COUNTY OF LETHBRIDGE
IN THE PROVINCE OF ALBERTA

BY-LAW NO. 1308

A BY-LAW OF THE COUNTY OF LETHBRIDGE BEING A BY-LAW PURSUANT TO SECTION 633(1) OF THE MUNICIPAL GOVERNMENT ACT, CHAPTER M.26.1

WHEREAS Bluestone Developments wish to develop a Grouped Country Residential Subdivision on a portion of the North East Section 18, Township 9, Range 22, West of the Fourth Meridian;

AND WHEREAS an application to reclassify the above land for Country Residential has also been submitted to County Council;

AND WHEREAS the Developer has submitted the "Seiller Estates Area Structure Plan" which will provide a framework for subsequent subdivision and development of the area;

NOW THEREFORE BE IT RESOLVED that the Council of the County of Lethbridge does hereby adopt the "Seiller Estates Area Structure Plan" attached as Appendix "A".

GIVEN first reading this 21st day of February, 2008.

Lorne Hickey
Reeve

Dennis Shigematsu
County Manager

GIVEN second reading this 3rd day of April, 2008.

Lorne Hickey
Reeve

Dennis Shigematsu
County Manager

GIVEN third reading this 3rd day of April, 2008.

Lorne Hickey
Reeve

Dennis Shigematsu
County Manager

X:\115Bylaws\001 Bylaws General\Bylaw 1308 ASP Seiller Estates - Bluestone Developments .doc

AREA STRUCTURE PLAN FOR SEILLER ESTATES SUBDIVISION
A PORTION OF NE ¼ 18-9-22-4

Submitted to
County of Lethbridge

Environmental
Agricultural
Structural
Civil
Municipal

PREPARED FOR:
Bluestone Developments
Box 474
Lethbridge, AB T1J-3Z1

PREPARED BY:
Hasegawa Engineering
A Division of 993997 Alberta Ltd.
1220 – 31st Street North
Lethbridge, AB T1H 5J8

HASEGAWA ENGINEERING
Consulting Professional Engineers
A Division of 993997 Alberta Ltd.

Environmental
Agricultural
Structural
Civil
Municipal

1220 31st Street North, Lethbridge, AB T1H 5J8
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February 4, 2008 **Our File #: 07-295**

Bluestone Developments
Box 474
Lethbridge, Alberta
T1J 3Z1

Re: Seiller Estates Subdivision Area Structure Plan

Dear Sir:

Attached please find the Area Structure Plan submitted for the proposed Seiller Estates subdivision located in the County of Lethbridge.

Please review this document and contact our office with any questions or comments. This document was prepared under my supervision.

Yours truly,

Mark Hasegawa, P.Eng.
HASEGAWA ENGINEERING
Consulting Professional Engineers
MAH/dd

Attachment

cc:

PERMIT TO PRACTICE
HASEGAWA ENGINEERING LTD
Signature
Date 2/4/08
PERMIT NUMBER: P 582
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

TABLE OF CONTENTS

1 INTRODUCTION

This document outlines the Area Structure Plan for the proposed Seiller Estates subdivision of NE ¼ 18-9-22-4 located in the county of Lethbridge No. 26. The 27-acre parcel under consideration currently is used as native grassland (refer to Figure 1). A Land Use zoning change application has been filed with the County of Lethbridge No. 26 to meet the by-law requirements for this Area Structure Plan. The proposed subdivision is surrounded by county land that is currently used for agricultural purposes.

The proposed land use is country residential with a 1 acre minimum lot size. This is intended to match the County's land use bylaw requirements.

The client is proposing to subdivide the property into 20 lots each being equal to or greater than 1 acre in area. The enclosed conceptual plan, survey data, engineering analysis, and architectural controls are designed to assure a quality subdivision.

2 PLANS AND DRAWINGS

In order to illustrate the location of the property, site drainage, and the proposed subdivision layout, seven figures have been prepared. The figures are provided in Appendix A and are as follows:

1. Location map
2. Contour map of subject property
3. Conceptual site plan of subdivision
4. Water tie into existing services
5. Existing north-south profiles
6. Existing east-west profiles 1
7. Existing east west profiles 2

These maps are conceptual in nature and to be used for planning purposes only. Upon ASP acceptance design drawing and plans will be prepared and submitted for review.

3 SERVICING

3.1 Sanitary Sewer System

Sanitary sewage will be handled individually on each lot with a private sewage disposal system. The soil characteristics, as detailed in the Geotechnical Evaluation of slope stability Report (Refer to Appendix B; EBA 2008), verifies the suitability of the soil for this type of a disposal system and supplies the base design criterion for the required septic fields. All septic designs must comply with the criteria set forth in Appendix B and County and Alberta Environment Criteria. AENV requirements indicate that the soil within the septic field foot print must be tested in two locations prior to installation.

3.1.1 Septic Systems

Five boreholes were advanced and percolation test performed on site (refer to Figure 2 and Appendix B for locations). The observed soil type was sandy, silty, stiff, brown plastic clay. The percolation results are shown in Table 1.

Table 1: Percolation Test Results

Percolation test location	Results (min/cm)	Safety Codes Council acceptable values (min/cm)
P001	3	2 - 25
P002	3	2 - 25
P003	10	2 - 25
P004	10	2 - 25
P005	15	2 - 25

These results indicate that the surface soils in this area generally satisfy Safety Code design standards (*Alberta Private Sewage System Hand Book).*

EBA also verified the depth to groundwater which cannot be within 1.5 meters below the septic field. The test holes were advanced to a depth of 3 m and no groundwater was observed in any of the holes. This indicates the water table is at levels conforming to the Alberta private sewage standards.

There will be no requirements for a sewer system since all treatment will occur within each lot.

3.2 Water System

The developer will provide a water main for water delivery to the property line of each lot within the development. This water system will be for potable water.

3.2.1 Potable Water

The source of water will be Monarch Shared Water Users Co-op. The access point to the Co-Op line location is shown on Figure 1 and 4. Bluestone Developments has purchased 60 water shares from the Coop (refer to attachment). There is adequate supply in the water system to supply water to the 20 lots proposed in this development.

The developer will provide a water main for water delivery to each lot within the development. Since the water supply line is low pressure and called a "drip system" onsite storage will be required to support fire flows and daily residential usage. As a result, Residents will use cisterns and pumps on their property to store water adequate water supply and provide pressure to their homes. The water to the Coop line is fed from the City of Lethbridge infrastructure and is already treated.

The developer will provide a 150 mm main from the water Coop turn out to the water line servicing the development as shown on Figure 3 and 4. The water main will also feed the lagoon designate to store water to support fire flow. The water distribution system within the development will be designed with 150 mm water mains to accommodate pressure flow if that service ever became available to the development. The water lines will be installed to meet County, City of Lethbridge and AENV standard.

3.2.2 Fire Protection & Landscape Water

The developer will ensure fire protection capability is provided for the property. This water will be provided from a water lagoon located onsite, as shown in Figure 3. The lagoon will be equipped with a County approved and properly designed dry hydrants that pull from the bottom of the lagoon. Separate water lines will be provided to service the two hydrants proposed for the development (refer to Figure 3)

The lagoon has been designed to accommodate 2-hr fire at a flow rate of 2000 lpm. The lagoon will hold a minimum of 350,000 l with a minimum depth of 3.5 m (refer to Figure 3). The lagoon will be properly fenced.

Landscape water will be provided through the Coop water shares. This will also allow the water to be used for irrigation and to water the lots.

3.3 Gas

ATCO will supply natural gas to the development. The existing line is located south of the property (refer to Figure 4) and has sufficient pressure for the subdivision. The developer will bring natural gas to each property line.

3.4 Electrical Power

Fortis will provide services to the proposed subdivision and underground services to each property line. An overhead power service is located south of the property and is shown on Figure 4.

3.5 Telephone

Telus will provide services to the lots, but each individual owner must apply for the service when building. There is an existing service in County road east of the property but it is not adequate in size to service 20 homes. An additional line will be required to allow for adequate service.

4 ROADS

Access to all lots will be from a new road created within the development (refer to Figure 3). The road onsite will meet County of Lethbridge No. 26 design criteria and will have a 20 meter right of way. Minimal area disturbance and natural drainage will be emphasized. The road surface will be paved with sides seeded to grass. The roadway will be adequate in width to accommodate local traffic and meet County requirements. An example of a design cross-section is included in Figure 4.

Each lot will have direct access, with culverts being the responsibility of each property owner. The road will be paved but there will be no curb and gutter but ditches on each side of the road. A cross-section of the proposed road structure is shown in Figure 4. In addition EBA has proposed Street sub grade preparation criteria to be used in road design (refer to Appendix B).

The developer also may propose to add surface pavement to County Range Road 22-5 from the entrance of this development to the intersection with Highway 509. This may not occur until after the development is mostly complete and more information on road design requirements will be reviewed prior to making the final decisions as to when and if this will occur. All design and construction will conform to County Standards and requirements.

5 SITE DRAINAGE AND GRADING

As can be seen in Figure 2, according to area topography information, the drainage on the site generally flows towards the northwest corner. All drainage onsite must conform to County, and Alberta Environmental requirements. Documents referred to when completing this analysis included the Alberta Environment Storm Water Management Guidelines (1999). This document also includes descriptions of Best Management Practices (BMPs) which are used to mitigate peak runoff values over the entire development and to minimize the need for centralized mitigation measures such wet ponds and dry ponds.

5.1 Site Drainage Results

A detailed drainage analysis was performed on this property to compare pre and post development surface runoff. Detailed results of the surface runoff analysis are provided in Appendix C.

This analysis was conducted using the "TR-55 Urban Hydrology for Small Watersheds" which is a model approved by AENV. Based on these results there should not be an increase in peak flow from pre to post development. Although this development is expected to result in approximately 20% impermeable surface, the overall peak runoff flow is mitigated due to increased flow paths, lowered grade on the lots as compared to pre-existing slopes, maintaining ditch grade at 1% or less and storage in the ditch system. Based on this analysis it appears that there is no need to create a retention pond.

As a precautionary measure, runoff from this development will also be chanelled to an existing storm pond located at the bottom of the coulee This pond was utilized for storm water retention during gravel mining activities.

5.2 Grading and Best Management Practices

Since the proposed land use is country residential, impact to the existing land will be kept to a minimum. As a result, grading will be kept to a minimum on this property. All developed areas with impermeable surfaces (or concentrated flows) and from the back edge of the house to the front of the lot, must be designed to flow toward the proposed road right of way. Areas of the back yards that are permeable and do not yield concentrated flows may be allowed to flow to the coulee crest. In addition, driveways designed to access the lots must be designed with a swale or culvert that will not restrict storm water flow in the ditch. Culverts must be properly designed by an engineer and will be constructed of reinforce concrete.

The following BMPs will also be implemented to minimize peak runoff from the property and to keep water quality of runoff within acceptable parameters.

1. Grading within 4 m of a structure must be at least 2% grade away from the structure.
2. All flow from developed areas where concentrated flow occurs (from the back of the house to the front of lot) must be designed to flow to the road right of way at a grade of no greater than 2%.
3. The slope of the road ditch is to be kept below 1%
4. The ditch system and discharge point is to be designed to allow for the storage of 1280 m3 of storm water during a large storm event

6 SOLID WASTE DISPOSAL

As part of the codes and covenants of the subdivision, regular trash disposal will be a requirement.

7 GEOTECHNICAL TESTING AND SLOP STABILITY

Geotechnical testing was conducted by EBA Engineers and Consultants. They also evaluated the soil for slope stability purposes. An overview of their results is presented in this section and the detailed report included in Appendix B.

The allowable development setback line is shown on Figure 3. This safe setback is also used as the property lines for lots adjacent the Coulee. The detailed results of the analysis are shown in Appendix B. The City of Lethbridge has completed extensive studies on slope stability adjacent the Oldman River Coulee. This work is summarized in the River Valley Area Redevelopment Plan (RVARP). EBA used this criteria in completing the analysis for this site.

Also included in the Geotechnical report are design considerations (as related to soil testing) for:

1. Shallow foundation design
2. Slab on grade design
3. Excavation and trenching backfill design
4. Concrete type and surface work
5. Frost protection
6. Seismic design

When preparing the design of the subdivision, these criteria are to be followed and EBA or Hasegawa to be consulted when appropriate.

8 ARCHITECTURAL CONTROLS

The following controls are designed to ensure an aesthetically pleasing environment. The intent is to create the subdivision such that it enhances the natural beauty of its surroundings. The following criteria will apply:

1. Earth tones and/or neutral colors, as determined by the Development Officer, are to be used on all physical structures.
2. Wire fences, chain link excepted, are not permitted.
3. Fences in front yards of residences need to be limited to one metre in height or less.
4. Each residence is to be a minimum of 1500 square feet on the main floor and is to be constructed on site. Mobile homes are not permitted.
5. Each property owner is to be responsible for upkeep of utility right-of-way along property frontage.

APPENDIX A

FIGURES

Notes:

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Contractor to check and verify all dimensions before construction, any errors and omissions reported to Engineer immediately.

Drawing not to be used for construction until so approved.

Do not scale Drawing.

All construction shall be in accordance with the latest code, may it be construction, mechanical, etc. code.

No	Revision	Date	By

Hasegawa Engineering

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Fax: 328-2728
email:hasgm@telusplanet.net

CALGARY OFFICE
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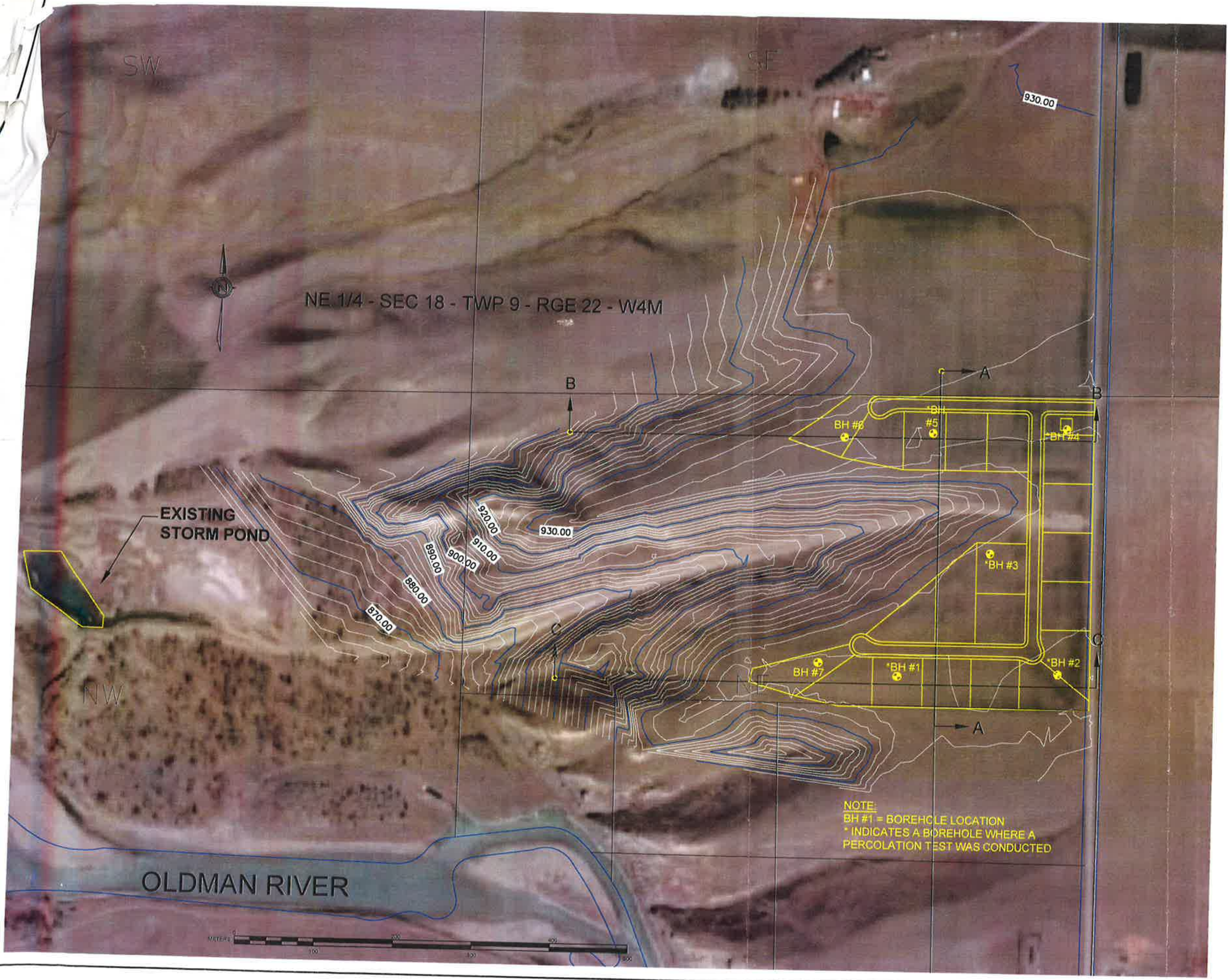
Calgary Alberta T2E 6Z2
Ph: 250-5261

CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
AREA MAP

DESIGN	HE
DRAWN	DPB
CHECKED	HE
APPROVED	HE
DATE DRAWN	FEB4,08
PROJECT NO	07295
SCALE	1:1000
SHEET NO	FIGURE 1



NOTE:
BH #1 = BOREHOLE LOCATION
* INDICATES A BOREHOLE WHERE A PERCOLATION TEST WAS CONDUCTED

G. HASEGAWA

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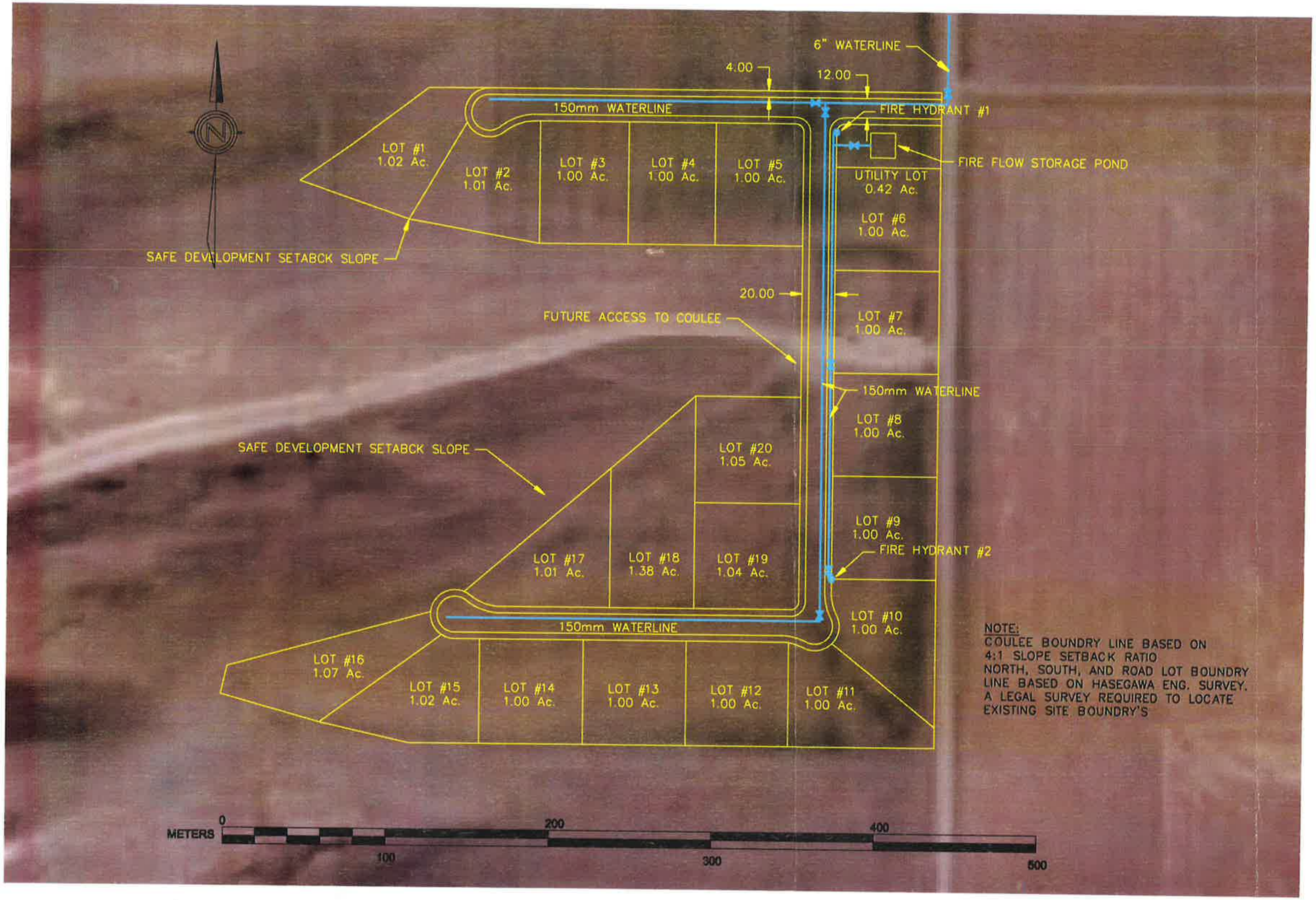
CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
EXISTING SITE CONTOURS

DESIGN HE
DRAWN DPB
CHECKED HE
APPROVED HE
DATE DRAWN FEB4,08

PROJECT NO 07295
SCALE 1:5000
SHEET NO FIGURE 2



NOTE:
COULEE BOUNDRY LINE BASED ON 4:1 SLOPE SETBACK RATIO
NORTH, SOUTH, AND ROAD LOT BOUNDRY LINE BASED ON HASEGAWA ENG. SURVEY.
A LEGAL SURVEY REQUIRED TO LOCATE EXISTING SITE BOUNDRY'S

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1	ROAD WIDENED TO 20M	AUG14,07	DPB

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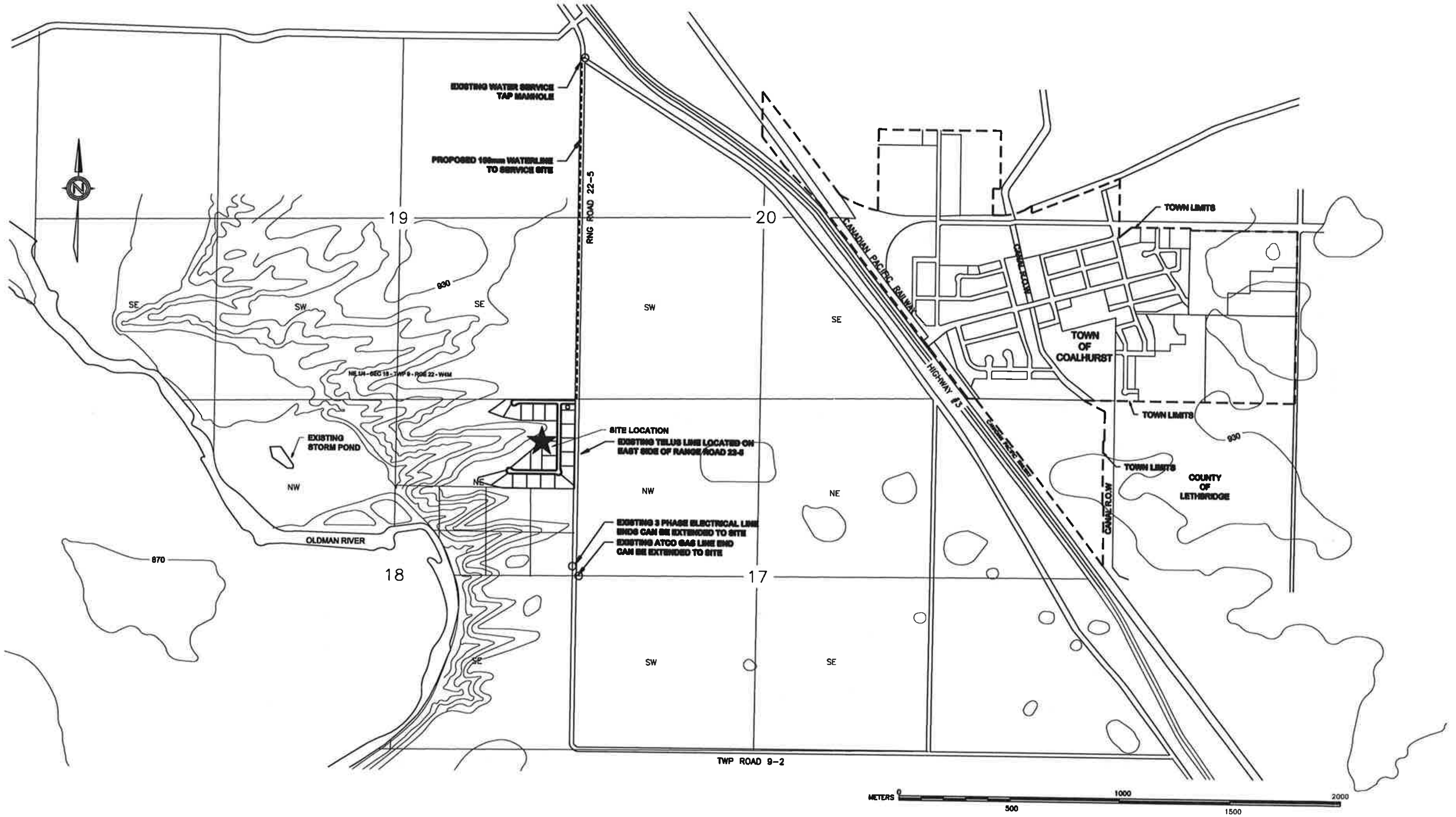
CLIENT
BLUESTONE DEVELOPMENTS

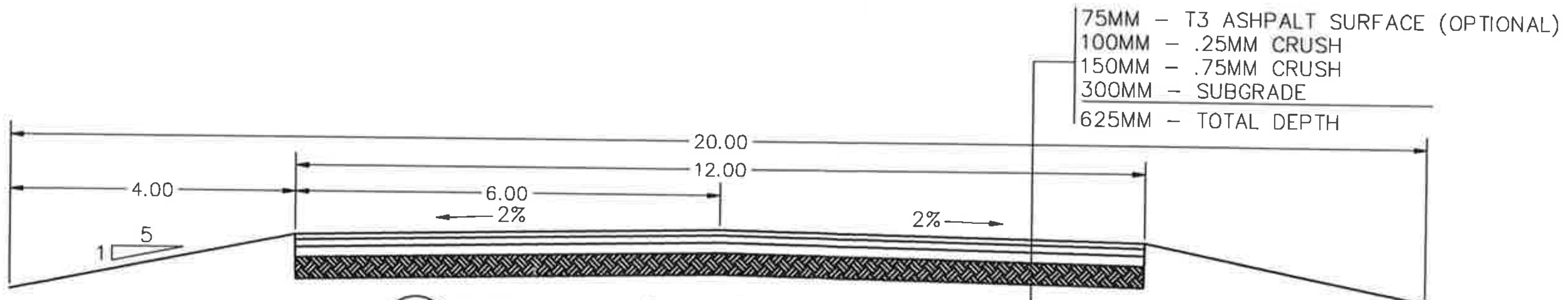
PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
CONCEPTUAL SITE PLAN

DESIGN
HE
DRAWN
DPB
CHECKED
HE
APPROVED
HE
DATE DRAWN
FEB4,08

PROJECT NO.
07295
SCALE
1:2500
SHEET NO.
FIGURE 3





RURAL RESIDENTIAL ROAD SECTION
SCALE: NTS

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Hasegawa Engineering

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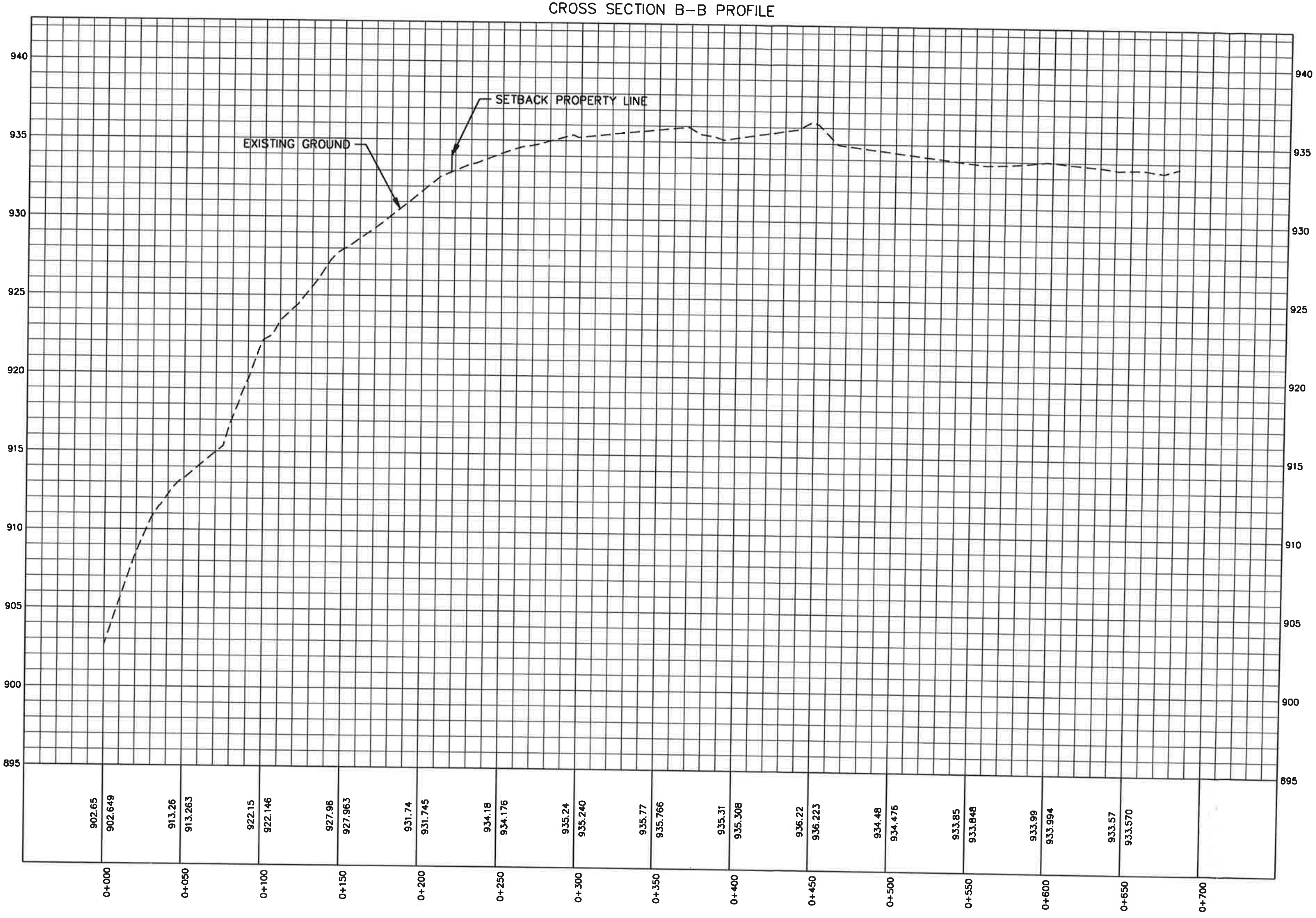
CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
UTILITY ACCESS LOCATIONS & RURAL ROAD DETAIL

DESIGN: HE	PROJECT NO. 07295
DRAWN: DPB	
CHECKED: HE	SCALE: 1:1000
APPROVED: HE	SHEET NO. FIGURE 4
DATE DRAWN: FEB4,08	

CROSS SECTION B-B PROFILE



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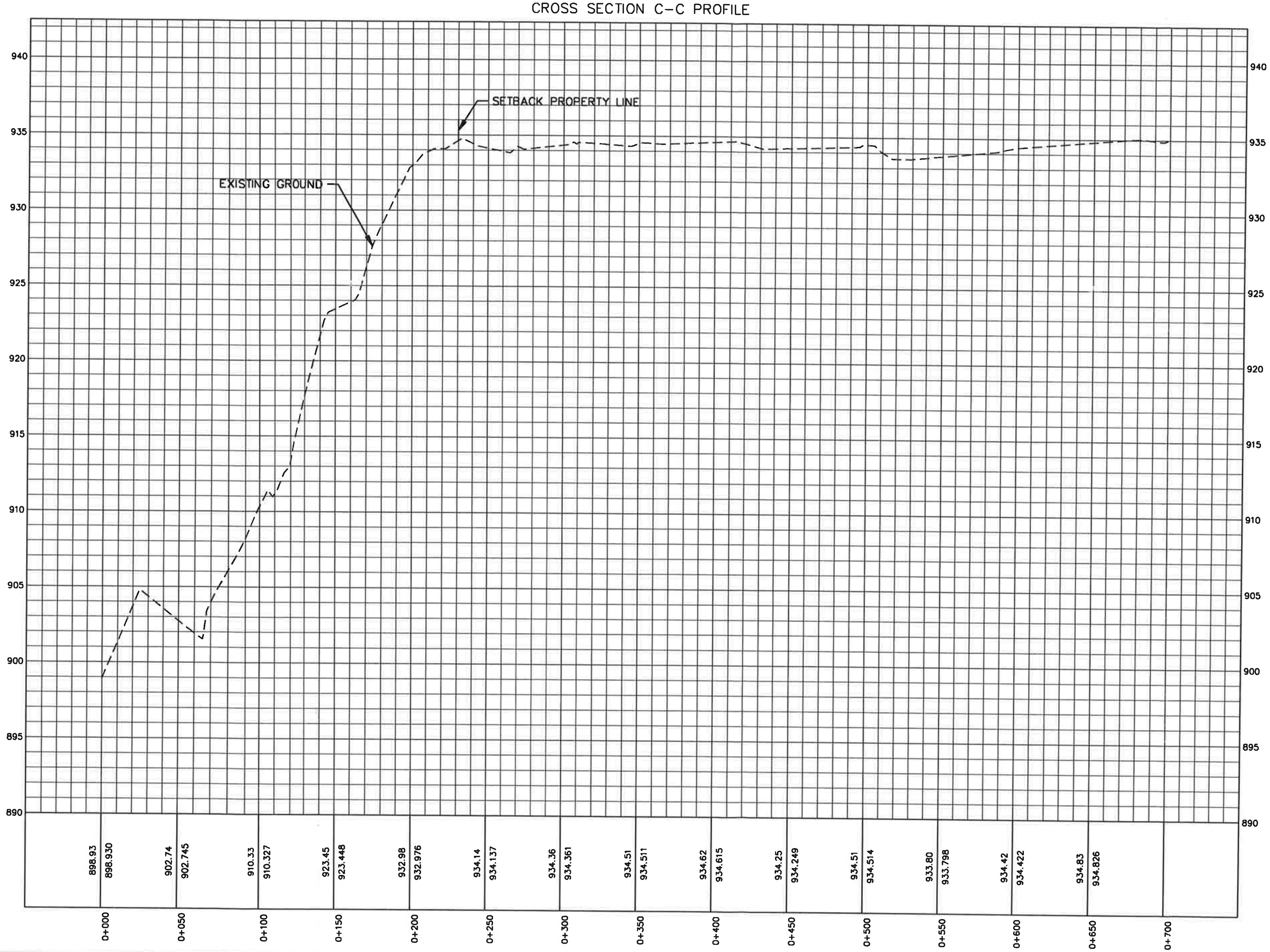
CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
SECTION B–B PROFILE

DESIGN HE	PROJECT NO. 07295
DRAWN DPB	SCALE 1:2500
CHECKED HE	SHEET NO. FIGURE 6
APPROVED HE	
DATE DRAWN FEB4,08	



CROSS SECTION C–C PROFILE

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1	ROAD WIDENED TO 20M	AUG14,07	DP

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CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
SECTION C–C PROFILE

DESIGN HE	PROJECT NO. 07295
DRAWN DPB	
CHECKED HE	SCALE 1:2500
APPROVED HE	SHEET NO.
DATE DRAWN FEB4,08	FIGURE 7

APPENDIX B

GEOTECHNICAL EVALUATION & SLOPE STABILITY REPORT

TOLLESTRUP CONSTRUCTION INC.

GEOTECHNICAL EVALUATION OF SLOPE STABILITY
GROUPED COUNTRY RESIDENTIAL SUBDIVISION
COALHURST, ALBERTA

L12101239

January 2008

EBA Engineering Consultants Ltd.
p. 403.329.9009 • f. 403.328.8817
442 - 10 Street N • Lethbridge, Alberta T1H 2C7 • CANADA

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FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical evaluation conducted by EBA Engineering Consultants Ltd. (EBA) for the proposed Grouped Country Residential Subdivision, to be located near Coalhurst, Alberta.

The scope of work for the geotechnical evaluation was described in a proposal issued to Mr. Randy Rimmer, of Tollestrup Construction Inc. (Tollestrup). The objective of this evaluation was to determine the general subsurface conditions in the area of the proposed development and to provide geotechnical recommendations, specifically with respect to development setback distances from the slopes within and adjacent to the subdivision as well as the suitability of the site soils for septic field disposal of residential wastewater. This report also addresses geotechnical issues with respect to the general subdivision development pertaining to foundations, grading, roadways, utilities, etc.

Authorization to proceed with this evaluation was provided by Mr. Rimmer.

2.0 PROJECT DETAILS AND SCOPE OF WORK

The legal land description of the property is a portion of NE ¼ Section 18-9-22-W4M, located south of Highway 3, within the County of Lethbridge, near Coalhurst, Alberta. The property to be developed is shown on Figure 1 (a recent orthophoto contour plan). Figure 1 also includes a proposed subdivision layout concept and land development limits, provided by Hasegawa Engineering Ltd. (Hasegawa). The land area is bounded to the north and south by undeveloped agricultural farmlands, to the west by the Oldman River Valley and to the east by Range Road 225. Adjacent to the site to the west, north and south are tributary coulee valleys draining towards the Oldman River.

Based on discussions with Tollestrup, it is EBA's understanding that the major component of the proposed development includes single family residential housing across most of the property limits. The foundation system for the residences will likely be shallow spread footings and a grade supported lower level floor slab, typical of other residential developments in the area. It is understood that the final lot layouts and road accesses will incorporate the development setback distances from the top of bank of these slopes as recommended in this report. The report also includes a preliminary assessment of the feasibility of septic disposal fields which are under consideration for the subdivision.

The proposed street developments will be designed and constructed to the current City of Lethbridge Infrastructure Services Engineering Standards, or equivalent, as deemed appropriate, for each area. A detailed pavement design for the respective street sections has not been requested as part of this evaluation and may be completed at a later date, pending review of any traffic studies completed.

Given the proximity of the adjacent slopes, the scope of work for this evaluation included visual site reconnaissance of the adjacent slope's stability, as well as a detailed slope stability analysis, with computer modeling software (Geoslope/Slope/W). Recommendations development setback limits from the Top of Bank[1] of the slopes were developed and are reported in subsequent sections. The slope stability evaluation was conducted in accordance with guidelines adopted by City of Lethbridge By-law # 5277, 'River Valley Area Redevelopment Plan' (RVARP), adopted on July 26, 2004. The requirements of this Bylaw are deemed relevant to this proposed development. As part of the RVARP guidelines, the evaluation considered the recommendations pertaining to safe development setbacks as detailed in the study by AMEC Earth and Environmental Limited (AMEC) entitled, "City of Lethbridge Phase II Development Setback Assessment Oldman River Valley Slopes", issued in November 2002 contained therein.

The agreed work scope for this evaluation also consisted of a review of existing geotechnical data for the property, as well as a review of historical mine workings (EUB) and historical aerial photographs.

The scope of work also included the installation of seven (7) geotechnical boreholes (for the general property development and slope stability evaluation). A laboratory program was completed to assist in classifying the subsurface soils and this report provides the following design and construction recommendations.

- Recommendations for development restrictions in consideration of the adjacent slopes, specifically including recommended development setback distances.
- Recommendations for the feasibility of septic disposal fields
- Recommendations for shallow footing foundations for the proposed residences.
- Recommendations for lot grading, backfill materials and compaction.
- Recommendations for utility line installation, including trench excavation, backfill, and compaction standards.
- Recommendations for subgrade preparation for street pavements.
- Recommended design and construction provisions for control of groundwater.
- Recommendations for concrete type.

---

[1] Top of Bank means the line where the general trend of the slope changes from greater than 15 percent to less than 15 percent, as determined by field survey.

3.0 SITE DESCRIPTION

3.1 SURFACE DESCRIPTION

As shown on Figure 1, the property was noted to be predominately undeveloped at the time of fieldwork. Based on EBA's knowledge of this property's history, including an aerial photograph review from the 1960s to present day, it is understood that the area has been utilized mainly for agricultural purposes.

The property at the time of fieldwork was surfaced in most areas with stubble crop. The ground surface was noted to be generally flat to slightly undulating, with overland drainage towards the Oldman River by way of the tributary coulee valleys. The upper prairie level of the property appears to have a ground surface Geodetic Elevation varying between approximately 932 m and 936 m.

To the west of the site is the east wall of the Oldman River Valley and at the west, northwest and southeast, three tributary coulee slopes drain into the main valley. It is these slopes adjacent to the development area that are the focus of this evaluation. Figure 2 depicts profiles of three typical sections of the slopes (A-A', B-B' and C-C', shown on Figure 1).

For this property, the height of the adjacent slopes to the west (shown on Figure 2) appears to vary between approximately 40 m to 45 m. The base elevation, along the edge of the Oldman River appears to be at approximately Elevation 830 m. For the purposes of this evaluation, the slope sections are described as follows.

Adjacent to the west perimeter of the property (Profiles A-A' and B-B'), the slopes are comprise a coulee draw which extends from the river valley inland towards the east. The road access to the river valley is located within the base of this coulee draw. The upper portion of slope Profile A-A' appears to average approximately 4.5 horizontal to 1 vertical down to an elevation of approximately 918 m, becoming steeper than 3H:1V below this elevation, whereas, slope Profile B-B' appears to generally average approximately 2.5 horizontal to 1 vertical. The river is located several hundred metres to the west of these slope sections. The slope faces are smoothly vegetated with grasses and weeds.

There is an indication of an historical slope instability along a portion of the slope approximately 200 m west of Profile B-B'. The slide mass comprises a back-scarp and slide mass on the face of the slope, typical of a rotational/translational slide, likely founded on an inter-glacial geologic deposition layer references as the Lenzie Silts (discussed in subsequent sections of this report). The upper portion of Profile B-B' appears to indicate relatively wet surface conditions, as a result of apparent overland drainage.

The other slope section type is illustrated by Profile C'C', adjacent to the northwest portion of the proposed development site. This slope section is flatter overall in relation to the other slopes analysed. The lower portion of this section of slope is deeply buried in colluvium (slope wash or slumped soil mass) down to the river level. Therefore, for Profile C-C', the upper portion of the slope is given more relevance for this evaluation. The slope face is covered with colluvium, with a profile in the order of approximately 5H:1V overall within the upper portions of the slopes. Several small surface slumps were noted within the slope face, the course of which is likely attributed to natural groundwater seepage.

3.2 HISTORICAL AERIAL PHOTOGRAPHIC REVIEW

As part of the evaluation, EBA reviewed aerial photographs taken of the project area between 1961 and 1991 (1961, 1975, 1984, 1985 and 1999). Relevant observations from these aerial photographs and recent site reconnaissance are presented in this section.

- 1961 – The subject property is undeveloped at this time. There appears to be multiple locations of drainage overflow over the edges of the slopes. There is evidence of several shallow surface slumps noted along the face of slopes. The slope failure previously noted west of Profile B-B' is present. There is some evidence of road construction within the coulee invert at this time. No other evidence of significant instability is noted.
- 1975 – Similar to that noted above. The shallow slope face failures are clearly evident and appear to be ongoing. The slope failure west of Profile B-B' is more clearly evident.
- 1984 – Similar to the 1970 photo
- 1999 to Present Day – Figure 1 approximately represents current conditions, as limited changes to the property are noted. Except for the single slope instability to the west of Profile B-B', there is no evidence of other significant instability of the slopes.

4.0 GEOTECHNICAL FIELD AND LABORATORY WORK

The initial fieldwork for this evaluation was carried out on October 19, 2007, using a truck mounted drill rig contracted from Chilako Drilling Services Ltd. of Coaldale, Alberta. The rig was equipped with 150 mm diameter solid stem continuous flight augers. EBA's field representatives was Mr. Jackson Meadows, C.E.T. The location of buried utilities was first carried out through Alberta First Call.

Seven (7) boreholes in total were drilled across the property area. These include two relatively deep boreholes (BH006 and BH007) drilled along the slope crest areas to depths of approximately 30.1 m and 30.5 m below ground surface, respectively. The remaining five boreholes were drilled across the general subdivision area to depths of 3.0 m (BH001 through BH005). The borehole locations are depicted on Figure 1.

In all of the boreholes, disturbed grab samples were obtained at 600 mm intervals. A solid soil coring technique was utilized for BH006. All soil samples were visually classified in the field and the individual soil strata and the interfaces between them were noted. The borehole logs are presented in Appendix B. An explanation of the terms and symbols used on the borehole logs is also included in Appendix B.

Slotted 25 mm diameter PVC standpipe was installed in all of the boreholes in order to monitor the groundwater level at each location. Auger cuttings were used to backfill around the standpipes and they were sealed at the ground surface with bentonite chips.

The locations of the boreholes were initially selected based on a proposed subdivision concept provided by Tollestrup. The locations and Geodetic Elevations of the existing ground surface at the borehole locations were subsequently determined by detailed surveying by Hasegawa. The borehole elevations are indicated on the borehole logs.

Classification tests, including natural moisture content, Atterberg Limits, and soluble sulphate content were subsequently performed in the laboratory on samples collected from the boreholes, to aid in the determination of engineering properties. The results of the laboratory tests are presented on the borehole logs in Appendix B.

The drilling program also included five percolation testholes (200 mm diameter) drilled to depths of approximately 900 mm (P001, through P005) in close proximity to the borehole locations, on October 19, 2007. The locations of these boreholes are also shown on Figure 1 and the testhole logs included in Appendix B of this report.

The percolation test conducted at each location included half filling the percolation testhole with water and allowing the testhole to saturate for a period of approximately 24 hours. On October 20, 2007, the percolation holes (P001 through P005) were refilled with water to approximately 0.45 m below existing ground surface and maintained at 0.45 m below existing ground surface for 2 hours. Commencing directly after this, the subsidence of the water was measured versus time by EBA (refilling to the same level every 30 minutes and measuring the drop in water level). The results of the percolation testing are discussed in subsequent sections of this report.

5.0 SUBSURFACE CONDITIONS

5.1 GEOLOGY

EBA has reviewed published reports regarding the geological history of the Lethbridge area. A brief summary, in descending order, of the general stratigraphy is presented below.

- Lacustrine Deposit; a fine-grained lacustrine deposit overlies the Buffalo Lake Till, with thickness varying from non-existent to 8 m.
- Buffalo Lake Till; characterized by a lack of cohesion which often leads to slumping of this deposit. A single period of consolidation has resulted in the development of vertical stress cracks, well oxidized, with some limited bedding.
- Lenzie Silts; unit consists of buff, stratified, calcareous silt and silty sand. The deposit includes black or grey varved clays and poorly sorted till-like colluvium with coarse fragments. This is a glacial lake deposit that formed in a peri-glacial (prior to deposition of Buffalo Till) lake environment as continental ice advanced. Overlying the cross-bedded sediments are lake clays deposited in thin, well-bedded laminae forming true

rythmites. The clay deposit may have developed as a glacier underwent a minor halt after advancing into the area.

- Labuma Till; columnar, massive till, which is hard as a result of consolidation pressure from overlying ice, deposited during Laurentide glaciation.
- Basal Till; massive till, hard, brown to grey.
- Saskatchewan Sands and Gravels; clean, well-sorted and bedded, rounded to subrounded river gravel deposit with a sandy matrix. It is noted that this layer is not expected as this area was not the site of a pre-glacial river valley.
- Oldman Formation Bedrock; relatively massive, sedimentary deposit in both brackish and freshwater environments (non-marine), light grey to light brownish grey in colour, contains cross bedded silty clay shales, siltstones, calcareous sandstones, ironstones, bentonitic clay and coal layers.

5.2 MINING ACTIVITY

Research was conducted to review the existence of mine workings within the boundary of the subject site. The literature search included documents contained within EBA's in-house library, including publications by ERCB (1988) (now EUB) and various other documents contained in EBA's library regarding the coal mining industry in the Lethbridge area. The literature does not indicate that the area was under-mined within the subdivision footprint.

5.3 SOIL STRATIGRAPHY

Specific details of the stratigraphy encountered at each borehole location are presented on the borehole logs. In addition, based on the information gathered during this evaluation and from EBA's geotechnical experience in the area, the general soil stratigraphy from prairie level to below the base of the slopes of significance to this evaluation is summarized in this section.

It should be noted that geological conditions are innately variable. Glacial deposits in particular are seldom spatially uniform. At the time of preparation of this report, information on subsurface stratigraphy is available only at discrete borehole locations. In order to develop recommendations from the information, it is necessary to make some assumptions concerning conditions other than that at borehole locations. Adequate monitoring should be provided during construction to check that these assumptions are reasonable.

Upper Stratigraphic Deposits

In general, the majority of the site was open and surfaced with stubble crop at the time of the fieldwork. The topsoil thickness across the property was generally determined to be approximately 100 mm to 150 mm. Underlying the topsoil layer, brown stained inorganic clay (B Horizon) is commonly encountered in this area, typically for a depth of approximately 150 mm. In addition, wind blown topsoil deposits of greater thickness may exist in areas downwind of topographic high areas. Variable thickness of topsoil should be expected across the site.

It is important to note that based on the proposed stripping methodology (i.e. equipment usage) the thickness of stripping may vary. The method of stripping should therefore be taken into account when determining stripping volumes. In addition, monitoring by geotechnical personnel to ensure approved subgrade materials are stripped to may reduce the risk of over stripping volumes due to excavation methodology.

Underlying the topsoil, a layer of lacustrine clay were encountered at the boreholes, to depths varying between approximately 3.0 m and 4.0 m below the present ground surface. The clay layer was described as silty, with some sand to sandy, damp to moist, low to medium plastic, and very stiff in consistency. The results of Atterberg Limit testing (one test) carried out on a clay soil sample indicated a Plastic Limit of 11 percent and a Liquid Limit of 29 percent, indicative of low plasticity. Moisture contents within the near surface clay were between 6 and 7 percent.

At BH005, BH006, and BH007, inclusions of sand were noted within the lacustrine deposit, with thicknesses of approximately 0.7 m to 1.0 m, extending to depths of approximately 4 m below ground surface. The sand was silty, with trace clay, fine to medium grained, damp, and compact to dense. Moisture contents in the sand were between 4 and 5 percent.

Underlying the lacustrine soil layers, glacial clay till (Buffalo Lake Till) was generally encountered, extending to depths of approximately 27 m and 15.2 m below ground surface at BH006 and BH007, respectively. The clay till layer was described as silty, with some sand to sandy, trace gravel, damp near ground surface, increasing to moist with depth, medium plastic with high plastic clay layers and very stiff in consistency. Coal and oxide particles were noted throughout much of this deposit. Thin wet sand seams and lenses were also noted in some zones in this layer.

The results of Atterberg Limit testing (four tests) carried out on clay till soil samples indicated Plastic Limits varying between 12 to 19 percent and Liquid Limits varying between 40 and 60 percent, indicative of medium to high plasticity. Soil moisture contents within the clay till typically varied between approximately 7 and 20 percent within the upper 10 m of the deposit, with higher moisture contents attributed to higher plastic zones within the clay till.

Underlying the clay till a layer of clay, underlain by a layer of sand was encountered, extending to the borehole termination depths. The clay and sand layers comprise the Lenzie Silts deposit. This geologic deposit is discussed in more detail in the following section.

Lower Stratigraphic Deposits

The previous descriptions of the upper soil layers are presented for consideration of the general subdivision development. Deeper soil information is available from BH006 and BH007 from this evaluation as well as from EBA's background review (including the

AMEC report). The following stratigraphic profile was used in the cross-section models for the slope stability analysis.

- Upper Clay Till (Buffalo Lake Till); silty, some sand to sandy, trace of gravel, medium plastic, brown, moist, coal specs and oxide staining. Surficial layers of medium plastic lacustrine clay (encountered to less than 4 m below existing grade) considered as geologically similar to till materials with respect to the soil parameters used in slope stability analysis (i.e. phi, c). The Atterberg Limit tests, conducted on bulk samples from this layer during this evaluation confirmed a medium to high plastic soil. The clay till was encountered to depths below ground surface of approximately 16 m to 27 m (approximate base Elevation of 909 m to 919 m).
- Lacustrine Deposit (Lenzie Silts); typically described as interbedded layers of sand, clay and silt with medium to high plastic clay lenses generally encountered near the top of this deposit. Specifically, at BH006 in particular, the Lenzie layer comprised an upper zone with high plastic clay inclusions and layers at a depth of approximately 27 m below ground surface. Based on the field data and review of the literature, the upper boundary of the Lenzie Silts layer is estimated to be within the range of approximately Elevation 905 m to 910 m. Based on BH006, the top of the Lenzie Layer is taken as 909 m, for this evaluation. Atterberg Limit tests confirmed a soil of medium plasticity (typical LL=30 to 50% ±, PI 10 to 35% ±), however, some pockets and/or lenses of clay were described as being of high plasticity than that shown by the laboratory tests. The lower portion of Lenzie Silts layer at BH006, and below the upper clay till at BH007, was comprised of sand to depth of approximately 30 m below ground surface (elevation of 905 m). The lower sand portion was described as silty, with trace clay, fine to medium grained, with silt and clay inclusions, damp and compact to dense.
- Lower Clay Till (Labuma Till); the lower till unit was not encountered during this drilling program at depths of approximately 30m below ground surface. However, this layer would be expected within an additional 5 m to 10 m depth, based on information reviewed in geotechnical literature. The lower unit is expected to be described as silty, some sand to sandy, occasional to some gravel, moist to very moist, medium plastic, and stiff to very stiff in consistency.
- Oldman Formation Bedrock; the elevation of the bedrock is estimated to be approximately 870 m to 890 m in this area. The bedrock within the upper zone likely comprise weathered clay shale (claystone), silty, highly weathered, and weak in strength, with sandstone layers/stringers throughout the layer.

It is noted that the elevation of contact with the Lenzie Silts layer, as well as the stratigraphic information below the deepest borehole drilled by EBA for this study compared favourably with the 2002 AMEC study (Section 1.0). The top of the Lenzie Silts layer encountered by EBA is consistent with the elevation ranges indicated in the AMEC report. Furthermore, the AMEC study provided reference to the base of the Labuma Till, the lack of a Saskatchewan Gravels layer, as well as data on the bedrock contact elevation for this area, which generally agrees with EBA's assessment of the local geology noted in this and previous sections of the report.

It is noted that given the depth of the valley (40 m) and the geometry of the coulee valleys, the bedrock strata does not influence the stability of the slopes adjacent to the site (i.e., as the bedrock appears to be deeply buried) with regards to the development limits recommended in this report.

Stratigraphic cross sections of the soil stratigraphy representing the conditions analyzed are presented on Figure 2 for profiles A-A', B-B', and C-C'. The elevations of the geologic layers noted in this section have been included in the slope stability analysis of this evaluation.

A more detailed description of the subsurface stratigraphy encountered on this specific site is provided on the borehole logs included in Appendix B.

5.4 GROUNDWATER CONDITIONS

Seepage and sloughing was generally not encountered during the borehole drilling program. The groundwater level was measured within the standpipes on October 20, 2007. The following table summarizes the groundwater monitoring data.

Borehole Number	Depth of Standpipe (m)	Ground Elevation of Borehole (m)	Groundwater Monitoring Data October 30, 2007	
			Depth to Groundwater (m)	Elevation of Groundwater (m)
001	3.0	934.19	Dry	--
002	3.0	934.45	Dry	--
003	3.0	932.38	Dry	--
004	3.0	934.16	Dry	--
005	3.0	935.36	Dry	--
006	30.1	935.82	Dry	--
007	30.5	934.29	Dry	--

It should be noted that perched groundwater levels will fluctuate seasonally and in response to climatic conditions and may be at different depths when construction commences. Groundwater levels should be monitored periodically prior to development. The intent is to provide an early indication of dewatering requirements during excavation for foundations or utility trenches.

Based on the groundwater data monitored and reported above, significant groundwater problems are not expected for the majority of shallow to moderate excavations expected for this development. Groundwater conditions have also been assumed for the slope stability analysis for this evaluation.

Further comments regarding groundwater issues are provided in subsequent sections.

6.0 SLOPE STABILITY EVALUATION

6.1 GENERAL

EBA's slope stability evaluation for this project comprised an analysis of the present stability of the coulee slopes abutting the west limits of the upland plain of the subject site, an analysis of the impact of development on the stability of the slopes, and an analysis of the impact of any potential slope instability on the development, i.e. setback requirements. These aspects are detailed in the following sections. The minimum Factor of Safety (FS) used to determine the setback requirements was 1.5. This FS is typically used for developments of this nature and is consistent with the requirements of the City of Lethbridge Bylaw # 5277 (RVARP).

The recommendations for stability analyses and appropriate development setback limits, as presented in the AMEC report (referenced in Section 1.0) were also reviewed by EBA and where applicable, incorporated as part of EBA's analyses. The analysis is discussed in the following sections.

6.2 PRESENT SLOPE STABILITY

The present stability of the slopes of this study has been evaluated based on site reconnaissance and analytical techniques using the computer program Slope/W (Morgenstern-Price and Bishop Methods) for circular and block failures.

Visual observations of the slopes in the project area indicate the slopes are currently stable, as evidenced by a lack of recent slope instability (air photo review) (i.e. Factor of Safety of 1.0 or slightly higher).

Soil strength parameters assumed by EBA were based on the results of moisture content and Atterberg Limit tests conducted by EBA on soil samples recovered from the development site and from other sites within the boundary of the City of Lethbridge within similar deposits. The historical data also includes triaxial test data obtained by EBA and others for other sites in the Lethbridge area and documented in the AMEC report. Groundwater conditions (pore pressure parameters), reasonably expected from the data collected in the fieldwork, laboratory program, and from information reviewed from past site studies were then selected by EBA to satisfy the observed conditions.

The soil strength and groundwater parameters selected for the analyses, modelling current conditions, are as follows. It should be noted that these parameters are in general agreement with those assumed in the AMEC stability analyses and have been developed from a collaboration of local geotechnical experience. The order presented is the stratigraphic profile from ground surface to below the base of the slopes being analyzed.

- Material: Upper Lacustrine Soils and Clay Till
  Unit Weight: 18 kN/m^3
  Cohesive Intercept c': 10 kPa
  Friction Angle ϕ': 27°
  Pore Water Pressure Parameter r_u = 0.1
- Materials: Lenzie Silts Deposit (interbedded layers of clay, silt, and sand)
  Top layer: Clay (CI-CH)
    Unit Weight: 18 kN/m^3
    Cohesive Intercept c': 0 kPa
    Friction Angle ϕ_p' (peak): 19°
    Pore Water Pressure Parameter r_u = 0.2
  Subsequent layers of Silt and Sand
    Unit Weight: 21 kN/m^3
    Cohesive Intercept c': 0 kPa
    Friction Angle ϕ': 33°
    Pore Water Pressure Parameter r_u = 0.1

It is noted that it was not necessary to include the lower clay till and the bedrock in the slope stability analysis as they were deemed to be deeply buried below the slopes in this area.

The current stability of the slopes adjacent to the project site has been evaluated by means of limit equilibrium analyses conducted on three cross-sections of the slope (Sections A-A', B-B', and C-C'). It is noted that moderate failures on the top of the Lenzie Silts layer was assumed as the governing slope failure condition. As noted, deep seated failures on the bedrock surface are not considered relevant at this location as the slope geometry makes this failure mechanism unlikely. The slope profiles for the cross-sections were taken from topographic elevation data provided to EBA by Hasegawa. Figure 1 depicts the location of the cross-sections and the slope elevation contours and stratigraphic cross sections are shown on Figure 2.

Slope stability analyses on the cross-sections, using the above parameters, indicate that the existing slopes are "meta-stable". Factors of Safety for shallow slope face failures are slightly higher than 1.0 for the slope areas. With respect to moderate depth instability affecting the slope crests (within the Lenzie Silts layer), the factor of safety varies between 1.0 and 1.2. From this analysis, it is confirmed that a theoretical slope failure on the Lenzie Silts layer appears to be the governing slope failure mechanism for the slopes of this study.

6.3 IMPACT OF DEVELOPMENT ON SLOPE STABILITY

The relatively steep river valley slopes in the Lethbridge area rely upon low degrees of soil saturation for stability. Any increase in the level of soil saturation reduces the stability of the slopes.

Development of the site will bring about changes in the factors which contribute to the present stability of the slopes. Evaporation of soil moisture will be reduced by the presence of ground cover such as buildings and roadway structures. Septic field disposal of wastewater, irrigation, and possible leakage of water from underground utilities in addition to the water retention within stormwater management facilities will increase the amount of water infiltrating the site subsoils. This combination of reduced evaporation of subsoil moisture and increased infiltration of water to the subsoils is considered to be the most significant influence of development on the factors that contribute to the present stability of the slopes. Increasing soil moisture content produces a reduction in the total cohesion as the apparent cohesion is reduced or lost and an increase in the pore pressure ratio reduces the effective stress. The result is a corresponding decrease in the factor of safety.

For post-development analyses, the pore pressure parameter r_u was revised to suit anticipated increases in soil moisture and a reduction in the cohesion in the upper till, for moderately deep failure surfaces within the Lenzie Silts layer. The revisions to the parameters developed in Section 6.2 were:

- Cohesion in the upper till layer was revised to 0 kPa for post-development analysis.
- The r_u value in the upper till was revised to 0.20.
- The r_u value in the upper clay layer of the Lenzie Silts was revised to 0.3.

6.4 CITY BYLAW SETBACK REQUIREMENTS

Recommendations presented in the AMEC report (incorporated into the updated City RVARP guidelines) which affect this development are related to slope instabilities sliding along the Lenzie Silts layer. Based on this study, it was determined that in the worst case, slopes above the Lenzie Silts layer will tend to retrogress back to an ultimate slope of approximately 4H:1V. Therefore, minimum development setback lines should incorporate a line drawn at a slope of 4H:1V from the point where this deposit exits the face of the slope back into the property. The report concludes that the point where this 4H:1V line intercepts the ground surface at upper prairie level should be taken as the development setback line with respect to the Lenzie Silts deposit.

This 4H:1V line has been shown for reference purpose on the sections considered and presented on Figure 2 (in section) and on Figure 1 (where the line intercepts the prairie level). It is noted that the determination of the 4H:1V line assumes the top of Lenzie Silts deposit elevation of 909 m, determined from this evaluation. In addition, a reasonable assumption of the thickness of colluvium covering the slope face was required in some cases, as shown on Figure 2.

With the assumptions noted in this report, a development setback line, generally in accordance with those noted in the City Bylaw documentation was determined. Section 7.0 presents the results of this analysis.

6.5 IMPACT OF SLOPE INSTABILITY ON THE DEVELOPMENT (SETBACK LINES)

The long-term stability of the slopes adjacent to the project site has also been evaluated by means of limit equilibrium analysis conducted on the cross-sections of the slopes noted.

The approach used in the stability analysis was to first establish the existing Factor of Safety against slope instability using the strength parameters indicated in Section 6.2. Successive points set back from the crest of the slopes were then selected and minimum factors of safety were calculated modelling current relatively dry slope conditions. This was followed by additional analysis to determine Factors of Safety when post-development groundwater levels and partially saturated slope conditions, respectively, were assumed (refer Section 6.3).

As noted above, the Development Setback Line presented on Figure 1 was established by using the City of Lethbridge RVARP guidelines in Bylaw # 5277 and based on the data and assumptions of this geotechnical evaluation. Based on the stability analysis conducted by EBA, it is confirmed that the development setback line established provides a minimum Factor of Safety of 1.5 against slope failure for the assumed, worst case, post development groundwater condition. The limits of the proposed development setback line established by EBA are described in Section 7.0.

7.0 RECOMMENDED DEVELOPMENT GUIDELINES

Analysis of the present stability of the slopes indicates a factor of safety against slope instability affecting the property at the 'Top of Bank' of between 1.5 (deep seated – Lenzie Silts), 1.2 (moderate seated) and 1.0 (shallow seated). This models the current condition of slopes.

For post-development conditions, the recommended 'Development Setback Line'[2] is as shown on Figure 1. Figure 2 presents the cross-section models for Sections A-A', B-B' and C-C'. Generally, the development setback distance has been determined by projecting a 4H:1V line back from where the Lenzie Silts layer is exposed at the slope face (RVARP Guidelines).

In summary, the recommended development setback lines are presented in Figure 1, and are based on the various analysis techniques described in the preceding sections. The setback distances have also been transitioned along the perimeter of the slope, based on three dimensional effects. It is recommended that the development setback lines be established by field survey given the setback distances determined by the topographic model

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[2] Development Setback Line: established by survey which subsequently is registered on a plan of subdivision which determines the extent of development in relation to the Top of Bank.

derived for this site by EBA. EBA should then be contacted to review and confirm the location of the development setback line prior to any development of the proposed land.

Precautionary measures which should be included in the design of the proposed development (with respect to slope stability issues) are outlined as follows:

- Any fill excavated during development should not be disposed of within the development restriction zone unless directed otherwise after a review by the project geotechnical engineer. The development restriction zone is the area of land between the development setback line and the top of bank.
- Positive grading should be provided to ensure surface drainage from the development is directed as either sheet flow over the crest of the slopes or away from the slopes into the stormwater management facility.
- All utilities and plumbing should be carefully installed and inspected to ensure they are in good working order.
- Septic field should be kept a minimum of 10 m from Top of Bank and not have any discharge towards or on the slopes in any concentrated manner.
- Normal, prudent design and construction procedures should be followed during development.
- The development recommendations of this geotechnical report should be closely adhered to.

The upper coulee slopes should be treated as a restricted development zone. This involves:

- No excavation on the valley slope without review by a geotechnical engineer.
- No clearing of vegetation.
- No fill to be placed on the crest of the slopes.
- Maintain vegetation cover along the crest and on the slope.

Notwithstanding the setback distances recommended, some sloughing and slope movements may occur. The development may result in a general increase in the degree of saturation of the site subsoils which may cause minor sloughing of the top portion of the slope. The setback distance is not intended to prevent failure of the slope but rather to prevent such failures from directly affecting developed areas of the site.

8.0 SUBDIVISION DEVELOPMENT

8.1 GENERAL

Specific recommendations that apply to this project are provided in the following subsections for shallow footings, basement construction and floor slabs, general site development and lot grading, groundwater issues, trench excavation and backfill, and concrete type (including commentary on concrete surfacing). Pavement structures for this development should be designed and constructed to the City of Lethbridge Infrastructure Services Engineering Standards and as such, are not presented in this report. However, recommendations for subgrade preparation within the proposed asphalt concrete surfaced roadways are discussed.

Development in close proximity to slopes has been discussed elsewhere in this report and the development restrictions must be considered.

A groundwater study has not been requested as part of this evaluation. It is EBA's understanding that weeping tiles for the residences will most likely include tie-ins to the storm sewer utility or to a surface discharge. Any discharge must not be directed towards the crest of the slopes.

The initial topsoil stripping depth is of particular importance. For such a development, following removal of the surficial organic topsoil, the majority of any underlying B Horizon layer (organic stained, but essentially inorganic clay) can likely remain in place during site stripping and incorporated into the fill mass during general site grading. Full-time monitoring by experienced personnel is recommended in order to avoid over-stripping and to ensure appropriate material mixing and placement.

Subgrade preparation is required in all subdivision development areas, including lot grading, as well as all paved areas to City of Lethbridge Standards as noted in this report. This includes stripping of topsoil and deleterious fill materials, scarification and moisture conditioning and compaction. The native medium plastic clay soils should be acceptable for site grading purposes in all areas. The clay surface appears to be variable with respect to its optimum moisture content and as such, moisture conditioning (wetting, mixing, and drying as necessary) will be required to reduce the swelling potential of this soil and to achieve the compaction standards recommended. Proof-rolling within roadways to detect soft areas is also recommended. The contractor should expect soil moisture variability around the property.

Shallow footings are considered feasible for residential developments in the subdivision, most likely in conjunction with full or partial basements. Further recommendations are provided in Section 8.7.

Slabs-on-grade for this project must consider the precautions recommended. For slabs-on-grade, including the subgrade preparation measures intended to improve slab performance.

All foundation design recommendations presented in this report are based on the assumption that an adequate level of monitoring will be provided during construction and that all construction will be carried out by suitably qualified contractors, experienced in foundation and earthworks construction. An adequate level of monitoring is considered to be:

- for shallow foundations and slabs; inspection of bearing surfaces prior to placement of concrete or mudslab and design review during construction;

- for earthworks; full-time monitoring and compaction testing.

All such monitoring should be carried out by suitably qualified persons, independent of the contractor. One of the purposes of providing an adequate level of monitoring is to check that recommendations, based on data obtained at discrete borehole locations, are relevant to other areas of the site.

8.2 SEPTIC DISPOSAL FIELDS

8.2.1 Percolation Test results

The following table provides the results of the field program and percolation test results.

Percolation Test	Subsurface Stratigraphy (0.3 m to 0.9 m)	Percolation Test Result (min/cm)
P001	Clay, silty, some sand to sandy, damp, medium plastic, very stiff, brown	3
P002	Clay, silty, some sand to sandy, damp, medium plastic, very stiff, brown	3
P003	Clay, silty, some sand to sandy, damp, medium plastic, very stiff, brown	10
P004	Clay, silty, some sand to sandy, damp, medium plastic, very stiff, brown	3
P005	Clay, silty, some sand to sandy, damp, medium plastic, very stiff, brown	15

8.2.2 Septic Disposal Field Design

The Safety Codes Council's, Alberta Private Sewage Systems Standard of Practice 1999, states that a subsurface effluent disposal system that uses the absorption of effluent into the soil for treatment and disposal, should absorb the effluent into the soil at a rate of:

- not faster than 5 minutes per 2.5 cm (2 minutes / cm); and
- not slower than 60 minutes per 2.5 cm (24 minutes / cm),

as determined by a percolation test. In addition, the natural separation between the point of effluent infiltration into the soil and the groundwater should be a minimum of 1.5 m.

The percolation test results ranged between 3 and 15 minutes/cm. These results indicate that the surface soils for design and construction of septic disposal fields generally satisfy the requirements of the Safety Code Council's guidelines.

Groundwater was not encountered within the standpipes installed during the geotechnical evaluation above depths of 3 m. Therefore, it is considered that the phreatic surface is generally a minimum 1.5 m below the disposal field elevations, which satisfies the Safety Codes Council guidelines.

Based on the results of this assessment, the use of septic disposal fields for the country residential developments is generally considered feasible. However, it is noted that the specific site selection of the proposed fields needs careful consideration by the septic field installer to satisfy the requirements of the Regulator Having Jurisdiction (Municipality, AENV, Alberta Labour). This requirement is in accordance with the provincial regulations, which state that two percolation tests are required within the final footprint of the field by the installer with tests results satisfying the recommended percolation limits. Following the site-specific testing, the septic disposal field should be designed and sized accordingly by the disposal field designer or alternate disposal system considered where the native soils are not considered suitable. It is further recommended that the design footprint of the residences be determined once the final disposal field is selected, to ensure the appropriate gravity flow or pumping requirements are satisfied.

During installation of the weeping trenches, the installer should pay close attention to the soil conditions encountered, to define the extent of any silt or sand pockets (areas subject to faster percolation rates) or medium plastic clay till (areas of slower percolation rates). These should be immediately reported to the disposal field designer for review prior to completion of the septic disposal field.

The information provided herein is intended to be a preliminary assessment of the feasibility of septic disposal fields for the proposed residential lot developments as per the provincial regulations. Site specific municipal regulations or septic field siting requirement guidelines with respect to the local health unit, if applicable, have not been addressed.

8.3 LOT GRADING

In general terms, lot grading should be designed and carried out to the current City of Lethbridge Infrastructure Services Engineering Standards. The particulars for this development are discussed as follows.

All lots should be initially graded for drainage at a minimum gradient of 2.0 percent. The existing surficial site soils comprising medium plastic clay and clay till, are suitable for use as 'landscape fill' materials or for use as 'general engineered fill' materials for lot grading, as defined in Appendix C. The moisture content of the site soil materials at surface generally appears to be both above and below the anticipated optimum moisture content for these soils in most areas. It is anticipated therefore, that moisture conditioning consisting of both wetting and drying will be required at the site for proper compaction. Although soil moisture variability should be expected, the earthwork contractor should, however, make his own estimate of the requirements and should consider such factors as weather and construction procedures.

Final grading for lots backing on to the crest of the slopes must ensure no concentrated flows of surface water is directed over the slopes without detailed engineering review for erosion measures.

General engineered fill materials for lot grading should be moisture conditioned to within a range of –1 percent of optimum to +2 percent of the optimum moisture content prior to compaction and compacted to a minimum of 98 percent of SPD.

Further recommendations regarding backfill materials and compaction are contained in Appendix C.

8.4 STREET SUBGRADE PREPARATION

Within all asphalt concrete surfaced paved areas, the upper 300 mm of native clay soils or prepared general engineered fill subgrade should be scarified and uniformly moisture conditioned to between minus 1 percent of optimum and 2 percent over optimum moisture content. The subgrade should then be uniformly compacted to a minimum of 98 percent of SPD.

Based on EBA's local experience, the contractor should be made aware that subgrade difficulties often arise at moisture contents of 3 percent over optimum, as noted in the current City of Lethbridge Standards, where siltier soils are encountered. Therefore, in practice, the moisture content within proposed paved areas should be limited to no more than 2 percent over optimum for acceptable subgrade support conditions.

Backfill to raise these areas to subgrade level should be general engineered cohesive fill materials, as defined in the report text or Appendix C, moisture conditioned and compacted as noted previously. The subgrade should be prepared and graded to allow drainage into catchbasins or crowned to drain to the road shoulders. Proof-rolling of the prepared surface is recommended to identify localized soft areas and for an indication of overall subgrade support characteristics.

It is imperative that positive surface drainage be provided to prevent ponding of water within the roadway structure and subsequent softening and loss of strength of the subgrade materials. Surrounding landscaping should be such that runoff water is prevented from ponding beside paved areas in order to avoid softening and premature failure of the pavement surface.

The soil moisture regime should be considered in achieving the above recommended standards for construction of the subgrades. If localized areas of soft subgrade soils are encountered, provisions may be required to subcut each area and replace with cohesive engineered fill, or alternatively, with granular (pit-run) fill with the use of a geotextile grid or geotextile fabric to strengthen the subgrade support characteristics. Further design information can be provided following initial proof-rolling of the subgrade soils. It should be noted that the use of red shale to stabilize soft areas is no longer recommended in the Lethbridge area.

8.5 EXCAVATIONS AND TRENCH BACKFILL

Excavations should be carried out in accordance with the Alberta Occupational Health and Safety (OH&S) Regulations.

For this project, the depths of excavations are anticipated to be shallow to moderate for such components as service trenches, and tie-ins (< 3.0 m). Excavations which are to be deeper than 1.5 m should have the sides shored and braced or the slopes should be cut back not steeper than 1.0 horizontal to 1.7 vertical for periods up to one month. Where excavations are open for longer than one month, the slopes should be cut back so they are not steeper than 1.0 horizontal to 1.0 vertical.

The maximum allowable sideslopes for utility trenches may not be governed by OH&S regulations, but by construction methodology for ensuring appropriate transition lengths from backfill soils to native soils. As an example, an appropriate transition of 1H:1V is normally recommended to avoid abrupt changes in subgrade stiffness and subsequent consolidation/cracking of the pavement structure. However, areas of multiple trenches, varying trench depth, and position of trenches (parallel or perpendicular to roadway alignments) need to be considered. EBA would be pleased to provide further specific recommendations, once final roadway/utility configurations are known.

It is considered unlikely that significant groundwater seepage will occur where construction is less than 2.5 m below the existing ground surface. Therefore, dewatering of most excavations should not be necessary. For the main utility trenches (deeper than 2.5 m), any seepage encountered, should be directed towards a sump for removal from the excavation.

Temporary surcharge loads, such as spill piles, should not be allowed within a distance from an unsupported excavation face equal to the depth of excavation. Mobile equipment should be kept back at least 2.0 m. All excavations should be checked regularly for signs of sloughing, especially after rainfall periods. Small earth falls from the sideslopes are a potential danger to workmen and must be guarded against.

The moisture content of the clay soils encountered across the site is generally variable with respect to the estimated Standard Proctor optimum moisture content for the materials. It is expected that such soils would be satisfactory as trench backfill material, however, may require moisture conditioning prior to reworking. It is anticipated therefore, that moisture conditioning consisting of both wetting and drying or mixing will be required for proper compaction. The earthwork contractor should, however, make his own estimate of the requirements and should consider such factors as weather and construction procedures.

Trenches must be backfilled in such a way as to minimize the potential differential settlement and/or frost heave movements. A minimum density of 98 percent of SPD is recommended for all trench backfill, at a moisture content of between -1 percent and +2 percent of optimum. The compacted thickness of each lift of backfill shall not exceed 150 mm. The upper 1.5 m of service trenches should be cut back at a maximum slope of 1.0 horizontal to 1.0 vertical to avoid an abrupt transition between backfill and in situ soil.

It should be noted that the ultimate performance of the trench backfill is directly related to the uniformity of the backfill compaction. In order to achieve this uniformity, the lift thickness and compaction criteria must be strictly enforced.

For frost protection, pipes buried with less than 2.0 m of soil cover (above top of pipe) should be protected with insulation to avoid frost effects that might cause damage to or breakage of the pipes. Rigid insulation placed under areas subject to vehicular wheel loadings should be provided with a minimum thickness of 600 mm of compacted granular base.

General recommendations regarding construction excavation, backfill materials and compaction are contained in Appendix C.

8.6 CONCRETE ISSUES

8.6.1 Concrete Type

The water soluble sulphate content of two representative soil samples recovered from the site (determined in a laboratory) were in excess of 0.2 percent. For this development, based on EBA's experience and CSA A23.1-04, the recommended concrete exposure classification for general usage should be Class S-2 (CSA A23.1-04, Table 3). For this exposure classification, alternatives include the usage of Type HS (Sulphate Resistant) Portland cement, or blends of cement and supplementary cementing materials, conforming to Type MSb and/or Type HSb cements (CSA A3001-03).

For all concrete exposed to soil and/or groundwater (i.e., including all building foundation concrete, all below grade concrete, and surface works concrete), a maximum water/cementing materials (W/CM) ratio of 0.45 is recommended. Based on EBA's experience with Alberta aggregates, a W/CM ratio of 0.45 normally corresponds to a 28-day compressive strength of 28 MPa or greater (32 MPa at 56-days).

Air entrainment of 4 to 6 percent by volume is recommended for all concrete exposed to freezing temperatures, native soils and/or groundwater. This should be increased to 5 to 7 percent for exterior flatwork.

8.6.2 Concrete Surface Works

With respect to surface works concrete (i.e., specifically concrete curbs and sidewalks), the recommendations provided in this report for subgrade preparation, including moisture conditioning and compaction, are intended to provide relative uniformity in the subgrade. The intention of uniformity, with respect to material type and moisture content, is to reduce the risk of differential concrete movements due to soil volume changes as a result of fluctuating moisture content. For these types of developments, a gradual increase in moisture content is common, resulting from precipitation, reduced evaporation, and irrigation. However, some differential movement and subsequent cracking of concrete surface works should be anticipated, typical for the Lethbridge area.

With respect to providing a layer of granular material beneath surface works concrete, there are both positive and negative consequences. In the positive sense, it must be assumed that the subgrade will be uniformly graded properly such that any moisture gaining access beneath the concrete within the granular layer would be drained away quickly to an area designed to accommodate excess moisture (i.e., roadway weeping tile tied into the storm system). If well drained, the provision of granular material also serves to reduce some differential distortions, when washed materials are used, and has been documented as helping to reduce longitudinal cracking.

On the negative side, if free drainage of the granular layer is not designed, constructed, and maintained, granular materials provide easy access for excess moisture to pond below the concrete, causing swelling of the medium to high plastic subgrade soils and/or consolidation of fill soils. There is also a risk of softening of the adjacent roadway pavement edges.

The risk of differential movement of the subgrade soils and the economic consequence for either option should be given due consideration by the municipal engineer.

8.7 SHALLOW FOUNDATIONS

Shallow foundations, if considered, should be constructed approximately 1.4 m below the final design exterior ground surface (frost protection requirement). At this depth the foundation subgrade soil generally consists of very stiff, damp to moist, medium to high plastic, clay or clay till.

The net allowable static bearing pressure for the design of strip and spread footings for residential construction at this depth may be taken as 75 kPa, on native, undisturbed clay soils, subject to other recommendations in this report. The allowable static bearing pressure is based on correlation between Standard Penetration Test 'N' values. The factor of safety used from ultimate bearing capacity was 3.0. Footing dimensions should be in accordance with the minimum requirements of the Alberta Building Code 1997 (Section 9.15.3 Footings). Bearing certification is recommended to ensure that the footings are placed on competent soils, satisfying the design bearing requirements of applicable codes and municipal bylaws.

It is recommended to use a smooth edge-trimming bucket or Grade-All for final excavation to the foundation subgrade elevation to minimize disturbance of the founding soils. The foundation concrete should be placed immediately following excavation to ensure the bearing clay soil does not dry out to below the plastic limit.

The anticipated foundation clay soils are expected to be prone to volume changes (both heave and consolidation) with varying moisture content. Therefore, a permanent weeping tile system is also recommended around the outside perimeter of the structure at the foundation elevation to maintain a consistent moisture profile of the founding soils. This will reduce the potential of differential movement (heave or consolidation) of the foundations. Weeping tile drainage is discussed in Subsection 8.10.

Settlement of footings designed and constructed in accordance with the above recommendations should be well within the normally tolerated values of 25 mm total and 20 mm differential.

Recommendations for minimum depth of cover for footings are presented under the heading 'Frost Protection' below. Further recommendations regarding shallow foundations are given in Appendix C.

8.8 FLOOR SLABS-ON-GRADE

Construction of floor slabs-on-grade for this project must consider the surficial lacustrine soils noted within the development area as well as the general engineered fill layers placed during site grading. Construction may be considered feasible, provided the following precautions and construction recommendations are followed.

In native soils areas, following removal of topsoil, the subgrade should be scarified to a minimum depth of 300 mm, and moisture conditioned to a range of optimum to 2 percent over optimum moisture content. Within areas of engineered fill, the exposed subgrade should be scarified for a minimum depth of 600 mm, considering the engineered clay fill soils (not containing deleterious materials) and moisture conditioned as noted above. The minimum compaction in each case should be 98 percent of Standard Proctor maximum dry density (SPD). The prepared subgrade should be proof-rolled and any soft or loose pockets detected should be reconditioned as recommended above or over-excavated and replaced with general engineered fill.

As required, all general engineered fill needed to bring the development area to design subgrade elevation should be uniformly moisture conditioned between within 2 percent of optimum moisture content. Recommendations for general engineered fill are provided in Appendix C. The minimum compaction should be 98 percent of SPD. The site soils are generally considered acceptable for use as general engineered fill, provided they are acceptably moisture conditioned.

A levelling course of clean well graded crushed gravel, at least 150 mm in compacted thickness, is recommended directly beneath the slabs-on-grade, unless a thicker course is required for structural purposes. The subgrade beneath slabs-on-grade should be protected at all times from moisture or exposure which may cause softening or disturbance of the subgrade soils. This applies during and after the construction period (and before and after replacement of the required general engineered fill). Should the exposed surface become saturated or disturbed, it should be reworked to achieve the above standards.

If the subgrade is properly prepared as noted above, floor slab movements should be limited to less than approximately 25 mm. Slabs-on-grade should be separated from bearing members to allow some differential movement. If this range of differential movement is unacceptable, the owner should consider a structurally supported floor.

Recommended procedures for proof-rolling and backfill materials and further recommendations for slabs-on-grade construction are included in Appendix C.

8.9 BASEMENT CONSTRUCTION

8.9.1 Basement Floor Slabs

Slab-on-grade construction for basements is considered feasible providing certain precautions are undertaken. All excavation should be carried out remotely using a smooth-mouth bucket or Grade-All at final grade in order to minimize disturbance of the base. Basement floor slabs should be supported by a minimum of 150 mm compacted, clean, free-draining granular material.

In areas where floor slabs bear on a clay subgrade, the clay at this site may swell following completion of the floor slabs. Therefore, some movement should be anticipated. Any light columns in the basement designed to support the main floor should be of the adjustable "telepost" type. If partitions are constructed in the basement, provision must be made so that, if the basement floor slab heaves, the partitions do not raise the main floor. A minimum allowance of 25 mm should be left between the top plates of basement partitions and the floor above them to accommodate heaving of the floor slab. This heaving allowance is less applicable for interior columns founded on spread footings.

The slab subgrade should be sloped to provide positive drainage to the edge of the slab (where the native soils are cohesive). A minimum drainage gradient of 0.5 percent is recommended.

Slabs-on-grade should be separated from bearing members to allow some differential movement. If differential movement is unacceptable, a structurally supported floor system or crawlspace may be considered.

General recommendations regarding floor slab construction are presented in Appendix C.

8.9.2 Basement Walls

All basement walls should be designed to resist lateral earth pressures in an "at-rest" condition. This condition assumes a triangular pressure distribution and may be calculated using the following:

$$P_o = K_o (\gamma H+q)$$

where:

P_o = lateral earth pressure "at-rest" condition (no wall movement occurs at a given depth)

K_o = co-efficient of earth pressure "at-rest" condition (use 0.5 for silt or clay backfill and 0.45 for sand and gravel backfill)

γ = bulk unit weight of backfill soil (use 19 or 21 kN/m³ for clay or granular backfill, respectively)

H = depth below final grade (m)

q = surcharge pressure at ground level (kPa)

It is assumed that drainage is provided for all basement walls through the installation of weeping tile and hydrostatic pressures will not be a factor in design.

Backfill around concrete basement walls should not commence before the concrete has reached a minimum two-thirds of its 28-day strength and first floor framing are in place or the walls are laterally braced. Only hand operated compaction equipment should be employed within 600 mm of the concrete walls. Caution should be used when compacting backfill to avoid high lateral loads caused by excessive compactive effort. A compaction standard of 95 percent of Standard Proctor maximum dry density (SPD) is recommended. To avoid differential wall pressures, the backfill should be brought up evenly around the walls. A minimum 600 mm thick engineered clay cap should be placed at the ground surface to minimize the infiltration of surface water.

8.10 FOUNDATION PERIMETER DRAINAGE REQUIREMENTS

It is understood that all residential weeping tiles will be tied into the storm sewer system and/or will have arrangements for surface discharge. An acceptable weeping tile system should consist of a perforated weeping tile, surrounded with a minimum of 150 mm thick blanket of washed rock (maximum size 20 mm), with the drain rock surrounded by non-woven geotextile. The weeping tile should have a minimum 0.5 percent slope leading to a sump to then discharge as noted above.

8.11 FROST PROTECTION

For protection against frost action, perimeter footings in heated structures should be extended to such depths as to provide a minimum soil cover of 1.4 m. Isolated or exterior footings in unheated structures should have a minimum soil cover of 2.1 m unless provided with equivalent insulation.

All piles in unheated areas should have full depth steel reinforcement and should be drilled to a minimum depth of 6 m. Grade beams spanning concrete piles should have a minimum 100 mm void space on the underside of the grade beam and around the pile caps to reduce the risk of interaction with the underlying soil, associated with frost heaving and/or swelling soils.

8.12 SEISMIC DESIGN

The Site Classification recommended for Seismic Site Response is Classification D, as noted in Table 4.1.8.4.a of the National Building Code of Canada (NBCC) 2005.

9.0 DESIGN AND CONSTRUCTION GUIDELINES

Recommended general design and construction guidelines are provided in Appendix C, under the following headings.

- Shallow Foundations
- Floor Slabs-on-Grade
- Construction Excavations
- Backfill Materials and Compaction
- Proof-Rolling

These guidelines are intended to present standards of good practice. Although supplemental to the main text of this report, they should be interpreted as part of the report. Design recommendations presented herein are based on the premise that these guidelines will be followed. The design and construction guidelines are not intended to represent detailed specifications for the works although they may prove useful in the preparation of such specifications. In the event of any discrepancy between the main text of this report and Appendix C, the main text should govern.

10.0 REVIEW OF DESIGN AND CONSTRUCTION

EBA should be given the opportunity to review details of the design and specifications, related to geotechnical aspects of this project, prior to construction.

Bearing surfaces, foundation installation, and deep excavations should be monitored by qualified geotechnical personnel during construction. EBA will provide these services, if requested.

11.0 LIMITATIONS

Recommendations presented herein are based on a geotechnical evaluation of the findings in seven geotechnical boreholes, five percolation testholes, historical air photo review, site reconnaissance, slope stability evaluation and a review of existing geotechnical data in EBA's records, including previous reports. The conditions encountered during the fieldwork are considered to be reasonably representative of the site. If, however, conditions other than those reported are noted during subsequent phases of the project, EBA should be notified and given the opportunity to review our current recommendations in light of new findings. Recommendations presented herein may not be valid if an adequate level of monitoring is not provided during construction.

This report has been prepared for the exclusive use of Tollestrup Construction Inc., and their agents, for specific application to the development described in Section 2.0 of this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No warranty is either expressed or implied.

For further limitations, reference should be made to the General Conditions in Appendix A of this report.

12.0 CLOSURE

We trust this report satisfies your present requirements. We would be pleased to provide further information that may be needed during design and to advise on the geotechnical aspects of specifications for inclusion in contract documents. Should you require additional information or monitoring services, please do not hesitate to contact our office.

Respectfully submitted,
EBA Engineering Consultants Ltd.

Prepared by:

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2008/02/01

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Feb 01, 2008

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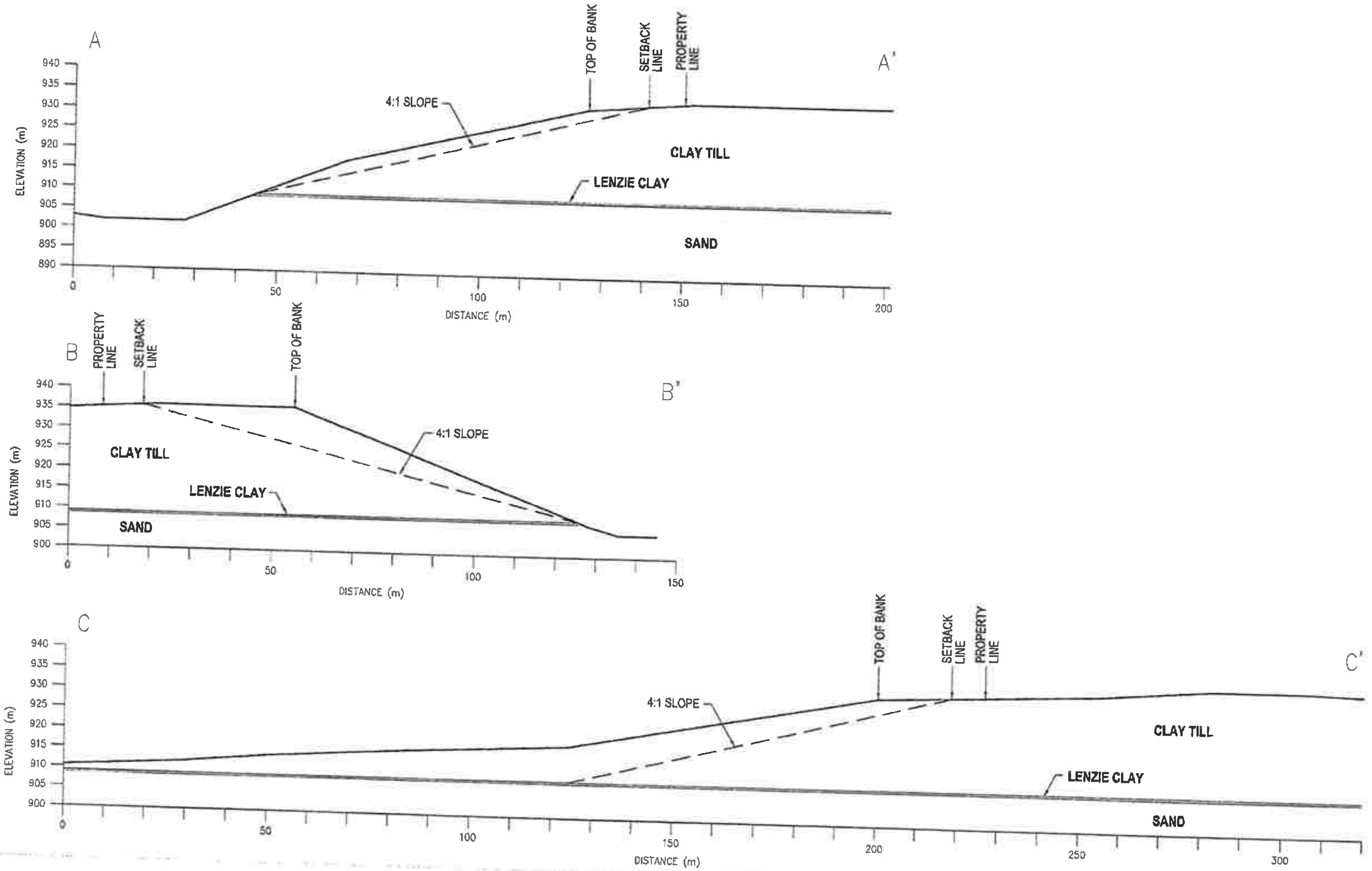
Senior Project Director

/cld

PERMIT TO PRACTICE
EBA ENGINEERING CONSULTANTS LTD.
Signature
Date Feb 01, 2008
PERMIT NUMBER: P245
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

FIGURES



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CLIENT

Tollestrup Construction
Incorporated

EBA Engineering
Consultants Ltd.

GROUPED COUNTRY RESIDENTIAL SUBDIVISION
NE 1/4 18-9-22 W4M, SOUTH OF COALHURST, AB

CROSS SECTIONS

PROJECT NO.	DWN	CKD	REV
L12101239	LCH	JAR	0
OFFICE	DATE		
EBA-Lethbridge	January 31, 2008		

Figure 2

APPENDIX

APPENDIX A GEOTECHNICAL REPORT – GENERAL CONDITIONS

GEOTECHNICAL REPORT – GENERAL CONDITIONS

This report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of EBA's client. EBA does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than EBA's client unless otherwise authorized in writing by EBA. Any unauthorized use of the report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of EBA. Additional copies of the report, if required, may be obtained upon request.

2.0 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

3.0 LOGS OF TESTHOLES

The testhole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

4.0 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

5.0 SURFACE WATER AND GROUNDWATER CONDITIONS

Surface and groundwater conditions mentioned in this report are those observed at the times recorded in the report. These conditions vary with geological detail between observation sites; annual, seasonal and special meteorologic conditions; and with development activity. Interpretation of water conditions from observations and records is judgmental and constitutes an evaluation of circumstances as influenced by geology, meteorology and development activity. Deviations from these observations may occur during the course of development activities.

6.0 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

7.0 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

8.0 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

9.0 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well as the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

10.0 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

11.0 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

12.0 SAMPLES

EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the client's expense upon written request, otherwise samples will be discarded.

13.0 STANDARD OF CARE

Services performed by EBA for this report have been conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practising under similar conditions in the jurisdiction in which the services are provided. Engineering judgement has been applied in developing the conclusions and/or recommendations provided in this report. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of this report.

14.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

15.0 ALTERNATE REPORT FORMAT

Where EBA submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed EBA's instruments of professional service), the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding. The hard copy versions submitted by EBA shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancies, the hard copy versions shall govern over the electronic versions. Furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed version archived by EBA shall be deemed to be the overall original for the Project.

The Client agrees that both electronic file and hard copy versions of EBA's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except EBA. The Client warrants that EBA's instruments of professional service will be used only and exactly as submitted by EBA.

The Client recognizes and agrees that electronic files submitted by EBA have been prepared and submitted using specific software and hardware systems. EBA makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

APPENDIX

APPENDIX B BOREHOLE LOGS

TERMS USED ON BOREHOLE LOGS

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (major portion retained on 0.075mm sieve): includes (1) clean gravels and sands, and (2) silty or clayey gravels and sands. Condition is rated according to relative density, as inferred from laboratory or in situ tests.

DESCRIPTIVE TERM	RELATIVE DENSITY	N (blows per 0.3m)
Very Loose	0 to 20%	0 to 4
Loose	20 to 40%	4 to 10
Compact	40 to 75%	10 to 30
Dense	75 to 90%	30 to 50
Very Dense	90 to 100%	greater than 50

The number of blows, N, on a 51mm O.D. split spoon sampler of a 63.5kg weight falling 0.76m, required to drive the sampler a distance of 0.3m from 0.15m to 0.45m.

FINE GRAINED SOILS (major portion passing 0.075mm sieve): includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as estimated from laboratory or in situ tests.

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH (kPa)
Very Soft	Less Than 25
Soft	25 to 50
Firm	50 to 100
Stiff	100 to 200
Very Stiff	200 to 400
Hard	Greater Than 400

NOTE: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above, because of planes of weakness or cracks in the soil.

GENERAL DESCRIPTIVE TERMS

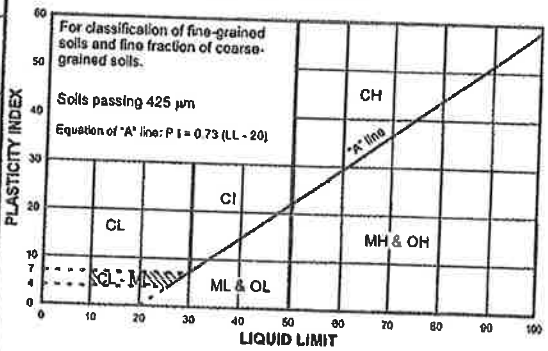
Slickensided	- having inclined planes of weakness that are slick and glossy in appearance.
Fissured	- containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
Laminated	- composed of thin layers of varying colour and texture.
Interbedded	- composed of alternate layers of different soil types.
Calcareous	- containing appreciable quantities of calcium carbonate.
Well Graded	- having wide range in grain sizes and substantial amounts of intermediate particle sizes.
Poorly graded	- predominantly of one grain size, or having a range of sizes with some intermediate size missing.

eba

MODIFIED UNIFIED SOIL CLASSIFICATION †

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	CLASSIFICATION CRITERIA		
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve*	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines Less than 5% Pass No. 200 sieve More than 12% Pass No. 200 sieve 5% to 12% Pass No. 200 sieve GW, GP, SW, SP GM, GC, SM, SC Bordering Classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4 $C_u = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures		Atterburg limits plot below "A" line or plasticity index less than 4	Atterburg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterburg limits plot above "A" line or plasticity index greater than 7	
	SANDS More than 50% of coarse fraction passes No. 4 sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands,little or no fines		$C_u = D_{60}/D_{10}$ Greater than 6 $C_u = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures		Atterburg limits plot below "A" line or plasticity index less than 4	Atterburg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterburg limits plot above "A" line or plasticity index greater than 7	
FINE-GRAINED SOILS 50% or more passes No. 200 sieve*	SILTS AND CLAYS Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	SILTS AND CLAYS Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
HIGHLY ORGANIC SOILS			PT	Peat, muck and other highly organic soils			

PLASTICITY CHART



*Based on the material passing the 3 in. (75 mm) sieve
†ASTM Designation D 2487, for identification procedure see D2488

SOIL COMPONENTS

FRACTION	SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
	PASSING	RETAINED	PERCENTAGE	DESCRIPTOR
GRAVEL				
coarse	75 mm	19 mm	>35 %	"and"
fine	19 mm	4.75 mm	21 to 35 %	"y-adjective"
SAND				
coarse	4.75 mm	2.00 mm	10 to 20 %	"some"
medium	2.00 mm	425 µm	>0 to 10 %	"trace"
fine	425 µm	75 µm		
SILT (non plastic) or CLAY (plastic)	75 µm		as above but by behavior	

OVERSIZE MATERIAL

Rounded or subrounded

COBBLES	75 mm to 200 mm
BOULDERS	> 200 mm

Not rounded

ROCK FRAGMENTS	>75 mm
ROCKS	> 0.76 cubic metre in volume

2046 - Revised July 07.cdr

PROJECT: GROUPED COUNTRY SUBDIVISION

LOCATION: NE 1/4 SEC18- 9-22-W4M

CITY: COALHURST, ALBERTA

CLIENT: TOLLESTRUP CONSTRUCTION INC.

DRILL METHOD: 150mm SOLID STEM AUGER

PROJECT ENGINEER: JIM RYAN

BOREHOLE NO: BH 001

PROJECT NO: L12101239

ELEVATION: 934.19m

SAMPLE TYPE ▨ DISTURBED ⧄ NO RECOVERY ☒ SPT ▤ A-CASING ▥ SHELBY TUBE ■ CORE

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT	PLASTIC M.C. LIQUID 20 40 60 80	◆ UNCONFINED (kPa) ◆ 50 100 150 200 ▲ POCKET PEN. (kPa) ▲ 100 200 300 400	Elevation (m)
0	TOPSOIL - clay, silty, sandy, moist, dark brown, roots & root hairs, organics					
	CLAY - silty, some sand to sandy, damp, very stiff, medium plastic, light brown, white precipitate					
1						934.0
	... damp to moist					
2						933.0
3	... damp					932.0
	End of Borehole @ 3.0m					
	No Seepage or Sloughing Slotted PVC Pipe Installed to 3.0m Borehole Measured Dry Oct. 30, 2007					
4						931.0
5						930.0
6						929.0
7						928.0
8						927.0
9						926.0
10						925.0
11						924.0
12						923.0
13						922.0
14						921.0
15						920.0
16						919.0

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LOGGED BY: JKM

REVIEWED BY: JAR

DRAWING NO: B6

COMPLETION DEPTH: 3m

COMPLETE: 10/19/2007

Page 1 of 1

PROJECT: GROUPED COUNTRY SUBDIVISION	CLIENT: TOLLESTRUP CONSTRUCTION INC.	BOREHOLE NO: BH 002
LOCATION: NE 1/4 SEC18- 9-22-W4M	DRILL METHOD: 150mm SOLID STEM AUGER	PROJECT NO: L12101239
CITY: COALHURST, ALBERTA	PROJECT ENGINEER: JIM RYAN	ELEVATION: 934.45m

SAMPLE TYPE ▨ DISTURBED ◿ NO RECOVERY ☒ SPT ☰ A-CASING ▥ SHELBY TUBE ▮ CORE

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT	PLASTIC M.C. LIQUID 20 40 60 80	◆UNCONFINED (kPa)◆ 50 100 150 200 ▲POCKET PEN. (kPa)▲ 100 200 300 400	Elevation (m)
0	TOPSOIL - clay, silty, sandy, moist, dark brown, roots & root hairs, organics					
	CLAY - silty, some sand to sandy, damp, very stiff, medium plastic, light brown, white preciptitates					934.0
1						
						933.0
2	... moist, brown					
						932.0
3	End of Borehole @ 3.0m					
	No Seepage or Sloughing Slotted PVC Pipe Installed to 3.0m Borehole Measured Dry Oct. 30, 2007					931.0
4						
						930.0
5						
						929.0
6						
7						928.0
8						927.0
9						926.0
10						925.0
11						924.0
12						923.0
13						922.0
14						921.0
15						920.0
16						919.0

EBA Engineering Consultants Ltd.	LOGGED BY: JKM	COMPLETION DEPTH: 3m
	REVIEWED BY: JAR	COMPLETE: 10/19/2007
	DRAWING NO: B7	Page 1 of 1

PROJECT: GROUPED COUNTRY SUBDIVISION	CLIENT: TOLLESTRUP CONSTRUCTION INC.	BOREHOLE NO: BH 003
LOCATION: NE 1/4 SEC18- 9-22-W4M	DRILL METHOD: 150mm SOLID STEM AUGER	PROJECT NO: L12101239
CITY: COALHURST, ALBERTA	PROJECT ENGINEER: JIM RYAN	ELEVATION: 932.38m

SAMPLE TYPE ▨ DISTURBED ▢ NO RECOVERY ☒ SPT ▤ A-CASING ▥ SHELBY TUBE ▮ CORE

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT	PLASTIC M.C. LIQUID 20 40 60 80	◆ UNCONFINED (kPa)◆ 50 100 150 200 ▲POCKET PEN. (kPa)▲ 100 200 300 400	Elevation (m)
0	TOPSOIL - clay, silty, sandy, moist, dark brown, roots & root hairs, organics CLAY - silty, some sand, damp, very stiff, medium plastic, light brown, white precipitates					932.0
1	... damp to moist, brown, thin sand lenses					931.0
2						930.0
3	End of Borehole @ 3.0m No Seepage or Sloughing Slotted PVC Pipe Installed to 3.0m Borehole Measured Dry Oct. 30, 2007					929.0
4						928.0
5						927.0
6						926.0
7						925.0
8						924.0
9						923.0
10						922.0
11						921.0
12						920.0
13						919.0
14						918.0
15						917.0
16						

EBA Engineering Consultants Ltd.	LOGGED BY: JKM	COMPLETION DEPTH: 3m
	REVIEWED BY: JAR	COMPLETE: 10/19/2007
	DRAWING NO: B8	Page 1 of 1

PROJECT: GROUPED COUNTRY SUBDIVISION	CLIENT: TOLLESTRUP CONSTRUCTION INC.	BOREHOLE NO: BH 004
LOCATION: NE 1/4 SEC18- 9-22-W4M	DRILL METHOD: 150mm SOLID STEM AUGER	PROJECT NO: L12101239
CITY: COALHURST, ALBERTA	PROJECT ENGINEER: JIM RYAN	ELEVATION: 934.16m

SAMPLE TYPE ▨ DISTURBED ◺ NO RECOVERY ☒ SPT ☰ A-CASING ▥ SHELBY TUBE ▮ CORE

BACKFILL TYPE ▨ BENTONITE ⁙ PEA GRAVEL ▥ SLOUGH ⁙ GROUT ▧ DRILL CUTTINGS ⁙ SAND

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT	PLASTIC M.C. LIQUID 20 40 60 80	◆ UNCONFINED (kPa)◆ 50 100 150 200 ▲POCKET PEN. (kPa)▲ 100 200 300 400	Elevation (m)
0	TOPSOIL - clay, silty, sandy, moist, dark brown, roots & root hairs, organics					934.0
	CLAY - silty, some sand to sandy, damp to moist, very stiff, medium plastic, brown, white precipitates					
1						933.0
2	... moist					932.0
3	End of Borehole @ 3.0m					931.0
	No Seepage or Sloughing Slotted PVC Pipe Installed to 3.0m Borehole Measured Dry Oct. 30, 2007					
4						930.0
5						929.0
6						928.0
7						927.0
8						926.0
9						925.0
10						924.0
11						923.0
12						922.0
13						921.0
14						920.0
15						919.0
16						

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LOGGED BY: JKM	COMPLETION DEPTH: 3m
REVIEWED BY: JAR	COMPLETE: 10/19/2007
DRAWING NO: B9	Page 1 of 1

PROJECT: GROUPED COUNTRY SUBDIVISION
LOCATION: NE 1/4 SEC18- 9-22-W4M
CITY: COALHURST, ALBERTA

CLIENT: TOLLESTRUP CONSTRUCTION INC.
DRILL METHOD: CONTINOUS CORE
PROJECT ENGINEER: JIM RYAN

BOREHOLE NO: BH 006
PROJECT NO: L12101239
ELEVATION: 935.82m

SAMPLE TYPE ▨ DISTURBED ◺ NO RECOVERY ☒ SPT ▤ A-CASING ▥ SHELBY TUBE ▮ CORE
BACKFILL TYPE ▨ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▧ DRILL CUTTINGS ⁘ SAND

SOIL DESCRIPTION

... high plastic

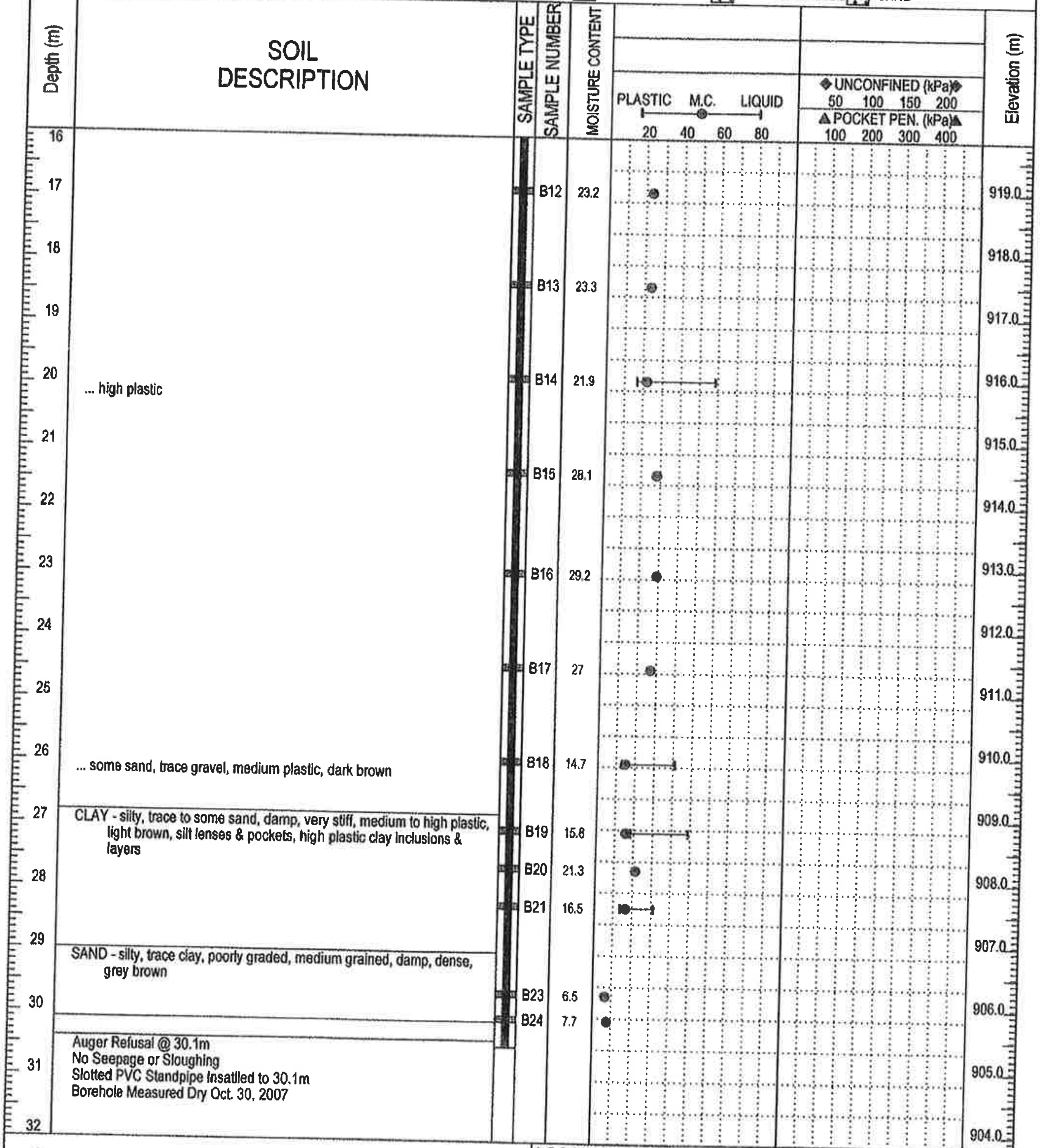
... some sand, trace gravel, medium plastic, dark brown

CLAY - silty, trace to some sand, damp, very stiff, medium to high plastic, light brown, silt lenses & pockets, high plastic clay inclusions & layers

SAND - silty, trace clay, poorly graded, medium grained, damp, dense, grey brown

Auger Refusal @ 30.1m
No Seepage or Sloughing
Slotted PVC Standpipe Insatlled to 30.1m
Borehole Measured Dry Oct. 30, 2007

SAMPLE NUMBER	MOISTURE CONTENT
B12	23.2
B13	23.3
B14	21.9
B15	28.1
B16	29.2
B17	27
B18	14.7
B19	15.6
B20	21.3
B21	16.5
B23	6.5
B24	7.7



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LOGGED BY: JKM
REVIEWED BY: JAR
DRAWING NO: B11

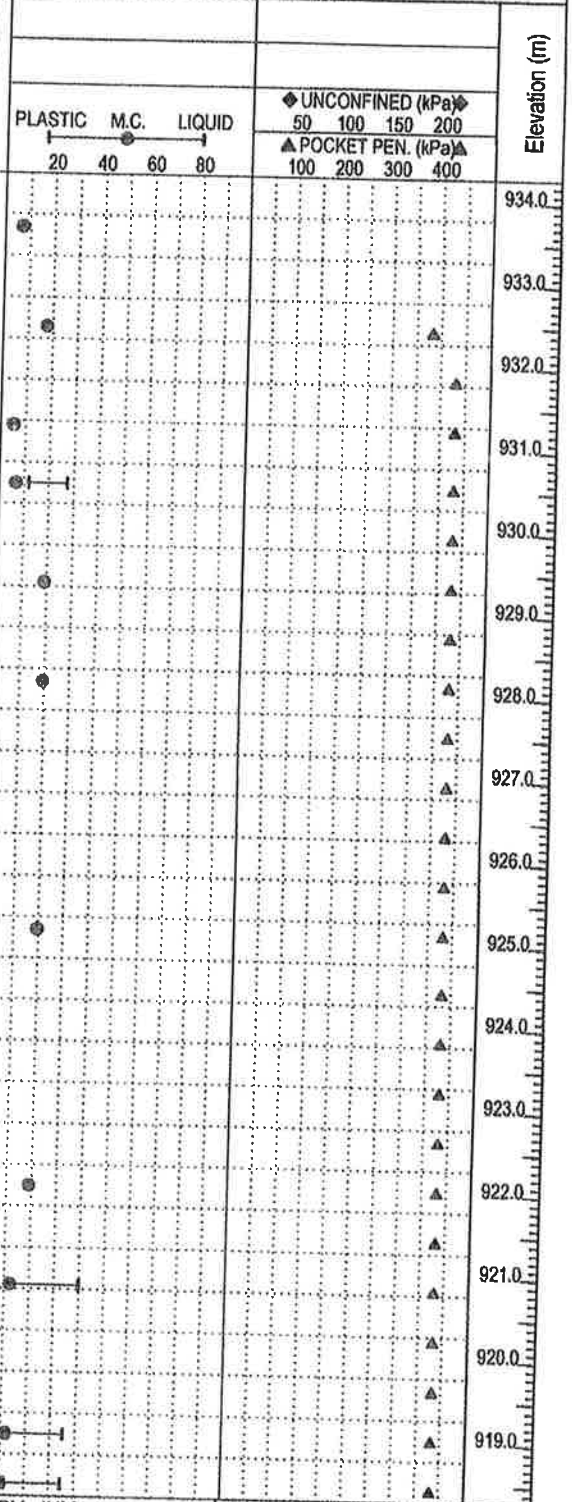
COMPLETION DEPTH: 30.1m
COMPLETE: 10/23/2007
Page 2 of 2

PROJECT: GROUPED COUNTRY SUBDIVISION	CLIENT: TOLLESTRUP CONSTRUCTION INC.	BOREHOLE NO: BH 007
LOCATION: NE 1/4 SEC18- 9-22-W4M	DRILL METHOD: 150mm SOLID STEM AUGER	PROJECT NO: L12101239
CITY: COALHURST, ALBERTA	PROJECT ENGINEER: JIM RYAN	ELEVATION: 934.29m

SAMPLE TYPE ▨ DISTURBED ◿ NO RECOVERY ☒ SPT ☰ A-CASING ▥ SHELBY TUBE ▮ CORE

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT
0	CLAY - silty, very sandy, damp, very stiff, low plastic, brown			
1		▨	B1	7.1
	... damp to moist			
		▨	B2	
2	CLAY - silty, trace to some sand, damp to moist, very stiff, medium plastic, brown, white precipitates	▨	B3	17.6
3	SAND - silty, trace clay, poorly graded, fine to medium grained, rounded, moist, compact to dense, brown	▨	B4	
	... damp	▨	B5	5.3
	CLAY - silty, sandy, damp, very stiff, low to medium plastic, light brown			
4		▨	B6	6.7
	CLAY (TILL) - silty, trace to some sand, damp, hard, medium to high plastic, brown with dark brown mottling, blocked, white precipitates, high plastic inclusions, thin silt lenses	▨	B7	
5		▨	B8	19
6		▨	B9	
		▨	B10	19.6
7		▨	B11	
		▨	B12	
8		▨	B13	
9		▨	B14	
		▨	B15	19.6
10	... coal and oxide specks	▨	B16	
11		▨	B17	
		▨	B18	
12		▨	B19	
	... silt pocket to 200mm @ 12.0m	▨	B20	18.9
13		▨	B21	
	... very stiff, medium plastic, light brown with dark brown mottling, thin sand lenses	▨	B22	12.3
14		▨	B23	
15		▨	B24	
	CLAY - silty, trace to some sand, damp, very stiff, medium plastic, light brown with dark brown mottling, silt lenses, high plastic clay inclusions & layers	▨	B25	11.8
16		▨	B26	10.4



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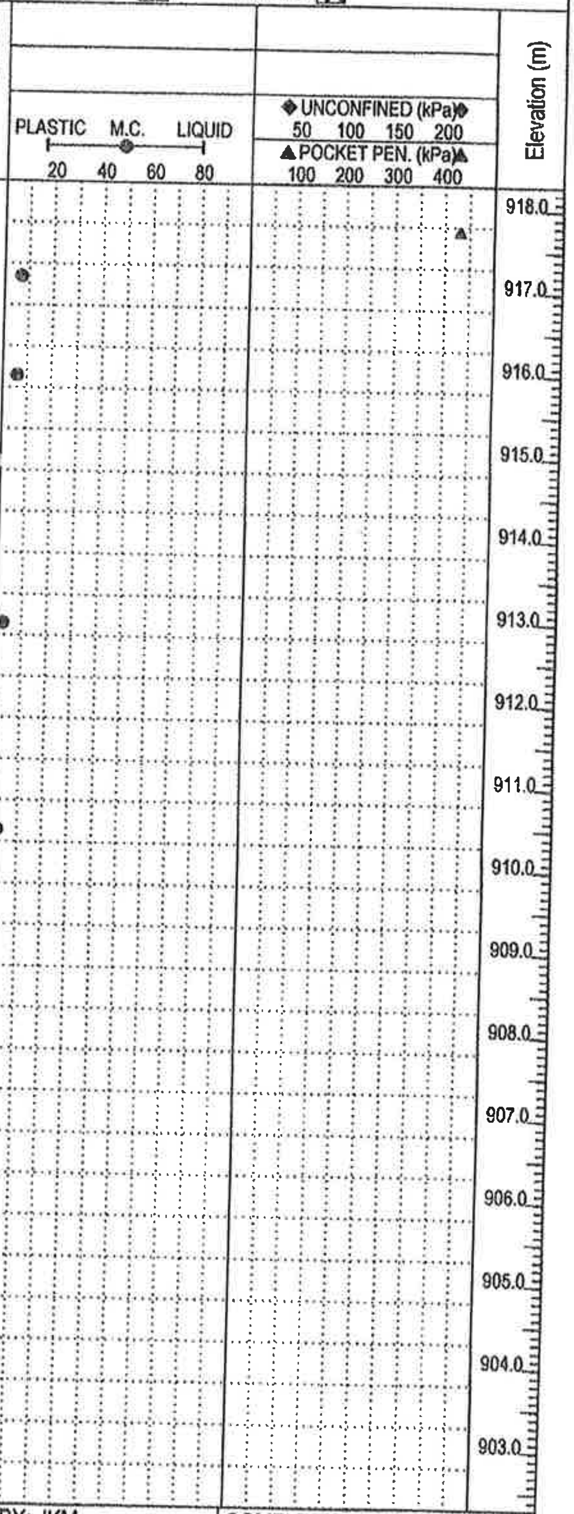
LOGGED BY: JKM	COMPLETION DEPTH: 30.5m
REVIEWED BY: JAR	COMPLETE: 10/24/2007
DRAWING NO: B12	Page 1 of 2

PROJECT: GROUPED COUNTRY SUBDIVISION	CLIENT: TOLLESTRUP CONSTRUCTION INC.	BOREHOLE NO: BH 007
LOCATION: NE 1/4 SEC18- 9-22-W4M	DRILL METHOD: 150mm SOLID STEM AUGER	PROJECT NO: L12101239
CITY: COALHURST, ALBERTA	PROJECT ENGINEER: JIM RYAN	ELEVATION: 934.29m

SAMPLE TYPE	▨ DISTURBED	◿ NO RECOVERY	☒ SPT	☰ A-CASING	▥ SHELBY TUBE	▮ CORE
BACKFILL TYPE	▨ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▧ DRILL CUTTINGS	⁘ SAND

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTURE CONTENT
16	SAND - silty, trace clay, poorly graded, fine to medium grained, damp, compact to dense, light brown			
		▨	B27	
17				
	... greyish			
		▨	B28	6.9
18		▨	B29	
		▨	B30	5.8
19				
	... browny grey	▨	B31	
		▨	B32	
20				
		▨	B33	
	... medium to coarse grained, dense			
21		▨	B34	
		▨	B35	2.4
22		▨	B36	
23		▨	B37	
		▨	B38	
24		▨	B39	2
		▨	B40	
25		▨	B41	
26		▨	B42	
		▨	B43	
27		▨	B44	
		▨	B45	1.6
28				
	... damp to moist, coal specks	▨	B46	
29		▨	B47	
		▨	B48	
30		▨	B49	
	End of Borehole @ 30.5m	▨	B50	1.7
31	No Seepage, Sloughing from 16.0m Slotted PVC Pipe Installed to 12.2m Borehole Measured Dry Oct. 30, 2007			
32				



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LOGGED BY: JKM	COMPLETION DEPTH: 30.5m
REVIEWED BY: JAR	COMPLETE: 10/24/2007
DRAWING NO: B12	Page 2 of 2

PROJECT: GROUPED COUNTRY SUBDIVISION	CLIENT: TOLLESTRUP CONSTRUCTION INC.	BOREHOLE NO: PH 002
LOCATION: NE 1/4 SEC18- 9-22-W4M	DRILL METHOD: 150mm SOLID STEM AUGER	PROJECT NO: L12101239
CITY: COALHURST, ALBERTA	PROJECT ENGINEER: JIM RYAN	

SAMPLE TYPE ▨ DISTURBED ◿ NO RECOVERY ☒ SPT ☰ A-CASING ▥ SHELBY TUBE ▮ CORE

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

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	MOISTURE CONTENT	PLASTIC M.C. LIQUID 20 40 60 80	◆UNCONFINED (kPa)◆ 50 100 150 200 ▲POCKET PEN. (kPa)▲ 100 200 300 400	Depth (ft)
0	TOPSOIL - clay, silty, sandy, moist, dark brown, roots & root hairs, organics					0
	CLAY - silty, some sand to sandy, damp, very stiff, medium plastic, light brown, white precipitates					
	End of Borehole @ 0.9m					
1	No Seepage or Sloughing					
1.5						5

EBA Engineering Consultants Ltd.

LOGGED BY: JKM	COMPLETION DEPTH: 0.9m
REVIEWED BY: JAR	COMPLETE: 10/19/2007
DRAWING NO: B2	Page 1 of 1

GEOTECHNICAL L12101239 TOLLESTRUP COUNTRY SUDIVISION COALHURST.GPJ EBA.GDT 08/02/01

APPENDIX

APPENDIX C RECOMMENDED GENERAL DESIGN AND CONSTRUCTION GUIDELINES

SHALLOW FOUNDATIONS

Design and construction of shallow foundations should comply with relevant Building Code requirements.

The term "shallow foundations" includes strip and spread footings, mat slab and raft foundations.

Minimum footing dimensions in plan should be 0.45 m and 0.9 m for strip and square footings, respectively.

No loose, disturbed or sloughed material should be allowed to remain in open foundation excavations. Hand cleaning should be undertaken to prepare an acceptable bearing surface. Recompaction of disturbed or loosened bearing surface may be required.

Foundation excavation and bearing surfaces should be protected from rain, snow, freezing temperatures, drying and the ingress of free water, during and after footing construction.

Footing excavations should be carried down into the designated bearing stratum.

After the bearing surface is approved, a mud slab should be poured to protect the soil and provide a working surface for construction, should immediate foundation construction not be intended.

All constructed foundations should be placed on unfrozen soils, which should be at all times protected from frost penetration.

All foundation excavations and bearing surfaces should be observed by a qualified geotechnical engineer to confirm that the recommendations contained in this report have been followed and that soil conditions are consistent with those assumed in the design.

Where over-excavation has been carried out through a weak or unsuitable stratum to reach into a suitable bearing stratum or where a foundation pad is to be placed above stripped natural ground surface, such over-excavation may be backfilled to subgrade elevation utilizing either structural fill or lean-mix concrete. These materials are defined under the separate heading "Backfill Materials and Compaction."

BACKFILL MATERIALS AND COMPACTION

Maximum density, as used in this section, means Standard Proctor Maximum Dry Density (ASTM Test D698) unless specifically noted otherwise. Optimum moisture content is as defined in this text.

"General engineered fill" materials should comprise clean, well-graded granular soils or inorganic, low-plastic cohesive soils. Such material should be placed in compacted lifts not exceeding 200 mm and compacted to not less than 98% of maximum density, at a moisture content at or slightly above optimum.

"Structural fill" materials should comprise clean, well-graded inorganic granular soils. Such fill should be placed in compacted lifts not exceeding 150 mm and compacted to not less than 98% of maximum density, at a moisture content near or slightly above optimum.

"Landscape fill" material may comprise soils without regard to engineering quality. Such soils should be placed in compacted lifts not exceeding 300 mm and compacted to a density of not less than 90% of maximum density.

Backfill adjacent to and above footings, abutment walls, basement walls, grade beams and pile caps or below highway, street or parking lot pavement sections should comprise general engineered fill materials as defined above.

Backfill supporting structural loads should comprise structural fill materials as defined above.

Backfill adjacent to exterior footings, foundation walls, grade beams and pile caps and within 300 mm of final grade should comprise low-plastic cohesive general engineered fill as defined above. Such backfill should provide a relatively impervious surface layer to reduce seepage into the sub-soil.

Backfill should not be placed against a foundation structure until the structure has sufficient strength to withstand the earth pressures resulting from placement and compaction. During compaction, careful observation of the foundation wall for deflection should be carried out continuously. Where deflection is apparent, the compactive effort should be reduced accordingly. In order to reduce potential compaction induced stresses, only hand held compaction equipment should be used in the compaction of fill within 500 mm of retaining walls or basement walls.

Backfill materials should not be placed in a frozen state or placed on a frozen subgrade. All lumps of materials should be broken down during placement.

Where the maximum-sized particles in any backfill material exceed 50% of the lift thickness or minimum dimension of the cross-section to be backfilled, such particles should be removed and placed at the other more suitable locations on site or screened-off prior to delivery to site.

Bonding should be provided between backfill lifts, if the previous lift has become desiccated. For the fine-grained materials, the previous lift should be scarified to 75 mm in depth followed by proper moisture conditioning and recompaction.

Recommendations for the specifications for various backfill types are presented below.

"Pit-run gravel" should conform to the following grading:

Sieve Sizes (Square Openings)	Percent Passing By Weight
200 mm	100 of Total Sample
150 mm	96 - 100 of Total Sample
75 mm	60 - 80 of Total Sample
25 mm	70 - 100 of Material Passing 75 mm Sieve
4.75 mm	25 - 63 of Material Passing 75 mm Sieve
1.18 mm	14 - 41 of Material Passing 75 mm Sieve
0.60 mm	7 - 30 of Material Passing 75 mm Sieve
0.15 mm	3 - 18 of Material Passing 75 mm Sieve
0.075 mm	2 - 9 of Material Passing 75 mm Sieve

Any grading variation from the above should be at the discretion of the Engineer; however, the percent of material passing the 0.075 mm sieve should not exceed 2/3 of the material passing the 0.6 mm sieve. The pit-run gravel should be free of any form of coating and any gravel containing clay, loam or other deleterious materials should be rejected. No oversized material should be tolerated.

"Crushed gravel" should conform to the following grading:

Sieve Sizes (Square Openings)	Percent Passing by Weight (Nominal Gravel Size)		
	100 mm	50 mm	25 mm
100 mm	100	—	—
75 mm	90 - 100	—	—
50 mm	—	100	—
40 mm	60 - 80	90 - 100	—
25 mm	—	—	100
20 mm	40 - 66	50 - 75	95 - 100
10 mm	25 - 54	25 - 52	60 - 80
4.75 mm	15 - 43	15 - 40	40 - 60
2.36 mm	10 - 35	10 - 33	28 - 48
0.60 mm	5 - 23	5 - 23	13 - 29
0.30 mm	—	—	9 - 21
0.15 mm	3 - 12	2 - 14	6 - 15
0.075 mm	2 - 10	1 - 10	4 - 10

Gravel:

100 mm Crushed Gravel: At least 13% by weight of the material retained on the 4.75 mm sieve should have two more fractured faces.

50 mm Crushed Gravel: At least 13% by weight of the material retained on the 4.75 mm sieve should have two more fractured faces.

25 mm Crushed Gravel: At least 50% by weight of the material retained on the 4.75 mm sieve should have two more fractured faces.

Any gravel containing deleterious material should be rejected.

"Coarse gravel" for bedding and drainage should conform to the following grading:

Sieve Sizes (Square Openings)	**Percent Passing By Weight (Nominal Gravel Size)**	
	50 mm	**40 mm**
50 mm	100	—
40 mm	90 - 100	100
25 mm	—	95 - 100
20 mm	35 - 70	—
15 mm	—	25 - 60
10 mm	10 - 30	—
4.75 mm	0 - 5	0 - 10
2.36 mm	—	0 - 5

"Coarse sand" for bedding and drainage should conform to the following grading:

Sieve Sizes (Square Openings)	**Percent Passing By Weight**
10 mm	100
4.75 mm	95 – 100
2.36 mm	80 – 100
1.18 mm	50 - 85
0.60 mm	25 - 60
0.30 mm	10 - 30
0.15 mm	2 - 10

"Lean-mix concrete" should be low strength concrete having a minimum 28-day compressive strength of 3.5 MPa.

PROOF-ROLLING

Proof-rolling is a method of detecting soft areas in an "as-excavated" subgrade for fill, pavement, floor or foundations or detecting non-uniformity of compacted embankment. The intent is to detect soft areas or areas of low shear strength not otherwise revealed by means of testholes, density testing or visual examination of the site surface and to check that any fill placed or subgrade meets the necessary design strength requirements.

Proof-rolling should be observed by qualified geotechnical personnel.

Proof-rolling is generally accomplished by the use of a heavy (15— 60 tonne) rubber-tired roller having four wheels abreast on independent axles with high contact wheel pressures [inflation pressures ranging from 550 kPa (80 psi) up to 1,030 kPa (150 psi)].

A heavily-loaded truck may be used in lieu of the equipment described in the paragraph above. The truck should be loaded to approximately 10 tonnes (22,000 lbs) per axle and a minimum tire pressure of 550 kPa (80 psi).

Ground speed to be maximum of 8 km/hr (133 m/min) (5 mph) (400 ft/min). Recommended speed is 4 km/hr (65 m/min) (2.5 mph) (200 ft/min).

The recommended procedure is two complete coverages with the Proof-rolling equipment in one direction and a second series of two coverages made at right angles to the first series; one "coverage" means that every point of the proof-rolled surface has been subjected to the tire pressure of a loaded wheel. Less rigorous procedures may be acceptable under certain conditions subject to the approval of an engineer.

Any areas of soft, rutted or displaced materials detected should be either recompacted with additional fill or the existing material removed and replaced with general engineered fill or properly moisture conditioned as necessary.

The surface of the grade under the action of the proof-rolling should be observed, noting visible deflection and rebound of the surface or shear failure in the surface of granular soils as ridging between wheel tracks.

If any part of an area indicates significantly more distress than other parts, the cause should be investigated, by, for example, shallow auger holes.

In the case of granular subgrades, distress will generally consist of either compression due to insufficient compaction or shearing under the tires. In the first case, proof-rolling should be continued until no further compression occurs. In the second case, the tire pressure should be reduced to a point where the subgrade can carry the load without significant deflection and subsequently, gradually increased to its specified pressure as the subgrade increases in shear strength under this compaction.

APPENDIX C

HYDROLOGICAL & SITE DRAINAGE ANALYSIS

Environmental
Agricultural
Structural
Civil
Municipal

HASEGAWA ENGINEERING

Consulting Professional Engineers
A Division of 993997 Alberta Ltd.

1220 31st Street North, Lethbridge, AB T1H 5J8
Bus: 328-2686 Fax: 328-2728 E-mail: hasgm@telusplanet.net

February 4, 2008

Our File #: 07-295

Nick Paladino
County of Lethbridge

Re: Seiller Estates Subdivision Site Drainage Analysis Report

---

Dear Sir:

Attached please find the Site Drainage Analysis Report submitted for the proposed Seiller Estates subdivision located in the County of Lethbridge.

Please review this document and contact our office with any questions or comments. This document was prepared under my supervision.

Yours truly,

Mark Hasegawa, P.Eng.
HASEGAWA ENGINEERING
Consulting Professional Engineers
MAH/dd

PERMIT TO PRACTICE
HASEGAWA ENGINEERING LTD
Signatur
Date 2/4/08
PERMIT NUMBER: P 582
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

Attachment

cc:

TABLE OF CONTENTS

1. INTRODUCTION

On behalf of Bluestone Developments, Hasegawa Engineering (HE) has completed this hydrological analysis at the subject site. The site is located as shown in Figure 1 and 2. The hydrological analysis includes the following major aspects:

1. Overall site layout and conditions
2. Site topography and runoff
3. Rainwater retention design (if required)

2. SITE CONDITIONS

The site is located directly west of the Town of Coalhurst, Alberta (refer to Figure 1). The site is adjacent Range Road 22-5 on the west, farm land to the north and south and the Oldman river valley to the west. The site surrounds a coulee access road and drains toward this access. The average grade from the south side of the access road is 1.5% toward the northwest and on the north side of the property flow is generally to the south west at 2.6%. Water does not appear to accumulate onsite at all. Most drainage from the site funnels to the access road and flows into the coulee. There is a small portion on the north side of the development that runs north onto the field.

According to the Alberta Geological survey, surficial soils consist of primarily Stream/slope wash, but portions of the development may be underlain by Fluvial- fine or Lucustrine – Fine.

In addition septic testing was performed for this site to determine the infiltration rates for septic design. The results of this analysis indicated that surfical soils onsite had a permeability of 3 to 15 min/cm (Appendix B; EBA 2008). This information was used to determine infiltration rates used in predicting surface runoff.

3. OFFSITE DRAINAGE

At this point there does not seem to be any major offsite runoff through the property. The land east of the property is separated by Range Road 22-5 and does not readily flow through the property. If offsite drainage was an issue, provisions would be made for a culvert to bypass the development and drain the land east into the coulee through the coulee access road. The land located north and south of the property appears to drain directly to the coulee and not pass through the proposed development.

4. SURFACE RUNOFF DESIGN CRITERIA

The total area of the onsite basin is 27 acres. In order to determine the volume of runoff from the subject site, surface runoff analysis was performed. Rainfall intensity data was obtained for the City of Lethbridge from the Atmospheric Environment Service, which is part of Environment Canada. The input data for each basin was determined using the site information. Runoff estimations were developed using the "TR-55 Urban Hydrology for Small Watersheds" runoff model. The basin was divided into sheet flow, shallow concentrated flow and stream flow regions. The model utilizes the information from each sub-basin area to develop a time of concentration. The model then calculates the peak flow and total runoff based on this input.

Inputs used to calculate runoff are included in Table A1 and A2 below. Analysis was conducted for both pre- and post- development conditions. The curve number used for pre-development flow was 69, which represents poor grassland for moderate Class B soils. The post-development curve number was 68, which represents urban development at 1 acre lots and poor soil classification. Key input data used for this analysis is included in Table A1 and A2. Note that the post development slope is half of the predevelopment slope. The road and drainage of the development will be designed to have more gradual slopes and as a result flatten peak runoff values. In addition the flow paths observed for post development conditions are longer since the water is forced to follow the ditch system and not flow directly to the coulee.

Table A1: Runoff Analysis Input Data

	Drainage Basin (acres)	**2 year 24 hour storm (inches)**	**100 year 24 hour storm (inches)**	**Average Slope (ft/ft)**	**Curve number (CN)**	**Percent impervious area**
Pre-development	27	2	4.25	0.02	69	0%
Post-development	27	2	4.25	0.01	68	20%

Table A2: Additional Design Information

	Overland flow length M	**Shallow concentrated flow length M**	**Channel flow length M**	**Average slope**	**Soil type**
Pre-development	30	150	80	0.02	B
Post-development	20	150	180	0.01	B

5. SURFACE RUNOFF RESULTS

The results of the surface runoff analysis are provided in this section. Based on these results there should not be an increase in runoff from pre to post development (refer to Table A3). This is due to, longer flow paths, the low curve number of the country residential development, storage in the ditch system and lowered grade on the lots. Based on this analysis it appears that there is no need to create a retention pond

Since there will be minimal grading as part of this development, lots adjacent the coulee will not be altered significantly from pre-development conditions. It is estimated that 50% of the drainage from these lots will flow to the coulee. Lots not adjacent the coulee will be designed to drain to the ditch on the proposed road.

Table A3: Runoff Analysis Input Data

	Time of concentration (Hours)	Peak flow (cfs)	Runoff volume (Inches per acre)	Runoff volume (Acre ft)	Volume of retention provided
Pre-development	0.25	7	1.4	3.15	NA
Post-development	0.2	6	1.4	3.15	NA

6. DRAINAGE STRUCTURE DESIGN AND GRADING

There should be no requirement to build additional drainage structures as a result of this development. Since this is a rural development, grading should be kept to a minimum. Grading on the lots adjacent the coulee should be designed to have development in the front of the proposed structure to flow to the right of way. Grading on the other roads should ensure drainage either flows to the proposed access right of way or the county ditch system. The ditch system in the road will be designed to not exceed an average slope of 1% and channel runoff to natural drain paths into the coulee.

In addition, the volume of water retention in the ditch creates storage during a rain event. In this case there is 1000 m of road and a ditch on each side. Using a 4:1 slope for the ditch and a depth of water of 0.4 m, there will be 1280 M^3 of storage in the ditch system.

7. CONCLUSIONS AND RECOMENDATIONS

Surface runoff analysis was conducted on the subject property. The runoff from the property appears to not increase due to low density residential development. It is recommended that all development with impervious surfaces (i.e. structures and driveways) be design to drain to the proposed access road. In addition, driveways designed to access the lots must be designed with a swale or culvert that will not restrict storm water flow in the ditch.

In order to minimize peak runoff values the design of this development should also include BMPs listed in the Alberta Environment Storm Water Management Guidelines (1999) and the Area Structure Plan.

Notes:

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Contractor to check and verify all dimensions before construction, any errors and omissions reported to Engineer immediately.

Drawing not to be used for construction until so approved.

Do not scale Drawing.

All construction shall be in accordance with the latest code, may it be construction, mechanical, etc. code.

No	Revision	Date	By

Hasegawa Engineering

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201,2816–21 Street NE
Calgary Alberta T2E 6Z2
Ph: 250–5261

CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
AREA MAP

DESIGN
HE

DRAWN
DPB

CHECKED
HE

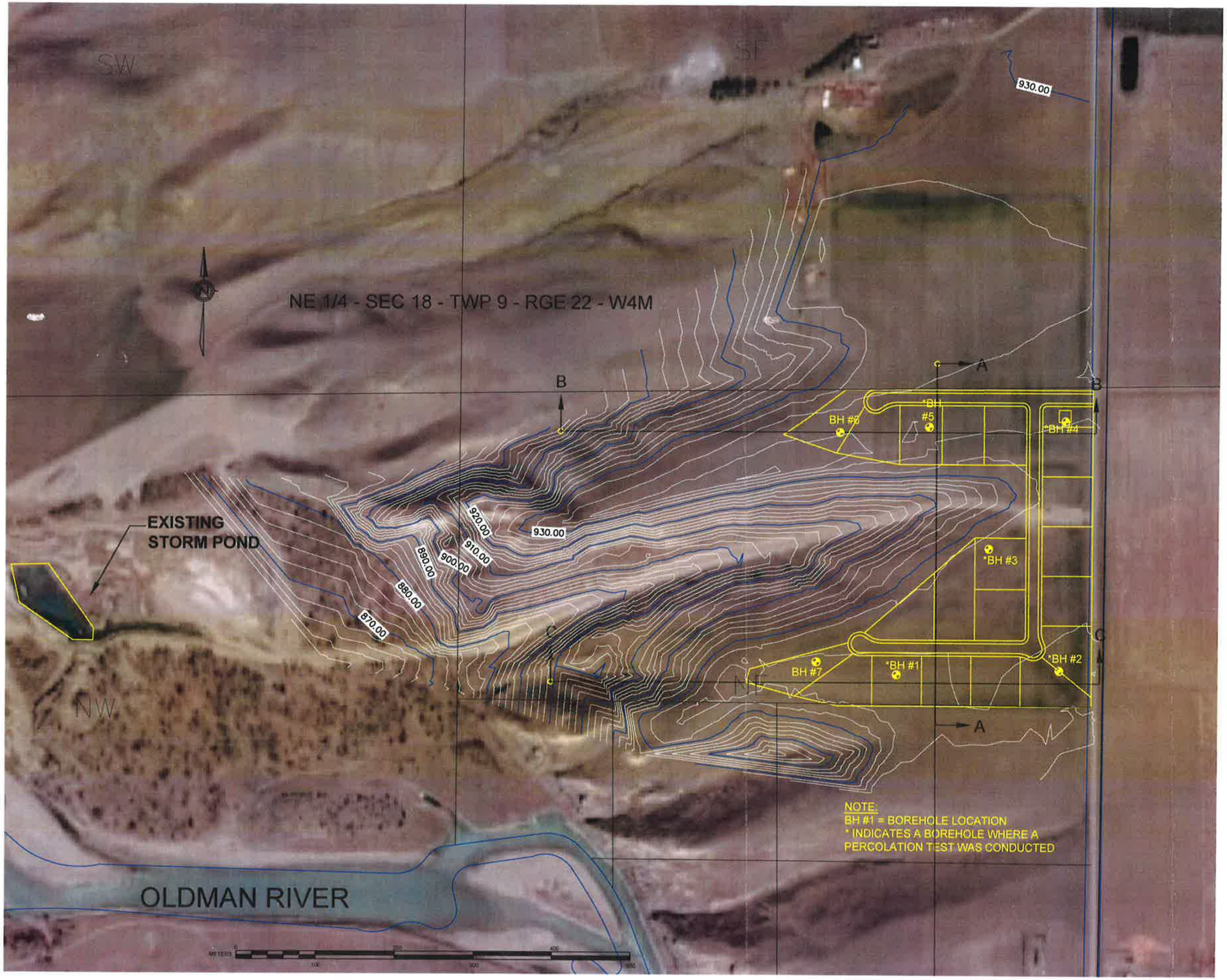
APPROVED
HE

DATE DRAWN
FEB4,08

PROJECT NO
07295

SCALE
1:1000

SHEET NO
FIGURE 1



NOTE:
BH #1 = BOREHOLE LOCATION
* INDICATES A BOREHOLE WHERE A PERCOLATION TEST WAS CONDUCTED

G. HASEGAWA

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Ph: 250-5261

CLIENT
BLUESTONE DEVELOPMENTS

PROJECT TITLE
SEILLER ESTATES

DRAWING TITLE
EXISTING SITE CONTOURS

DESIGN: HE
DRAWN: DPB
CHECKED: HE
APPROVED: HE
DATE DRAWN: FEB4,08
PROJECT NO: 07295
SCALE: 1:5000
SHEET NO: FIGURE 2

APPENDIX D

LAND TITLE & WATER SHARES

CERTIFIED COPY OF
CERTIFICATE OF TITLE

S

LINC	SHORT LEGAL
0022 190 227	4;22;9;18;;15,16

TITLE NUMBER: 041 416 423
TRANSFER OF LAND
DATE: 02/11/2004

AT THE TIME OF THIS CERTIFICATION

TOLLESTRUP CONSTRUCTION INC..
OF BOX 474
LETHBRIDGE
ALBERTA

IS THE OWNER OF AN ESTATE IN FEE SIMPLE
OF AND IN

MERIDIAN 4 RANGE 22 TOWNSHIP 9
SECTION 18
LEGAL SUBDIVISIONS 15 TO 16 INCLUSIVE
EXCEPTING THEREOUT ALL MINES AND MINERALS

SUBJECT TO THE ENCUMBRANCES, LIENS AND INTERESTS NOTIFIED BY MEMORANDUM UNDERWRITTEN OR ENDORSED HEREON, OR WHICH MAY HEREAFTER BE MADE IN THE REGISTER.

ENCUMBRANCES, LIENS & INTERESTS

REGISTRATION NUMBER	DATE (D/M/Y)	PARTICULARS
741 091 031	27/09/1974	IRRIGATION ORDER/NOTICE THIS PROPERTY IS INCLUDED IN THE LETHBRIDGE NORTHERN IRRIGATION DISTRICT
041 229 630	19/06/2004	CAVEAT RE : AGREEMENT FOR SALE CAVEATOR - TOLLESTRUP CONSTRUCTION INC., C/O KLASSEN VANDERBERG 400,7015 MACLEOD TR SW CALGARY ALBERTA T2H2K6 AGENT - ROBERT S VANDERBERG

THE REGISTRAR OF TITLES CERTIFIES THIS TO BE AN ACCURATE REPRODUCTION OF THE CERTIFICATE OF TITLE REPRESENTED HEREIN THIS 02 DAY OF NOVEMBER ,2004

(CONTINUED)

CERTIFICATE OF TITLE

TITLE NUMBER: 041 416 423

SUPPLEMENTARY INFORMATION

CONSIDERATION: SEE INSTRUMENT
MUNICIPALITY: COUNTY OF LETHBRIDGE
REFERENCE NUMBER:
031 352 067
AREA:
32.4 HECTARES (80 ACRES) MORE OR LESS
TOTAL INSTRUMENTS: 002

GOVERNMENT OF THE PROVINCE OF ALBERTA

CERTIFIED COPY OF
CERTIFICATE OF TITLE

S

LINC
0022 190 235

SHORT LEGAL
4;22;9;10;;13,14

TITLE NUMBER: 041 416 423 +1
TRANSFER OF LAND
DATE: 02/11/2004

AT THE TIME OF THIS CERTIFICATION

TOLLESTRUP CONSTRUCTION INC..
OF BOX 474
LETHBRIDGE
ALBERTA

IS THE OWNER OF AN ESTATE IN FEE SIMPLE
OF AND IN

MERIDIAN 4 RANGE 22 TOWNSHIP 9
SECTION 10
LEGAL SUBDIVISIONS 13 TO 14 INCLUSIVE
EXCEPTING THEREOUT ALL MINES AND MINERALS

SUBJECT TO THE ENCUMBRANCES,LIENS AND INTERESTS NOTIFIED BY MEMORANDUM UNDERWRITTEN OR ENDORSED HEREON,OR WHICH MAY HEREAFTER BE MADE IN THE REGISTER.

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741 091 031	27/09/1974	IRRIGATION ORDER/NOTICE THIS PROPERTY IS INCLUDED IN THE LETHBRIDGE NORTHERN IRRIGATION DISTRICT
041 229 629	19/06/2004	CAVEAT RE : AGREEMENT FOR SALE CAVEATOR - TOLLESTRUP CONSTRUCTION INC.. C/O KLASSEN VANDERBERG 400,7015 MACLEOD TR SW CALGARY ALBERTA T2H2K6 AGENT - ROBERT S VANDERBERG

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(CONTINUED)

CERTIFICATE OF TITLE

TITLE NUMBER: 041 416 423 +1

SUPPLEMENTARY INFORMATION

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MUNICIPALITY: COUNTY OF LETHBRIDGE
REFERENCE NUMBER:
031 352 067 +1
AREA:
32.4 HECTARES (80 ACRES) MORE OR LESS
TOTAL INSTRUMENTS: 002