of reservoir. The exterior weeping tile system should be independent of the interior weeping tile system. Two separate sumps are recommended for interior and exterior subdrainage systems.

6.7 DRAINAGE AND GROUNDWATER ISSUES

- Rainwater and snow melt dispersed on the property from the roof leaders should not be allowed to accumulate against the foundation walls. To ensure positive drainage, the soil surface of all lots should be constructed sloping away from all buildings. This will require positive lot grading of at least 5 percent away from the foundation walls for a minimum distance of 1.5 meters. In cases where the lot drainage runs from the back of the lot to the front, runoff should be kept 1.2 meters away from the house. In order to ensure no flow paths for water from the roof leaders occur adjacent to the foundation walls, the following two alternatives are proposed:

  - A concrete splash pad, placed beneath the downspouts, a minimum of 1.2 m long and firmly anchored to the house foundation can be used.

  or

  - A permanent downspout extension could be used to carry water away from the foundation wall

- At least the top 1.0 m of backfill around the basement walls should be an impermeable clay material. The native clay till may be suitable for this purpose.

- During house construction, temporary excavation dewatering may be required in isolated areas until the permanent weeping tile system is operational. In instances where the excavation base intercepts the seasonal high groundwater elevation, subgrade softening may occur, requiring the use of a washed rock base. It is recommended that if free water is noted in any basement excavations at this site, a geotechnical engineering firm should be contacted to inspect the site and provide the recommended action to handle the short-term groundwater issue.

6.8 SITE GRADING AND DRAINAGE

Excess water should be drained from the site as quickly as possible both during and after construction. The finished grade around the hotel building should be laid out such that the surface water drains away from the building by the shortest route. The upper 0.5 m of backfill around the building should consist of compacted clay to act as a seal against the ingress of runoff water. The clay should extend to a distance of 3 m around the building and be graded to a slope of 2 percent away from the building.

6.9 LATERAL EARTH PRESSURE

6.9.1 Lateral Earth Pressure on Concrete Walls at Rest

It is expected that the basement walls will be subjected to significant lateral earth pressure. The concrete walls should be designed as unyielding walls. The earth pressures and groundwater pressures can be estimated using the geostatic and hydrostatic pressure distributions for the soil above and below the water level and the total stress would be the static superposition of the stresses. The lateral earth pressures acting against the walls may be computed using the following expression:

$$P = K_0 \times (Q) + K_0 \times \gamma \times (H_1) + \left[K_0 \times \gamma' \times (H_2) + 9.81 \times (H_2)\right] \quad \text{(KPa)}$$

Where:

P = Lateral Earth Pressure at depth H below the ground level (kPa)
Q = Surcharge loading at the ground surface (kPa)

K_0 = Coefficient of lateral earth pressure at rest for the soil
γ = Total unit weight of backfill compacted to 95 percent SPMDD (kN/m^3)
γ' = Submerged unit weight of backfill compacted to 95 percent SPMDD (kN/m^3)
H_1 = Thickness of the soil layer above the groundwater level
H_2 = Thickness of the soil layer below the groundwater level ($H - H_1$)

Recommended design values for total unit weights and coefficient of lateral earth pressures at rest conditions for different types of backfill are provided in table 4.

TABLE 4: LATERAL EARTH PRESSURE AT REST FOR STATIC CONDITIONS

Type of Backfill	Total Unit Weight (kN/m^3)	Coefficient of Lateral Earth Pressure At Rest
Granular Material	21	0.40
Clay/fill	20	0.60

The point of action of the resultant earth pressure at rest is defined in such a way to provide static equivalent load (usually at 0.33 H for a uniform condition, where H is the height above the bottom of the wall). The above expression makes no allowance for additional horizontal forces due to frost or seismic effects behind the wall, assuming that frost protection system will be utilized. If no frost protection is provided, the lateral earth pressures should be increased by a factor of 2.

The preceding relationship makes no allowance for additional horizontal forces due to frost pressures to build up behind the wall on the assumption that frost protection and a weeping drain system will be utilized. If no frost protection is provided, the lateral earth pressures should be increased by a factor of 2. If no weeping tiles are provided, then hydrostatic pressures should be added to the above expression assuming a design groundwater table elevation at final grade.

In addition, the above expression assumes nominal compaction of up to 95 percent of SPMDD adjacent to the wall. If a higher degree of compaction is proposed, the design lateral earth pressure will adopt a triangular-trapezoidal direction. Stewart Weir can provide further details upon request.

It must be recognized that due to the relatively low degree of compaction of backfill, subsidence of the backfill will occur with associated deflection and possible cracking of structures such as sidewalks and pavements. Additionally, subsidence of backfill may result in loss of grade adjacent to the structure. The use of granular backfill will minimize potential backfill consolidation.

6.9.2 Lateral Earth Pressures Due to Seismic Loads

The resultant earth pressure due to seismic effects is applied at 0.6 H where H is height above bottom of wall. Recommended design values for total unit weights and coefficient of lateral earth pressures at due to seismic effects for different types of backfill are provided in table 5.

TABLE 5: LATERAL EARTH PRESSURE PARAMETERS DUE TO SEISMIC LOADS

Type of Backfill	Total Unit Weight (kN/m^3)	Coefficient of Lateral Earth Pressure due to Seismic Effects
Granular Material	21	0.44
Clay/Fill	20	0.66

6.10 FLOOR SLABS

6.10.1 Floor Slab Subgrade Preparation

All uncontrolled fill containing topsoil should be removed from under the floor slab area. Following the removal, the building footprint area should be proof rolled to identify soft areas. The entire area of the clay subgrade should be scarified to a depth of 150 mm and re-compacted to at least 98 percent of Standard Proctor Maximum Dry Density (SPMDD) within ± 2 percent of its optimum moisture content.

Conventional slab-on-grade floors may be supported on new engineered fill or the existing native clay till. There is some risk of slab heaving for slabs supported on medium plastic clay soils. The risk is reduced provided good surface drainage measures are implemented such that the surface runoff is directed away from the structure and the potential of leaky water lines is minimized. Provisions should be implemented to accommodate floor slab heaving since the risk cannot be readily quantified in addition to the fact that the swelling potential is dependent on the availability of free water. Provisions should include independent partitions, walls, columns, and grade beams.

Any backfill required to raise the subgrade elevation should be completed using gravel, sand or low plastic clay compacted to at least 98 percent of SPMDD, at moisture content within 2 percent of the Optimum Moisture Content (OMC). Imported fill or potential existing soils should be placed in lifts not exceeding 150 mm in compacted thickness.

A minimum of 200 mm of clean, well-graded crushed gravel is recommended beneath the floor slab. Coarse material greater than 50 mm in diameter should be avoided directly beneath the floor slab to prevent stress concentrations in the slab. The granular coarse material should be compacted to a uniform dry density of about 100 percent of SPMDD within ± 2 percent of its optimum moisture content. A recommended typical gradation for stable granular material, for use under the floor slab is provided in table 6.

TABLE 6: GRADATION REQUIREMENTS FOR GRANULAR BACKFILL

Sieve	Percent Passing
20,000 µm	100
10,000 µm	35-77
5,000 µm	15-55
1,250 µm	0-30
80 µm	0-10

The percent fracture by weight (2 faces) should be at least 40 percent. Other appropriate materials, which fall outside the above recommended gradation limits, may be suitable but should be evaluated by a geotechnical engineer prior to use.

6.10.2 Exterior Concrete Aprons

It is recommended that exterior concrete aprons and sidewalks be supported on engineered fill that is compacted in 150 mm thick lifts to at least 98 percent of SPMDD. It is recommended that the concrete aprons be sloped at least 1.5 to 2.0 percent for proper surface drainage.

6.11 CONCRETE

The soluble sulphate analysis tests conducted on six selected soil samples from borehole locations at depths ranging from 1.5 to 3.75 m below ground surface revealed a “negligible to severe” potential for sulphate attack on concrete in contact with native soils at this site. Therefore, all concrete in contact with the native soils at this site should be made from CSA Type 50 (HS), Sulphate resistant cement possessing a minimum 56-day compressive strength of 32 MPa. The maximum water cement ratio should be 0.45. An air entrainment agent of 5 to 7 percent is recommended for improved workability and durability. Any new fill brought to the site should be tested for sulphate concentrations.

6.12 UNDERGROUND UTILITIES

The following general recommendations are provided for underground utilities:

- The subsurface soil conditions in the borehole locations are generally adequate in the clay till. As the groundwater table is high the amount of groundwater infiltration is expected and will depend on the relative percentage of silt and sand at any given location and depth. The clay till encountered at the site is generally at or below optimum moisture, therefore some moisture conditioning might be required to achieve proper compaction.

- Temporary surcharge loads, such as soil spill-piles, should not be allowed within 2.0 meters of an unsupported excavation face while mobile vehicles should be kept back at least 1.5

meter. All excavations should be checked regularly for signs of sloughing or failures, especially after rainfall periods.

- To minimize pipe loading, trench widths should be minimal but compatible with safe construction operations. The trench width must be wide enough to accommodate pipe bedding and compaction equipment.

- Based on information obtained from the borehole logs, trench excavation are mainly expected to be in clay till but may encounter sand layers and gravel depending on the depth of the trench.

- Groundwater seepage will likely be encountered during excavations especially within sand layers. Temporary dewatering measures may be required during utility installation. Pumping water from trenches during installation should be sufficient to maintain the working conditions. In general, flow rates into trenches from clay till should be of magnitude that can be handled by common trench grading practices and sumps and pumps where necessary. Delays in construction will likely occur. For underground services below the water table, increased construction cost will occur because of the need for drying of the wet soils.

- The type of material encountered and the proposed depth of the trench will largely govern the temporary excavation slope requirements. For trenches excavated in the firm to stiff high plastic clay till, the following maximum temporary trench slopes may be used for short term excavations (up to about 7 days).

TABLE 7: RECOMMENDED TRENCH SLOPE ANGLES

Depth of Trench (m)	Recommended Trench Slope Angle
Native Clay till up to 4 m	0.6 H : 1 V
Greater than 4 m to 10 m	1 H : 1 V
Greater than 8 m	1.5 H : 1 V

Trench slopes greater than 8 m should be benched to comply with the above guidelines. It is recommended that the trenching be carried out in relatively short lengths and all trenches should be backfilled at the end of each day. Flatter slope may be required in areas of wet sand and or soft to firm clay till. Alternatively, the excavations could be made vertical with proper trench bracing such as portable trench shields.

- The excavated soil should be kept back from the top of the trench by at least the depth of excavation. Personal should not be allowed in the open trench during installation without proper precautions being taken. In all cases excavations should be consistent with occupational health and safety regulations.

6.12.1 Pipe Bedding

- All soft, loosened and disturbed material should be removed from the trench base before placement of bedding. The pipe should be bedded and installed according to the manufacture's specifications. Care should be taken that the pipe is not in contact with rigid objects such as cobbles or rocks as this will cause a stress concentration in the pipe and may result in breakage.

- Granular bedding and pipe bedding procedures should adhere to the Town of Wainwright specifications. It is recommended that a minimum thickness of 150 mm of granular bedding be placed below the pipe. The bedding material should also be placed around the pipe and should extend at least 300 mm above the crown of the pipe. The material should be placed in 150 mm lifts and compacted uniformly to at least 95 percent of the standard proctor maximum dry density.

- To overcome the installation difficulties which may be encountered within some areas of the site, where ingression of groundwater and/or poor bearing conditions may be a problem (BH 10-14 & BH 10-15), it is recommended that a washed drain rock and geotextile separator be utilized for the pipe bedding in these areas of poor pipe bedding conditions. The washed rock and geotextile exact dimensions should be determined in the field during construction.

- It is recommended that a minimum thickness of 150 mm of granular bedding be placed below the pipe. The bedding material should also be placed around the pipe and should extend at least 300 mm above the crown of the pipe. The material should be placed in 150 mm lifts and compacted uniformly to at least 95 percent of the standard proctor maximum dry density. The granular bedding should meet Town of Wainwright specifications.

- In the event that the trench base is situated in firm clay till (pocket penetrometer (PP) readings less than 100 kPa), or saturated sands below the water table where the pipe support conditions may be poor (such as in proposed school area at BH 10-14 & BH 10-15), special bedding procedures may be required to improve pipe support conditions and reduce future settlements of the pipes. The recommended approach consist of sub excavations of the base and placement of a gravel pad of about 300 mm minimum thickness wrapped in a geotextile fabric in the base of the trench for support of the pipe bedding. This technique has been found to provide a better working surface in the trench base and also facilitates trench drainage during pipe installation.

6.12.2 Backfilling

- It should be noted that the ultimate performance of the trench backfill is directly related to the consistency and uniformity of the backfill compaction, as well as the underground construction procedures. In order to achieve this uniformity, the soil lift thickness and compaction criterion should be strictly enforced.

- In general the clay till encountered in the boreholes is expected to range from 2 to 6 percent above the corresponding plastic limits for the material. Therefore drying of the backfill to suitable moisture contents or mixing with drier materials will be required to meet town of Wainwright moisture content and compaction limits. A minimum of 97 percent of SPMDD is recommended for all trench backfill, with the exception of the top 1.5 meters, which should be compacted to 100 percent of SPMDD. The compacted thickness of each lift should not exceed 150 millimetres.

All contractors bidding on the underground portion of this project should be made aware that drying of the native material will be a requirement during trench backfill operations.

- The backfill material beneath and above the pipe should be an approved bedding sand material where conditions allow. This material should be hand placed and hand tamped with care taken to fill the underside of the pipe

- The remainder of the trench above the bedding zone may be backfilled with the excavated on site materials that are free of debris or organics.

- Trench backfill should not be placed frozen, contain snow or ice inclusions, or be placed at temperature below freezing. Heavy compaction equipment should not be allowed to operate above the placed pipe until 1 m of backfill has been placed and compacted above the pipe.

6.13 MANHOLES

- It is recommended that excavation for new manholes should follow the Town of Wainwright roadway specification for trench excavation.

- Manholes may be founded directly on the native undisturbed inorganic soils. In areas of soft base conditions (especially boreholes 10-14 and 10-15), considerations should also be given to the use of a gravel pad wrapped in geotextile or alternatively a lean concrete base below the base of the excavation. It is recommended that the native clay or sand backfill be placed uniformly around the manhole in 150 mm lifts and compacted to not less than 97 percent of the SPMDD, except for the top 1.5 m, which should be compacted to a minimum of 100 percent of SPMDD.

- Due to high groundwater table encountered at this site, buoyancy of the manholes due to hydrostatic uplift pressure on the base should be checked by referring to the nearest available borehole information to determine the potential groundwater levels. If required one method of providing the necessary uplift resistance is to widen the base of the manholes beyond the manhole vertical walls.

6.14 STORM WATER MANAGEMENT POND

The location of the storm water pond is shown on the site plan in the Appendix as given by the client. A subsurface soil encountered in the majority of boreholes is comprised of clay till with inter-bedded sand layers, which should yield moderate to high permeability characteristics for water retention purposes. Topsoil should be stripped and stockpiles for use on pond side slopes or other landscape areas.

Boreholes 10-4 and 10-5 were drilled at the proposed location for the storm water pond. A sand layer was encountered below 5.20 m in borehole 10-4 and extended to the termination depth. Adjacent boreholes, 10-7 and 10-10 also encountered inter-bedded sand layers. If sand layers are encountered within the storm water pond, it is recommended to cover the sand with a clay liner. Clay till at the site is considered to be favourable as liner material.

Clay till used as replacement of sub-excavated material or for the liner material should be compacted in layers not exceeding 200 mm loose thickness and uniformly compacted to at least 95 percent of the SPMDD. The moisture content of the clay should be within ± 2 percent of the corresponding optimum moisture content. It is expected that moisture conditioning (drying) will be required.

The low to medium plastic native clay till encountered to depths up to 9 m is considered suitable for the construction of storm water management ponds since its permeability is estimated less than 10-6 cm/s. Where layers of permeable sand are encountered during excavations it will be necessary to sub-excavate and replace the sub-excavated material with clay to a suggested minimum depth of 600 mm below base level. The native low to medium clay till from the upper zones of the excavations are expected to be suitable for this purpose, however it will need to be disked and dried to achieve acceptable levels of compaction.

Side slopes constructed within the native clay till should be sub-excavated to a minimum of 300 mm and re-compacted to a minimum of 95 percent of SPMDD. The purpose of this procedure is to break up any small sand/silt layers that may be present within the clay till.

All cut slopes used in the construction of the ponds should be constructed no steeper than 4H:1V. Flatter slopes may be required for safety or aesthetic reasons. Fill slopes should be constructed not steeper than 3H:1V. This could be reviewed further once pond details such as depths and locations are finalized.

Some form of erosion and ice protection will be required for the side slopes. This can be accomplished by a riprap cover extending along the slope. The riprap should be placed 0.5 meters (measured vertically) above and below the normal water level. Slopes above the normal water level to the elevation of the high water level should be landscaped with grass as part of the pond construction.

The soil being excavated from the storm water pond is suitable for fill material; although some drying will be required as they are in excess of optimum moisture content.

6.15 ROADWAYS AND SIDEWALKS

6.15.1 Subgrade Preparation

- The existing topsoil and other deleterious materials should be removed prior to construction of roads, sidewalks and other surface utilities. The surface soil conditions will generally be satisfactory.

- Achieving an acceptable road subgrade may not be possible using standard subgrade preparation methods in isolated areas. Underground construction activity may cause pumping and subsequent reduction in bearing capacity. In these areas where standard methods cannot achieve an acceptable base, additional subgrade treatment methods will be required. The use of cement stabilization is a potential option, and would likely require 300 millimetres of thickness to perform satisfactorily. In extreme isolated cases, 500 millimetres of cement stabilization may be required. The envisioned cement content is 20-40 kg/m^2 with the exact content to be determined in the field during construction. The amount of drying performed during trench backfill compaction will directly affect the subgrade performance during roadway construction. Another design option is to sub-cut the very moist soils and import dry clay or gravel. Observations during underground construction will help determine the subgrade treatment required. All subgrade soils should be proof rolled after final compaction and any areas showing visible deflections should be inspected and repaired.

- It is recommended that in all cases, the subgrade should be inspected by Stewart Weir personnel during construction to determine the recommended subgrade treatment.

- In cases of fill areas, the fill material below the upper 150 millimetres should be compacted to 97 percent of SPMDD. Any fill areas should be placed in lifts of 150 mm maximum thickness. All compaction in the high water Table areas should be non-vibratory methods. The native clay till encountered at the site is generally suitable for use as a fill material with adequate moisture control.

- Surface water will often collect within the granular subgrade, causing subgrade softening and pavement damage. The use of drainage holes installed into the catch basins, immediately below the bottom of the granular lift, is recommended in order to provide drainage for any water collecting within the granular base.

- The near surface clay soils encountered in most areas exhibits a moderate swelling potential. It is recommended that subgrade soils not be allowed to dry excessively when exposed.

- All crushed gravel base course and surface layer materials should be placed and compacted to 100 percent of SPMDD. Asphalt concrete pavement should be compacted to at least 98 percent of the recommended Marshall density of the mix design being utilized. The materials should meet the minimum requirements as established by the Town of Wainwright.

- Fills within roadway right of way should be uniformly compacted to a minimum of 95 percent of SPMDD within 2 percent of the optimum moisture content.

- Roadways should be graded to reduce the water seepage within the subgrade. A minimum pavement grade of 0.5 percent is recommended.

6.15.2 Roadway Design Recommendations

Roadway subgrades are expected to consist of low to medium plastic clay till. Moisture contents within the low to medium plastic clay till are generally 2 to 6 percent above the optimum; hence the subgrade will require drying and moisture conditioning to achieve the required compaction. Depending on the time of the year, and moisture conditions of the subgarde, it may be necessary to sub-excavate any wet soil and replace with dry clay fill or alternatively cement stabilization of the subgrade.

The following additional recommendations apply to design and construction of subgrades:

1. Subgrades areas that become softened as a result of construction traffic or weather conditions should be sub-excavated and replaced with inorganic low to medium plastic clay or clean granular fill.

2. The subgrade may be proof rolled to detect soft/wet zones. Soft material should be sub-excavated and replaced with well compacted clay or cement stabilizes as required.

3. Any additional fill required to raise the road to subgarde level may consist of inorganic low to medium plastic clay or clean gravel fill. The back fill should be placed and uniformly compacted in 150 mm compacted lifts to at least 95 percent of the SPMDD.

4. The upper 150 mm of the subgarde should be scarified, cement stabilized at a minimum application rate of 10 kg/m^2, uniformly compacted to 100 percent of the SPMDD of the material. The actual dosage rate should be decided at the time of construction depending on subgrade conditions.

5. It is recommended that the finished subgrade surface be sloped at a minimum of 1 percent toward catch basin, gutters or perimeter ditches. The purpose of this is to drain any subsurface water from the subgarde and thereby prevent ponding of water on the pavement subgarde that could result in swelling, softening and/or possible frost heaving of the clay subgarde.

6. It is recommended that all areas behind the back of curb/sidewalk (where used) be backfilled as soon as possible to avoid water permeating into the subgrade from free standing water puddles which can cause subgrade softening.

It should be noted that the clay till encountered on site are moderately susceptible to frost and to swelling or shrinkage in response to changes in moisture content. Some movement should therefore be expected in the subgarde soils due to seasonal changes. Damage resulting from such movements should be considered in any maintenance program.

Table 8, following, shows the two stage pavement design that may be applied to the proposed roadways and lanes. An estimated California Bearing Ratio (CBR) of 3 percent was used in the design as well as a design life of 20 years.

TABLE 8: RECOMMENDED PAVEMENT DESIGN

Structure	Minimum Thickness (mm)
Local Residential	35 mm asphalt concrete @ FAC 65 mm asphalt concrete
	250 mm crushed gravel (20 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade
Minor Collector	35 mm asphalt concrete @ FAC 65 mm asphalt concrete
	275 mm crushed gravel (20 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade
Major Collector	35 mm asphalt concrete @ FAC 90 mm asphalt concrete
	300 mm crushed gravel (20 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade
Residential Lane	100 mm of crushed gravel (20 mm) 250 mm crushed gravel (50 mm)
	150 mm of compacted subgrade to 100 percent SPMDD or Cement stabilized clay subgrade

All crushed gravel base course and surface layer materials should be placed and uniformly compacted to 100 percent of SPMDD. Asphalt should be compacted to town to Wainwright specifications.

7.0 CLOSURE

This report was based on the findings at fifteen borehole locations within the proposed residential subdivision. It should be noted that surficial deposits are generally variable in nature. Should different subsoil and groundwater conditions be encountered during construction, Stewart Weir should be notified immediately and the recommendations submitted herein will be reviewed, and revised if necessary.

The construction of foundations should be inspected by a qualified engineer or technologist. Similarly, the construction of any structural fill on which subgrade support will be required should be monitored by both on-site visual inspection and compaction tests.

Proposed school area covered under boreholes 10-14 and 10-15 indicated the presence of moist to wet and soft clay till and resulted in lower “PP” and “N” values. Further geotechnical investigation is recommended in the subject area, when location and size of the proposed school is finalized.

This report was prepared for the exclusive use of **Town of Wainwright** and is the authorized user for the specific application to the project described in the report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

APPENDIX

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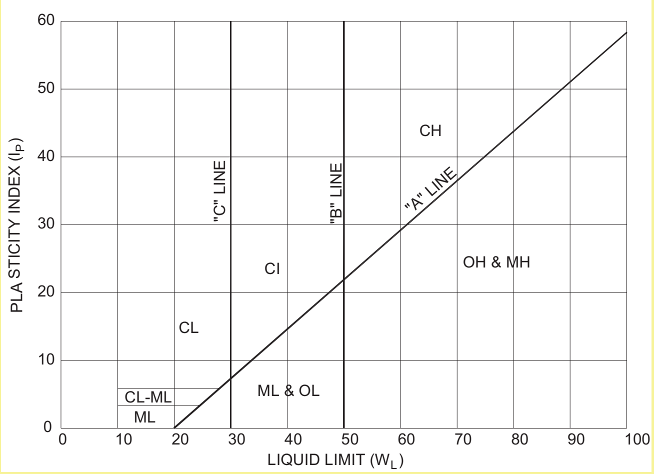
TABLE 1

<table>
<tr><th colspan="10">UNIFIED SOIL CLASSIFICATION SYSTEM
(Modified by PFRA)</th></tr>
<tr><th colspan="3">MAJOR DIVISIONS</th><th colspan="2">GROUP SYMBOLS</th><th>LOG SYMBOLS</th><th>TYPICAL NAMES</th><th colspan="3">LABORATORY CLASSIFICATION CRITERIA</th></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS
(More than half of material is larger than 80μm sieve size)</td><td rowspan="4">GRAVELS
(More than half of coarse fraction is larger than the 5000μm sieve size)</td><td rowspan="2">CLEAN GRAVELS
(Little or no fines)</td><td colspan="2">GW</td><td>▨</td><td>Well-graded gravels, gravel-sand mixtures, little or no fines</td><td rowspan="8">Determine the amount of sand and gravel from the grain size curve
Depending on percent of fines (fraction smaller than the 80μm sieve), coarse-grained soils are classified as follows:
Less than 5%.......................GW, GP, SW, SP
More than 12%...................GM, GC, SM, SC
5 to 12%.............................Borderline cases requiring dual symbols**</td><td colspan="2">$C_u = D_{60}/D_{10}$ is greater than 4:
$C_c = (D_{30})^2/(D_{10}xD_{60})$ is between 1 and 3</td></tr>
<tr><td colspan="2">GP</td><td>▨</td><td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td><td colspan="2">Not meeting all gradation requirements for GW</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES
(Appreciable amount of fines)</td><td>GM*</td><td>d
u</td><td>▨</td><td>Silty gravels, gravel-sand-silt mixtures</td><td>Atterberg Limits below "A" Line or P.I. Less than 4</td><td rowspan="2">Above "A" Line with P.I. Between 4 and 7 are borderline cases requiring use of dual symbols</td></tr>
<tr><td colspan="2">GC</td><td>▨</td><td>Clayey gravels, gravel-sand-clay mixtures</td><td>Atterberg Limits above "A" Line with P.I. greater than 7</td></tr>
<tr><td rowspan="4">SANDS
(More than half of coarse fraction is smaller than the 5000μm sieve size)</td><td rowspan="2">CLEAN SANDS
(Little or no fines)</td><td colspan="2">SW</td><td>▨</td><td>Well-graded sands, gravelly sands, little or no fines</td><td colspan="2">$C_u = D_{60}/D_{10}$ is greater than 6:
$C_c = (D_{30})^2/(D_{10}xD_{60})$ is between 1 and 3</td></tr>
<tr><td colspan="2">SP</td><td>▨</td><td>Poorly graded sands, gravelly sands, little or no fines</td><td colspan="2">Not meeting all gradation requirements for SW</td></tr>
<tr><td rowspan="2">SANDS WITH FINES
(Appreciable amount of fines)</td><td>SM*</td><td>d
u</td><td>▨</td><td>Silty Sands; sand-silt mixtures</td><td>Atterberg Limits below "A" Line or P.I. Less than 4</td><td rowspan="2">Limits plotting in hatched zone with P.I. Between 4 and 7 are borderline cases requiring use of dual symbols</td></tr>
<tr><td colspan="2">SC</td><td>▨</td><td>Clayey sands; sand-clay mixtures</td><td>Atterberg Limits above "A" Line with P.I. greater than 7</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS
(More than half of material is smaller than 80μm sieve size)</td><td rowspan="3">CLAYS
(Above "A" Line on PLASTICITY CHART: negligible organic content</td><td>W_L<30%</td><td colspan="2">CL</td><td>▨</td><td>Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays</td><td colspan="3" rowspan="7">PLASTICITY CHART


(NOTE: For soils passing No. 400 sieve & sieve size in μm)</td></tr>
<tr><td>30<W_L<50 %</td><td colspan="2">CI</td><td>▨</td><td>Inorganic clays of medium plasticity, gravelly clays, sandy clays, silty clays</td></tr>
<tr><td>W_L>50%</td><td colspan="2">CH</td><td>▨</td><td>Inorganic clays of high plasticity, fat clays</td></tr>
<tr><td rowspan="2">SILTS
(Below "A" Line; negligible organic content)</td><td>W_L<50%</td><td colspan="2">ML</td><td>▨</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td></tr>
<tr><td>W_L>50%</td><td colspan="2">MH</td><td>▨</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td></tr>
<tr><td rowspan="2">ORGANIC SILTS AND CLAYS
(Below "A" Line)</td><td>W_L<50%</td><td colspan="2">OL</td><td>▨</td><td>Organic silts and organic silty clays of low plasticity</td></tr>
<tr><td>W_L>50%</td><td colspan="2">OH</td><td>▨</td><td>Organic clays of medium to high plasticity, organic silts</td></tr>
<tr><td colspan="2">HIGHLY ORGANIC SOILS</td><td colspan="2">Pt</td><td>▨</td><td>Peat and other highly organic soils</td><td colspan="3">Strong colour or odor and fibrous textures</td></tr>
</table>

▨	BEDROCK	▨	CONGLOMERATE
▨	SANDSTONE	▨	COAL
▨	SHALE	▨	OVERBURDEN
▨	LIMESTONE	☐	TOPSOIL

NOTE

* Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits: suffix d is used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

** Borderline classifications, used for soils processing characteristics of two groups, are designated by combination of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.

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TABLE 2

ENGINEERING USES AND CHARACTERISTICS OF SOILS BASED ON THE UNIFIED SOIL CLASSIFICATION SYSTEM

Group Symbol Letter	Important Properties When Compacted			Workability As A Construction Material	Standard Maximum Dry Density (kg/m³) and Voids Ratio	Compaction Characteristics	Potential Frost Action
	Permability Characteristics k (cm per sec.)	Sheer Strength (when saturated)	Compressibility And Expansion (when saturated)				
GW	Pervious	Excellent	Almost none	Excellent	2000-2160 0.35	Excellent, tractor, rubber tired, steel wheeled roller	none to very slight
GP	Very Pervious	Good	Almost none	Good	1840-2000 0.45	Good to Excellent, tractor, rubber tired, steel wheeled roller	none to very slight
GM	Semi-pervious to pervious	Good	Very slight to slight	Good	1920-2160 0.40	Good to Excellent with close control, tractor, rubber tired, sheepsfoot roller	slight to medium
GC	Impervious	Good to Fair	Slight	Good	1840-2160 0.30	Excellent, rubber tired, sheepsfoot roller	slight to medium
SW	Pervious	Excellent	Almost none	Excellent	1760-2080 0.40	Excellent, tractor, rubber tired, equipment	none to very slight
SP	Pervious	Good	Almost none	Fair	1600-1920 0.70	Good to Excellent, tractor rubber tired equipment	none to very slight
SM	Semi-pervious to impervious	Good	Very slight to medium	Fair	1760-2000 0.60	Good to Excellent with close control, tractor, rubber tired, sheepsfoot roller	slight to high
SC	Impervious	Good to Fair	Slight to medium	Good	1680-2000 0.35	Excellent, sheepsfoot roller, rubber tired, equipment	slight to high
ML	Semi-pervious to impervious	Fair	Slight to medium	Fair	1520-1920 0.70	Good to poor, close control essential, rubber tired roller, sheepsfoot roller	medium to very high
CL	Impervious	Fair	Medium	Good to Fair	1520-1920 0.70	Fair to Good, sheepsfoot roller, rubber tired roller	medium to high
OL	Semi-pervious to impervious	Poor	Medium to High	Fair	1280-1600 0.90	Fair to poor, sheepsfoot roller	medium to high
CI	Impervious	Fair to Poor	Medium to High	Fair	1440-1840 0.80	Fair to poor, sheepsfoot roller	medium to high
MH	Semi-pervious to impervious	Fair to Poor	High	Poor	1120-1520 0.70	Poor to very poor, sheepsfoot roller	medium to very high
CH	Impervious	Poor	High	Poor	1200-1680 0.80	Fair to poor, sheepsfoot roller	medium
OH	Impervious	Poor	High	Poor	1040-1600 0.70	Poor to very poor, sheepsfoot roller	medium
Pt	Semi-pervious to impervious, variable		Very High			Compaction not practical	slight

TABLE 3

LIQUIDITY INDEX AND SOIL TYPES TABLE

(Liqiudity Index as established by PFRA)

LIQUIDITY INDEX	CL	CI	CH	ML&MH	SM	SC
0.0 - 0.20	CAN BE EXCAVATED WITH RUBBER TIRED EQUIPMENT WITH NO DIFFICULTY					
0.21 - 0.40	INCREASINGLY MORE DIFFICULT TO EXCAVATE WITH RUBBER TIRED EQUIPMENT					
0.41 - 1.0	MUST USE A BACKHOE OR DRAGLINE TO EXCAVATE MATERIAL					
RECOMMENDED MOISTURE COMPACTION RANGE FROM OPTIMUM MOISTURE CONTENTAND A LIQUIDITY INDEX LESS THAN 0.2	FROM -1% TO +1%	FROM OPT. TO +1%	FROM OPT. TO +3%	FROM OPT. TO -3%	FROM -1% TO -2%	FROM -1% TO +1%

The Liquidity Index of a soil is the relationship of the natural moisture content of a soil to moisture contents of that soil at both its Plastic and Liquid Limits. It may be described as how close the natural moisture content is to these limits. The Liquidity Index equals 0.0 at the Plastic Limit and 1.0 at the Liquid Limit. It is negative below the Plastic Limit (below optimum) and is over 1.0 when the soil is in a fully liquid state. The Liquidity Index formula is:

$$\text{Liquidity Index} = \frac{\text{Natural Moisture Content} - \text{Plastic Limit}}{\text{Plasticity Index}}$$

The Liquidity Index may be used to anticipate excavation difficulties in deep cuts. It may also be used to predict practical depth limits for borrow pits, considering that the moisture content of a does not vary appreciably over long periods of time below a depth of about 3.5 meters.

If the Liquidity Index is below 0.20, then rubber tired equipment will have no difficulty excavating the soil. As the Liquidity Index increases, excavating the soil becomes more difficult.

Once the Liquidity Index increases to above 0.40, backhoes or draglines would have to be used to excavate the soil.

This table reflects the AT Specification Amendment AMC_S246 to be used in all tenders with a grading component (as of 2010).

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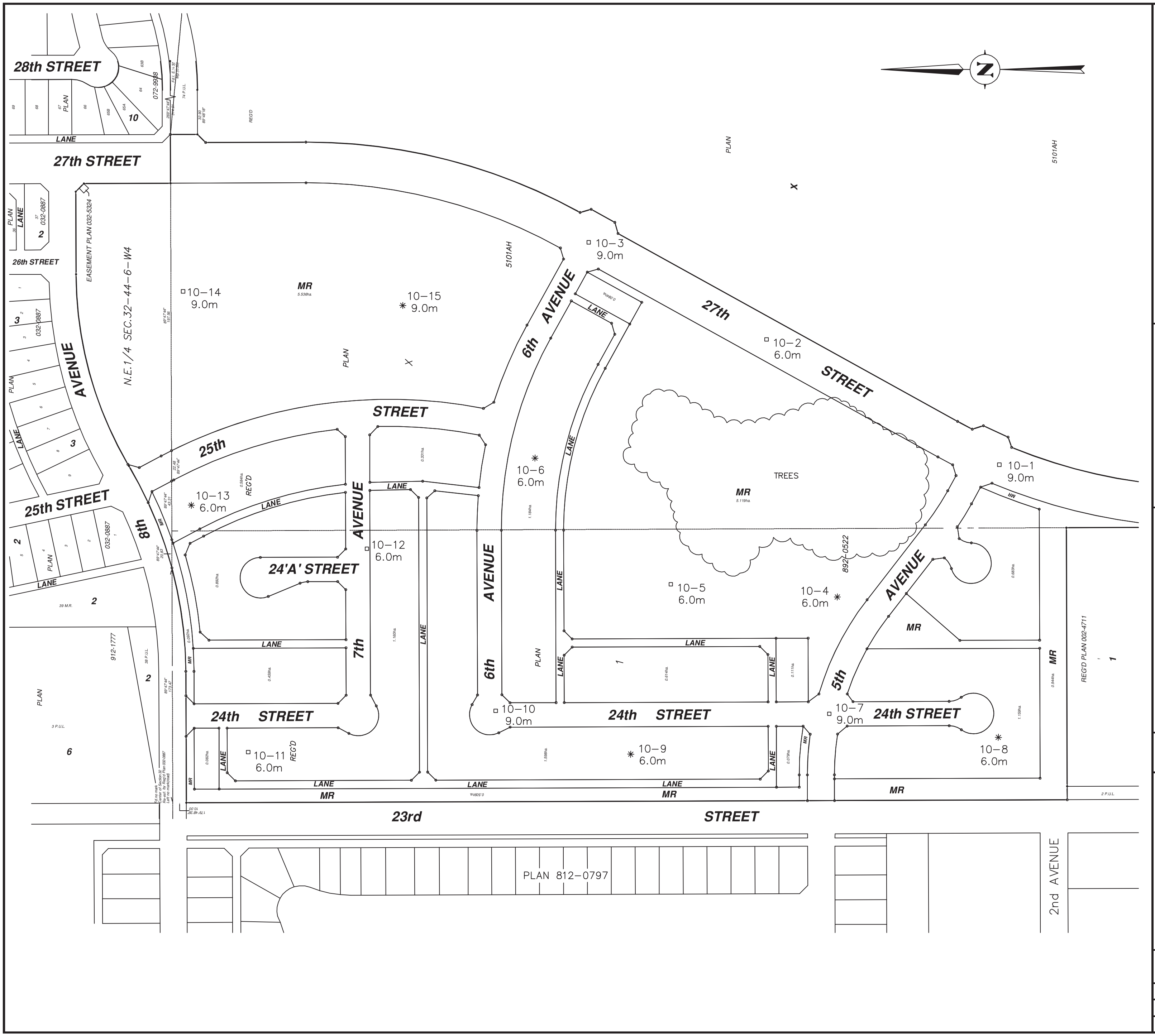
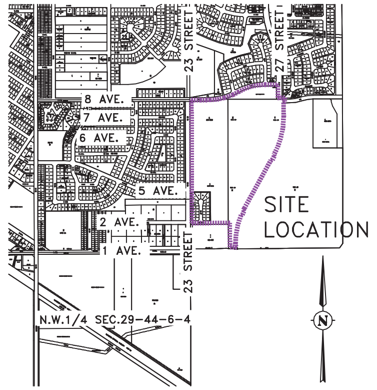
TABLE 4

Group Symbol	Potential Erosion Resistance		Potential Frost Action	
GW	Excellent	E	none to very slight	N-VS
GP	Good to Excellent	G-E	none to very slight	N-VS
GM	Fair to Good	F-G	slight to medium	S-M
GC	Good to Excellent	G-E	slight to medium	S-M
SW	Excellent	E	none to very slight	N-VS
SP	Excellent	E	none to very slight	N-VS
SM	Fair to Poor	F-P	slight to high	S-H
SC	Good to Excellent	G-E	slight to high	S-H
ML	Poor	P	medium to very high	M-VH
CL	Good to Fair	G-F	medium to high	M-H
OL	Fair	F	medium to high	M-H
CI	Fair to Good	F-G	medium to high	M-H
MH	Poor	P	medium to very high	M-VH
CH	Excellent	E	medium	M
OH	Good to Excellent	G-E	medium	M
Pt	Poor	P	slight	S





LOCATION PLAN
SCALE 1:20.000

LEGEND

BOREHOLES WITHOUT STAND PIPE PIEZOMETER . . . . ✳
BOREHOLES WITH STAND PIPE PIEZOMETER . . . . . . □

OWNER

TOWN OF WAINWRIGHT

SITE PLAN
SHOWING
BOREHOLES LOCATION
WITHIN
URBAN RESEDENTAL SUBDIVISION
(ENSTROM)
IN THE
S.E. 1/4 SEC. 32
TWP. 44 – RGE. 6 – W. 4MER.
WAINRIGHT – ALBERTA

Stewart Weir
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780.410.2580

DWG NAME: 33962–Site Plan	DATE: August 10, 2010
DRAFTED BY: G.L.	SCALE: 1:2000
CHECKED BY: A.F.	FILE No.: ED60.33962

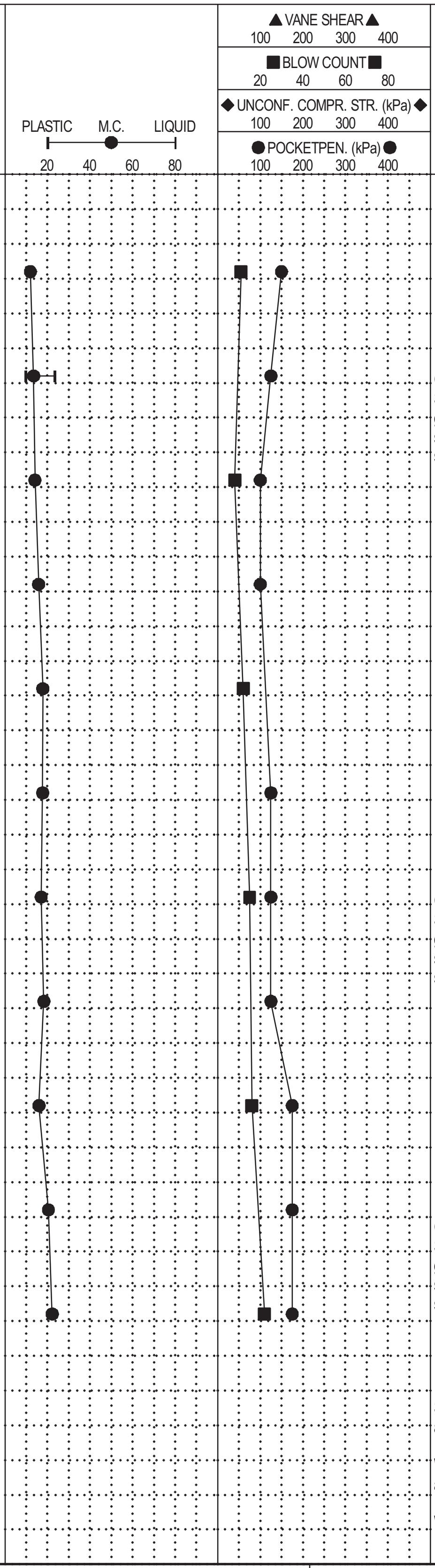
PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-01**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE: ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⬚ PEA GRAVEL ▥ SLOUGH ▩ GROUT ▨ DRILL CUTTINGS ⁘ SAND

Depth (m) Water Level	USC	SOIL DESCRIPTION
0	TS	TOPSOIL (150mm), black, damp
	CL	CLAY TILL, sandy, some silt, medium brown, moist, low plasticity, est. 2% above opt., stiff, trace gravel, salts, coal specks
		@ 2.3m - rust specks
▼		
		@ 5.25m - trace water seepage
		@ 6.8m - som rust stains
		@ 8.45m - sand lense (5-10cm), medium to fine grained, brown, wet, medium dense
		End of borehole at 8.71m Backfilled with drill cuttings. Standpipe installation depth = 8.71m
10		

SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm
☒	G1	3-4-7
☰	G2	
☒	G3	3-4-4
☰	G4	
☒	G5	3-6-6
☰	G6	
☒	G7	4-5-10
☰	G8	
☒	G9	6-8-8
☰	G10	
☒	G11	8-11-11



OTHER DATA

Grab Sample # 2
Sieve Analysis :
gravel 3.2%
sand 43.8%
silt&clay 53.0%

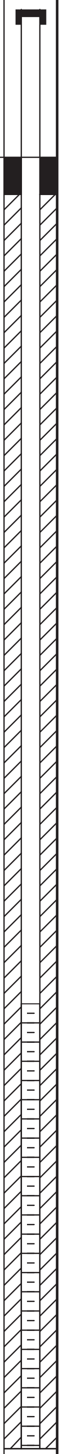
Grab Sample # 7
Sieve Analysis :
gravel 1.7%
sand 38.1%
silt&clay 60.2%

Grab Sample # 11
Sieve Analysis :
gravel 0.0%
sand 22.9%
silt&clay 77.1%

Slough at 8.0m
at completion

Water Level at 7.3m
at completion

Water Level at 2.80m
11 days later



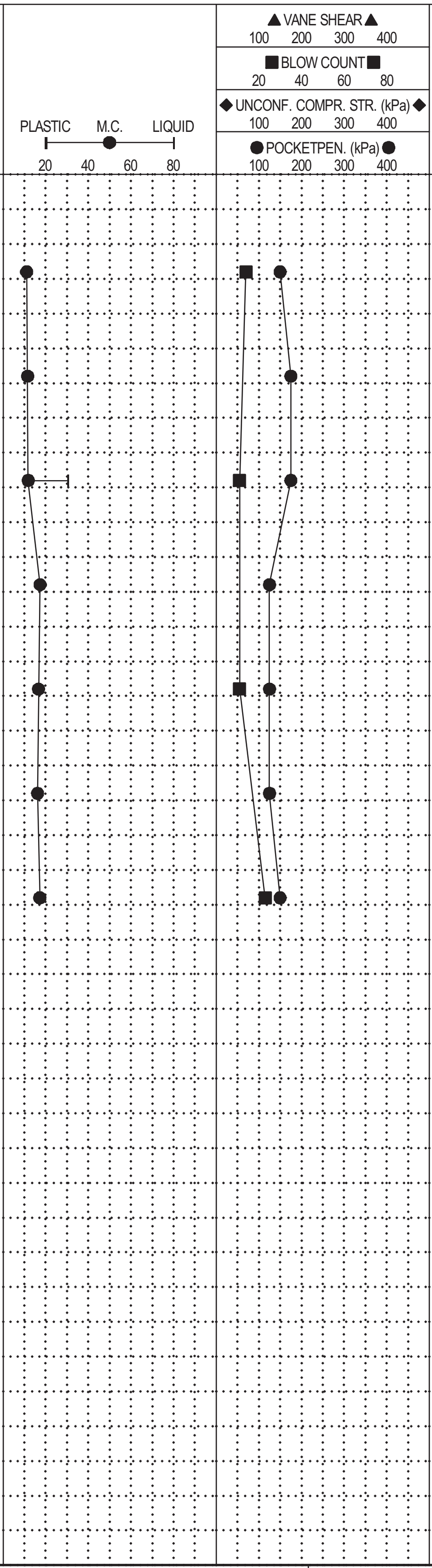
Stewart Weir

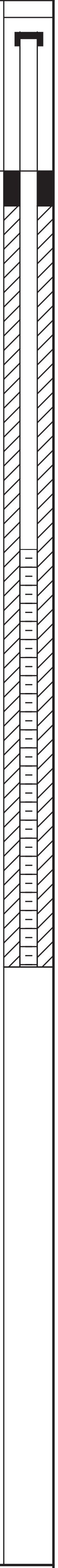
LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-02**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE	■ SHELBY TUBE	◪ CORE SAMPLE	☒ SPT SAMPLE	☰ GRAB SAMPLE	▥ NO RECOVERY	
BACKFILL TYPE	■ BENTONITE	⁙ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▨ DRILL CUTTINGS	⁙ SAND

Depth (m) Water Level	USC	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS	TOPSOIL (150mm), black, damp				
		CLAY TILL, sandy, some silt, medium brown, medium to low plasticity, est. opt., stiff, trace gravel, salts, coal specks.	☒	G12	5-7-7	
1			☰	G13		Grab Sample # 13 Sieve Analysis : gravel 0.0% sand 45.1% silt&clay 54.9%
2		@ 2.25m - rust specks	☒	G14	4-5-6	Grab Sample # 14 Sieve Analysis : gravel 1.2% sand 37.6% silt&clay 61.2%
3	CI-CL		☰	G15		
▼			☒	G16	3-5-6	
4			☰	G17		
5		@ 5.25m - some water seepage	☒	G18	8-11-12	Grab Sample # 18 Sieve Analysis : gravel 0.6% sand 92.9% silt&clay 6.5%
		@ 5,35m - sand lense (200-250mm), medium to fine grained, light brown, wet, water seepage, medium dense				
6		End of borehole 5.71m Backfilled with drill cuttings. Standpipe installation depth = 5.71m				
7						
8						Slough at 5.0m at completion Water Level at 4.0m at completion
9						Water Level at 3.45m 11 days later
10						





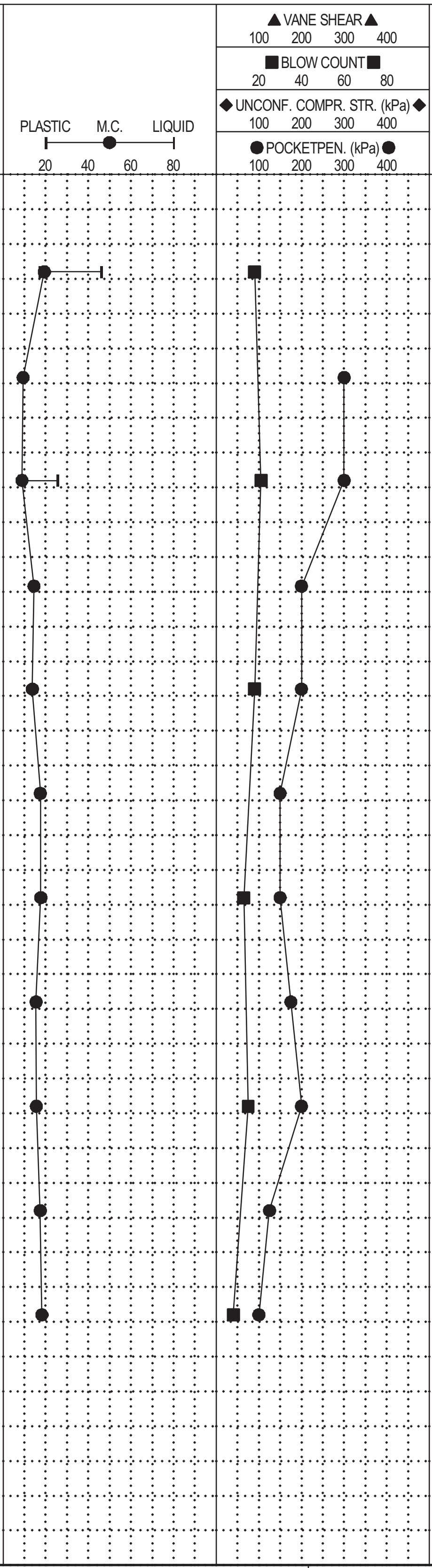
Stewart Weir	LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
	REVIEWED BY: LD	COMPLETION DATE: 8/4/10
		Page 1 of 1

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-03**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE: ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ⁘ SAND

Depth (m) Water Level	USC	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS	TOPSOIL (150mm), black, damp				
	CI	CLAY TILL, sandy, some silt, medium brown, moist, medium plasticity, est. 1% above opt., stiff	☒	G19	5-8-10	Grab Sample # 19 Sieve Analysis : gravel 0.0% sand 10.9% silt&clay 89.1%
1		@ 0.85m - dark brown, sandier, very stiff				
			☰	G20		
	CL	@ 1.5-3.0m - low plasticity, est. 1% below opt.				
2			☒	G21	6-10-11	Grab Sample # 21 Sieve Analysis : gravel 0.3% sand 42.4% silt&clay 57.3%
		@ 2.35m - rust specks				
3			☰	G22		
	CI	@3.0m - medium plasticity, est. 2% above opt., stiff				SULPHATE TEST BH 10-03 G21 @ 2.25m - MODERATE
▼			☒	G23	5-8-10	
4		@ 3.8m - stiff				
			☰	G24		
5			☒	G25	4-5-8	
6			☰	G26		
			☒	G27	6-6-9	
7		@ 7.0m - stiff to firm				Grab Sample # 28 Sieve Analysis : gravel 0.4% sand 34.4% silt&clay 65.2%
			☰	G28		
8			☒	G29	3-4-4	
						NO SLOUGH NOTED at completion
9		End of borehole 8.71m Backfilled with drill cuttings. Standpipe installation depth = 8.71m				Borehole dry at completion Water Level at 3.80m 11 days later
10						

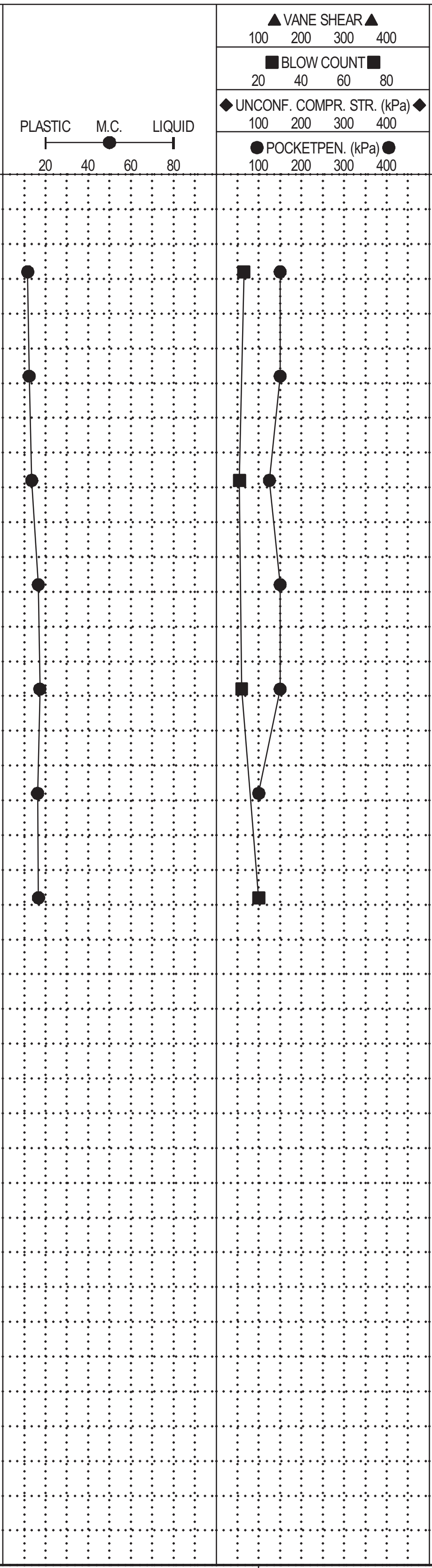


LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-04**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (100mm), black, damp				
			CLAY TILL, sandy, some silt, medium brown, moist, low plasticity, est. 2% above opt., stiff, trace gravel, salts, coal specks				
				☒	G30	4-5-8	
1							
				☰	G31		
2							
				☒	G32	4-5-6	
	CL		@ 2.3-5.0m - rust inclusions (<1cm diam)				
3				☰	G33		Grab Sample # 33 Sieve Analysis : gravel 0.8% sand 32.9% silt&clay 66.3%
▼							
				☒	G34	4-5-7	
4							
				☰	G35		Grab Sample # 35 Sieve Analysis : gravel 0.3% sand 39.1% silt&clay 60.6%
5							
	SP-SM		SAND, medium to fine grained, rusty brown, wet, water seepage, medium dense	☒	G36	7-8-12	Grab Sample # 36 Sieve Analysis : gravel 2.0% sand 87.9% silt&clay 10.1%
6			End of borehole 5.71m Backfilled with drill cuttings				
7							
8							Slough at 4.8m at completion Water Level at 3.3m at completion
9							
10							



Stewart Weir

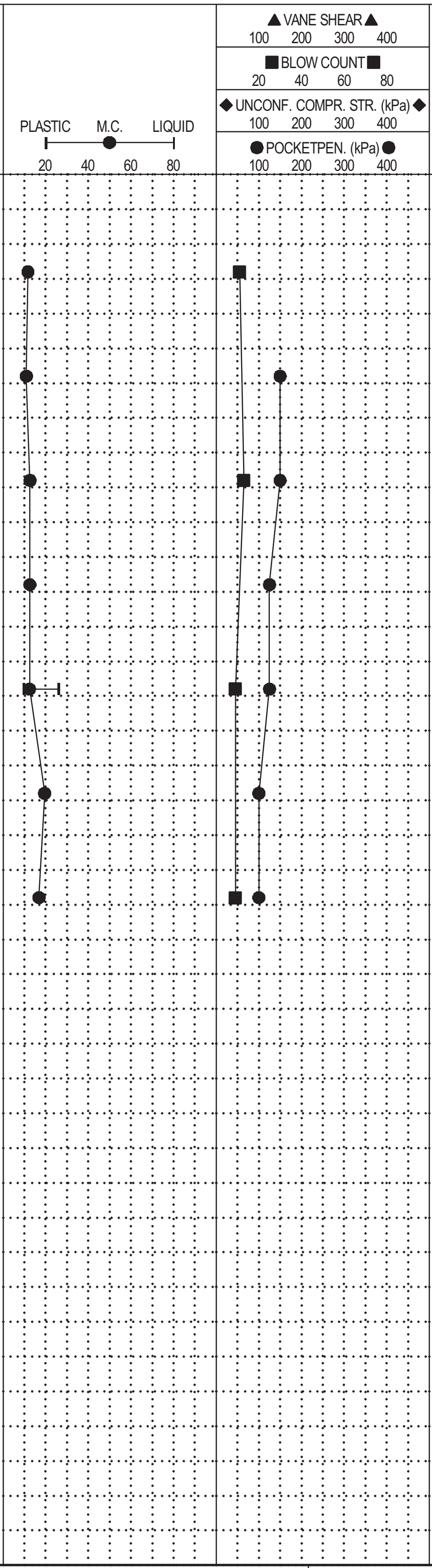
LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

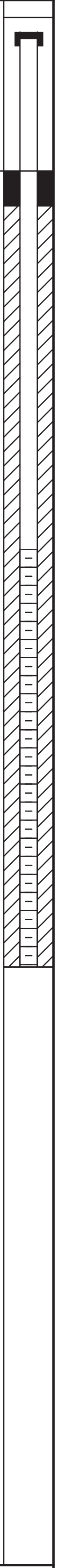
PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-05**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⛶ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁙ SAND

Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (150mm), black, moist				
		▨	CLAY TILL, sandy, some silt, medium brown, moist, low plasticity, est. 2% above opt., stiff, trace gravel, salts, coal specks				
				☒	G37	3-5-6	
1							
				☰	G38		
2							
				☒	G39	4-6-7	
			@ 2.5m - stiff to firm, occasional rust specks				
3	CL			☰	G40		
▼							
				☒	G41	3-4-5	Grab Sample # 41 Sieve Analysis : gravel 2.1% sand 41.7% silt&clay 56.2%
4							
				☰	G42		
5							
				☒	G43	3-4-5	Grab Sample # 43 Sieve Analysis : gravel 0.0% sand 34.5% silt&clay 65.5%
6			End of borehole 5.71m Backfilled with drill cuttings. Standpipe installation depth = 5.71m				
7							SULPHATE TEST BH 10-05 G41 @ 3.75m - MODERATE
8							NO SLOUGH NOTED at completion
							Borehole dry at completion
9							Water Level at 3.30m 11 days later
10							





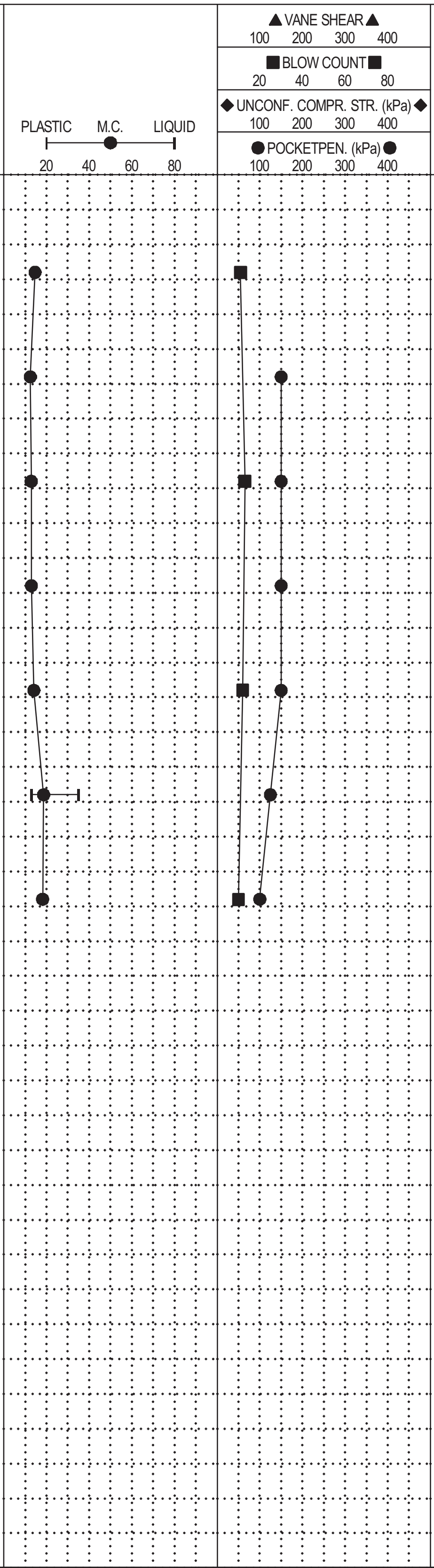
Stewart Weir

LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-06**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (160mm), black, moist				
		▨	CLAY TILL, sandy, some silt, medium brown, moist, medium plasticity, est. opt., stiff, trace gravel, salts, coal specks				
				☒	G44	3-5-6	
1			@ 1.0m - rust specks, some salts				
				☰	G45		
2				☒	G46	3-5-8	
			@ 2.4m - occasional rust (<1cm diameter)				
3	CI			☰	G47		
				☒	G48	4-4-8	
4			@ 4.0m - stiff to firm				
				☰	G49		Grab Sample # 49 Sieve Analysis : gravel 0.5% sand 33.3% silt&clay 66.2%
5				☒	G50	3-4-6	
6			End of borehole 5.71m Backfilled with drill cuttings.				
7							
8							Borehole dry at completion
9							
10							



Stewart Weir

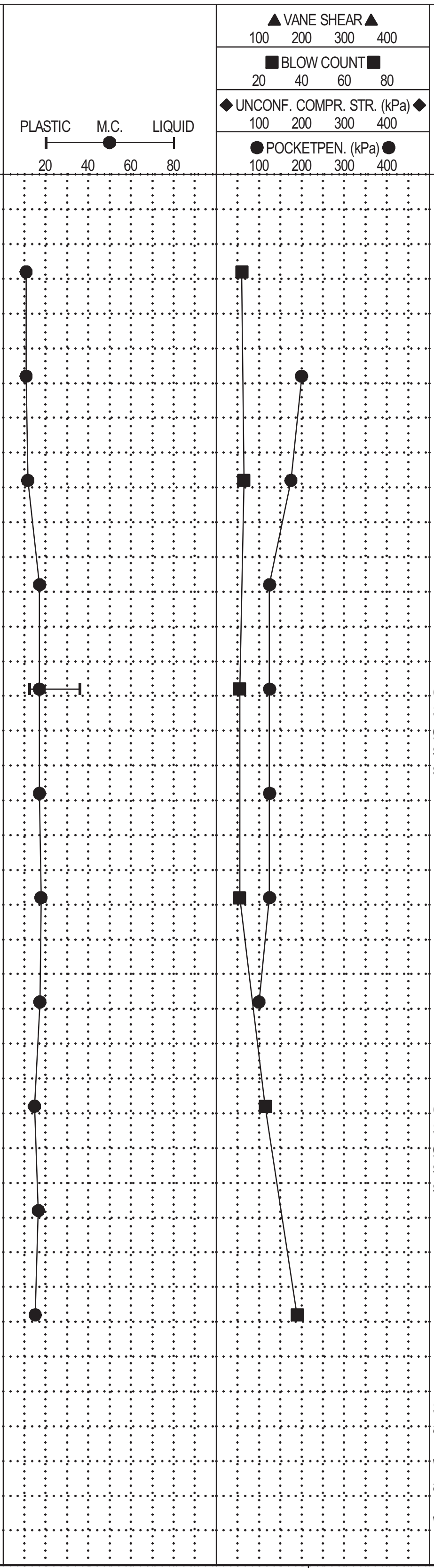
LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-07**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE	■ SHELBY TUBE	◪ CORE SAMPLE	☒ SPT SAMPLE	☰ GRAB SAMPLE	▥ NO RECOVERY	
BACKFILL TYPE	■ BENTONITE	⁚ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▨ DRILL CUTTINGS	⁙ SAND

Depth (m) Water Level	USC	SOIL DESCRIPTION
0	TS	TOPSOIL (180mm), black, damp
0.18 – 6.2	CI	CLAY TILL, sandy, some silt, medium brown, moist, medium plasticity, est. 2% below opt., stiff, trace gravel, salts, coal specks @1.5m - dark brown, trace rust specks @3.0m - est. 5% above opt.
▼ 3.2		
6.2 – 8.71	SP-SM	SAND, medium to fine grained, brown, wet, water seepage, medium dense, trace gravel pieces @ 6.75m - hole sloughing in
8.71		End of borehole 8.71m Backfilled with drill cuttings. Standpipe installation depth = 6.0m

SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
☒	G51	3-6-6	
☰	G52		
☒	G53	4-6-7	
☰	G54		
☒	G55	3-5-6	Grab Sample # 55 Sieve Analysis : gravel 4.2% sand 31.4% silt&clay 64.4%
☰	G56		
☒	G57	4-4-7	
☰	G58		
☒	G59	8-10-13	Grab Sample # 59 Sieve Analysis : gravel 2.6% sand 91.3% silt&clay 6.1%
☰	G60		
☒	G61	14-16-22	



OTHER DATA:

Slough at 5.8m at completion

Water Level at 3.7m at completion

Water Level at 3.20m 11 days later

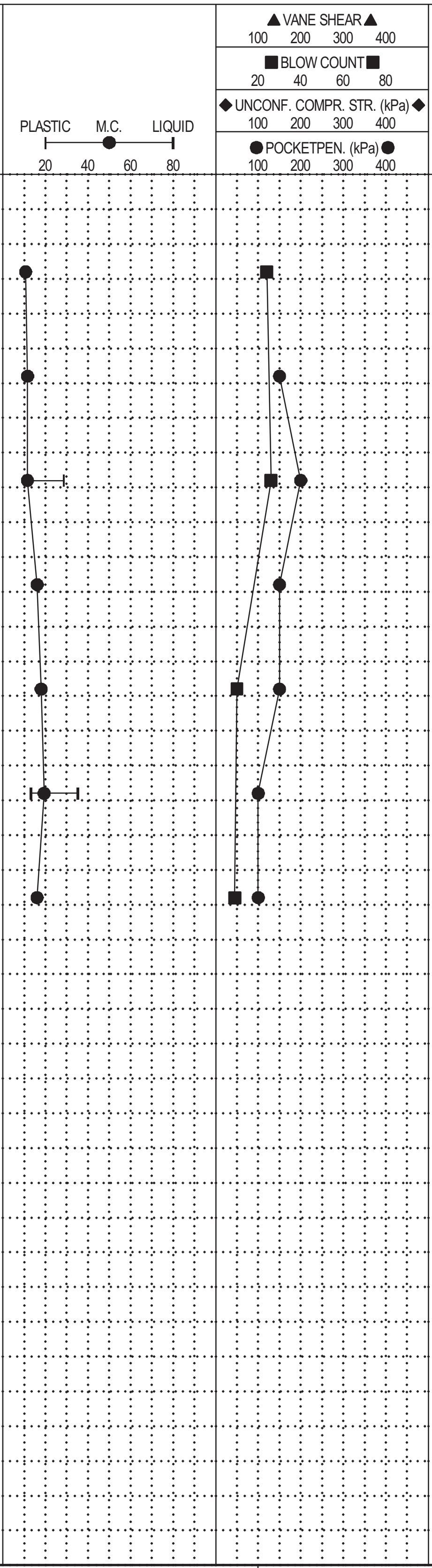
Stewart Weir

LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-08**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, damp				
		▨	CLAY TILL, sandy, some silt, medium brown, moist, low to medium plasticity, est. 2% above opt., stiff, trace gravel, some salts, coal specks	☒	G62	4-12-12	
1							SULPHALTE TEST BH 10-8 G64 @ 2.25m - SEVERE
				☰	G63		
2			@ 2.0m - rust specks	☒	G64	6-12-14	Grab Sample # 64 Sieve Analysis : gravel 0.0% sand 40.7% silt&clay 59.3%
3	CL-CI			☰	G65		
				☒	G66	2-5-5	
4							
				☰	G67		Grab Sample # 67 Sieve Analysis : gravel 0.0% sand 33.0% silt&clay 67.0%
5				☒	G68	3-4-5	
							Borehole dry at completion
6			End of borehole 5.71m Backfilled with drill cuttings.				
7							
8							
9							
10							



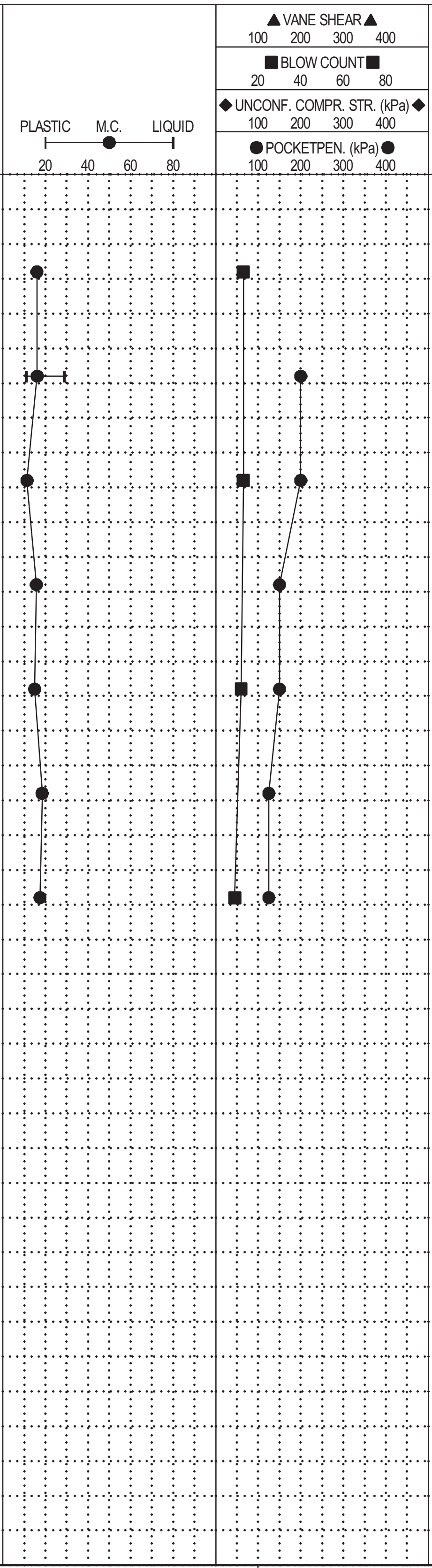
Stewart Weir

LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision
LOCATION: SE 32-44-06 W4
BOREHOLE NO: **10-09**
CLIENT: Town of Wainwright
NORTHING:
CONSULTANT PROJECT NO: ED60.33962
DRILL/METHOD: SOLID STEM AUGER
EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, damp				
	CL-CI	▨	CLAY TILL, sandy, some silt, medium brown, moist, low to medium plasticity, est. 6% above opt., stiff, trace gravel, some salts, coal specks				
				☒	G69	4-6-7	
1							SULPHATE TEST BH 10-9
				☰	G70		Grab Sample # 70 Sieve Analysis : gravel 10.7% sand 33.5% silt&clay 55.8%
2				☒	G71	4-4-9	
			@ 2.1 to 5.71m - occasional rust specks				
3				☰	G72		
				☒	G73	3-6-6	
4				☰	G74		
5				☒	G75	3-4-5	
							Borehole dry at completion
6			End of borehole 5.71m Backfilled with drill cuttings.				
7							
8							
9							
10							



Stewart Weir

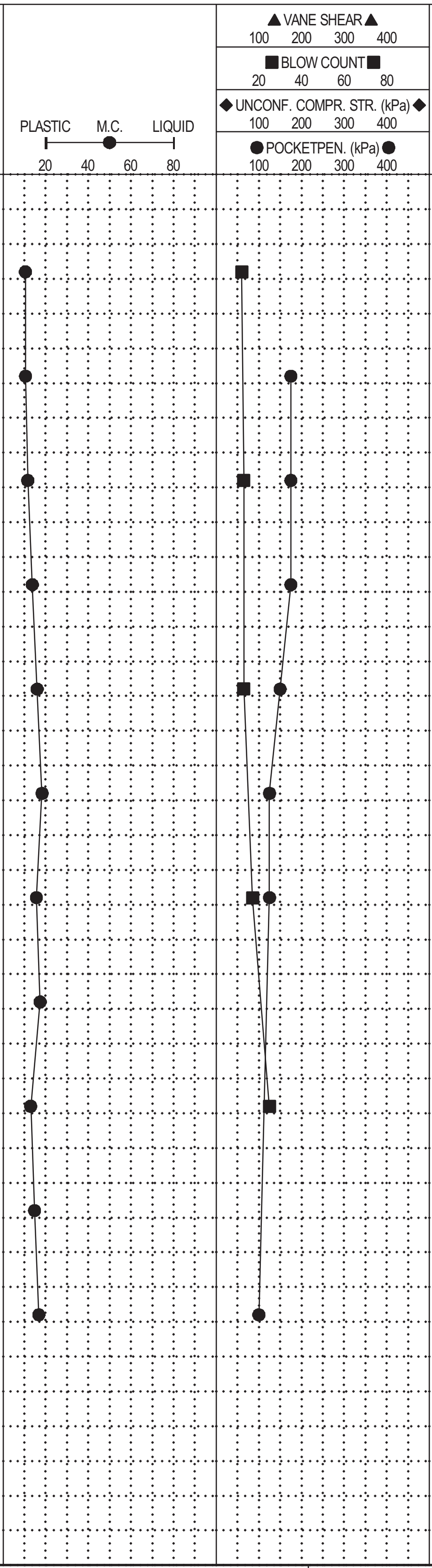
LOGGED BY: DJ
REVIEWED BY: LD
COMPLETION DEPTH: 5.71 m
COMPLETION DATE: 8/4/10

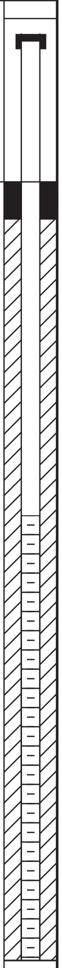
PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-10**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⬚ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ⁘ SAND

Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, moist				
	CI-CL	▨	CLAY TILL, sandy, some silt, medium to dark brown, moist, medium to low plasticity, est. opt., stiff, trace gravel, some salts, coal specks	☒	G76	5-6-6	
1			@ 1.0m - dark brown, moist, some salts	☰	G77		Grab Sample # 77 Sieve Analysis : gravel 1.2% sand 42.9% silt&clay 55.9%
2			@ 2.25m to 5.3m - occasional rust specks, trace salts	☒	G78	4-6-7	
▼ 3				☰	G79		
4				☒	G80	4-6-7	
				☰	G81		
5				☒	G82	5-8-9	
6	SM	⁘	SAND, silty, medium to fine grained, light brown, wet, water seepage, medium dense, hole sloughing in	☰	G83		Grab Sample # 83 Sieve Analysis : gravel 0.0% sand 71.8% silt&clay 28.2%
7				☒	G84	5-11-14	
				☰	G85		
8	CI-CL	▨	CLAY TILL, sandy, some silt, medium brown, moist, medium to low plasticity, est. opt., stiff to firm, trace gravel, coal specks	☰	G86		
9			End of borehole 8.71m Backfilled with drill cuttings. Standpipe installation depth = 5.25m				Slough at 4.1m at completion Water Level at 2.1m at completion Water Level at 2.90m 11 days later
10							





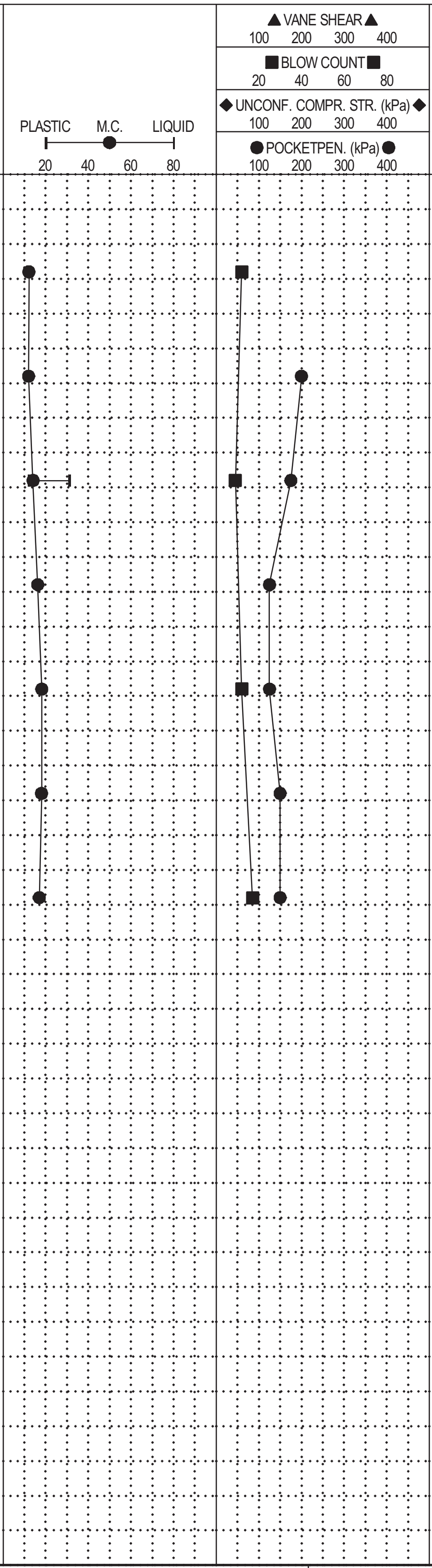
Stewart Weir

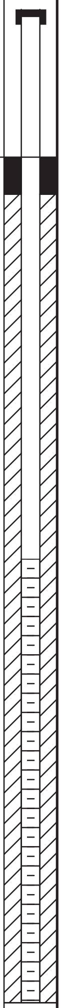
LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-11**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⁚ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▨ DRILL CUTTINGS ⁘ SAND





Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, damp				
		▨	CLAY TILL, sandy, some silt, medium to dark brown, moist, medium to low plasticity, est. 2% above opt., stiff, trace gravel, some salts, coal specks	☒	G87	5-6-6	
1							SULPHATE TEST BH 10-11 G89 @2.25m - NEGLIGIBLE
				☰	G88		
2			@ 2.0 to 5.71m - occasional rust specks & rust inclusions (<1cm diameter)	☒	G89	2-4-5	Grab Sample # 89 Sieve Analysis : gravel 4.1% sand 34.1% silt&clay 61.8%
3 ▼	CI-CL			☰	G90		
				☒	G91	4-6-6	
4							
				☰	G92		
5				☒	G93	6-9-8	
6			End of borehole 5.71m Backfilled with drill cuttings. Standpipe installation depth = 5.71m				NO SLOUGH NOTED at completion Borehole dry at completion Water Level at 3.0m 11 days later
7							
8							
9							
10							

Stewart Weir

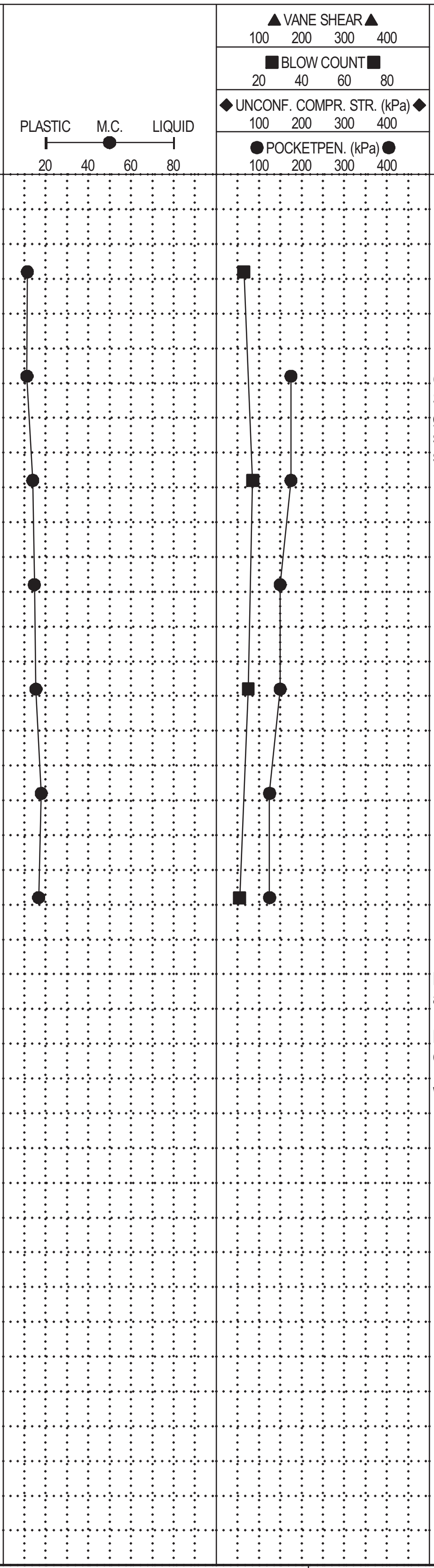
LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

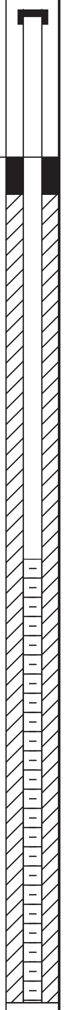
PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-12**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE: ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE: ■ BENTONITE ⁚ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁙ SAND

Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, damp				
		▨	CLAY TILL, sandy, some silt, medium brown, moist, medium to low plasticity, est. opt., stiff, trace gravel, salts, coal specks	☒	G94	3-5-8	
1			@ 1.0m - dark brown, stiff to very stiff				
				☰	G95		Grab Sample # 95 Sieve Analysis : gravel 1.7% sand 40.4% silt&clay 57.9%
2			@ 2.0 to 5.71m - occasional rust specks	☒	G96	9-8-9	
3	CI-CL			☰	G97		
				☒	G98	5-6-9	
4 ▼							
				☰	G99		
5							
				☒	G100	3-5-6	
6			End of borehole 5.71m Backfilled with drill cuttings. Standpipe installation depth = 5.71m				NO SLOUGH NOTED at completion Borehole dry at completion Water Level at 4.10m 11 days later
7							
8							
9							
10							



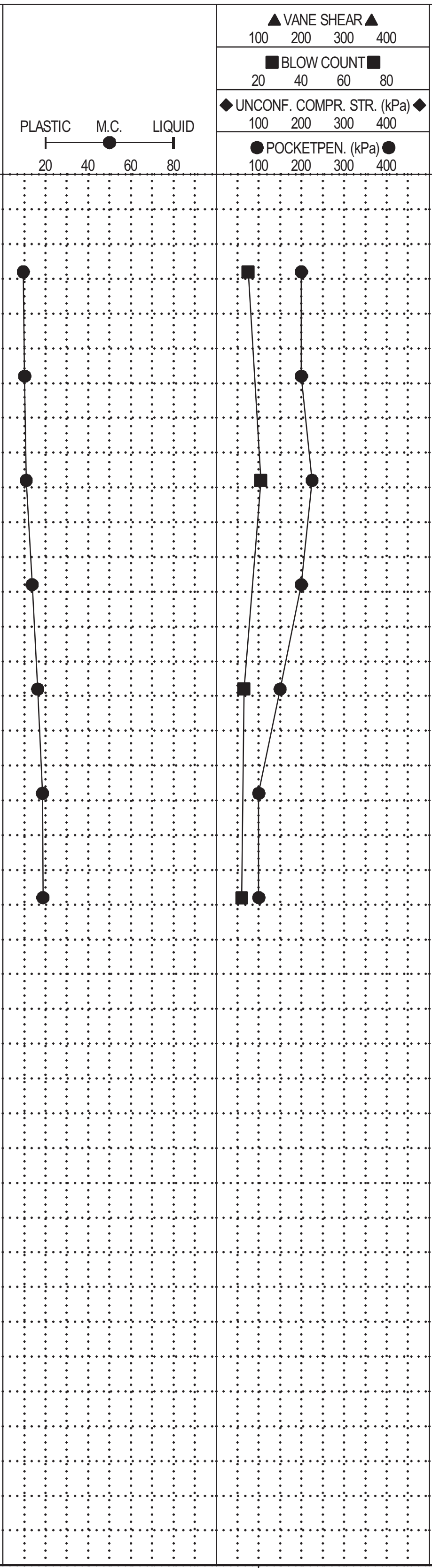


LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-13**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

Depth (m)	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (200mm), black, damp				
		▨	CLAY TILL, sandy, some silt, dark brown, moist, medium to low plasticity, est. opt. to 3% above opt., stiff to very stiff trace gravel, salts, coal specks	☒	G101	8-7-8	
1		▨	@ 0.85m - occasional rust specks				
		▨		☰	G102		Grab Sample # 102 Sieve Analysis : gravel 0.0% sand 37.9% silt&clay 62.1%
2		▨		☒	G103	7-10-11	
3	CI-CL	▨		☰	G104		
		▨		☒	G105	4-7-6	
4		▨	@ 4.0m - stiff to firm				
		▨		☰	G106		
5		▨		☒	G107	7-5-7	
							Borehole dry at completion
6			End of borehole 5.71m Backfilled with drill cuttings.				
7							
8							
9							
10							



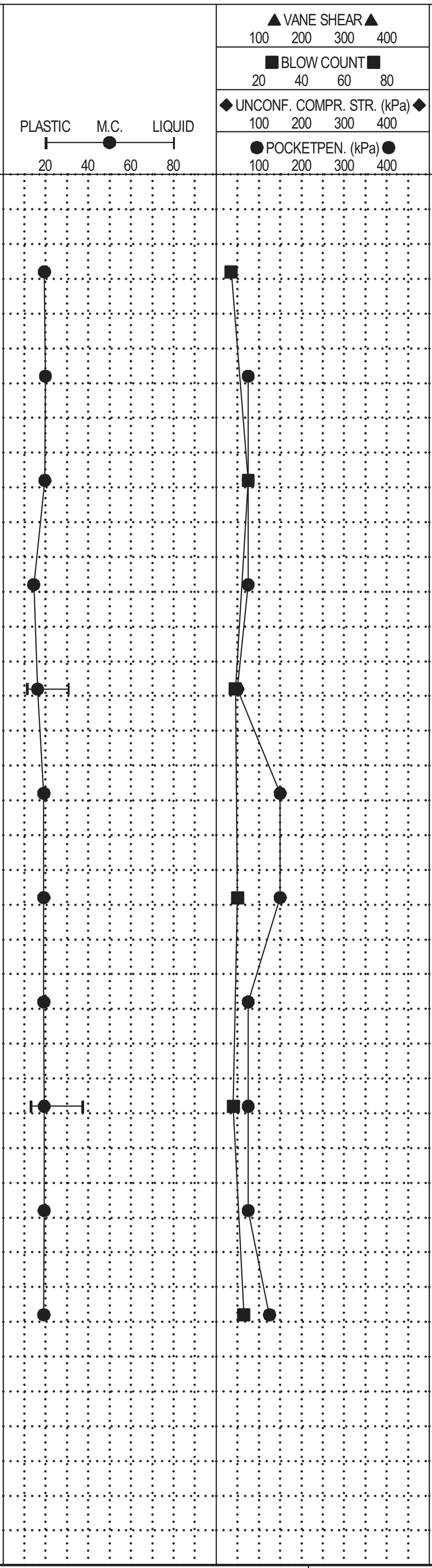
Stewart Weir	LOGGED BY: DJ	COMPLETION DEPTH: 5.71 m
	REVIEWED BY: LD	COMPLETION DATE: 8/4/10
		Page 1 of 1

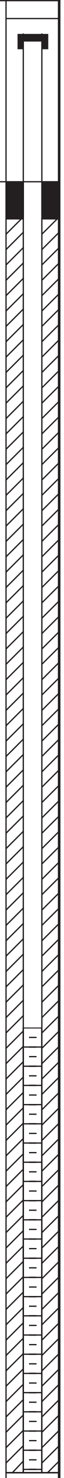
PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-14**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◪ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⬚ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND

Depth (m) Water Level	USC	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS	TOPSOIL (180mm), black, damp				
		CLAY TILL, sandy, some silt, medium to dark brown, moist to wet, medium to low plasticity, est. opt. to 6% above opt., firm, trace gravel	☒	G108	1-3-4	
1						
			☰	G109		Grab Sample # 109 Sieve Analysis : gravel 0.0% sand 53.1% silt&clay 46.9%
		@ 1.7m - occasional salts				
2 ▼						
			☒	G110	5-6-9	
3			☰	G111		SULPHATE TEST BH 10-14 G112 @3.75m - NEGLIGIBLE
		@ 3.2m - sandy, firm to soft				
			☒	G112	3-4-5	Grab Sample # 112 Sieve Analysis : gravel 0.0% sand 36.8% silt&clay 63.2%
4						
	CI-CL	@ sand lense, water seepage (25-35cm), medium to fine grained, medium dense	☰	G113		
5						
		@ 5.1m - medium plasticity	☒	G114	3-5-5	
6			☰	G115		
			☒	G116	3-4-4	Grab Sample # 116 Sieve Analysis : gravel 0.0% sand 31.6% silt&clay 68.4%
7						
			☰	G117		
		@ 7.75m - grey, stiff				
8			☒	G118	7-5-8	Slough at 8.3m at completion
		@ 8.25 to 8.71m - brown				Water Level at 5.6m at completion
9		End of borehole 8.71m Backfilled with drill cuttings. Standpipe installation depth = 8.71m				Water Level at 2.0m 11 days later
10						





Stewart Weir

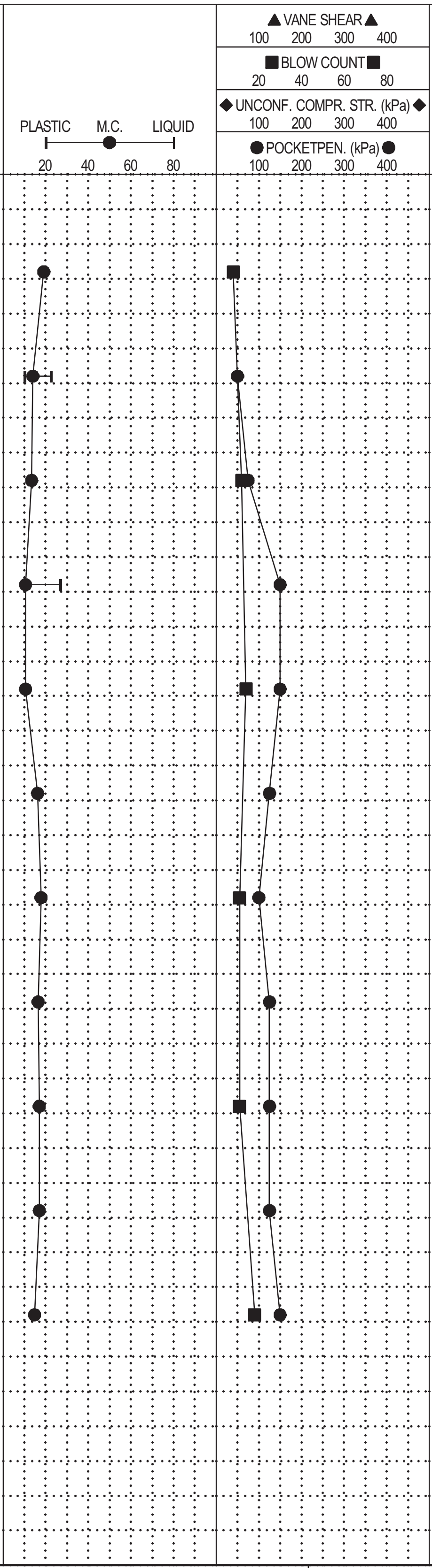
LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

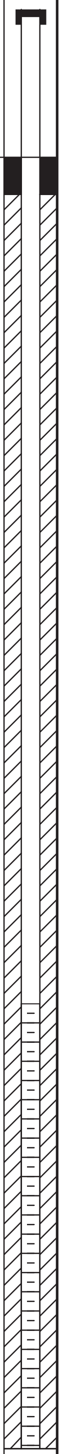
PROJECT: Proposed Urban Residential Subdivision	LOCATION: SE 32-44-06 W4	BOREHOLE NO: **10-15**
CLIENT: Town of Wainwright		NORTHING:
CONSULTANT PROJECT NO: ED60.33962	DRILL/METHOD: SOLID STEM AUGER	EASTING:

SAMPLE TYPE ■ SHELBY TUBE ◩ CORE SAMPLE ☒ SPT SAMPLE ☰ GRAB SAMPLE ▥ NO RECOVERY

BACKFILL TYPE ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ⁘ SAND

Depth (m) Water Level	USC	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	BLOWS /150 mm	OTHER DATA
0	TS		TOPSOIL (180mm), black, damp				
		▨	CLAY TILL, sandy, some silt, medium to dark brown, moist, low plasticity, est. opt. to 4% above opt., stiff, rust stains, trace gravel				
				☒	G119	4-4-4	Grab Sample # 119 Sieve Analysis : gravel 0.0% sand 34.5% silt&clay 65.5%
1							
				☰	G120		
2							Grab Sample # 120 Sieve Analysis : gravel 0.0% sand 48.1% silt&clay 51.9%
▼				☒	G121	5-5-7	
3				☰	G122		Grab Sample # 122 Sieve Analysis : gravel 1.1% sand 43.2% silt&clay 55.7%
				☒	G123	5-6-8	
4							
	CL			☰	G124		
5				☒	G125	4-6-5	Grab Sample # 125 Sieve Analysis : gravel 0.0% sand 30.5% silt&clay 69.5%
6				☰	G126		
				☒	G127	4-5-6	
7							
				☰	G128		
8				☒	G129	5-8-10	
							Slough at 8.6m at completion
9			End of borehole 8.71m Backfilled with drill cuttings. Standpipe installation depth = 8.71m				Water Level dry at completion Water Level at 2.25m 11 days later
10							





Stewart Weir

LOGGED BY: DJ	COMPLETION DEPTH: 8.71 m
REVIEWED BY: LD	COMPLETION DATE: 8/4/10

PROJECT: Proposed Urban Residential Subdivision | CLIENT: Town of Wainwright | PROJECT NO: ED60.33962

Borehole No.	Depth (m)	Atterberg Limits			Field Moisture Content (%)	Liquidity Index	Estimated Optimum Moisture (%)	Estimated Maximum Density (kg/m^3)	Soil Class	Potential Frost Action	Potential Erosion Resistance
		Liquid Limit	Plastic Limit	Plasticity Index							
10-01	0.00								TS		
10-01	0.65				12.0				CL	M-H	G-F
10-01	1.40	23	10	13	13.5	0.28	10	2040	CL	M-H	G-F
10-01	2.15				14.1				CL	M-H	G-F
10-01	2.90				15.8				CL	M-H	G-F
10-01	3.65				17.8				CL	M-H	G-F
10-01	4.40				17.8				CL	M-H	G-F
10-01	5.15				17.0				CL	M-H	G-F
10-01	5.90				18.3				CL	M-H	G-F
10-01	6.65				16.0				CL	M-H	G-F
10-01	7.40				20.4				CL	M-H	G-F
10-01	8.15				22.3				CL	M-H	G-F
10-02	0.00								TS		
10-02	0.65				11.1				CI-CL	M-H	F-G
10-02	1.40				11.5				CI-CL	M-H	F-G
10-02	2.15	30	11	19	11.8	0.03	11	1960	CI-CL	M-H	F-G
10-02	2.90				17.4				CI-CL	M-H	F-G
10-02	3.65				16.7				CI-CL	M-H	F-G
10-02	4.40				16.2				CI-CL	M-H	F-G
10-02	5.15				17.2						
10-03	0.00								TS		
10-03	0.65	46	17	29	19.4	0.08	18	1720	CI	M-H	F-G
10-03	1.41				9.4				CL	M-H	G-F
10-03	2.15	26	10	16	8.9	-0.06	10	2010	CL	M-H	G-F
10-03	2.91				14.5				CI	M-H	F-G
10-03	3.65				13.8				CI	M-H	F-G
10-03	4.40				17.5				CI	M-H	F-G
10-03	5.15				17.7				CI	M-H	F-G
10-03	5.90				15.5				CI	M-H	F-G
10-03	6.65				15.6				CI	M-H	F-G
10-03	7.40				17.5				CI	M-H	F-G
10-03	8.15				18.3				CI	M-H	F-G
10-04	0.00								TS		
10-04	0.65				11.6				CL	M-H	G-F
10-04	1.40				12.3				CL	M-H	G-F
10-04	2.15				13.5				CL	M-H	G-F
10-04	2.90				16.6				CL	M-H	G-F
10-04	3.65				17.3				CL	M-H	G-F
10-04	4.40				16.2				CL	M-H	G-F
10-04	5.15				16.7				SP-SM	S-H	F-P
10-05	0.00								TS		
10-05	0.65				11.7				CL	M-H	G-F
10-05	1.40				11.0				CL	M-H	G-F

Potential Frost Action : None (N), Very Slight (VS), Slight (S), Medium (M), High (H), Very High (VH)

Potential Erosion Resistance : Excellent (E), Good (G), Fair (F), Poor (P)

Sheet 1 of 4

Stewart Weir

SUMMARY OF LABORATORY RESULTS

PROJECT: Proposed Urban Residential Subdivision | CLIENT: Town of Wainwright | PROJECT NO: ED60.33962

Borehole No.	Depth (m)	Atterberg Limits			Field Moisture Content (%)	Liquidity Index	Estimated Optimum Moisture (%)	Estimated Maximum Density (kg/m^3)	Soil Class	Potential Frost Action	Potential Erosion Resistance
		Liquid Limit	Plastic Limit	Plasticity Index							
10-05	2.15				12.7				CL	M-H	G-F
10-05	2.90				12.6				CL	M-H	G-F
10-05	3.65	26	10	16	12.4	0.17	10	2010	CL	M-H	G-F
10-05	4.40				19.5				CL	M-H	G-F
10-05	5.15				16.9				CL	M-H	G-F
10-06	0.00								TS		
10-06	0.65				14.7				CI	M-H	F-G
10-06	1.40				12.4				CI	M-H	F-G
10-06	2.15				12.9				CI	M-H	F-G
10-06	2.90				12.9				CI	M-H	F-G
10-06	3.65				14.1				CI	M-H	F-G
10-06	4.40	35	13	22	18.7	0.26	13	1900	CI	M-H	F-G
10-06	5.15				18.2				CI	M-H	F-G
10-07	0.00								TS		
10-07	0.65				10.8				CI	M-H	F-G
10-07	1.40				10.8				CI	M-H	F-G
10-07	2.15				11.6				CI	M-H	F-G
10-07	2.90				17.2				CI	M-H	F-G
10-07	3.65	36	12	24	17.0	0.20	13	1910	CI	M-H	F-G
10-07	4.40				17.1				CI	M-H	F-G
10-07	5.15				17.8				CI	M-H	F-G
10-07	5.90				17.3				CI	M-H	F-G
10-07	6.65				14.8				SP-SM	S-H	F-P
10-07	7.40				16.6				SP-SM	S-H	F-P
10-07	8.15				15.1				SP-SM	S-H	F-P
10-08	0.00								TS		
10-08	0.65				10.7				CL-CI	M-H	G-F
10-08	1.40				11.6				CL-CI	M-H	G-F
10-08	2.15	28	11	17	11.6	0.03	10	1970	CL-CI	M-H	G-F
10-08	2.90				16.1				CI	M-H	F-G
10-08	3.65				18.0				CI	M-H	F-G
10-08	4.40	35	13	22	19.3	0.29	13	1910	CI	M-H	F-G
10-08	5.15				16.1				CI	M-H	F-G
10-09	0.00								TS		
10-09	0.65				16.0				CL-CI	M-H	G-F
10-09	1.40	29	11	18	16.1	0.29	10	1970	CL-CI	M-H	G-F
10-09	2.15				11.3				CL-CI	M-H	G-F
10-09	2.90				15.7				CL-CI	M-H	G-F
10-09	3.65				14.9				CL-CI	M-H	G-F
10-09	4.40				18.4				CL-CI	M-H	G-F
10-09	5.15				17.4				CL-CI	M-H	G-F
10-10	0.00								TS		
10-10	0.65				10.5				CI-CL	M-H	F-G

Potential Frost Action : None (N), Very Slight (VS), Slight (S), Medium (M), High (H), Very High (VH)
Potential Erosion Resistance : Excellent (E), Good (G), Fair (F), Poor (P)

Stewart Weir

SUMMARY OF LABORATORY RESULTS

PROJECT: Proposed Urban Residential Subdivision CLIENT: Town of Wainwright PROJECT NO: ED60.33962

Borehole No.	Depth (m)	Atterberg Limits			Field Moisture Content (%)	Liquidity Index	Estimated Optimum Moisture (%)	Estimated Maximum Density (kg/m^3)	Soil Class	Potential Frost Action	Potential Erosion Resistance
		Liquid Limit	Plastic Limit	Plasticity Index							
10-10	1.40				10.5				CI-CL	M-H	F-G
10-10	2.15				11.6				CI-CL	M-H	F-G
10-10	2.90				13.8				CI-CL	M-H	F-G
10-10	3.65				16.0				CI-CL	M-H	F-G
10-10	4.40				18.3				CI-CL	M-H	F-G
10-10	5.15				15.6				CI-CL	M-H	F-G
10-10	5.90				17.4				SM	S-H	F-P
10-10	6.65				13.0				SM	S-H	F-P
10-10	7.40				14.8				SM	S-H	F-P
10-10	8.15				16.8				CI-CL	M-H	F-G
10-11	0.00								TS		
10-11	0.65				12.1				CI-CL	M-H	F-G
10-11	1.40				12.0				CI-CL	M-H	F-G
10-11	2.15	31	12	19	14.1	0.10	12	1940	CI-CL	M-H	F-G
10-11	2.90				16.3				CI-CL	M-H	F-G
10-11	3.65				18.2				CI-CL	M-H	F-G
10-11	4.40				18.1				CI-CL	M-H	F-G
10-11	5.15				17.0				CI-CL	M-H	F-G
10-12	0.00								TS		
10-12	0.65				11.4				CI-CL	M-H	F-G
10-12	1.40				11.2				CI-CL	M-H	F-G
10-12	2.15				13.9				CI-CL	M-H	F-G
10-12	2.90				14.7				CI-CL	M-H	F-G
10-12	3.65				15.4				CI-CL	M-H	F-G
10-12	4.40				18.0				CI-CL	M-H	F-G
10-12	5.15				16.7				CI-CL	M-H	F-G
10-13	0.00								TS		
10-13	0.65				9.6				CI-CL	M-H	F-G
10-13	1.40				10.2				CI-CL	M-H	F-G
10-13	2.15				11.0				CI-CL	M-H	F-G
10-13	2.90				13.7				CI-CL	M-H	F-G
10-13	3.65				16.3				CI-CL	M-H	F-G
10-13	4.40				18.6				CI-CL	M-H	F-G
10-13	5.15				18.8				CI-CL	M-H	F-G
10-14	0.00								TS		
10-14	0.65				19.4				CI-CL	M-H	F-G
10-14	1.40				19.9				CI-CL	M-H	F-G
10-14	2.15				19.6				CI-CL	M-H	F-G
10-14	2.90				14.4				CI-CL	M-H	F-G
10-14	3.65	31	11	20	16.2	0.26	11	1960	CI-CL	M-H	F-G
10-14	4.40				19.2				CI	M-H	F-G
10-14	5.15				19.1				CI	M-H	F-G
10-14	5.90				19.1				CI	M-H	F-G

Potential Frost Action : None (N), Very Slight (VS), Slight (S), Medium (M), High (H), Very High (VH)

Potential Erosion Resistance : Excellent (E), Good (G), Fair (F), Poor (P)

SUMMARY OF LABORATORY RESULTS

PROJECT: Proposed Urban Residential Subdivision	CLIENT: Town of Wainwright	PROJECT NO: ED60.33962

Borehole No.	Depth (m)	Atterberg Limits Liquid Limit	Plastic Limit	Plasticity Index	Field Moisture Content (%)	Liquidity Index	Estimated Optimum Moisture (%)	Estimated Maximum Density (kg/m^3)	Soil Class	Potential Frost Action	Potential Erosion Resistance
10-14	6.65	37	13	24	19.3	0.26	13	1890	CI	M-H	F-G
10-14	7.40				19.3				CI	M-H	F-G
10-14	8.15				19.1				CI	M-H	F-G
10-15	0.00								TS		
10-15	0.65				19.1				CL	M-H	G-F
10-15	1.40	23	10	13	14.0	0.31	10	2040	CL	M-H	G-F
10-15	2.15				13.4				CL	M-H	G-F
10-15	2.90	27	10	17	10.6	0.03	10	2000	CL	M-H	G-F
10-15	3.65				10.5				CL	M-H	G-F
10-15	4.40				16.2				CL	M-H	G-F
10-15	5.15				17.9				CL	M-H	G-F
10-15	5.90				16.4				CL	M-H	G-F
10-15	6.65				17.0				CL	M-H	G-F
10-15	7.40				17.0				CL	M-H	G-F
10-15	8.15				14.8				CL	M-H	G-F

Potential Frost Action : None (N), Very Slight (VS), Slight (S), Medium (M), High (H), Very High (VH)
Potential Erosion Resistance : Excellent (E), Good (G), Fair (F), Poor (P)

SUMMARY OF LABORATORY RESULTS

ACRONYMS AND ABBREVIATIONS

ACP	Asphalt Concrete Pavement
AGS	Alberta Geological Survey
AT	Alberta Transportation
ASBC	Asphalt Stabilized Base Course (also called Cold Mix)
CSBC	Cement Stabilized Base Course
EPS	End Product Specification
EUB	Alberta Energy and Utilities Board
FDR	Full Depth Reclamation
GBC	Granular Base Course
GPS	Global Positioning System
HMA	Hot Mix Asphalt
HWY	Highway
MQA	Managed Quality Assurance
OMC	Optimum Moisture Content
PGAC	Performance Grade Asphalt Cement
PPE	Personal Protective Equipment
PFRA	Prairie Farm Rehabilitation Administration
QA	Quality Assurance
QC	Quality Control
RAP	Reclaimed Asphalt Pavement
SP	Safety Plan
SW	Stewart Weir
TAS	Traffic Accommodation Strategy
TerraCem	Specialized Blend of Cementitious Materials (Cement, Flyash, Kilndust) manufactured by Lafarge
USC	Unified Soil Classification
UCS	Unconfined Compressive Strength
UTM	Universal Transverse Mercator
WHMIS	Workplace Hazardous Materials Information System

PP	Pocket Penetrometre
N	Standard Penetration
SPMDD	Standard Proctor Maximum Dry Density