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Structural • Environmental •
Hydrogeology

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October 17, 2014 140208

Alium Investments Ltd.
338 Dufferin Street
Toronto
M6A 3A4

RE: SLOPE STABILITY ASSESSMENT AND RETAINING WALL DESIGN
Site Plan Control Application - City of Ottawa File Number: D07-12014-0038
Proposed Greely Commercial Center
5640 Bank Street, 7107 Marco Street, 701 Mitch Owens Road
Ottawa, Ontario

Dear Sir:

Kollaard Associates Inc. was retained to undertake a slope stability assessment and retaining wall design for the slope along the south side of the proposed Greely Commercial Center Site commonly known as 5640 Bank Street, 7107 Marco Street, 701 Mitch Owens Road, located in Greely, Ottawa, Ontario. The purpose of this assessment and design is to address the concern with respect to the stability of the existing slope along the south side of the site and to provide retaining wall design for the proposed retaining wall to be located along this slope.

BACKGROUND INFORMATION

The site area proposed for development in the current site plan application consists of the eastern about 5.6 hectare portion of a 14 hectare irregularly shaped parcel of land at the southwest corner of the intersection of Bank Street and Mitch Owens Road. This about 5.6 hectare portion will be referred to as the site. For the purposes of this letter, Bank Street is considered to be oriented along a north south axis.

The subject property is currently vacant and was formerly used as a sand and gravel pit. Extraction of sand and gravel from the pit has resulted in and/or increased the slope along the south side of the site. A portion of the sand and gravel pit has been filled in resulting a relatively level flat land at the base of the slope on which the proposed Greely Commercial Centre development will be located.

The existing slope and proposed retaining wall are located along the south side of the site. Existing and proposed grades have been provided on the Grading Plan prepared by WMI & Associates Limited drawing No. GR Project 11-183. Revision 2 dated February 4, 2014. An AutoCad version of this drawing was provided to Kollaard Associates and has been copied into the attached drawing 140208-PLAN. This drawing illustrates the area in question and provides existing and proposed slope conditions.

Review of Information obtained from BAE & Associates Environmental Inc.

Two geotechnical reports were prepared for the site by BAE & Associates Environmental Inc. in support of the proposed development for the site plan application. The first report was dated April 11, 2012 and the second report was dated August 9th 2013. It is understood that the report did not address the slope stability of the slope along the south side of the site or the design of the retaining wall. Copies of these reports were provided by the client to Kollaard Associates. From the borehole logs provided with the report dated April 11, 2012, the subsurface soil conditions for the level area below the slope consist of between about 0 and 8 metres of fill overlying grey brown fine to coarse silty sand followed by glacial till and bedrock. Standard penetration test results within the sand and glacial till ranged from 32 to greater than 100 blows per 0.3 metres indicating the glacial till is dense to very dense.

The geotechnical report completed August 9th 2013 provided the factual data obtained during the additional field investigation for the report in Table 1. Boreholes BH#15, BH#20, BH#26, BH#27 and BH#29 were put down on the level ground in proximity to the base of the slope. The table 1 indicates auger refusal in glacial till at between 1.2 and 3.0 metres below the existing ground surface at these borehole locations.

Review of Information obtained from Kollaard Associates Inc. Geotechnical Investigation

A geotechnical investigation and report was completed by Kollaard Associates Inc. for the site. The field work for this investigation was carried out on August 25 to 29, 2014 and September 22, 2014. From August 25 to 29, 2014, twenty-one boreholes, numbered BH1 to BH21 were put down at the site using a track mounted drill rig equipped with a hollow stem auger owned and operated by Marathon Drilling of Greely, Ontario. An additional test hole BH22 was advanced by hand on August 28, 2014. On September 22, 2014, five test pits numbered TP1 to TP5 were put down at the site using a track mounted excavator supplied and operated by a local contractor.

The boreholes and test pits collectively referred to as test holes indicate that the fill thickness across the site varies about 1 to greater than 8 metres were explored. The fill thickness is less along the east and south sides of the relatively flat level area.

Two test holes were put down at the top of the slope to confirm the subsurface conditions at the top of slope. The first borehole, put down using a track mounted drill rig, BH6-5111 was ended with practical refusal at a depth of 3.15 metres below the existing ground surface in very dense glacial till. The second test hole BH22-5112 was put down by hand excavation to a depth of 0.74 metres. The second test hole was abandoned at this depth as the subsurface materials encountered were too dense to penetrate further by hand. A 10 mm diameter probe could not be advanced into the undisturbed material at the bottom of the borehole. Based on the particle size analysis of the material encountered at the bottom of BH6-5111, the native soils at the top of the slope consist of silty sand glacial till.

Subsurface Investigation for Slope Stability

Kollaard Associates Inc. supervised the excavation of 4 test pits at the base of the slope on April 25, 2014. The test pits were put down using a backhoe supplied and operated by a local excavating contractor. The test pits were put down on the face of the slope beginning at about 1 to 2 metres above the level ground surface at the toe of the slope. The test pits were advanced to about 1 to 1.5 metres below the level ground surface. The test pits were extended approximately 1.8 to 2.5 metres into the face of the slope. The subsurface conditions were classified based on visual and tactile examination of the materials exposed on the sides and bottom of the test pits. The ground water conditions were observed in the open test pits at the time of excavating. The test pits were loosely backfilled with the excavated materials upon completion of the field work. A description of the subsurface conditions encountered at the test pits is given in the attached Record of Test pits sheets.

Observations of the slope face and information obtained from the test pits indicate that the subsurface conditions of the slope in general consists of a layer of fill overlying a thin layer of topsoil followed by grey brown dense to very dense silty sand with some gravel cobbles and boulders (Glacial Till).

The following was observed within the test pits during excavation.

- It was difficult to advance the test pits beyond 1.0 metres into the glacial till with the backhoe due to the density of the glacial till.
- Significant quantities of cobbles and boulders were present with depth within the glacial till.

- The backhoe could not advance test pit TP3 beyond 1.2 metres below the level ground surface due to the presence of boulders and density of the glacial till.

It is noted that the ground surface conditions encountered in the test pits put down by Kollaard Associates Inc. are consistent with the subsurface conditions identified along the base of the slope in the geotechnical reports prepared by BAE & Associates Environmental Inc.

FIELD OBSERVATIONS

The undersigned of Kollaard Associates Inc visited the site on April 17, 2014 to observe the conditions of the existing slope. During the site visit measurements of the slope were obtained using a hand clinometre and level and were compared to the existing topographical survey information provided on the grading plan prepared by WMI & Associates Limited. The information obtained during site visit was in keeping with the topographic information provided on the grading plan prepared by WMI & Associates Limited.

In general the slope along the south side of the site begins at about 20 metres west of the east property line and has a height of about 1.5 metre with an inclination from horizontal of about 6 to 7 degrees. The height and inclination of the slope increase to the west over a distance of about 200 metres as follows:

- At about 70 metres from the east property line, the slope has a height of about 2.5 metres and an inclination from horizontal of about 9.5 degrees
- At about 100 metres from the east property line, the slope has a height of about 3.3 metres and an inclination from horizontal of about 12 degrees
- At about 130 metres from the east property line, the slope has a height of about 4.3 metres and an inclination from horizontal of about 21 degrees
- At about 170 metres from the east property line, the slope has a height of about 7.0 metres and an inclination from horizontal of about 28 degrees
- Between about 210 metres from the east property line and the west edge of the site, the slope has a height of about 7.7 to 7.8 metres and an inclination from horizontal of about 26 to 31 degrees

The ground surface immediately above the slope consists of near horizontal table land.

A review of the slope conditions indicated that a thin veneer of fill has been placed on the slope in many places. It is expected that the fill is a result of waste materials being deposited off the top edge of the slope. The slope and table land immediately adjacent the slope is well vegetated with grass and trees ranging from saplings to trees of 0.3 metres DBH or greater. There was no evidence of slope instability or previous slope movement in the form of tension cracks or terracing of the ground surface or in the vegetation on the slope.

SLOPE STABILITY

As previously indicated the slope varies in height from about 1.5 to 7.8 metres and in inclination from about 7 degrees to 31 degrees from horizontal. Four sections were identified as shown in the attached drawing 140208-Plan to model the various slope conditions along the south slope at the site. The four selected sections were chosen based on engineering judgement to represent higher, steeper sections along the slope to ensure then critical slope stability conditions were assessed.

GeoStudio: Slope/W (2012) slope stability software was used to model the existing conditions of the slope at the site and determine the worst case factor of safety. The Slope/W model incorporated the slope's approximate dimensions, soil types and properties to complete a two-dimensional slope analysis. The Morgenstern-Price method was used to compute the factor of safety for each section.

For seismic (earthquake) loading, the potential for instability was evaluated using a simple "pseudo-static" model where a horizontal force is applied to the failure mass. This horizontal force is proportional to the weight of the failure mass and is determined using a "seismic coefficient", which is typically taken as half the design peak horizontal ground acceleration for the region of Ottawa (Barhaven) as specified in the 2012 Ontario Building Code, of 0.32. A seismic coefficient kh of 0.16 was therefore used.

The soil parameters used in the analyses were based on experience with similar soils in eastern Ontario was well as published correlations** with the results of the information obtained from the field investigations of both Kollaard Associates Inc and BAE & Associates Environmental Inc.

*** Terzaghi, Peck and Mesri – 1996 Soil Mechanics in Engineering Practice, 3rd Edition Figure 19.6*

The soil parameters used in the analyses are:

Material	Effective Angle of Internal Friction (degrees)	Effective Cohesion (kPa)	Unit Weight (kN/m^3)
Silty Clay Fill	27	3	19
Silty Sand Fill	35	1	20
Glacial Till	40	1	20
Compacted Granular "A" backfill	38	0	21

It is noted that the effective angle of internal friction of the glacial till used is somewhat higher than the typical range of 30 to 35 degrees indicated in the City of Ottawa slope stability guidelines. It is noted however, dense to very dense glacial till is often considered to be impenetrable. Further, published correlations between standard penetration resistance and density with respect to the effective angle of internal friction indicate that values of 40 to greater than 45 would be appropriate.** While not present in the existing slope conditions, the compacted Granular "A" material will be used as backfill behind the proposed retaining wall.

The water table was conservatively set at between 0.8 to 1.5 meters below the existing ground surface to model close to fully saturated conditions. It is noted that ground water was not encountered within the test pits on April 25, 2014. Ground water was measured on September 12, 2014 at depths of 2.8 metres and greater within the stand pipes installed in select boreholes put down for the geotechnical investigation by Kollaard Associates Inc.

*** Terzaghi, Peck and Mesri – 1996 Soil Mechanics in Engineering Practice, 3rd Edition Figure 19.6*

EXISTING SLOPE GEOMETRY

Static Conditions

The slope stability analysis results for the existing slope geometry at the site is indicated on the attached Kollaard Associates Inc. drawings 140208-SL1 and 140208-SL2.

The overall slope stability factor for each section analyzed is as follows:
Section 7055 – min FS = 1.5
Section 7065 – min FS = 1.5
Section 7077 – min FS = 1.5
Section 7089 – min FS = 3.6

Under static conditions, slopes with a factor of safety of 1.1 to 1.3 are considered marginally stable, slopes with a factor of safety of greater than 1.3 are considered stable, and slopes with a factor of safety of 1.5 and greater are considered to be adequately stable for the construction of dwellings located close to the slope crests.

Based on the existing minimum factors of safety for each section, the slope is considered to be stable under static conditions with no setbacks from the top of slope.

Seismic Conditions

The slope sections used for the static analysis were re-assessed for seismic conditions using pseudo-static analysis.

The overall slope stability factor under seismic loading for each section analyzed is as follows:
Section 7055 – min FS = 1.1
Section 7065 – min FS = 1.1
Section 7077 – min FS = 1.1
Section 7089 – min FS = 1.9

From the City of Ottawa slope stability guidelines, a minimum factor of safety of 1.1 is suggested for seismic slope stability analysis.

Based on the existing minimum factors of safety for each section, the slope is considered to be stable under static conditions with no setbacks from the top of slope.

PROPOSED SLOPE GEOMETRY (GLOBAL STABILITY OF RETAINING WALL)

The slope sections previously assessed to model the stability of the existing slope geometry were re-assessed with the proposed retaining wall in place. Parameters for the Granular 'A' backfill were provided in above.

It is noted that the retaining wall design presented in the proposed grading plan by WMI & Associates Limited was modified with respect to the proposed top of retaining wall elevation. The top of wall elevation was reduced in combination with cutting the slope above the retaining wall.

The slope stability analysis results for the proposed slope geometry with the retaining wall in place at the site is indicated on the Kollaard Associates Inc. drawings 140208-SL3 and 140208-SL4.

The global slope stability factor for each section analyzed under static load conditions is as follows:
Section 7055 – min FS = 1.6
Section 7065 – min FS = 1.6
Section 7077 – min FS = 1.7
Section 7089 – min FS = 2.6

The global slope stability of the retaining wall used for the static analysis were re-assessed for seismic conditions using pseudo-static analysis.

The overall slope stability factor under seismic loading for each section analyzed is as follows:
Section 7055 – min FS = 1.1
Section 7065 – min FS = 1.1
Section 7077 – min FS = 1.2
Section 7089 – min FS = 1.7

Since the minimum factor of safety under static conditions is above 1.5 and under seismic conditions is above 1.1, the proposed retaining wall is considered to be globally stable. Therefore, the slope along the south side of the site with the proposed geometry is considered to be stable.

It is noted that the existing structures on the properties above the slope consist of lightly loaded garden style sheds or auxiliary garages and swimming pools. The loading from these structures was considered in the analysis as concentrated loading or line loads at the top of the slope. It was assumed that these structures would result in a line load of 40 kN at a distance of 3.5 metres from the property line.

The loading from these structures did not impact the stability of the slope as the structures were located beyond the slip surfaces resulting in the minimum factor of safety. As such, the proposed geometry will not have an impact on the stability of the slope with respect to the existing structures above the slope.

RETAINING WALL DESIGN

The retaining wall design presented in this letter has been completed using Stone Strong Segmental Blocks. It is considered that the blocks are filled with 20 mm or 25 mm clear stone. The fill immediately behind the blocks will consist of Granular "A" material compacted to a minimum of 98% of standard proctor maximum dry density. The block wall will be placed on a prepared engineered granular pad consisting of Granular "A" material compacted to a minimum of 100% SPDD. The allowable bearing capacity of the soil subgrade below the retaining wall is 250 kPa for serviceability limit states design. It is assumed that the retaining wall will be placed on an engineered granular pad founded on undisturbed glacial till. Standard Penetration tests completed within the glacial till provided values of N = 40 to 70 blows per 300 mm. *(K. Terzaghi and R.B. Peck, Soil Mechanics in Engineering Practice, 1967).*

As previously indicated, during the analysis, it was assumed that the structures on the adjacent property would result in a line load of 40 kN at a distance of 3.5 metres from the property line.

For the purposes of the retaining wall construction it is considered that the glacial till between the original ground surface and 1.2 metres below the original ground surface may be taken as soil Type 2 with respect to Ontario Regulation 213/91. The glacial till below 1.2 metres below the original ground surface may be taken as soil Type 1.

The strong stone blocks used in the design consist of:
24SF Block – face of 0.9 metres height and 2.4 metres width, depth of 1.12 metres.
6SF Block – face of 0.45 metres height and 1.2 metres width, depth of 1.12 metres.
24-86 Block – face of 0.9 metres height and 2.4 metres width, depth of 2.18 metres.

It is noted that there are other large dimension retaining wall block products available that have similar properties to the blocks used in this assessment. As such an alternative product may be proposed by the contractor awarded the construction of the retaining wall. Prior to construction a detailed engineering design (shop drawings) of the retaining wall incorporating final proposed grades and selected retaining wall block product should be provided by the contractor selected to construct the retaining wall for review and approval by the project engineer.

The wall at sections 7055, 7065 and 7077 was designed with two 24-86 block units followed by three 24SF block units and 1 6SF block Unit. There was is no geogrid reinforcement required. The stability of the retaining wall against overturning, sliding, bearing capacity failure, internal overturning and internal sliding for both normal and Seismic conditions were calculated by means of a copy writed program provided by Stone Strong Systems and used with permission from Stone Strong Systems. Results are attached following the text of this report. Results were verified by the undersigned using an excel spread sheet program.

The assessment of the retaining wall at the above sections produced the following factors of safety.

Against Overturning	– 2.1 to 2.3
Against Sliding	– 2.1 to 2.2
Against Bearing Capacity Failure	– 2.0
Against Internal Overturning	– 2.1
Against Internal Sliding	– 2.2 to 2.3
Against Seismic Overturning	– 2.1 to 2.2
Against Seismic Sliding	– 2.3
Against Seismic Internal Overturning	– 1.9 to 2.2
Against Seismic Internal Sliding	– 2.4 to 2.5

The wall at section 7089 was designed with two 24SF block units and 1 6SF block Unit. There was is no geogrid reinforcement required.

The assessment of the retaining wall at the above section produced the following factors of safety.

Against Overturning	– 3.8
Against Sliding	– 3.2
Against Bearing Capacity Failure	– 6.5
Against Internal Overturning	– 3.7
Against Internal Sliding	– 3.8
Against Seismic Overturning	– 2.7
Against Seismic Sliding	– 2.7
Against Seismic Internal Overturning	– 2.7
Against Seismic Internal Sliding	– 3.1

Since all of the factors of safety in the assessment against overturning and sliding are above 2.0 and the allowable bearing capacity is in excess of the pressure on the subgrade due to the wall, the retaining wall as designed is considered stable and adequate for the proposed design.

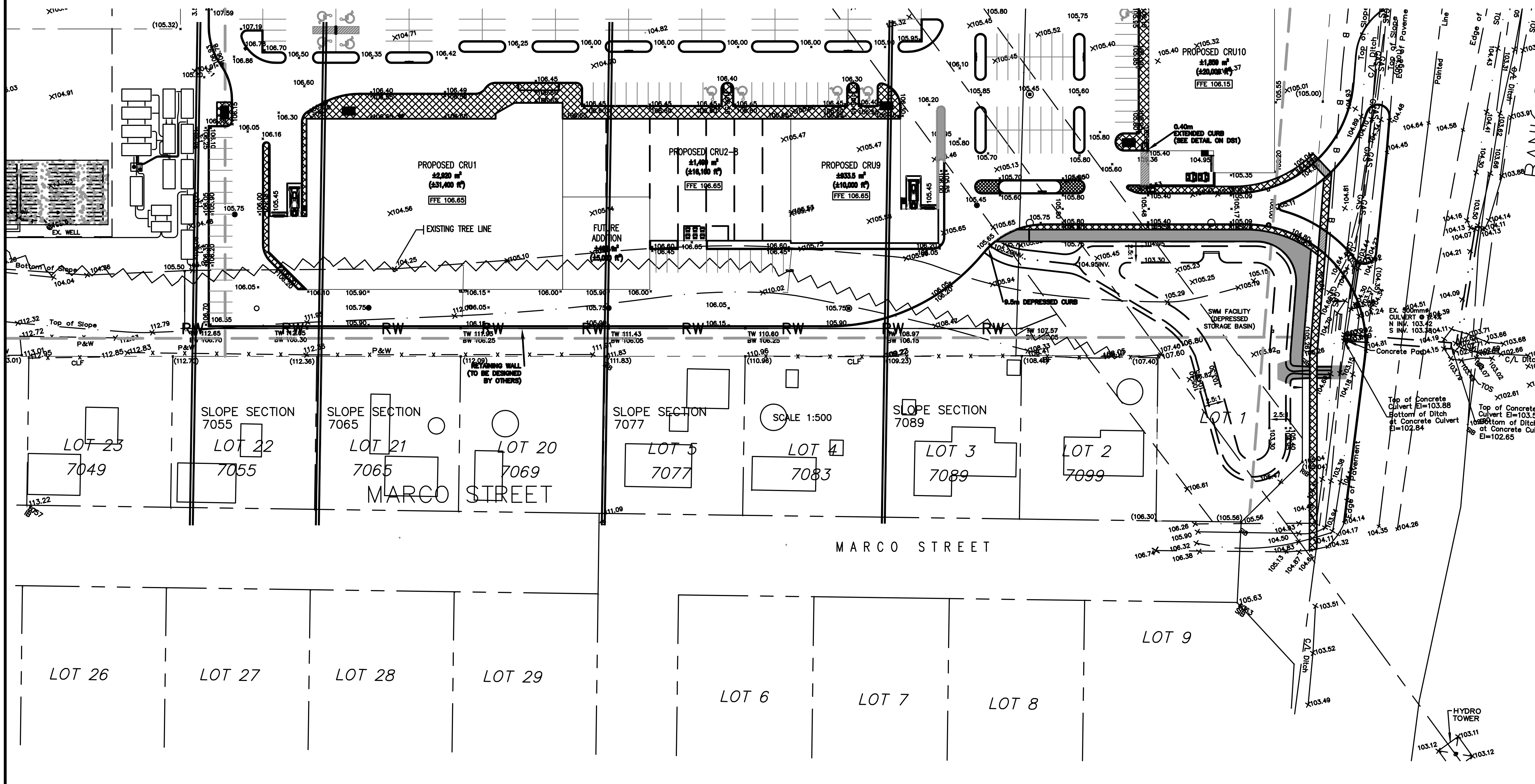
SLOPE STABILITY AND RETAINING WALL DESIGN RESULTS

Based on our field observations, review of the available information and the results of the slope stability analysis and retaining wall design assessment, it is stated that the existing slope along the south side of the site is stable in both the present geometry and in the proposed geometry. The proposed retaining wall is suitable to support the intended slope cut and is adequate for the proposed grading plan design as amended with the proposed slope cut above the retaining wall.

We trust that this report provides sufficient information for your present purposes. If you have any questions concerning this information or if we can be of further assistance to you, please do not hesitate to contact our office.

Regards,
Kollaard Associates Inc.

Steve DeWit, P.Eng



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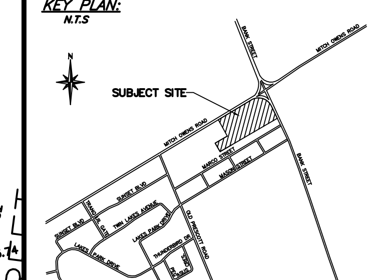
KOLLAARD ASSOCIATES INCORPORATED

DRAWING NUMBER:
140208-PLAN

KEY PLAN:
N.T.S



NOTES:

1. THE PLAN PROVIDED IN THIS DRAWING HAS BEEN OBTAINED FROM WMI & ASSOCIATES LIMITED.
2. THE PLAN HAS BEEN REFERENCED INTO THIS DRAWING WITH NO MODIFICATIONS TO THE EXISTING TOPOGRAPHY, BUILDING LOCATION OR RETAINING WALL LOCATION.
3. THE PROPOSED BOTTOM OF WALL ELEVATION IS NOT INTENDED TO BE MODIFIED BY THE INFORMATION PROVIDED ON THIS DRAWING.
4. IT IS PROPOSED TO REDUCE THE HEIGHT OF THE PROPOSED RETAINING WALL BY CUTTING THE SLOPE ABOVE THE RETAINING WALL.
5. THE CUT ILLUSTRATED ABOVE THE RETAINING WALL IS APPROXIMATE AND WILL VARY ALONG THE LENGTH OF THE RETAINING WALL.
6. THE PROPOSED SECTIONS HAVE BEEN PROVIDED TO ILLUSTRATE THE GLOBAL STABILITY FOR THE RETAINING WALL AT EACH SECTION INDICATED.
7. THE MINIMUM FACTOR OF SAFETY WITH RESPECT TO GLOBAL STABILITY OF THE RETAINING WALL IS FS = 1.6. AS SUCH THE RETAINED SLOPE IS CONSIDERED STABLE.
8. THE STRUCTURE LOCATIONS INDICATED ON THE ADJACENT PROPERTIES WERE OBTAINED FROM AERIAL PHOTOGRAPHY AND CONFIRMED BY SURVEYING.
9. THE STRUCTURE ON LOT 5 STREET ADDRESS 7077 IS A LIGHTLY LOADED GARAGE/SHED AND IS LOCATED GREATER THAN 2 METRES BEYOND THE MINIMUM SLIP SURFACE.

LICENSED PROFESSIONAL ENGINEER
MAY 05 2014
S.E. deWit
100079612
PROVINCE OF ONTARIO

REV.	NAME	DATE	DESCRIPTION
1	sd	Oct. 27, 2014	response to comments

Kollaard Associates
Engineers
PO, BOX 189, 210 PRESCOTT ST KEMPTVILLE ONTARIO K0G 1J0
(613) 860-0923 FAX (613) 258-0475
info@kollaard.ca
http://www.kollaard.ca

CLIENT:
ALIUM INVESTMENTS LTD

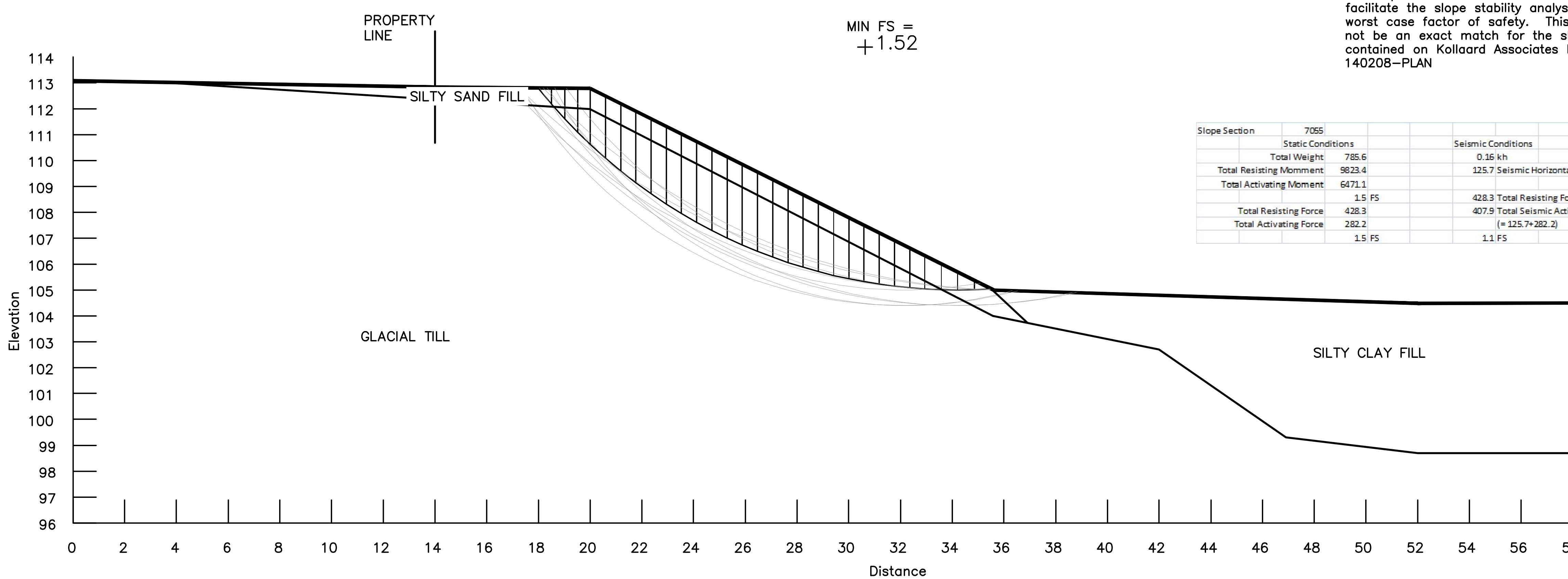
PROJECT:
PROPOSED GREELY COMMERCIAL CENTER

LOCATION:
5640 BANK STREET, 7107 MARCO STREET, 701 MITCH OWENS ROAD GREELY, CITY OF OTTAWA, ONTARIO

DESIGNED BY:	DATE:
SD	MAY 5, 2014
DRAWN BY:	SCALE:
SD	Not to Scale

KOLLAARD FILE NUMBER:
140208

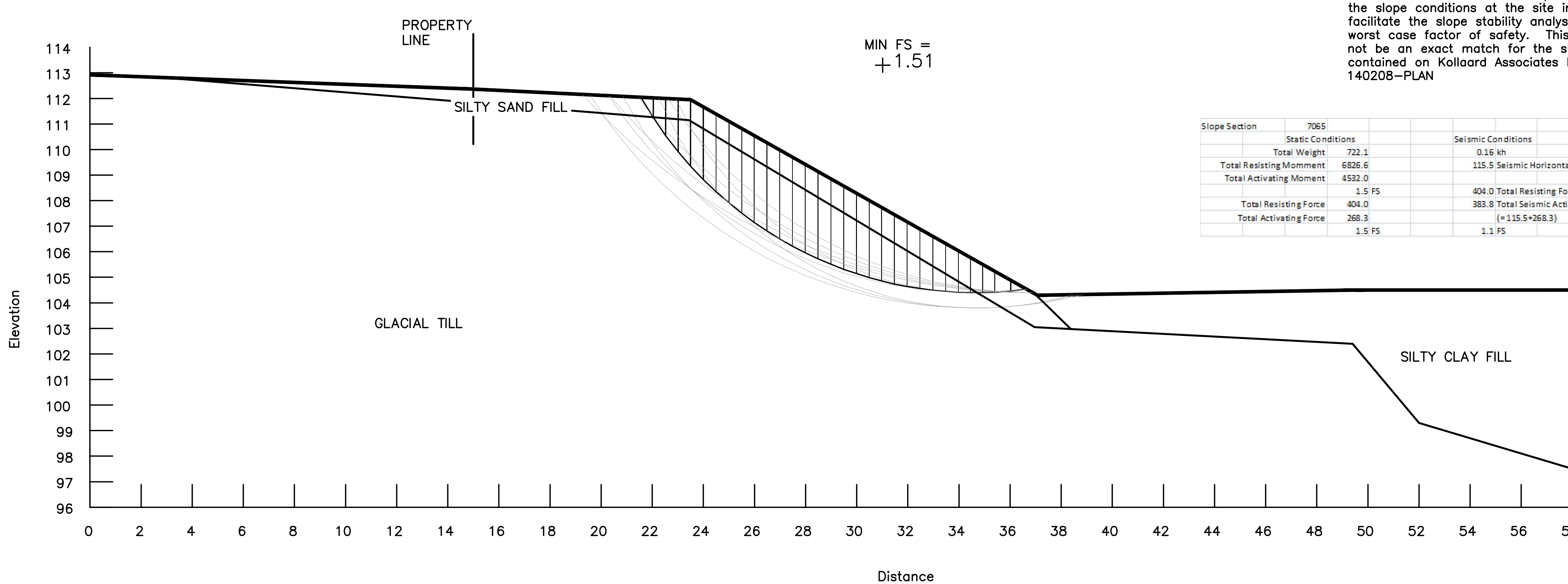
These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208–PLAN



Slope Section	7055				
	Static Conditions			Seismic Conditions	
Total Weight	785.6			0.16	kh
Total Resisting Momment	9823.4			125.7	Seismic Horizontal Force
Total Activating Moment	6471.1				
	1.5	FS		428.3	Total Resisting Force
Total Resisting Force	428.3			407.9	Total Seismic Activating Force
Total Activating Force	282.2				(= 125.7+282.2)
	1.5	FS		1.1	FS

SECTION 7055

These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208–PLAN



Slope Section	7065				
	Static Conditions			Seismic Conditions	
Total Weight	722.1			0.16	kh
Total Resisting Momment	6826.6			115.5	Seismic Horizontal Force
Total Activating Moment	4532.0				
	1.5	FS		404.0	Total Resisting Force
Total Resisting Force	404.0			383.8	Total Seismic Activating Force
Total Activating Force	268.3				(=115.5+268.3)
	1.5	FS		1.1	FS

SECTION 7065

DRAWING NUMBER:
140208 –SL1

CONSTRUCTION NOTES

1. Dimensions and elevations are in metric as shown. Do not scale drawing.

2. Any changes made to drawings must be verified and approved by Kollaard Associates Inc.

3. This drawing is intended to indicate the minimum factor of safety at the indicated sections along the slope at the south side of the site known as 5640 Bank Street, 7107 Marco Street, and 701 Mitch Owens Road.

REV.	NAME	DATE	DESCRIPTION
1.	SD	OCT.27.14	ADD ADDITIONAL INFORMATION

Kollaard Associates
Engineers

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K0G 1J0 FAX (613) 258–0475
http://www.kollaard.ca
(613) 860–0923
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CLIENT:
Alium Investments Ltd

PROJECT:
Slope Stability Analysis

DRAWING:
Minimum Factor of Safety

LOCATION:
Proposed Greely Commercial Center
Ottawa, Ontario
City of Ottawa File #
D07–12014–0038

DESIGNED BY:	DATE:
sd	MAY 5, 2014
DRAWN BY: sd	SCALE: 1:200

KOLLAARD FILE NUMBER:
140208

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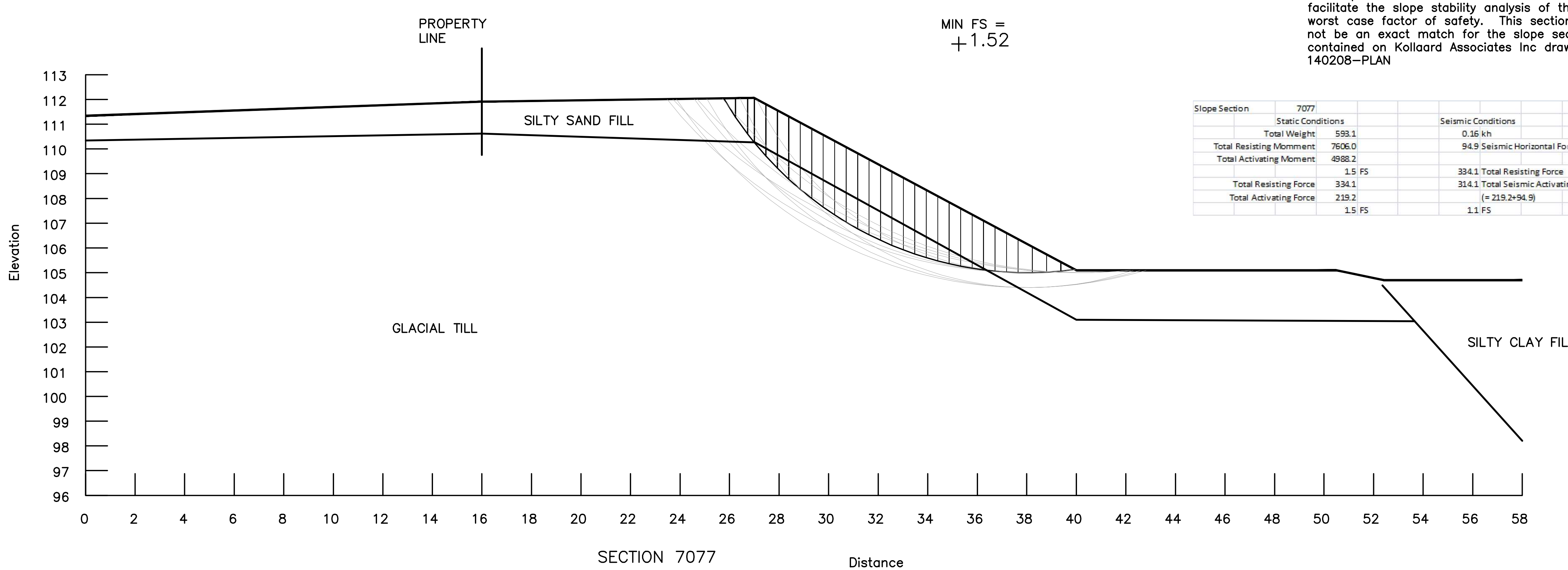
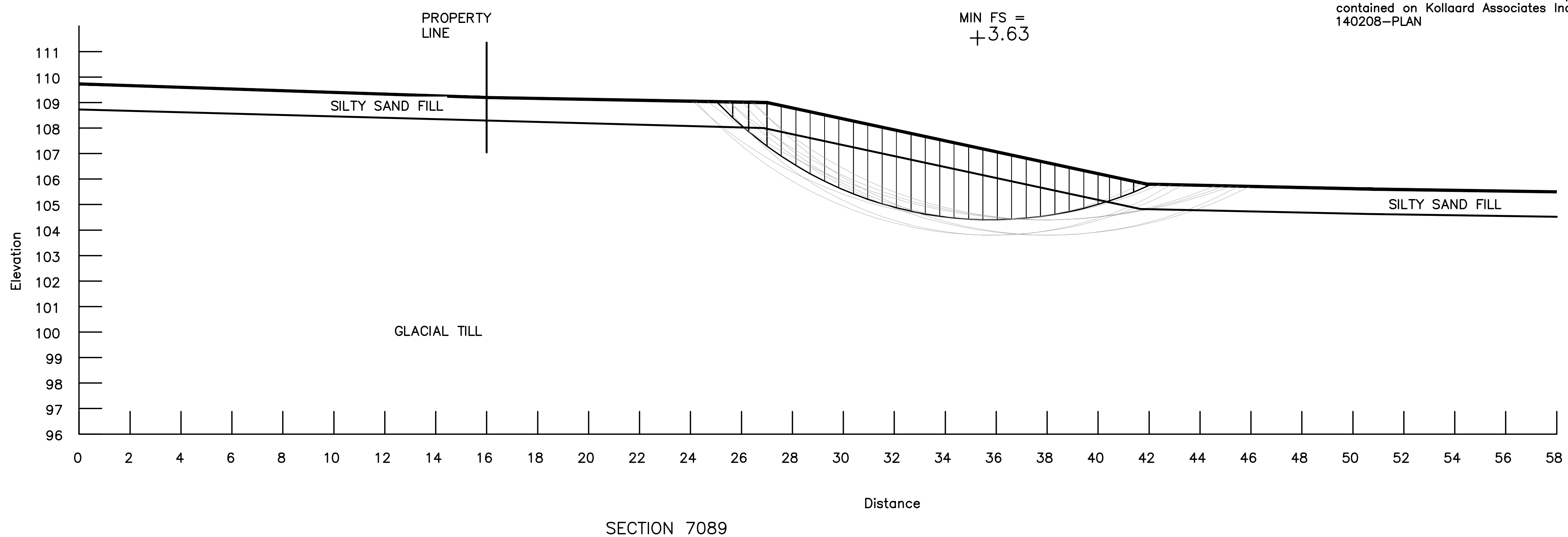


Slope Section	7077			
Static Conditions			Seismic Conditions	
Total Weight	593.1		0.16	kh
Total Resisting Momment	7606.0		94.9	Seismic Horizontal Force
Total Activating Moment	4988.2			
	1.5	FS	334.1	Total Resisting Force
Total Resisting Force	334.1		314.1	Total Seismic Activating Force
Total Activating Force	219.2			(= 219.2+94.9)
	1.5	FS	1.1	FS

SECTION 7077

These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208—PLAN



SECTION 7089

DRAWING NUMBER: 140208 —SL2

CONSTRUCTION NOTES

1. Dimensions and elevations are in metric as shown. Do not scale drawing.

2. Any changes made to drawings must be verified and approved by Kollaard Associates Inc.

3. This drawing is intended to indicate the minimum factor of safety at the indicated sections along the slope at the south side of the site known as 5640 Bank Street, 7107 Marco Street, and 701 Mitch Owens Road.

LICENSED PROFESSIONAL ENGINEER
OCT.27.2014
S.E. deWit
100079612
PROVINCE OF ONTARIO

REV.	NAME	DATE	DESCRIPTION
1.	SD	OCT.27.14	ADD ADDITIONAL INFORMATION

Kollaard Associates
Engineers
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K0G 1J0 FAX (613) 258—0475
http://www.kollaard.ca
(613) 860—0923
info@kollaard.ca

CLIENT: Alium Investments Ltd

PROJECT: Slope Stability Analysis

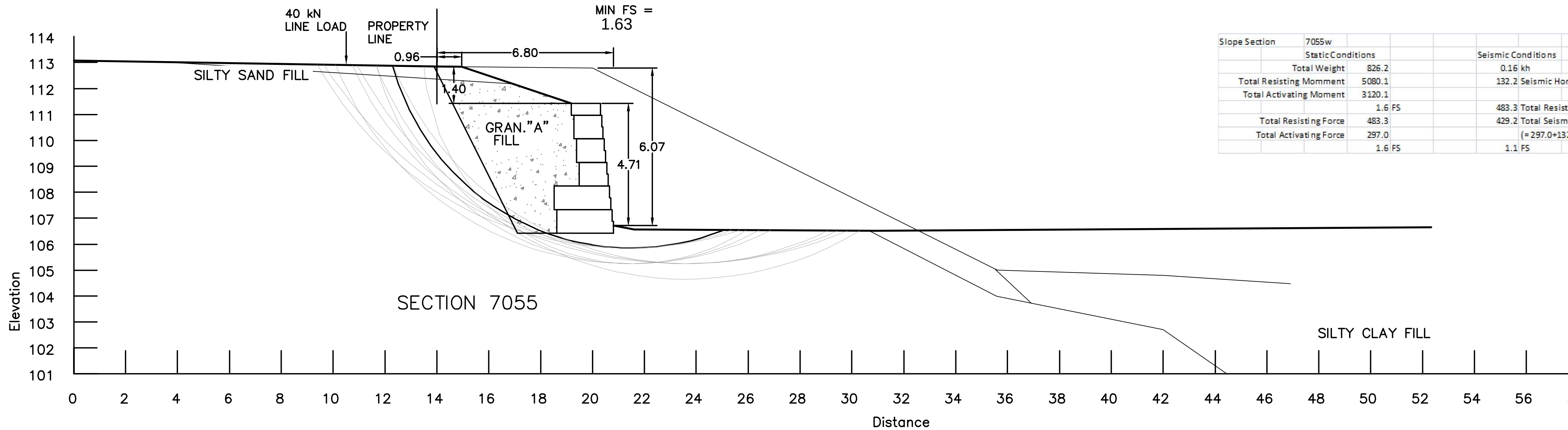
DRAWING: Minimum Factor of Safety

LOCATION: Proposed Greely Commercial Center Ottawa, Ontario City of Ottawa File # D07—12014—0038

DESIGNED BY:	DATE:
sd	MAY 5, 2014
DRAWN BY: sd	SCALE: 1:200

KOLLAARD FILE NUMBER: 140208

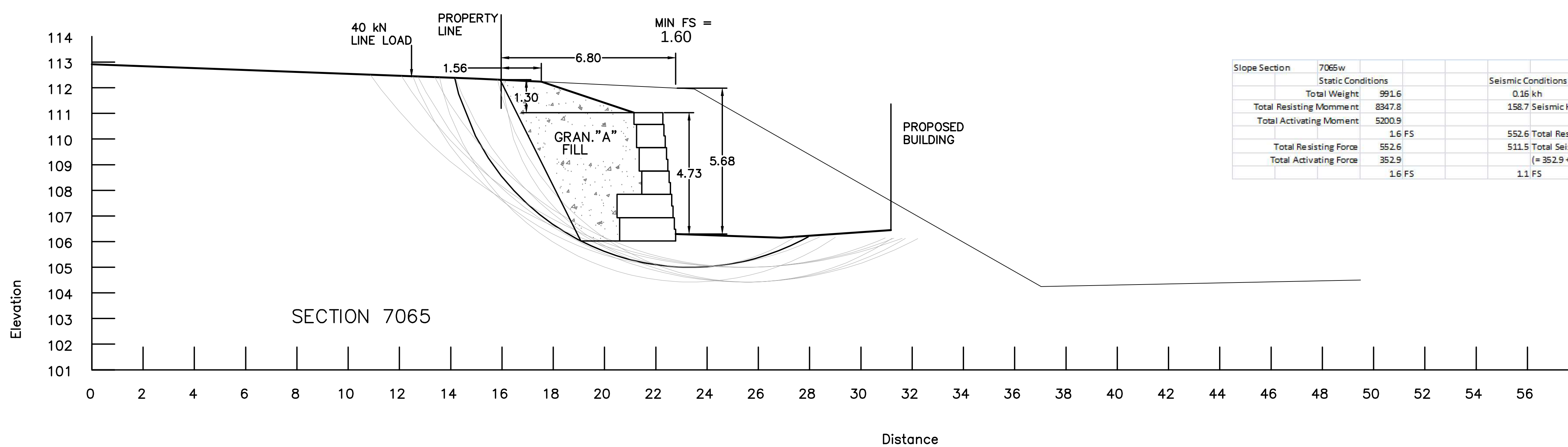
These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208–PLAN



Slope Section	7055w				
	Static Conditions			Seismic Conditions	
Total Weight	826.2			0.16	kh
Total Resisting Momment	5080.1			132.2	Seismic Horizontal Force
Total Activating Moment	3120.1				
	1.6	FS		483.3	Total Resisting Force
Total Resisting Force	483.3			429.2	Total Seismic Activating Force
Total Activating Force	297.0				(=297.0+132.2)
	1.6	FS		1.1	FS

SECTION 7055

These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208–PLAN



Slope Section	7065w				
	Static Conditions			Seismic Conditions	
Total Weight	991.6			0.16	kh
Total Resisting Momment	8347.8			158.7	Seismic Horizontal Force
Total Activating Moment	5200.9				
	1.6	FS		552.6	Total Resisting Force
Total Resisting Force	552.6			511.5	Total Seismic Activating Force
Total Activating Force	352.9				(= 352.9 + 158.7)
	1.6	FS		1.1	FS

SECTION 7065

DRAWING NUMBER: 140208 –SL3

CONSTRUCTION NOTES

1. Dimensions and elevations are in metric as shown. Do not scale drawing.

2. Any changes made to drawings must be verified and approved by Kollaard Associates Inc.

3. This drawing is intended to indicate the minimum factor of safety at the indicated sections along the slope at the south side of the site known as 5640 Bank Street, 7107 Marco Street, and 701 Mitch Owens Road.

REV.	NAME	DATE	DESCRIPTION
1.	SD	OCT.27.14	ADD ADDITIONAL INFORMATION

Kollaard Associates
Engineers
PO, BOX 189, 210 PRESCOTT ST
KEMPTVILLE ONTARIO
K0G 1J0 FAX (613) 258–0475
http://www.kollaard.ca
(613) 860–0923
info@kollaard.ca

CLIENT: *Alium Investments Ltd*

PROJECT: *Slope Stability Analysis*

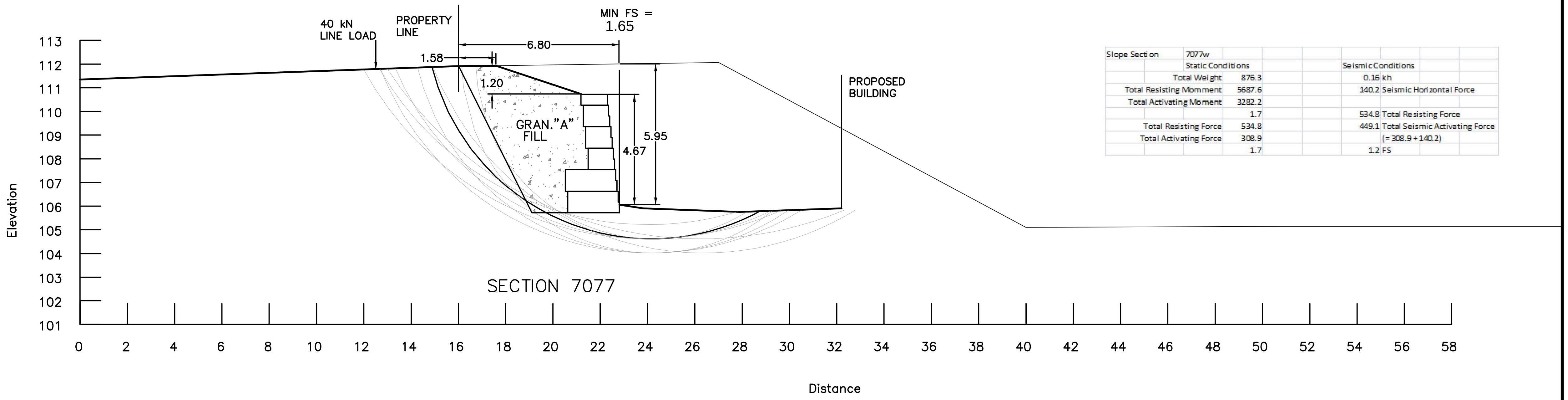
DRAWING: *Minimum Factor of Safety*

LOCATION: *Proposed Greely Commercial Center Ottawa, Ontario City of Ottawa File # D07–12014–0038*

DESIGNED BY:	*DATE:*
sd	MAY 5, 2014
DRAWN BY: sd	*SCALE:* 1:200

KOLLAARD FILE NUMBER: 140208

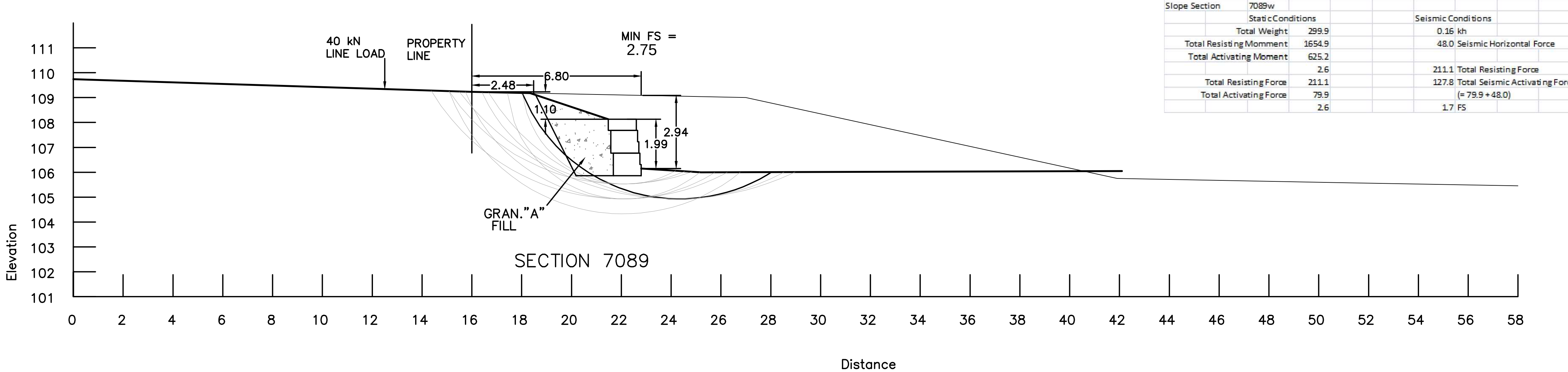
These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208–PLAN



Slope Section	7077w		
	Static Conditions	Seismic Conditions	
Total Weight	876.3	0.16	kh
Total Resisting Momment	5687.6	140.2	Seismic Horizontal Force
Total Activating Moment	3282.2		
	1.7	534.8	Total Resisting Force
Total Resisting Force	534.8	449.1	Total Seismic Activating Force
Total Activating Force	308.9		(= 308.9 + 140.2)
	1.7	1.2	FS

SECTION 7077

These sections shown are simplified to represent the slope conditions at the site in order facilitate the slope stability analysis of the worst case factor of safety. This section may not be an exact match for the slope section contained on Kollaard Associates Inc drawing 140208–PLAN



Slope Section	7089w		
	Static Conditions	Seismic Conditions	
Total Weight	299.9	0.16	kh
Total Resisting Momment	1654.9	48.0	Seismic Horizontal Force
Total Activating Moment	625.2		
	2.6	211.1	Total Resisting Force
Total Resisting Force	211.1	127.8	Total Seismic Activating Force
Total Activating Force	79.9		(= 79.9 + 48.0)
	2.6	1.7	FS

SECTION 7089

DRAWING NUMBER:
140208 –SL4

CONSTRUCTION NOTES

1. Dimensions and elevations are in metric as shown. Do not scale drawing.

2. Any changes made to drawings must be verified and approved by Kollaard Associates Inc.

3. This drawing is intended to indicate the minimum factor of safety at the indicated sections along the slope at the south side of the site known as 5640 Bank Street, 7107 Marco Street, and 701 Mitch Owens Road.

LICENSED PROFESSIONAL ENGINEER
OCT.27.2014
S.E. deWit
100079612
PROVINCE OF ONTARIO

REV.	NAME	DATE	DESCRIPTION
1.	SD	OCT.27.14	ADD ADDITIONAL INFORMATION

Kollaard Associates
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K0G 1J0 FAX (613) 258–0475
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(613) 860–0923
info@kollaard.ca

CLIENT:
Alium Investments Ltd

PROJECT:
Slope Stability Analysis

DRAWING:
Minimum Factor of Safety

LOCATION:
Proposed Greely
Commercial Center
Ottawa, Ontario
City of Ottawa File #
D07–12014–0038

DESIGNED BY: sd
DATE: MAY 5, 2014
DRAWN BY: sd
SCALE: 1:200
KOLLAARD FILE NUMBER:
140208

STONE STRONG GRAVITY CALCULATIONS - ver 4.9

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

STONE STRONG® systems

10/27/14 10:54

Notes

Gravity Analysis slope cut above wall.

Wall Configuration

block	w (mm)	h (m)	setback (mm) face	setback (mm) tail	block units W_b (kN)	block units x_b (mm)	unit fill W_a (kN)	unit fill x_a (mm)	soil wedge W_s (kN)	soil wedge x_s (mm)	CIP Extension w_e (mm)	CIP Extension h_t
6	1118	0.46	505	-561	5.8	988	5.2	1,102	0.4	1,605		
24	1118	0.91	404	-662	10.9	892	10.7	1,034	5.3	1,622		
24	1118	0.91	303	-763	10.9	791	10.7	933	10.6	1,664		
24	1118	0.91	202	-864	10.9	690	10.7	832	15.9	1,707		
24-86	2184	0.91	101	101	14.0	1,067	28.1	1,247	0.0	0		
24-86	2184	0.91	0	0	14.0	966	28.1	1,146	0.0	0		
OK!	2184	5.03	505	-561	66.7	903	93.3	1101	32.3	1677		

backfill height 5.03 m   ω= 6.31 deg
exposed height 4.80 m   ω'= -6.37 deg

Retained Soil
γ 20.00 kN/m³   interface friction angle
ϕ 38 deg   δ 28.5 deg

Aggregate Unit Fill
γ 21 kN/m³

Foundation Soil
allowable bearing pressure (net) 250 kPa (if specified)
γ 20.00 kN/m³
c' kPa
ϕ 38 deg
base embedment 225 mm
base thickness 225 mm
agg/conc/reinf base agg μ_b 0.69

Seismic Load PGA 0.32 G   kh 0.15

Toe Slope H:1V slope

Backfill Slope & Surcharge

		backslope		LL surcharge	
length 1	4 m (horizontal)	3.00	H:1V slope		kPa
length 2	2 m (horizontal)		H:1V slope		kPa
length 3	0.5 m (horizontal)		H:1V slope	80.00	kPa
length 4	5 m (horizontal)		H:1V slope		kPa

Tier Height
m
m
m

effective slope 6.15 H:1V slope   failure plane α 53.44 deg   avg LL q 8 kPa
β 9.2 deg   zone of influence 6.50 m

STONE STRONG GRAVITY CALCULATIONS - ver 4.9

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

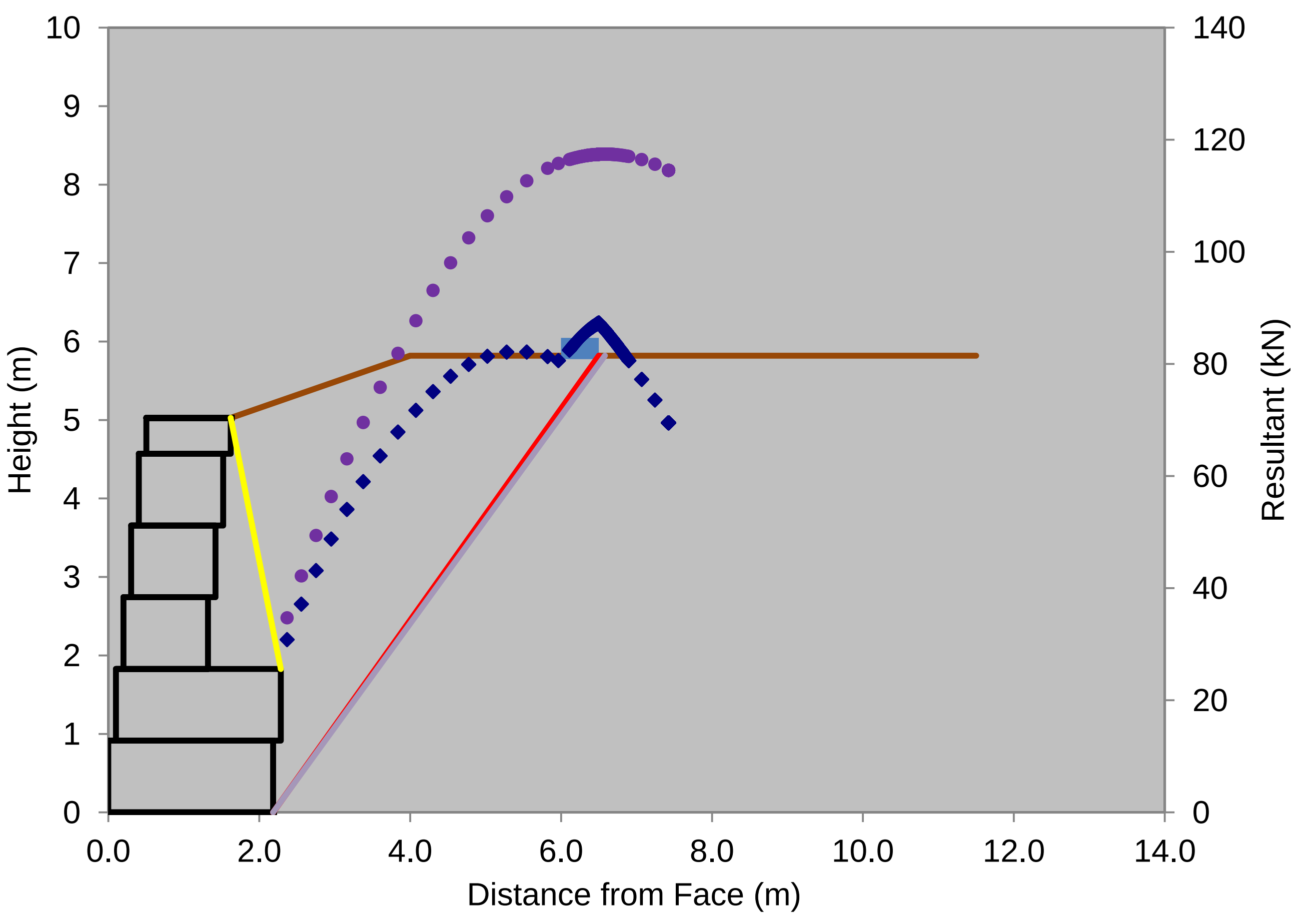
Analysis

		Q_{lh} =	9.26 kN			e=	0.30 m
K_a =	0.301	Q_{lv} =	6.45 kN	ΔK_{AE} =	0.164	B_f' =	1.76 m
P_h =	62.40 kN	R_s =	162.91 kN	P_{IR} =	28.88 kN	e_{eq}=	0.20 m
P_v =	43.48 kN	q_{ult} =	1,813 kPa	P_{AEh} =	33.97 kN	$B_f'_{eq}$ =	1.96 m

Results

Overturning:	Desired FS = 1.5		**Actual FS= 2.21**	***OK!***
Sliding:	Desired FS = 1.5		**Actual FS= 2.27**	***OK!***
Bearing Capacity: (net)	q_{all} = 250 kPa		q_c = 133 kPa	***OK!***
Seismic Overturning:	Desired FS = 1.13		**Actual FS= 2.30**	***OK!***
Seismic Sliding:	Desired FS = 1.13		**Actual FS= 2.42**	***OK!***
Seismic Bearing: (net)	q_{all} = 333 kPa		q_c = 133 kPa	***OK!***

Ground Surface & Trial Wedge Plot



STONE STRONG GRAVITY CALCULATIONS - ver 4.9

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

STONE STRONG® systems

10/27/14 10:56

Notes

Gravity Analysis slope cut above wall.

Wall Configuration

block	w (mm)	h (m)	setback (mm) face	setback (mm) tail	block units W_b (kN)	block units x_b (mm)	unit fill W_a (kN)	unit fill x_a (mm)	soil wedge W_s (kN)	soil wedge x_s (mm)	CIP Extension w_e (mm)	CIP Extension h_t
6	1118	0.46	505	-561	5.8	988	5.2	1,102	0.4	1,605		
24	1118	0.91	404	-662	10.9	892	10.7	1,034	5.3	1,622		
24	1118	0.91	303	-763	10.9	791	10.7	933	10.6	1,664		
24	1118	0.91	202	-864	10.9	690	10.7	832	15.9	1,707		
24-86	2184	0.91	101	101	14.0	1,067	28.1	1,247	0.0	0		
24-86	2184	0.91	0	0	14.0	966	28.1	1,146	0.0	0		
OK!	2184	5.03	505	-561	66.7	903	93.3	1101	32.3	1677		

backfill height 5.03 m — ω= 6.31 deg
exposed height 4.73 m — ω'= -6.37 deg

Retained Soil

γ 20.00 kN/m^3 — interface friction angle
ϕ 38 deg — δ 28.5 deg

Aggregate Unit Fill

γ 21 kN/m^3

Foundation Soil

γ 22.00 kN/m^3 — base embedment 300 mm
allowable bearing pressure — c' kPa — base thickness 225 mm
(net) 250 kPa (if specified) — ϕ 38 deg — agg/conc/reinf base agg — μ_b 0.69

Seismic Load PGA 0.32 G — kh 0.15

Toe Slope H:1V slope

Backfill Slope & Surcharge

		backslope	LL surcharge
length 1	4 m (horizontal)	3.00 H:1V slope	kPa
length 2	2 m (horizontal)	H:1V slope	kPa
length 3	0.5 m (horizontal)	H:1V slope	80.00 kPa
length 4	5 m (horizontal)	H:1V slope	kPa

Tier Height
m
m
m

effective slope 6.15 H:1V slope — failure plane α 53.44 deg — avg LL q 8 kPa
β 9.2 deg — zone of influence 6.50 m

STONE STRONG GRAVITY CALCULATIONS - ver 4.9

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

10/27/14 10:56

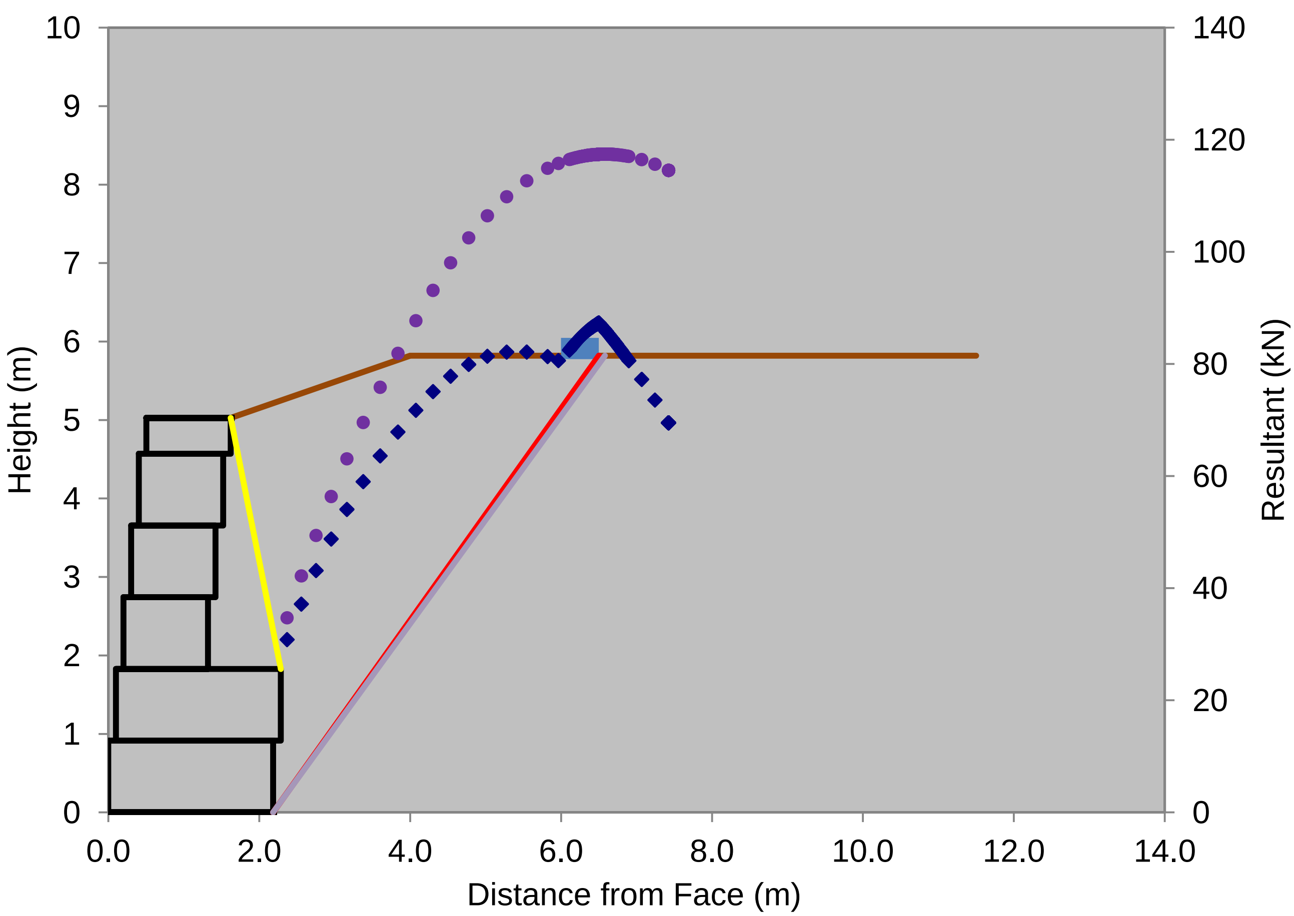
Analysis

		Q_{lh} =	9.26 kN			e=	0.30 m
K_a =	0.301	Q_{lv} =	6.45 kN	ΔK_{AE} =	0.164	B_f' =	1.76 m
P_h =	62.40 kN	R_s =	162.91 kN	P_{IR} =	28.88 kN	e_{eq}=	0.20 m
P_v =	43.48 kN	q_{ult} =	2,075 kPa	P_{AEh} =	33.97 kN	$B'_{f\,eq}$ =	1.96 m

Results

Overturning:	Desired FS =	1.5	**Actual FS=**	**2.21**	***OK!***
Sliding:	Desired FS =	1.5	**Actual FS=**	**2.27**	***OK!***
Bearing Capacity:					
(net)	q_{all} =	250 kPa	q_c =	131 kPa	***OK!***
Seismic Overturning:	Desired FS =	1.13	**Actual FS=**	**2.30**	***OK!***
Seismic Sliding:	Desired FS =	1.13	**Actual FS=**	**2.42**	***OK!***
Seismic Bearing:					
(net)	q_{all} =	333 kPa	q_c =	131 kPa	***OK!***

Ground Surface & Trial Wedge Plot



STONE STRONG GRAVITY CALCULATIONS - ver 4.9

STONE STRONG systems®

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

10/27/14 10:56

Notes

Gravity Analysis slope cut above wall.

Wall Configuration

block	w (mm)	h (m)	setback (mm) face	setback (mm) tail	block units W_b (kN)	block units x_b (mm)	unit fill W_a (kN)	unit fill x_a (mm)	soil wedge W_s (kN)	soil wedge x_s (mm)	CIP Extension w_e (mm)	CIP Extension h_t
6	1118	0.46	505	-561	5.8	988	5.2	1,102	0.4	1,605		
24	1118	0.91	404	-662	10.9	892	10.7	1,034	5.3	1,622		
24	1118	0.91	303	-763	10.9	791	10.7	933	10.6	1,664		
24	1118	0.91	202	-864	10.9	690	10.7	832	15.9	1,707		
24-86	2184	0.91	101	101	14.0	1,067	28.1	1,247	0.0	0		
24-86	2184	0.91	0	0	14.0	966	28.1	1,146	0.0	0		
OK!	2184	5.03	505	-561	66.7	903	93.3	1101	32.3	1677		

backfill height 5.03 m — ω= 6.31 deg
exposed height 4.80 m — ω'= -6.37 deg

Retained Soil

γ 20.00 kN/m^3 — interface friction angle
ϕ 38 deg — δ 28.5 deg

Aggregate Unit Fill

γ 21 kN/m^3

Foundation Soil

γ 20.00 kN/m^3 — base embedment 225 mm
allowable bearing pressure — c' kPa — base thickness 225 mm
(net) 250 kPa (if specified) — ϕ 38 deg — agg/conc/reinf base agg — μ_b 0.69

Seismic Load

PGA 0.32 G — kh 0.15

Toe Slope

H:1V slope

Backfill Slope & Surcharge

		backslope	LL surcharge	Tier Height
length 1	4 m (horizontal)	3.00 H:1V slope	kPa	m
length 2	2 m (horizontal)	H:1V slope	kPa	m
length 3	0.5 m (horizontal)	H:1V slope	80.00 kPa	m
length 4	5 m (horizontal)	H:1V slope	kPa	

effective slope 6.15 H:1V slope — failure plane α 53.44 deg — avg LL q 8 kPa
β 9.2 deg — zone of influence 6.50 m

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

STONE STRONG systems ®

10/27/14 10:56

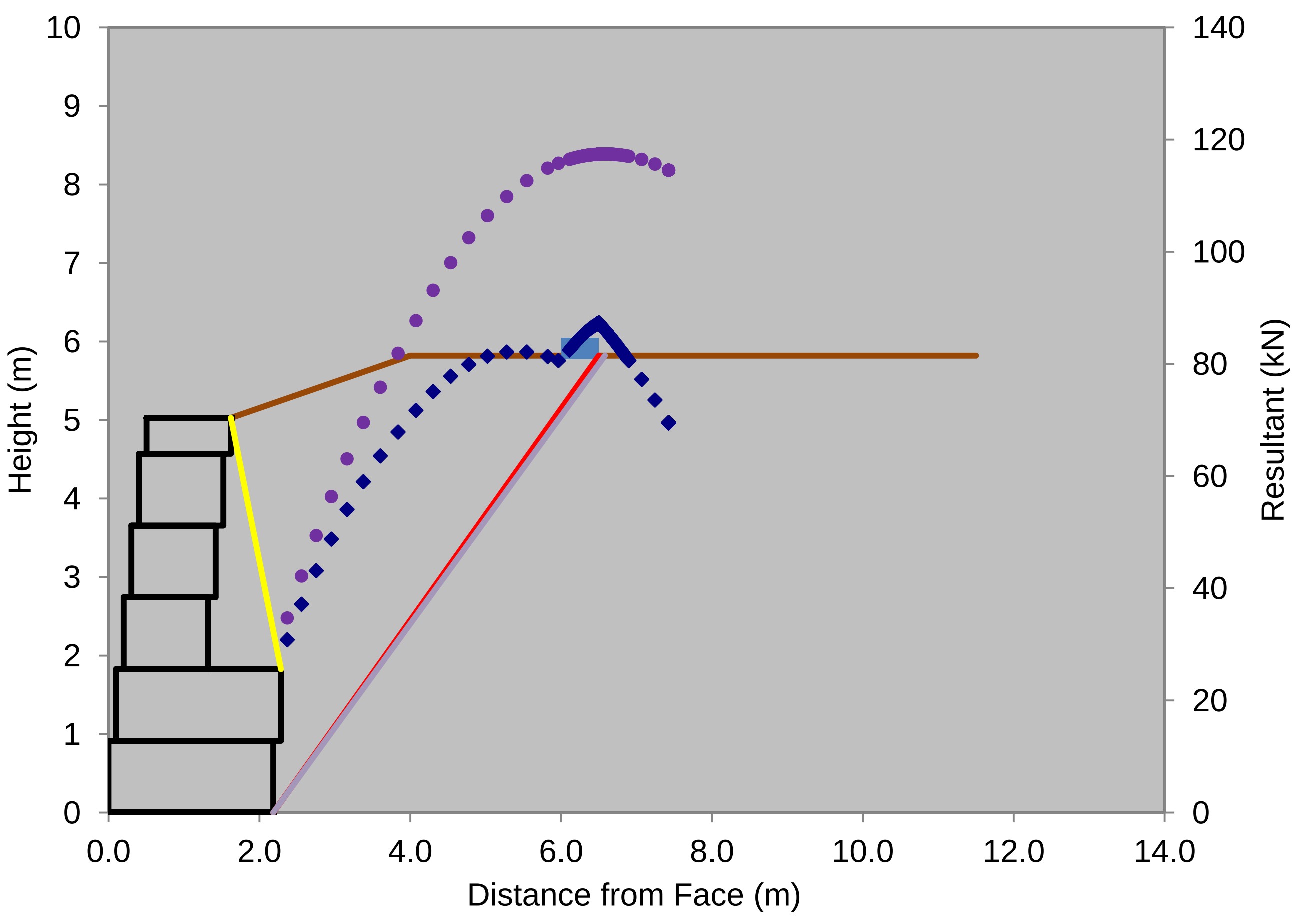
Analysis

		Q_{lh} =	9.26 kN			e=	0.30 m
K_a =	0.301	Q_{lv} =	6.45 kN	ΔK_{AE} =	0.164	B_f' =	1.76 m
P_h =	62.40 kN	R_s =	162.91 kN	P_{IR} =	28.88 kN	e_{eq}=	0.20 m
P_v =	43.48 kN	q_{ult} =	1,813 kPa	P_{AEh} =	33.97 kN	$B_f'{}_{eq}$ =	1.96 m

Results

Overturning:	Desired FS =	1.5	**Actual FS=**	**2.21**	***OK!***
Sliding:	Desired FS =	1.5	**Actual FS=**	**2.27**	***OK!***
Bearing Capacity:					
(net)	q_{all} =	250 kPa	q_c =	133 kPa	***OK!***
Seismic Overturning:	Desired FS =	1.13	**Actual FS=**	**2.30**	***OK!***
Seismic Sliding:	Desired FS =	1.13	**Actual FS=**	**2.42**	***OK!***
Seismic Bearing:					
(net)	q_{all} =	333 kPa	q_c =	133 kPa	***OK!***

Ground Surface & Trial Wedge Plot



STONE STRONG GRAVITY CALCULATIONS - ver 4.9

STONE STRONG® systems

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

10/27/14 11:06

Notes

Wall Configuration

block	w (mm)	h (m)	setback (mm) face	setback (mm) tail	block units W_b (kN)	block units x_b (mm)	unit fill W_a (kN)	unit fill x_a (mm)	soil wedge W_s (kN)	soil wedge x_s (mm)	CIP Extension w_e (mm)	CIP Extension h_t
6	1118	0.46	202	202	5.8	685	4.9	799				
24	1118	0.91	101	101	10.9	589	10.2	731				
24	1118	0.91	0	0	10.9	488	10.2	630				
OK!	1118	2.29	202	202	27.7	570	25.2	704	0.0	0		

backfill height 2.29 m   ω= 6.31 deg
exposed height 2.06 m   ω'= 6.31 deg

Retained Soil
γ 20.00 kN/m³
φ 38 deg
interface friction angle δ 19.0 deg

Aggregate Unit Fill
γ 20 kN/m³

Foundation Soil
allowable bearing pressure (net) 250 kPa (if specified)
γ 21.00 kN/m³
c' kPa
φ 38 deg
base embedment 225 mm
base thickness 225 mm
agg/conc/reinf base agg μ_b 0.69

Seismic Load
PGA 0.32 G   kh 0.15

Toe Slope
H:1V slope

Backfill Slope & Surcharge

		backslope	LL surcharge
length 1	4 m (horizontal)	3.00 H:1V slope	kPa
length 2	6 m (horizontal)	H:1V slope	kPa
length 3	0.5 m (horizontal)	H:1V slope	80.00 kPa
length 4	m (horizontal)	H:1V slope	kPa

Tier Height
m
m
m

effective slope 3.00 H:1V slope   failure plane α 55.83 deg   avg LL q 0 kPa
β 18.4 deg   zone of influence 3.05 m

STONE STRONG GRAVITY CALCULATIONS - ver 4.9

STONE STRONG® systems

Project Name: Greely Commercial Center
Location: Greely Ontario
Job#: 140208
Section: Section 7055
Calc by: Steven deWit

Analysis			Q_{lh} =	0.00 kN			e=	0.03 m
	K_a =	0.218	Q_{lv} =	0.00 kN	ΔK_{AE} =	0.133	B_f' =	1.23 m
	P_h =	11.08 kN	R_s =	38.05 kN	P_{IR} =	7.96 kN	e_{eq}=	0.05 m
	P_v =	2.50 kN	q_{ult} =	1,474 kPa	P_{AEh} =	6.79 kN	$B'_{f\,eq}$ =	1.20 m

Results	**Overturning:**	Desired FS =	1.5	**Actual FS=**	**3.90**	***OK!***
	Sliding:	Desired FS =	1.5	**Actual FS=**	**3.43**	***OK!***
	Bearing Capacity:					
	(net)	q_{all} =	250 kPa	q_c =	40 kPa	***OK!***
	Seismic Overturning:	Desired FS =	1.13	**Actual FS=**	**2.75**	***OK!***
	Seismic Sliding:	Desired FS =	1.13	**Actual FS=**	**2.81**	***OK!***
	Seismic Bearing:					
	(net)	q_{all} =	333 kPa	q_c =	47 kPa	***OK!***

Ground Surface & Trial Wedge Plot