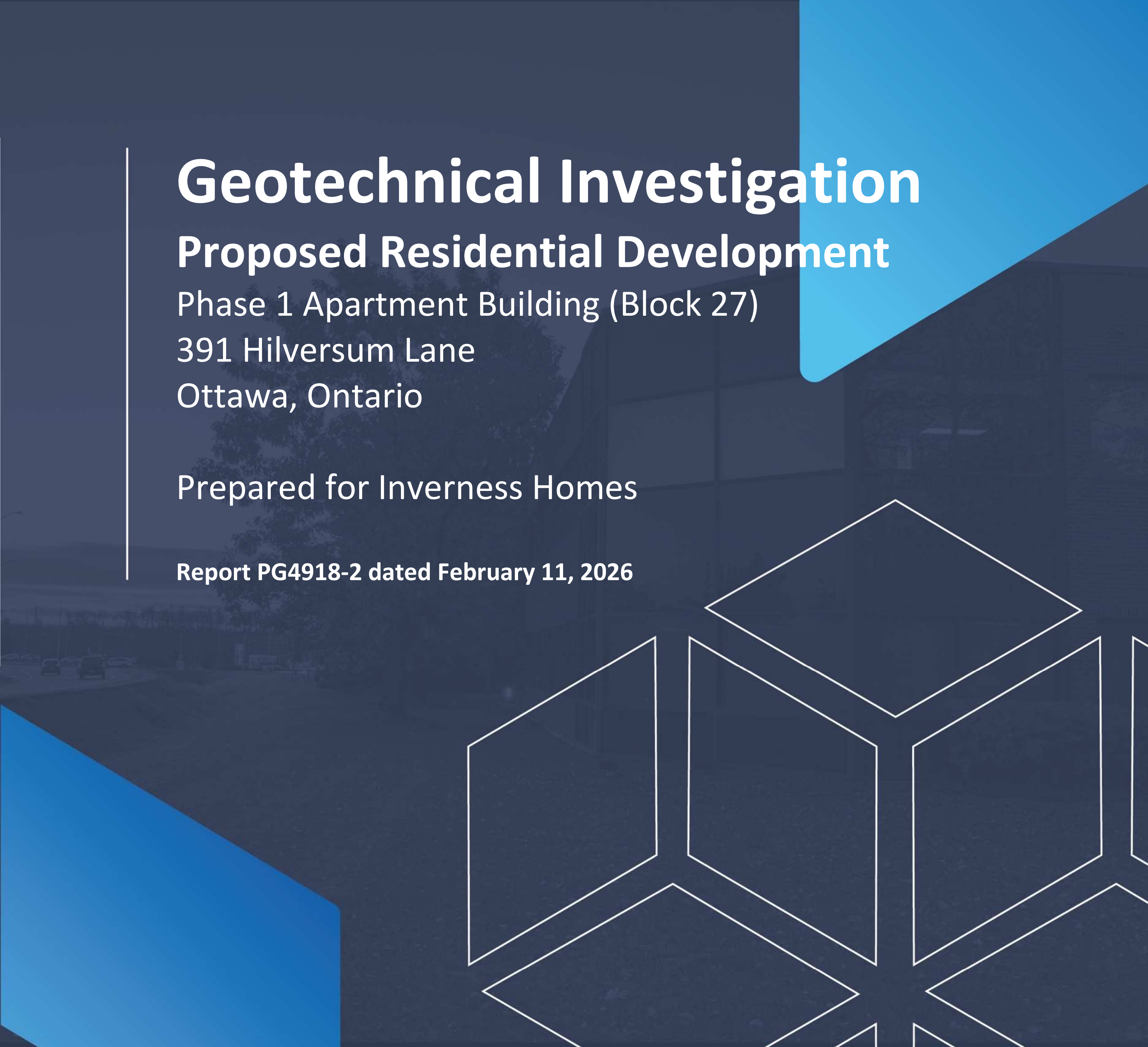
PATERSON GROUP



Geotechnical Investigation

Proposed Residential Development

Phase 1 Apartment Building (Block 27)
391 Hilversum Lane
Ottawa, Ontario

Prepared for Inverness Homes

Report PG4918-2 dated February 11, 2026

Table of Contents

Appendices

Appendix 1	Soil Profile & Test Data Sheets Symbols & Terms Groundwater Monitoring Results Atterberg Limits Results Grain Size Distribution & Hydrometer Testing Results Linear Shrinkage Testing Results Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Figures 10-13 & 46-49 & 56-58 - Slope Stability Analysis Sections Drawing PG4918-6 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Inverness Homes to conduct a geotechnical investigation for the proposed residential development to be located at 391 Hilversum Lane in the City of Ottawa (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsurface soil and groundwater conditions by means of boreholes.
- ☐ Provide geotechnical recommendations for the design of the proposed development, including construction considerations which may affect its design.

This report has been prepared specifically and solely for the aforementioned project which is described herein. The report contains the geotechnical findings and includes recommendations pertaining to the design and construction of the subject development, as understood at the time of writing this report.

2.0 Proposed Development

Based on the available plans, it is understood that the proposed Phase 1 development will consist of a 3-storey apartment building with 1 underground level. It is also understood that the development at this parcel will include associated access lanes, landscaped areas, and a one-storey clubhouse structure. It is further understood that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was carried out on January 27, 2026, and consisted of advancing 3 boreholes (BH 1-26 through BH 3-26) to a maximum depth of 9.75 m below the existing ground surface.

Previous geotechnical and hydrogeological field investigations were carried out at 147 Langstaff Drive in October 2008, July 2019, August 2023, and March 2024 by Paterson to inform the design of the subject residential development. A total of 8 boreholes from the previous investigations (BH 3, BH 4, BH 4-19, BH 5-19, BH 3-23, BH 4-23, BH 2-24, and BH 3-24), advanced to a maximum depth of 9.8 m below the existing ground surface, were located within, or in close proximity to, the subject Block 27 site.

The borehole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The borehole locations are presented on Drawing PG4918-6 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The borehole procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were either recovered directly from the auger flights (AU), or collected using a 50 mm diameter split-spoon (SS) sampler. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm. This testing was done in general accordance with ASTM D1586-11 - Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) during the current and previous investigations at borehole BH 3-26. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Subsurface conditions observed in the boreholes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the borehole locations.

Groundwater

Groundwater monitoring wells were installed in boreholes BH 1-26 and BH 2-26, and borehole BH 3-26 was fitted with a flexible PVC standpipe to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

The groundwater observations are discussed in Section 4.3 and are presented on the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and ground surface elevations at each borehole location, from the current geotechnical investigation (BH 1-26 through BH 3-26) and previous investigation (BH 3-23 and BH 4-23) were surveyed by Paterson using a high precision, handheld GPS unit and referenced to a geodetic datum.

The borehole locations and elevations from the geotechnical investigations conducted in 2019 and 2024 (BH 4-19 and BH 5-19, and BH 2-24 and BH 3-24) were surveyed in the field by Robinson Land Development. These boreholes are referenced to a geodetic datum.

The borehole elevations from the initial geotechnical investigation (BH 3 and BH 4) were inferred from surrounding spot elevations provided on the site survey, which are referenced to a geodetic datum. The locations of the boreholes are presented on Drawing PG4918-6 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the geotechnical investigations were examined in our laboratory to review the results of the field logging.

A total of 1 linear shrinkage analysis, 1 grain size distribution test, and 2 Atterberg limits tests were completed on selected soil samples recovered from the current investigation. Moisture content testing was also completed on all recovered soil samples from the current investigation. The results of the testing are presented in Section 4.2, and are also provided on the Soil Profile and Test Data sheets in Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.9.

4.0 Observations

4.1 Surface Conditions

The subject site located at Block 27 is currently vacant and undeveloped. An approximately 4 to 6 m deep ravine, with mature trees lining the top of the slope, is located approximately 15 to 25 m east of Block 27. The site is further bordered by Hilversum Lane and future residential development blocks to the south and west, and by future parklands to the north. The ground surface across Block 27 is relatively flat at an approximate geodetic elevation of 102 to 103 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations within, and in the surrounding area, of Block 27 consists of a silty clay deposit. It should be noted that the topsoil was stripped from this site in 2025.

The silty clay deposit generally consists of a hard to stiff silty clay extending to approximate depths of 4.6 to 5.9 m below the existing ground surface, underlain by a stiff to firm silty clay deposit. Silty sand to sandy silt seams were encountered throughout the silty clay deposit at various locations and depths throughout the borehole locations. A 1.1 m thick layer of sandy silt and a deposit of clayey silt were encountered directly above and below the silty clay deposit, respectively, at the location of borehole BH 3. Further, a deposit of silty sand was encountered underlying the silty clay at a depth of about 6 m below the existing ground surface at the locations of boreholes BH 1-26 and BH 4.

Bedrock

Practical refusal to the DCPT testing was encountered at a depth of 20.6 m below the existing ground surface in borehole BH 3-26. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, bedrock in the area consists of interbedded limestone and shale of the Verulam Formation, with an overburden drift thickness ranging between 15 and 50 m. The referenced geological mapping can be found at the Geological Survey of Canada website, Open File 5311, 2008. (https://doi.org/10.4095/226165).

Atterberg Limits Testing

Atterberg limits testing was completed on 4 selected silty clay samples recovered from boreholes BH 1-26 and BH 3-26 from the current investigation, in addition to BH 3 - 23 and BH 4-23 from previous investigations. The results of the Atterberg limits tests are presented in Table 1 below and on the Atterberg limits results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
Current Investigation						
BH 1-26 SS2	1.07	52	21	31	30.5	CH
BH 3-26 SS3	1.83	55	23	32	30.9	CH
Previous Investigations						
BH 3-23 SS2	1.2	58	25	33	30.6	CH
BH 4-23 SS2	1.2	50	22	28	33.9	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CL: Inorganic Clays of Low Plasticity CH: Inorganic Clays of High Plasticity

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was also completed on 3 selected soil samples from the current and previous investigations. The result of the grain size distribution analysis is presented in Table 2 below and on the Grain Size Distribution sheets in Appendix 1.

Table 2 – Grain Size Distribution Results

Sample	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
Current Investigation					
BH 1-26 SS2	1.07	0.0	12.9	40.1	47.0
Previous Investigations					
BH 3-23 SS3	1.83	0.0	1.6	62.4	36.0
BH 4-23 SS4	1.83	0.0	28.2	46.8	25.0

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

Shrinkage Testing

Linear shrinkage testing was completed on a sample recovered during the current investigation for Block 27 at a depth of 1.07 m from borehole BH 2-26, which yielded a shrinkage limit of 21.16 and a shrinkage ratio of 1.774. The results of the linear shrinkage testing are attached to this memorandum.

Linear shrinkage testing was additionally completed on a sample recovered from the previous investigation at a depth of 1.83 m from borehole BH 3-23, and yielded a shrinkage limit of 16.69 and a shrinkage ratio of 1.885.

4.3 Groundwater

Groundwater levels (GWL) from the current investigation were measured in boreholes BH 1-26 through BH 3-26 on February 3, 2026. Additionally, groundwater levels (GWL) from previous investigations were measured in boreholes BH 1-23 through BH 6-23 on August 29, 2023, boreholes BH 1-19 through BH 7-19 on July 18, 2019, and in boreholes BH 1 through BH 6 on November 3, 2008. Groundwater levels were recorded at each borehole location and are presented in Table 3 below and on the Soil Profile and Test Data sheets in Appendix 1.

Table 3 – Summary of Groundwater Levels Readings

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
Current Investigation				
BH 1-26	103.15	7.00	96.15	February 3, 2026
BH 2-26	102.20	8.02	94.18	
BH 3-26	102.26	8.44	93.82	
Previous Investigations				
BH 3-23	104.40	NE	-	August 29, 2023
BH 4-23	102.25	NE	-	
BH 4-19	104.31	2.07	102.24	July 18, 2019
BH 5-19	100.75	Blocked	-	
BH 3	102.60	6.31	96.29	November 3, 2008
BH 4	102.80	6.95	95.85	

Notes:
- Current borehole elevations (BH 4-19 and BH 5-19) surveyed by Robinson Land Development are understood to be referenced to a geodetic datum.
- The ground surface elevation at the remaining borehole locations were surveyed using a handheld GPS using a geodetic datum.
- NE: Not encountered within the depth of the piezometer/monitoring well.

Further reference can be made to the groundwater monitoring results included in Appendix 1 for BH 2-24 and BH 3-24 from previous investigations.

It should be noted that groundwater levels are subject to seasonal fluctuations; therefore, the groundwater levels could vary at the time of construction.

Additional groundwater readings will be obtained in Spring 2026 in order to determine the seasonal high groundwater level.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed apartment building and clubhouse. It is recommended that the proposed building be founded on conventional spread footings placed on an undisturbed, compact silty sand and/or an undisturbed, hard to stiff silty clay bearing surface.

Due to the presence of a silty clay deposit, permissible grade raise restrictions are recommended for this site.

A Limit of Hazard Lands setback has been defined along the existing ravine, as presented on Drawing PG4918-6 - Test Hole Location Plan. This is discussed further in Section 6.8.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, areas to be filled, paved areas, pipe bedding and other settlement-sensitive structures.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, compact silty sand or undisturbed, hard to stiff silty clay bearing surface, or on engineered fill which is placed directly over the undisturbed, compact silty sand or undisturbed, stiff silty clay, can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. All footings shall have a minimum dimension of 0.5 m.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to placement of concrete for footings. The undisturbed, compact silty sand or undisturbed, stiff silty clay bearing surface should be inspected by qualified geotechnical personnel from Paterson prior to foundation construction.

The bearing resistance value at SLS for shallow footings bearing on the above-noted soils will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty sand or silty clay bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil or engineered fill of the same or higher capacity as the bearing soil.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, the recommended permissible grade raise for Block 27 is **3.0 m**.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

A seismic site response **Class X_D** should be used for the design of the proposed building at Block 27, in accordance with the Ontario Building Code (OBC) 2024.

The results of the Atterberg limits testing indicate that the cohesive silty clay soils at the subject site have plasticity indices in excess of 12. Therefore, in accordance with the criteria established in Section 18.6.3.7.2 of the Canadian Foundation Engineering Manual – 5th Edition (2023), the silty clay deposit at this site is not considered susceptible to liquefaction.

5.5 Basement Slab / Slab-on-Grade Construction

With the removal of all topsoil and deleterious fill from within the footprints of the proposed buildings, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

It is understood that the basement level for the proposed building will be used as a parking level. Therefore, it is recommended that the rigid pavement structure provided in Table 4, presented in Section 5.7, be used for the design. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

In consideration of the groundwater conditions at the site, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lowest level slab of the apartment building.

For structures with slab-on-grade construction, such as the clubhouse, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. The proposed clubhouse will not have below-grade space, therefore a sub-slab drainage system is not required as this floor slab will not come into contact with groundwater.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab, where required.All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for basement walls of the proposed buildings, where present. However, the conditions can be represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3.

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$, where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}) for a Site Class X_D at the subject site is 0.339 g according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\} / P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per the OBC 2024.

5.7 Pavement Structure

Pavement Structure Over Overburden

For design purposes, it is recommended that the rigid pavement structure for the underground parking level of the apartment building consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below.

Table 4 – Recommended Rigid Pavement Structure – Underground Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 – 32MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
Subgrade – Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

For design purposes, the pavement structure presented in the following tables is recommended for the design of car-only parking areas and access lanes.

Table 5 – Recommended Pavement Structure – Driveways and Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II
Subgrade – Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 6 – Recommended Pavement Structure – Local Roads

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used at Block 27. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on maintaining the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials, it is recommended to install subdrains during the pavement construction, as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for the proposed apartment building. The system should consist of a 100 mm diameter, geotextile-wrapped, perforated and corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

A geocomposite drainage board, such as Delta Drain 6000, should be installed over the exterior below-grade foundation walls of the apartment building and connected to the perimeter drainage system. The exterior foundation walls can then be backfilled with the site excavated materials, provided that they are maintained in an unfrozen state and at a suitable moisture content for compaction. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

It should be noted that a geocomposite drainage board, perimeter drainage and sub-slab drainage are not required for the proposed clubhouse, as this structure will not have below-grade space.

For the proposed basement level of the apartment building, sub-slab drainage will be required to control water infiltration. For preliminary design purposes, it is recommended that 100 mm diameter perforated pipes be placed at approximate 6 m centres underlying the basement floor. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. It should be noted that sub-slab drainage is not required for the proposed clubhouse, as it will not have below-grade space.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided in this regard.

A minimum of 2.1 m thick soil cover, or an equivalent combination of soil cover and foundation insulation, should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the City of Ottawa. These recommendations are for standard, open cut excavation placed services.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) stiff to very stiff silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

The installation of seepage barriers or clay seals, as per City of Ottawa Standard Drawing S8, is recommended at strategic locations (site boundaries and 60 m spacing) along the site services that are 4.0 m below the finished ground surface to reduce potential post-development groundwater lowering.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be relatively low and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), provided that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of 3 to 4 weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

6.6 Winter Construction

The subsurface conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

Precautions should be taken if winter construction is considered for this project. In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Tree Planting Restrictions

Paterson completed soils testing to determine the applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size analysis and shrinkage testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the underside of footing elevation and a 3.5 m depth below finished grade. The results of the testing are attached to the current memorandum.

A low to medium sensitivity clay soil was encountered between the anticipated underside of footing elevations and 3.5 m below finished grade as per City Guidelines within Block 27. Based on our Atterberg limits test results, the modified plasticity index does not exceed 40% in this area.

Therefore, large trees (mature height over 14 m) can be planted within Block 27, provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the conditions noted below are met.

- ☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated in the procedural changes below.

☐ A small tree must be provided with a minimum 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally uncompacted when backfilling in street tree planting locations.

☐ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

☐ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

☐ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

6.8 Slope Stability Assessment

The slope conditions along the ravine were reviewed by Paterson field personnel as part of the previous geotechnical investigations. Three (3) slope cross-sections were studied directly adjacent to Block 27 as the worst-case scenarios, based on the height and steepness of the slopes at those locations. The cross-section locations are presented on Drawing PG4918-6 - Test Hole Location Plan attached to the current memorandum.

The existing slope face of the ravine located along the northeast boundary of Block 27 is currently vegetated and tree covered, with lower bank angles ranging from 0 to 90 degrees. Signs of minor erosion were observed during our site review, such as localized sloughing. The watercourse was also observed to be in a confined channel which is able to support routine flows. The localized areas of erosion are indicated on Drawing PG4918-6 – Test Hole Location Plan, attached to this memorandum.

A 0.5 to 1 m wide watercourse was observed at the base of the ravine. The watercourse was observed to be less than 75 mm deep with little to no flow. Two existing stormwater management ponds (SWMP) were observed along the existing watercourse throughout the area of the site at 175 Langstaff Drive. The first SWMP is located north of Langstaff Drive and the second is located south of Langstaff Drive within the area of the subject site.

Slope Stability Analysis

A slope stability analysis was carried out to determine the required construction setbacks from the top of the slope based on factors of safety of 1.5 under static conditions and 1.1 under seismic conditions. Toe erosion and access allowances were also considered in the determination of Limit of Hazard Lands and are discussed in the following sections. The proposed Limit of Hazard Lands and top of slope are shown on Drawing PG4918-6 - Test Hole Location Plan attached to the current memorandum.

The analyses of the stability of the slopes were carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method and the Morgenstern-Price method, which are widely used and accepted analysis methods. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety (F.o.S.) of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a F.o.S. greater than one is usually required to ascertain that the risks of failure are acceptable. Minimum F.o.S. of 1.5 and 1.1 are generally recommended for static and seismic conditions, respectively, where the failure of the slope would endanger permanent structures.

The cross-sections were analyzed based on the existing conditions observed during our site visit and review of the available topographic mapping. In developing the slope geometry at each cross-section location, spot elevations were used preferentially over contours, where present. Where spot elevations were not present, contours were used.

Proposed conditions were analyzed based on the available grading plans prepared by others. The slope stability analysis was completed at each slope cross-section under worst-case-scenario by assigning cohesive soils under fully saturated, groundwater flow conditions being parallel to the slope face. Subsoil conditions at the cross-sections were inferred based on nearby boreholes, our observations along the slopes during our site walks, and general knowledge of the area's geology.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation, and also from the Slope Stability Study of the Ottawa River (Ontario Geological Survey Miscellaneous Paper 112) prepared by A.S. Poschmann, K.E. Klassen, M.A. Klugman, and D. Goodings (1983). The effective strength soil parameters used for static analysis are presented in Table 7 below.

Table 7 – Effective Strength Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Brown and Grey Silty Clay	17	33	7
Silty Sand	20	35	1
Grade Raise Fill	18	33	5
Engineered Fill	22	0	30

For the silty clay, typically a higher cohesion of 10 kPa is typically used for the deeper silty clay under static analysis conditions, as compared to a cohesion of 7 kPa for the upper stiff to very stiff silty clay. This is due to the weathering process which creates numerous micro-fissures within the upper portion of the silty clay profile, and which reduces the cohesion as compared to the deeper, grey silty clay. As the very stiff to stiff silty clay extends to depths of approximately 8 m, based on the boreholes completed at this site, the reduced cohesion value was utilized for the full depth of the silty clay profile in our slope stability analyses, which is considered to be a conservative measure.

The total strength parameters for seismic analysis were chosen based on the in situ, undrained shear strengths recovered within the boreholes completed at the time of our geotechnical investigation and based on our general knowledge of the geology in the area. The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 8 on the next page:

Table 8 - Total Strength Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
Silty Clay	17	0	100 - 50*
Silty Sand	20	35	1
Grade Raise Fill	18	33	5
Engineered Fill	22	0	30

Note: For the stiff to very stiff silty clay, an undrained shear strength of 100 kPa was used for the upper 5 m, 100 kPa decreasing to 50 kPa from 5 to 7 m depth, 50 kPa increasing to 100 kPa from 7 to 9 m depth, and 100 kPa below a 9 m depth.

The borehole(s) used to develop the subsurface profile at each cross-section location are listed in Table 9 below:

Table 9 - Boreholes Used To Determine Subsurface Profile

Cross-Section	Borehole(s)
B-B (West)	BH 3
H-H (West)	BH 3
I-I	BH 3, BH 4-19

Where more than one borehole is listed in Table 9, the subsurface conditions were interpolated between the borehole locations.

Static Loading – Existing and Proposed Conditions

The results for the slope stability analyses under existing and proposed static conditions at Sections B, H, and I on the west side of the ravine are shown on Figures 10, 12, 46, 48, 56 and 58 and are attached to the present memorandum. The factor of safety was found to be greater than or equal to 1.5 at Sections B and H. However, Section I requires a setback of 3.2 m from top of slope to obtain a factor of safety greater than 1.5. A marginal factor of safety of 1.505 was calculated at the west side of Section B under static conditions, therefore, a stable slope allowance of 3.6 m was provided here as a conservative measure.

Seismic Loading – Existing and Proposed Conditions

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.17 g (50% of PGA = 0.34g for Site Class X_D based on the OBC

2024) was considered for all slopes. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The results of the slope stability analyses under existing and proposed, seismic conditions at Sections B, H, and I are shown on Figures 11, 13, 47, 49, 57 and 59, attached to the present memorandum. The results indicate that the factors of safety are greater than 1.1 under seismic conditions. Based on these results, the slopes are considered to be stable under seismic loading. Therefore, when considering seismic loading, no geotechnical setback from the top of the slope is required to achieve a factor of safety of 1.1 for the Limit of the Hazard Lands.

Geotechnical Setbacks – Limit of Hazard Lands

Signs of erosion were noted in localized areas along the lower portion of the slope face that confines the watercourse. Some minor sloughing failures were noted in the lower portion of the slope, leaving some exposed tree roots. Although signs of historic erosion were observed during our site visits, such as localized sloughing, the watercourse was observed to be in a confined channel which is able to support routine flows.

The general characteristics of the central and western ravines are also discussed in detail in the report titled “Fluvial Geomorphological and Erosion Hazard Assessment” prepared for this site by Geomorphix dated January 21, 2022.

Based on our observations and the recommendations in the above-noted report prepared by Geomorphix, a 2 m toe erosion allowance is deemed appropriate for most of the ravine, based on the cohesive nature of the soils, the observed erosion areas and the watercourse depth and width. However, a 5 m toe erosion allowance has been recommended by Geomorphix in certain localized areas on the west side of the ravine, where more significant toe erosion has been observed. These areas of more significant toe erosion are shown on the attached Drawing PG4918-6 - Test Hole Location Plan. It is considered that these toe erosion allowances, in addition to an access allowance of 6 m, are required from the top of slope.

The Limit of Hazard Lands, which include these allowances, are indicated on Drawing PG4918-6 - Test Hole Location Plan attached to the present memorandum. Further, the Limit of Hazards Lands setback at each cross-section location is summarized in Table 10:

Table 10 - Limit of Hazard Lands Setbacks

Cross-Section	Stable Slope Allowance [m]	Toe Erosion Allowance [m]	Access Allowance [m]	Limit of Hazard Lands Setback [m]
B-B (West)	3.6	5	6	14.6
H-H (West)	-	2	6	8
I-I	3.2	5	6	14.2

Given that the soils generally consist of very stiff to stiff clays, and the slopes have heights less than 8 m, the slopes at the subject site are not considered susceptible to retrogressive landslides.

The geotechnical top of slope, from which the Limit of Hazard Lands should be referenced, was identified on-site by a senior Paterson geotechnical engineer, as marked by wooden stakes, and was immediately surveyed by Robinson Consultants. As such, the top of slope has been accurately referenced.

The transitions between setback allowances, where present, were interpolated between the slope stability analysis cross-sections based on review of the ravine depth and slope geometry.

6.9 Corrosion Potential and Sulphate

One (1) soil sample from the recent investigation was submitted for analytical testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site.

The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a mild to slightly aggressive corrosive environment. Based on the analytical testing results, there are no risks associated with the chloride content, pH, and/or resistivity which would require mitigation measures.

7.0 Recommendations

It is recommended that the following be completed as the prior to and during the construction phase of the project:

- ☐ Review finalized grading plan(s) from a geotechnical perspective.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to placing backfilling materials.
- ☐ Field density tests to ensure that the specified level of compaction has been achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson's recommendations should also be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole logs are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests to be notified immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Inverness Homes or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Feb. 11, 2026
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Otillia McLaughlin B.Eng.

Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ Inverness Homes (e-mail copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

LINEAR SHRINKAGE TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

391 Hilversum Land, Carp, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 341258.90 **NORTHING:** 5022954.06 **ELEVATION:** 103.15

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** January 27, 2026

FILE NO. : PG4918

HOLE NO. : BH 1-26

SAMPLE DESCRIPTION

GROUND SURFACE

Hard to very stiff, brown **SILTY CLAY**

- 100 mm silty sand seam at 1.7 m depth

- 100 mm silty sand seam at 2.6 m depth

6.02m [97.13m]

Compact, brown **SILTY SAND**

- Trace clay to 6.7 m depth

- With clay by 7.6 m depth

7.82m [95.33m]

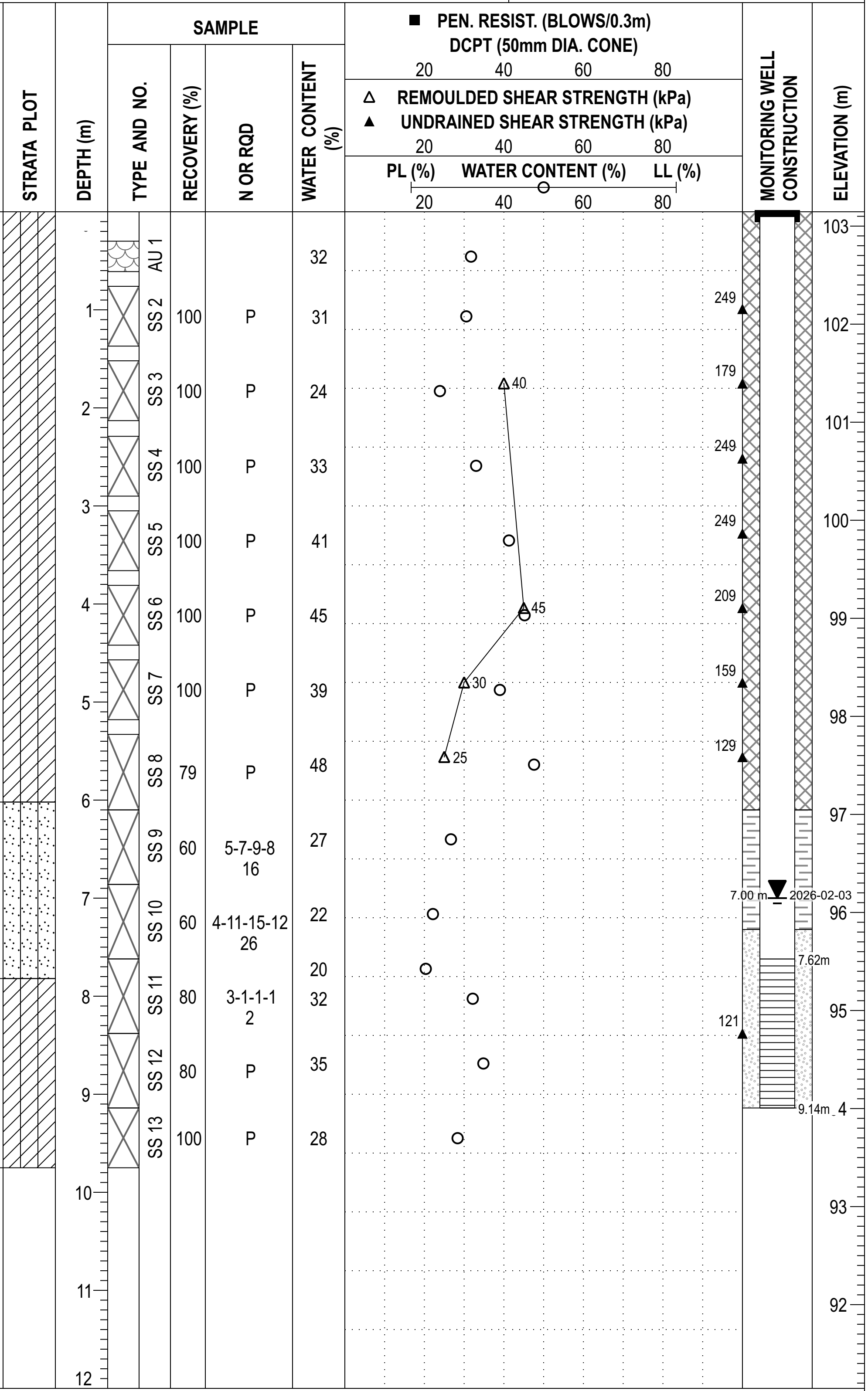
Stiff, grey **SILTY CLAY**

- With sand by 8.4 m depth

9.75m [93.40m]

End of Borehole

GWL at 7.00 m depth - February 3, 2026



TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			32
SS 2	100	P	31
SS 3	100	P	24
SS 4	100	P	33
SS 5	100	P	41
SS 6	100	P	45
SS 7	100	P	39
SS 8	79	P	48
SS 9	60	5-7-9-8 16	27
SS 10	60	4-11-15-12 26	22
SS 11	80	3-1-1-1 2	20 32
SS 12	80	P	35
SS 13	100	P	28

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

391 Hilversum Land, Carp, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 341288.18 **NORTHING:** 5022934.93 **ELEVATION:** 102.20

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** January 27, 2026

FILE NO. : PG4918

HOLE NO. : BH 2-26

SAMPLE DESCRIPTION

GROUND SURFACE

Hard to very stiff, brown **SILTY CLAY**

-Thin intermitent sand seams from 0.9 m to 1.5 m depth

- Thin intermitent sand seams from 4.6 m to 6.1 m depth

- With sand by 6.1 m depth

6.45m [95.75m]

Stiff, grey **SILTY CLAY**, trace to some sand

9.75m [92.45m]

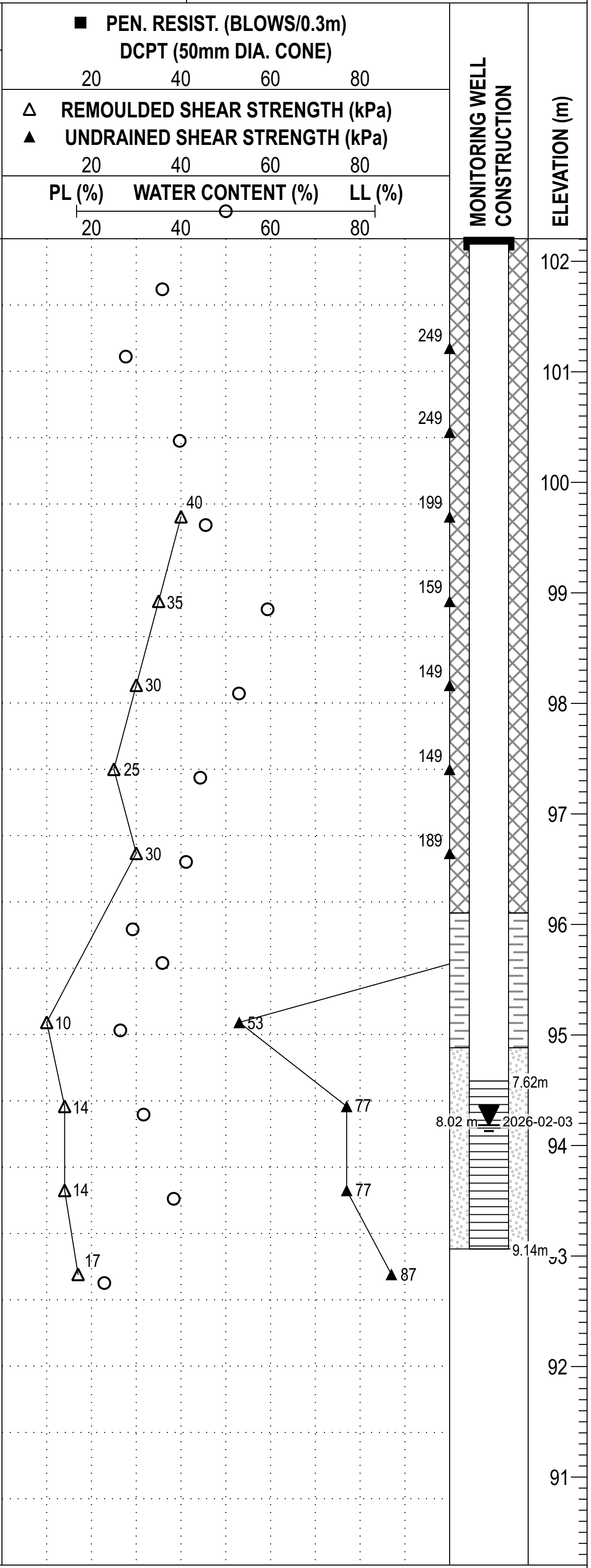
End of Borehole

GWL at 8.02 m depth - February 3, 2026

SAMPLE			
TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			36
SS 2	100	P	28
SS 3	100	P	40
SS 4	100	P	46
SS 5	100	P	59
SS 6	100	P	53
SS 7	100	P	44
SS 8	100	P	41
SS 9	100	P	29 36
SS 10	100	P	26
SS 11	100	P	32
SS 12	100	P	38
SS 13	100	P	23

STRATA PLOT

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12



P:/AutoCAD Drawings/Test Hole Data Files/PG49xx/PG4918/data.sqlite 2026-02-03, 16:15 Paterson_Template JP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

391 Hilversum Land, Carp, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 341307.13 **NORTHING:** 5022923.31 **ELEVATION:** 102.26

PROJECT: Proposed Residential Development

ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** January 28, 2026

FILE NO. : PG4918

HOLE NO. : BH 3-26

SAMPLE DESCRIPTION

GROUND SURFACE

Hard to very stiff, brown **SILTY CLAY**

- Thin intermitent sand seams from 0.8 m to 2.1 m depth

6.02m [96.24m]

Stiff, grey **SILTY CLAY**, trace to some sand

- Thin intermitent sand seams throughout
- 150 mm sand seam at 7.0 m depth

9.75m [92.51m]

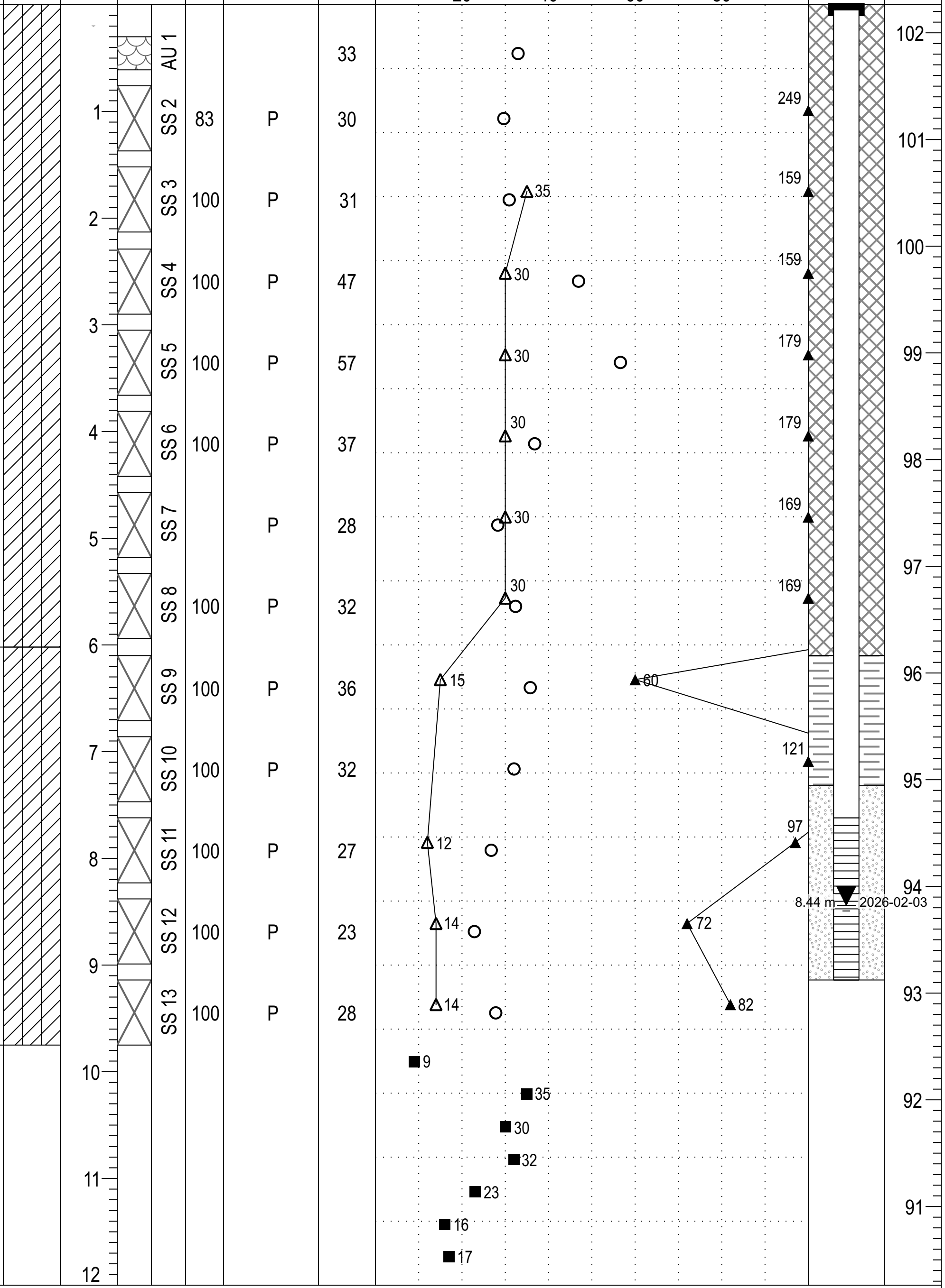
Dynamic Cone Penetration Test commenced at 9.75 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			33
SS 2	83	P	30
SS 3	100	P	31
SS 4	100	P	47
SS 5	100	P	57
SS 6	100	P	37
SS 7		P	28
SS 8	100	P	32
SS 9	100	P	36
SS 10	100	P	32
SS 11	100	P	27
SS 12	100	P	23
SS 13	100	P	28

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

391 Hilversum Land, Carp, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 341307.13 **NORTHING:** 5022923.31 **ELEVATION:** 102.26

PROJECT: Proposed Residential Development

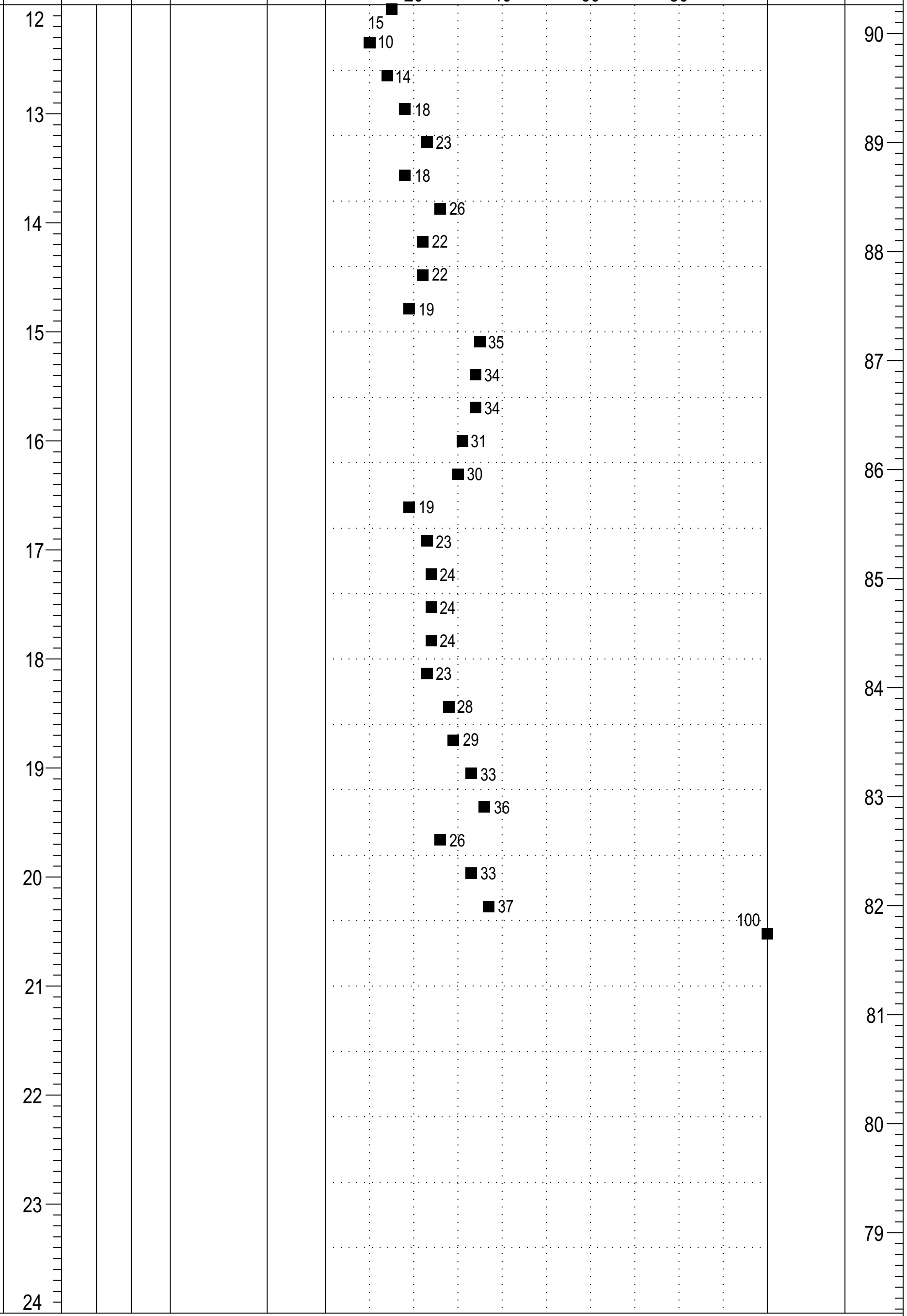
ADVANCED BY: Track Mounted Drill Rig

REMARKS: Datum: NAD1983 (Canada) Geoid: HT2-2010 **DATE:** January 28, 2026

FILE NO. : PG4918

HOLE NO. : BH 3-26

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE TYPE AND NO.	SAMPLE RECOVERY (%)	SAMPLE N OR RQD	SAMPLE WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80 △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80 PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
Dynamic Cone Penetration Test 20.62m [81.64m]									
End of Borehole Practical refusal to DCPT at 20.62 m depth GWL at 8.44 m depth - February 3, 2026									



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 147 Langstaff Drive
Ottawa, Ontario

EASTING: 341268.32 **NORTHING:** 5022923.884 **ELEVATION:** 102.69

DATUM: Ground surface elevations provided by Robinson Land Development.

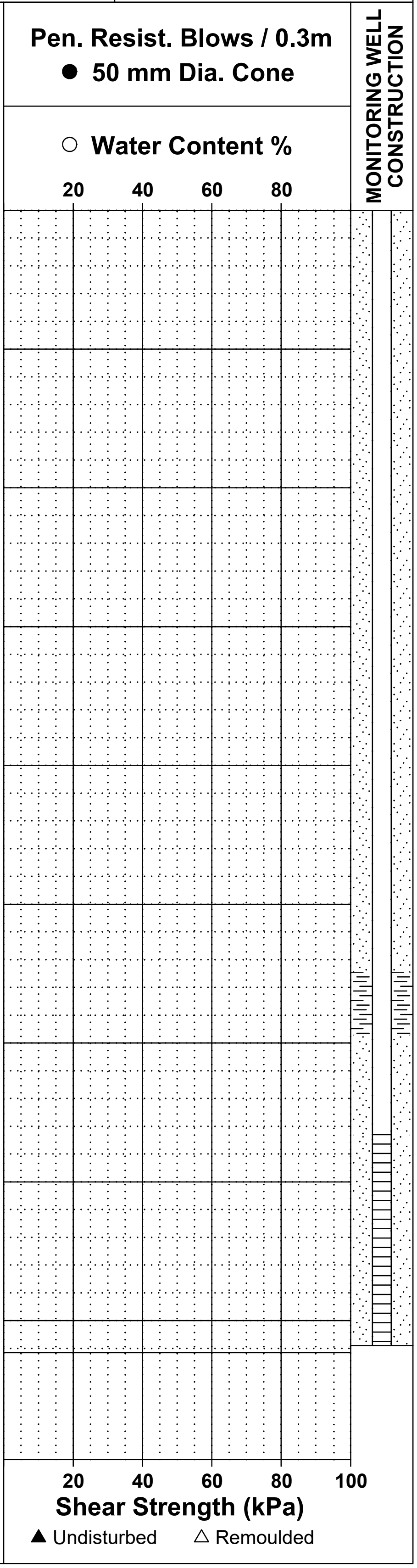
REMARKS:

BORINGS BY: CME-55 Low Clearance Drill **DATE:** March 7, 2024

FILE NO. PG4918

HOLE NO. BH 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL and organics 0.15						0	102.69
Hard to very stiff brown **SILTY CLAY**							
						1	101.69
		SS	1	92	7		
						2	100.69
						3	99.69
		SS	2	92	8		
						4	98.69
		SS	3	92	5		
- Intermittent silt seams @ 5.03m						5	97.69
5.49							
Stiff grey **SILTY CLAY**							
						6	96.69
		SS	4	92	1		
						7	95.69
		SS	5	92	1		
						8	94.69
8.23							
End of Borehole							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Dev. - 147 Langstaff Drive
Ottawa, Ontario

EASTING: 341245.506 **NORTHING:** 5023027.668 **ELEVATION:** 103.9

DATUM: Ground surface elevations provided by Robinson Land Development.

REMARKS:

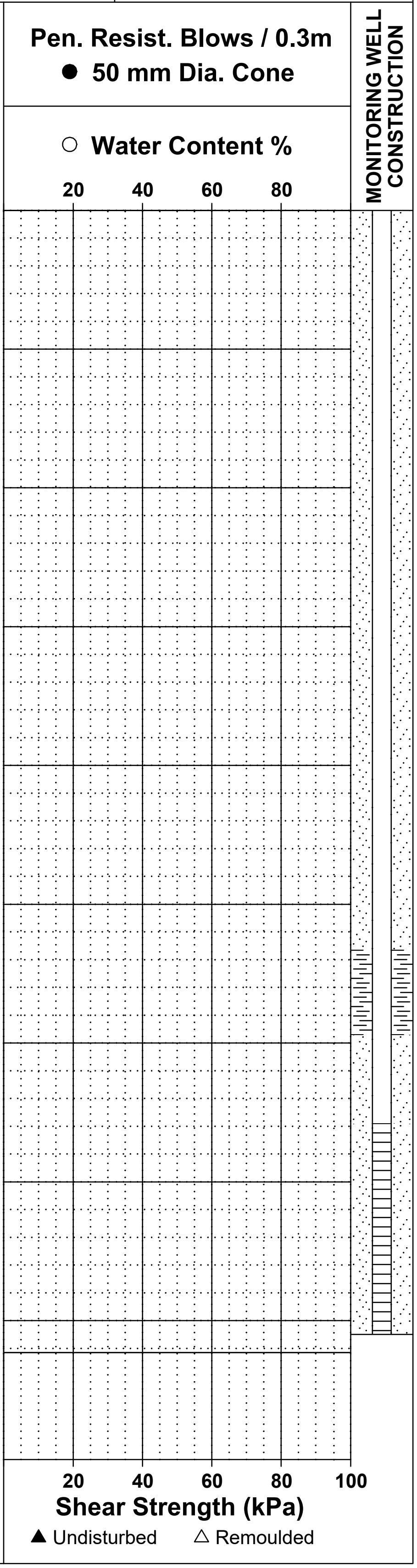
BORINGS BY: CME-55 Low Clearance Drill **DATE:** March 7, 2024

FILE NO. PG4918

HOLE NO. BH 3-24

SAMPLE DESCRIPTION		SAMPLE			
GROUND SURFACE	Depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
TOPSOIL and organics	0.15				
Hard to very stiff **SILTY CLAY**		SS	1	92	4
		SS	2	92	4
		SS	3	92	3
	5.49				
Stiff grey **SILTY CLAY** with intermittent silt seams		SS	4	92	1
- Clayey silt seams from 7.77m to 8.08m	8.23	SS	5	92	5
End of Borehole					

DEPTH (m)	ELEV. (m)
0	103.90
1	102.90
2	101.90
3	100.90
4	99.90
5	98.90
6	97.90
7	96.90
8	95.90



PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

147 Langstaff Drive, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 341205.574 **NORTHING:** 5023031.171 **ELEVATION:** 104.4

PROJECT: Proposed Residential Development

BORINGS BY: CME 55 Low Clearance Power Auger

REMARKS:

DATE: August 23, 2023

FILE NO. **PG4918**

HOLE NO. BH 3-23

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 25 50 75 100	Piezometer Construction
Ground Surface EL 104.4 m											
TOPSOIL 0.3 m EL 104.1 m		AU1	●			28.6	0				
Hard to very stiff, brown SILTY CLAY - silty sand seams from 0.8 to 1.8m depths		SS2	▽	100	21	30.6	1				
		SS3	▽	100	14	40.6	2				
		SS4	▽	100	5	38.8					
2.9 m EL 101.5 m							3				
End of Borehole (BH dry - August 29, 2023)							4				
							5				
							6				
							7				
							8				
							9				
							10				

RSLog / Geotechnical Borehole - Geodetic / paterson-group / admin / September 12, 2023 02:17 PM

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

147 Langstaff Drive, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 341249.73 **NORTHING:** 5022890.763 **ELEVATION:** 102.25

PROJECT: Proposed Residential Development

BORINGS BY: CME 55 Low Clearance Power Auger

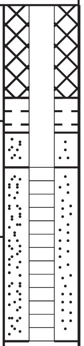
REMARKS:

DATE: August 23, 2023

FILE NO. **PG4918**

HOLE NO. BH 4-23

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE No.	SAMPLE Type	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 25 50 75 100	Peak Shear Strength (kPa) 0 25 50 75 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 25 50 75 100	Piezometer Construction
Ground Surface EL 102.25 m											
TOPSOIL 0.3 m EL 101.95 m		AU1	●			29.5	0				
Hard to very stiff, brown SILTY CLAY		SS2	▽	100	12	33.9	1				
- intermittent silty sand seams throughout		SS3	▽	100	9	32.5	2				
		SS4	▽	100	6	44.3					
2.9 m EL 99.35 m							3				
End of Borehole							4				
(BH dry - August 29, 2023)							5				
							6				
							7				
							8				
							9				
							10				



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
147 Langstaff Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Robinson Land Development.

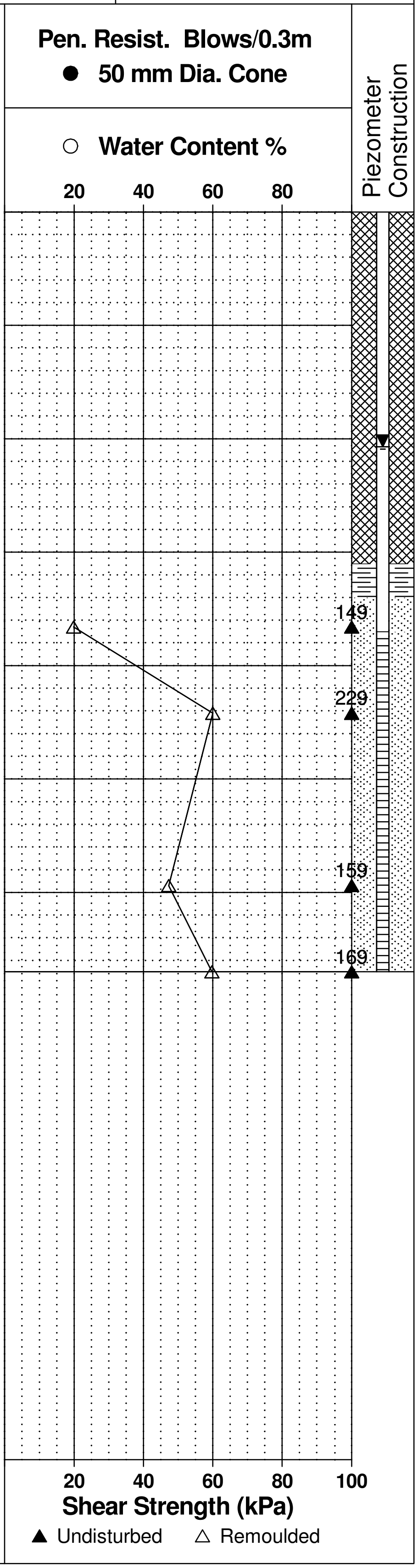
FILE NO. PG4918

REMARKS

HOLE NO. BH 4-19

BORINGS BY CME 55 Power Auger **DATE** 2019 July 11

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	104.31
TOPSOIL 0.20	AU	1				
	SS	2	100	14	1	103.31
	SS	3	100	9	2	102.31
	SS	4	100	6		
Very stiff, brown **SILTY CLAY,** trace sand					3	101.31
					4	100.31
- grey by 4.6m depth	SS	5	100	4	5	99.31
					6	98.31
6.70						
End of Borehole						
(GWL @ 2.07m - July 18, 2019)						



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
147 Langstaff Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Robinson Land Development.

FILE NO. PG4918

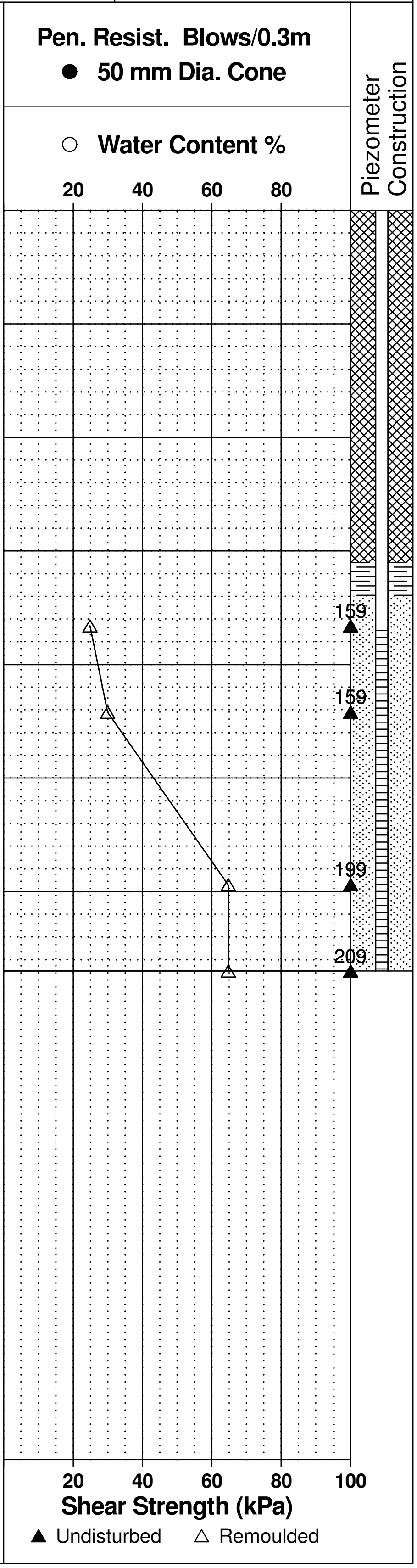
REMARKS

HOLE NO. BH 5-19

BORINGS BY CME 55 Power Auger

DATE 2019 July 12

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.75
TOPSOIL 0.20		AU	1				
		SS	2	88	11	1	99.75
		SS	3	100	10	2	98.75
		SS	4	100	9		
Very stiff, brown **SILTY CLAY**						3	97.75
						4	96.75
- grey by 4.6m depth		SS	5	100	7	5	95.75
						6	94.75
6.70							
End of Borehole (Piezometer blocked - July 18, 2019)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Langstaff Drive
Ottawa, Ontario

DATUM Borehole elevation inferred from nearby spot elevations on site survey, and is referenced top a geodetic datum.

REMARKS

BORINGS BY CME 55 Power Auger

DATE October 23, 2008

FILE NO. PG1773

HOLE NO. BH 3

SOIL DESCRIPTION		TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25	AU	1		
Compact, brown **SANDY SILTY**, trace clay	1.37	SS	2	100	12
Very stiff to stiff, brown **SILTY CLAY** - firm and grey by 5.9m depth	8.00	SS	3	100	9
		SS	4	100	5
		SS	5	100	5
		SS	6	100	4
		SS	7	100	5
		SS	8	100	4
		SS	9	100	2
		SS	10	100	2
		SS	11	100	2
Grey **CLAYEY SILT**	9.75	SS	12	100	2
		SS	13	100	2

End of Borehole

(GWL @ 6.31m - Nov. 3, 2008)

DEPTH (m)	ELEV. (m)
0	102.60
1	101.60
2	100.60
3	99.60
4	98.60
5	97.60
6	96.60
7	95.60
8	94.60
9	93.60

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - Langstaff Drive
Ottawa, Ontario

DATUM Borehole elevation inferred from nearby spot elevations on site survey, and is referenced top a geodetic datum.

REMARKS

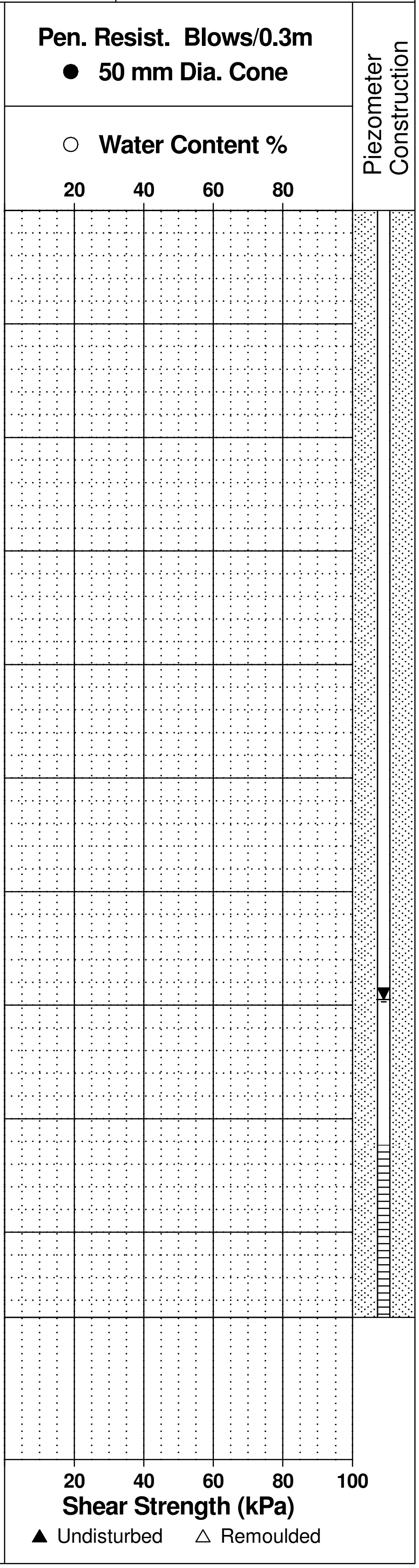
BORINGS BY CME 55 Power Auger

DATE October 23, 2008

FILE NO. PG1773

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL 0.25						0	102.80
		AU	1				
		SS	2	100	11	1	101.80
		SS	3	100	10	2	100.80
		SS	4	100	6		
Very stiff to stiff, brown **SILTY CLAY** with sand		SS	5	100	4	3	99.80
		SS	6	100	5	4	98.80
		SS	7	100	10	5	97.80
5.94		SS	8	100	6		
		SS	9	100	31	6	96.80
		SS	10	25	36	7	95.80
Dense to compact, grey **SILTY SAND**		SS	11	25	26	8	94.80
		SS	12	33	25	9	93.80
9.75		SS	13	42	22		
End of Borehole (GWL @ 6.95m - Nov. 3, 2008)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)
Very Soft	<12
Soft	12-25
Firm	25-50
Stiff	50-100
Very Stiff	100-200
Hard	>200

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called "mechanical breaks") are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))

TW - Thin wall tube or Shelby tube

PS - Piston sample

AU - Auger sample or bulk sample

WS - Wash sample

RC - Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: 1 < Cc < 3 and Cu > 4
Well-graded sands have: 1 < Cc < 3 and Cu > 6
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

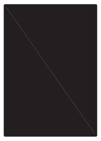
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

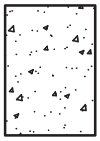
PERMEABILITY TEST

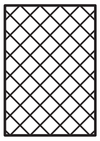
k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

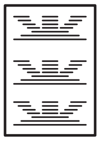
SYMBOLS AND TERMS (continued)

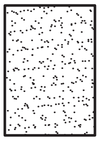
STRATA PLOT

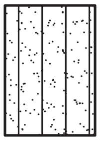

Topsoil

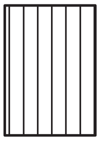

Asphalt

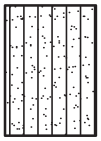

Fill

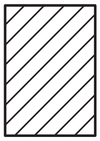

Peat

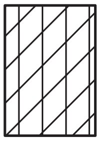

Sand

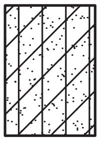

Silty Sand

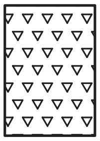

Silt

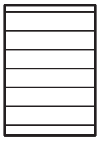

Sandy Silt

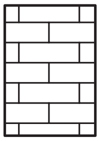

Clay


Silty Clay


Clayey Silty Sand

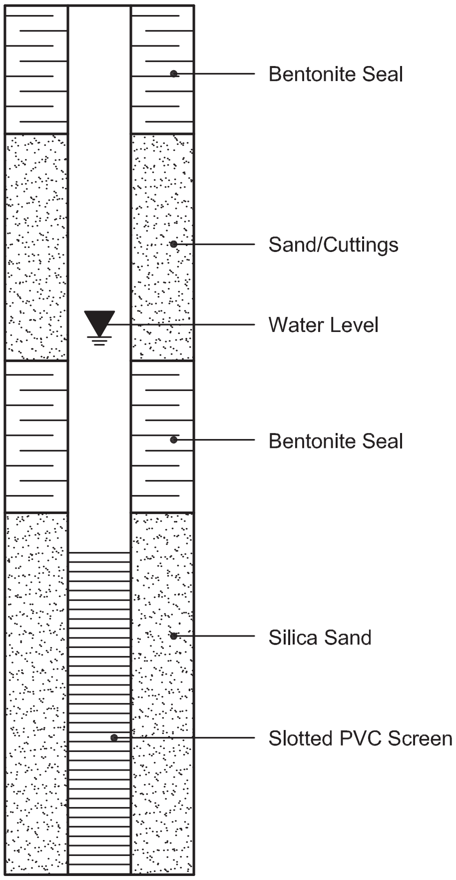

Glacial Till


Shale


Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

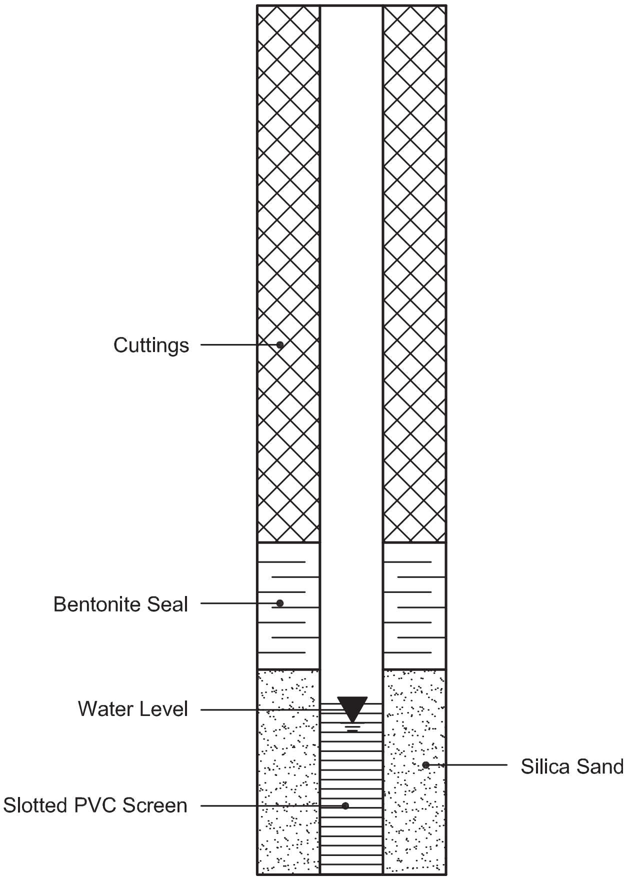
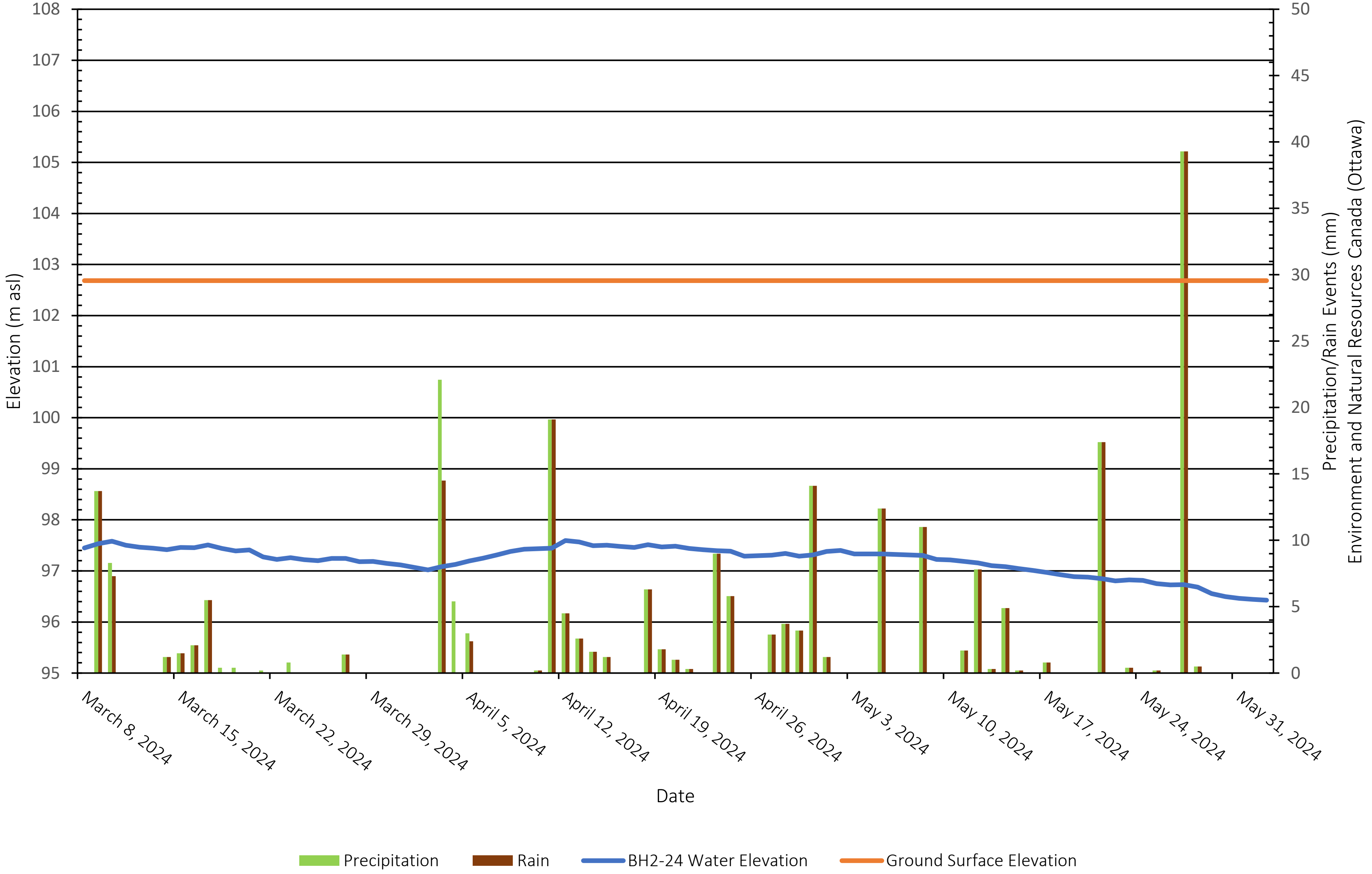


Figure 4: BH2-24 - Monitoring Well Water Elevations



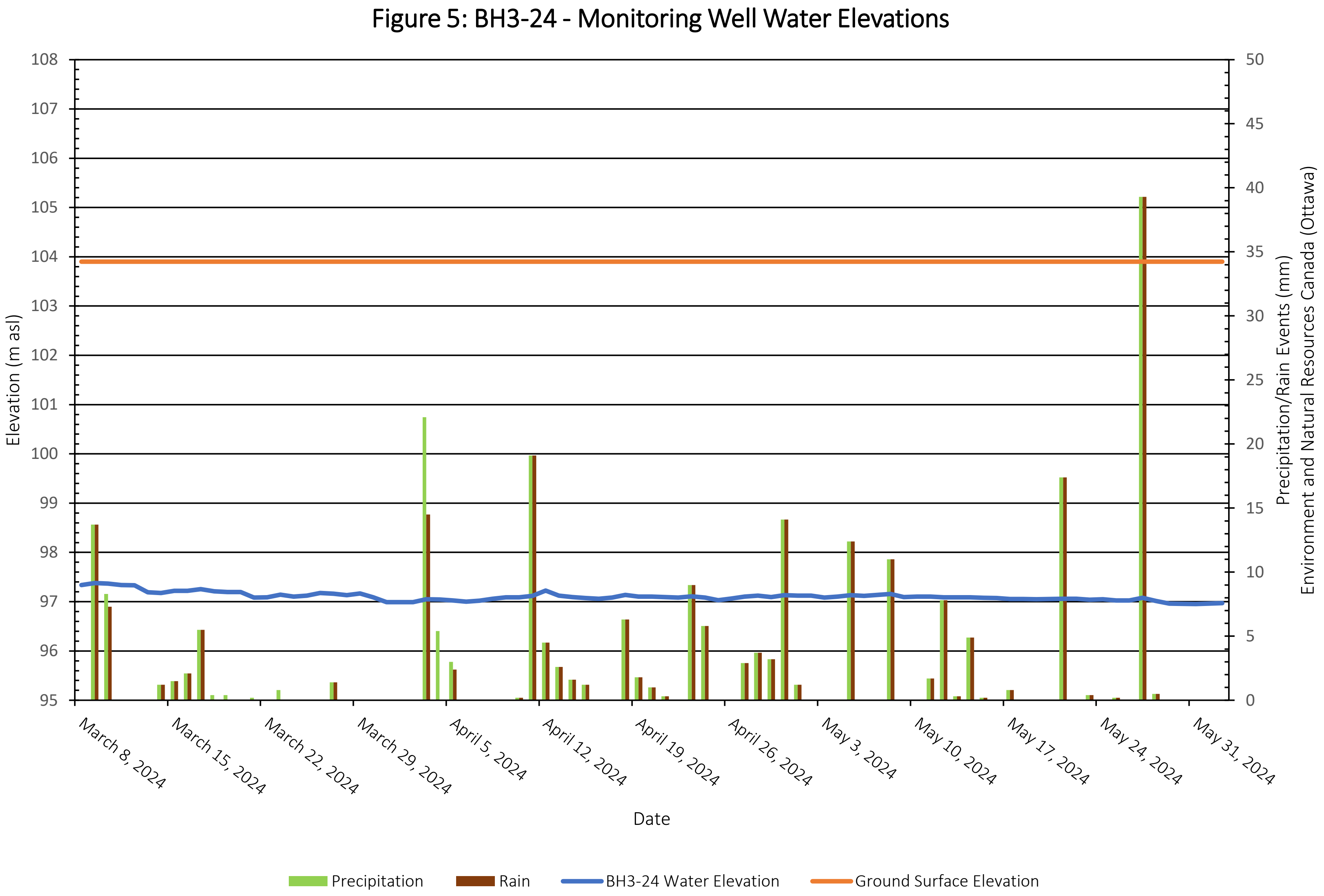


Figure 5: BH3-24 - Monitoring Well Water Elevations
Elevation (m asl)
108
107
106
105
104
103
102
101
100
99
98
97
96
95
Precipitation/Rain Events (mm)
Environment and Natural Resources Canada (Ottawa)
50
45
40
35
30
25
20
15
10
5
0
March 8, 2024
March 15, 2024
March 22, 2024
March 29, 2024
April 5, 2024
April 12, 2024
April 19, 2024
April 26, 2024
May 3, 2024
May 10, 2024
May 17, 2024
May 24, 2024
May 31, 2024
Date
Precipitation
Rain
BH3-24 Water Elevation
Ground Surface Elevation

ATTERBERG LIMITS
LS-703/704

CLIENT:	Inverness Homes	FILE NO.:	PG4918
PROJECT:	147 Langstaff Drive	DATE SAMPLED:	27-Jan
LOCATION:	BH1-26 SS2, 2'6" - 4'6"	DATE REPORTED:	04-Feb
LAB NUMBER:	65120		

LIQUID LIMIT DETERMINATION

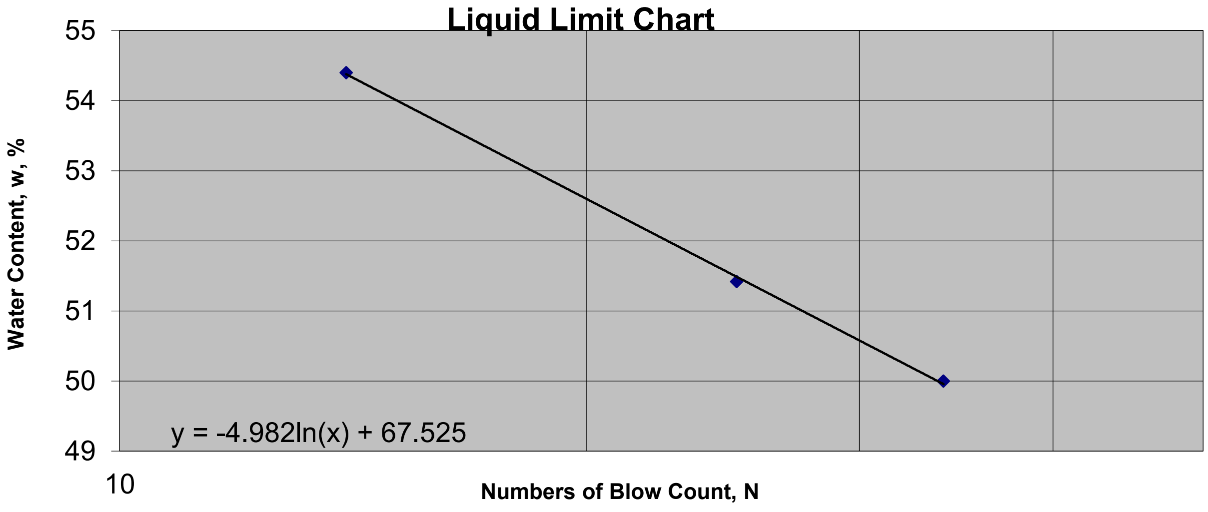
CAN NO.	**17**	**32**	**33**				
WT. OF CAN	4.30	4.34	4.29				
WT. OF SOIL & CAN	13.78	12.85	11.97				
WT. OF DRY SOIL & CAN	10.44	9.96	9.41				
WT. OF MOISTURE	3.34	2.89	2.56				
WT. OF DRY SOIL & CAN	6.14	5.62	5.12				
WATER CONTENT, w, %	**54.4**	**51.42**	**50**				
NO. OF BLOWS, N	14	25	34				

PLASTIC LIMIT DETERMINATION

CAN NO.	**4**	**10**
WT. OF CAN	19.97	19.77
WT. OF SOIL & CAN	28.31	28.08
WT. OF DRY SOIL & CAN	26.85	26.65
WT. OF MOISTURE	1.46	1.43
WT. OF DRY SOIL & CAN	6.88	6.88
WATER CONTENT, w, %	**21.22**	**20.78**

RESULTS

LIQUID LIMIT	**52**
PLASTIC LIMIT	**21**
PLASTICITY INDEX	**31**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

ATTERBERG LIMITS
LS-703/704

CLIENT:	Inverness Homes	FILE NO.:	PG4918
PROJECT:	147 Langstaff Drive	DATE SAMPLED:	27-Jan
LOCATION:	BH3-26 SS3, 5' - 7'	DATE REPORTED:	04-Feb
LAB NUMBER:	65121		

LIQUID LIMIT DETERMINATION

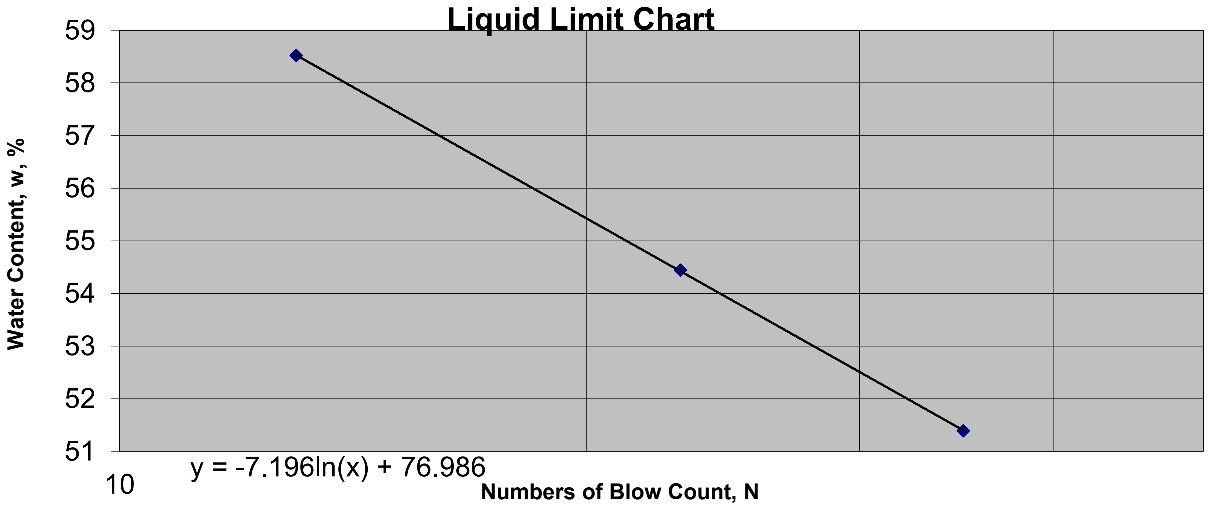
CAN NO.	**11**	**13**	**18**				
WT. OF CAN	8.60	8.63	8.65				
WT. OF SOIL & CAN	17.81	18.36	17.37				
WT. OF DRY SOIL & CAN	14.41	14.93	14.41				
WT. OF MOISTURE	3.4	3.43	2.96				
WT. OF DRY SOIL & CAN	5.81	6.3	5.76				
WATER CONTENT, w, %	**58.52**	**54.44**	**51.39**				
NO. OF BLOWS, N	13	23	35				

PLASTIC LIMIT DETERMINATION

CAN NO.	**9**	**14**
WT. OF CAN	19.34	19.95
WT. OF SOIL & CAN	27.11	27.93
WT. OF DRY SOIL & CAN	25.68	26.43
WT. OF MOISTURE	1.43	1.50
WT. OF DRY SOIL & CAN	6.34	6.48
WATER CONTENT, w, %	**22.56**	**23.15**

RESULTS

LIQUID LIMIT	**55**
PLASTIC LIMIT	**23**
PLASTICITY INDEX	**32**

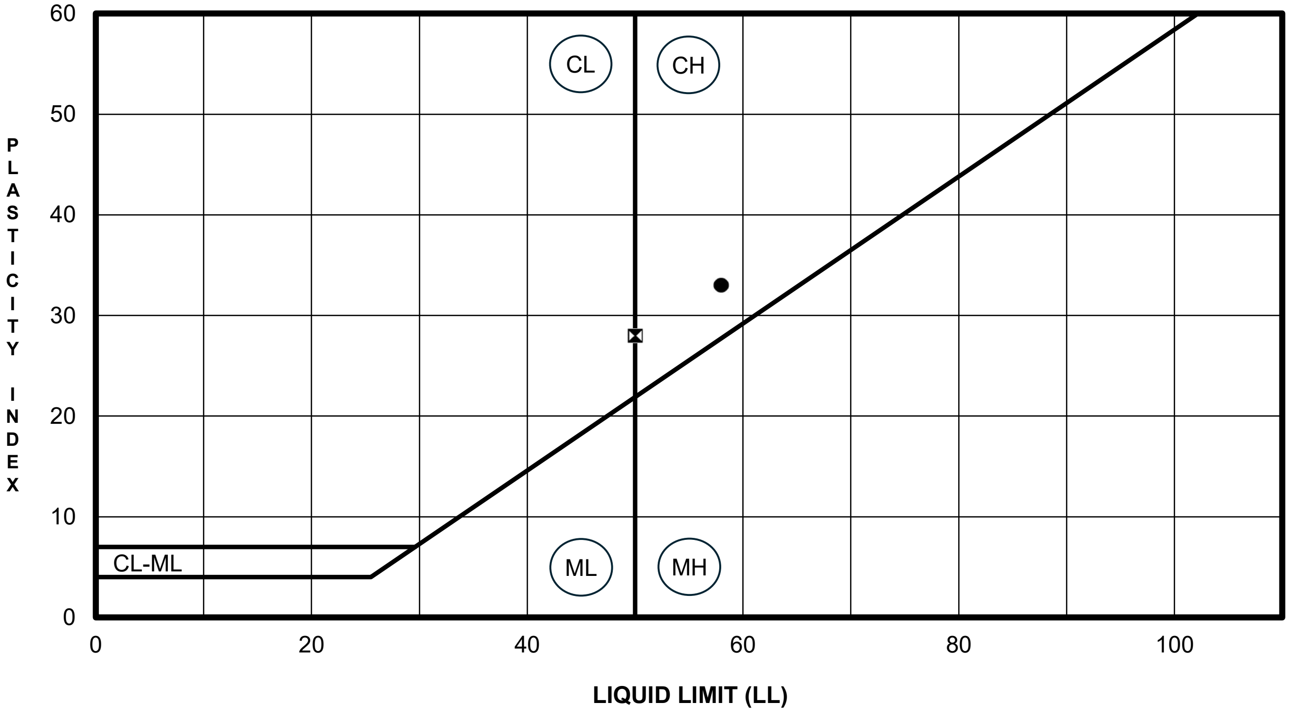


TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

ATTERBERG LIMITS' RESULTS



Specimen Identificacation			LL	PL	PI	Fines	Classification
●	**BH 3-23**	**SS3**	**58**	**25**	**33**		**CH - Inorganic clays of high plasticity**
☒	**BH 4-23**	**SS4**	**50**	**22**	**28**		**CH - Inorganic clays of high plasticity**

CLIENT Riverside South Developments Corporation

FILE NO. PG5131

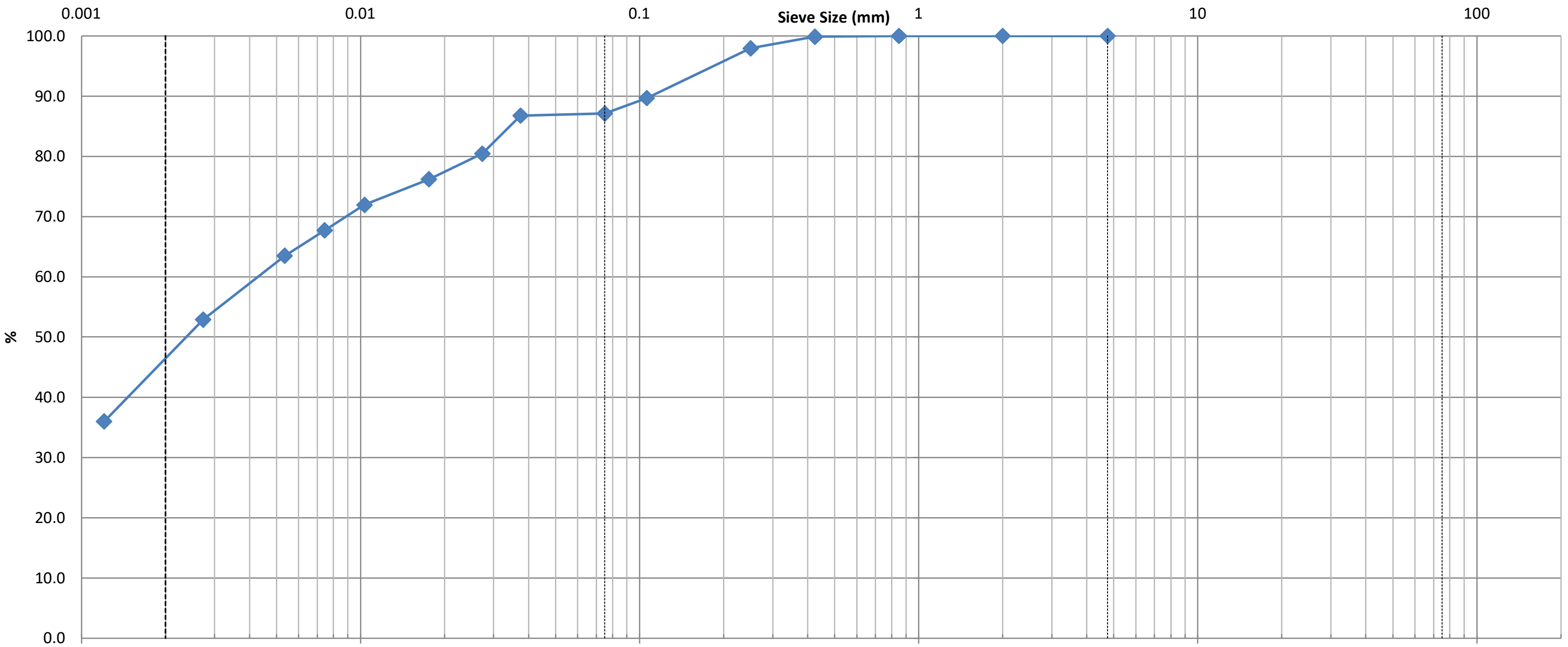
PROJECT Geotechnical Investigation - Phase 17-2 Riverside South Residential Development - Spratt Road

DATE 14-Sep-23

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Inverness Homes	DEPTH:	2'6" - 4'6"	FILE NO:	PG4918
CONTRACT NO.:		BH OR TP No.:	BH1-26 SS2	LAB NO:	65122
PROJECT:	147 Langstaff Drive			DATE RECEIVED:	29-Jan-26
				DATE TESTED:	29-Jan-26
DATE SAMPLED:	27-Jan-26			DATE REPORTED:	4-Feb-26
SAMPLED BY:	-			TESTED BY:	C.M.



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		30.1%					

D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
				0.0	12.9	40.1	47.0

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	Inverness Homes	DEPTH:	2'6" - 4'6"	FILE NO.:	PG4918
PROJECT:	147 Langstaff Drive	BH OR TP No.:	BH1-26 SS2	DATE SAMPLED:	27-Jan-26
LAB No. :	65122	TESTED BY:	C.M.	DATE RECEIVED:	29-Jan-26
SAMPLED BY:	-	DATE REPT'D:	04-Feb-26	DATE TESTED:	29-Jan-26

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
115.3	2.700

INITIAL WEIGHT	50.00	HYGROSCOPIC MOISTURE		
WEIGHT CORRECTED	46.72	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	6.74	AIR DRY	123.40	123.40
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	115.30	115.30
		CORRECTED	0.934	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75	**0.00**	0.0	**100.0**
2.0	**0.00**	0.0	**100.0**
Pan	**115.30**		
0.850	**0.00**	0.0	**100.0**
0.425	**0.07**	0.1	**99.9**
0.250	**1.04**	2.1	**97.9**
0.106	**5.16**	10.3	**89.7**
0.075	**6.45**	12.9	**87.1**
Pan	**6.74**		

SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

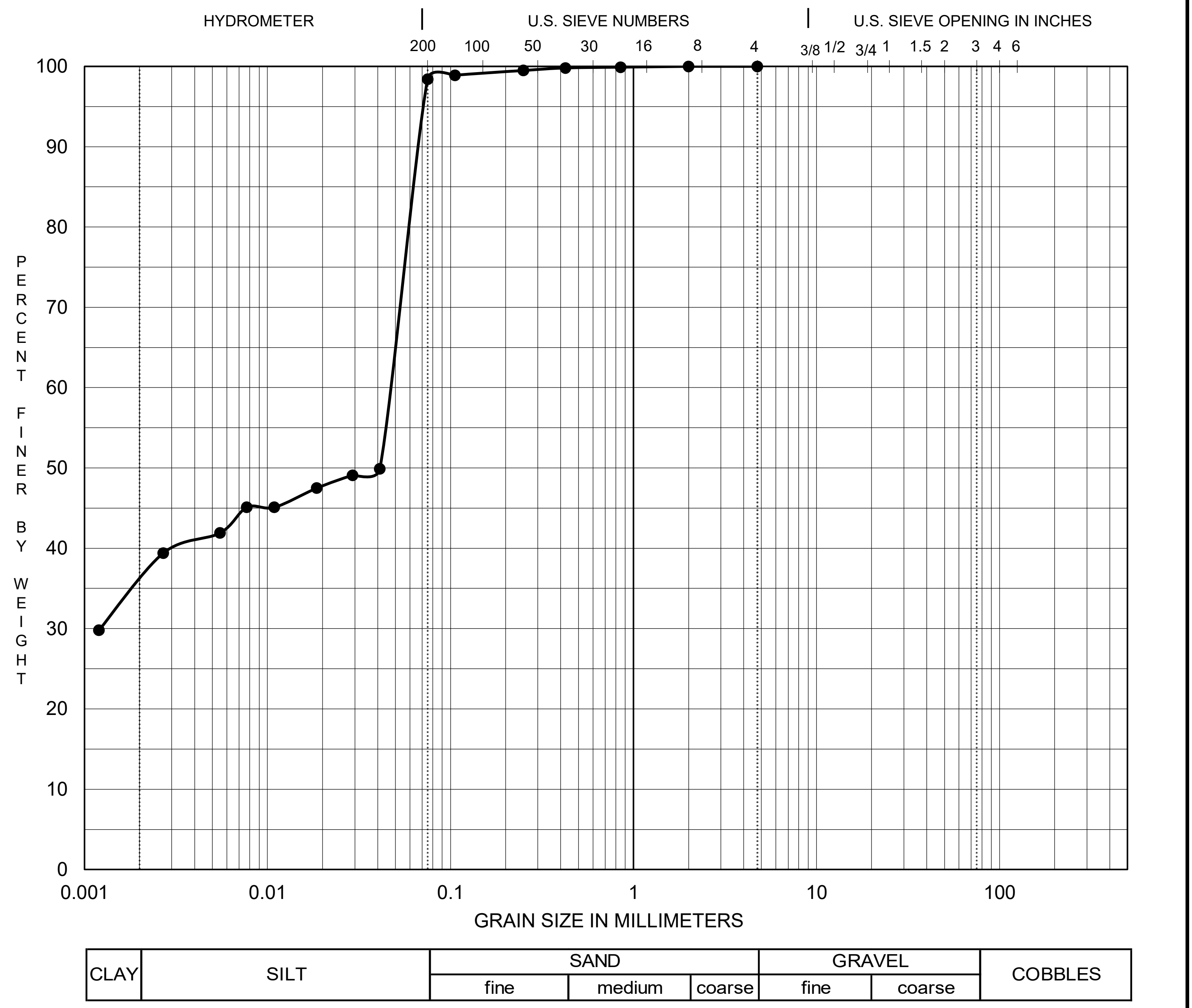
ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:00	47.0	6.0	23.0	0.0374	86.8	**86.8**
2	9:01	44.0	6.0	23.0	0.0273	80.4	**80.4**
5	9:04	42.0	6.0	23.0	0.0176	76.2	**76.2**
15	9:14	40.0	6.0	23.0	0.0103	72.0	**72.0**
30	9:29	38.0	6.0	23.0	0.0074	67.7	**67.7**
60	9:59	36.0	6.0	23.0	0.0054	63.5	**63.5**
250	13:09	31.0	6.0	23.0	0.0027	52.9	**52.9**
1440	8:59	23.0	6.0	23.0	0.0012	36.0	**36.0**

COMMENTS:

Moisture Content = 30.1%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

GRAIN SIZE DISTRIBUTION



	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-23**	**SS3**	**CH - Inorganic clays of high plasticity**	**36.1**	**58**	**25**	**33**		
⊠									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-23**	**SS3**					**0.0**	**1.6**	**62.4**	**36.0**
⊠										
▲										
★										

CLIENT **Riverside South Developments Corporation**

FILE NO. **PG5131**

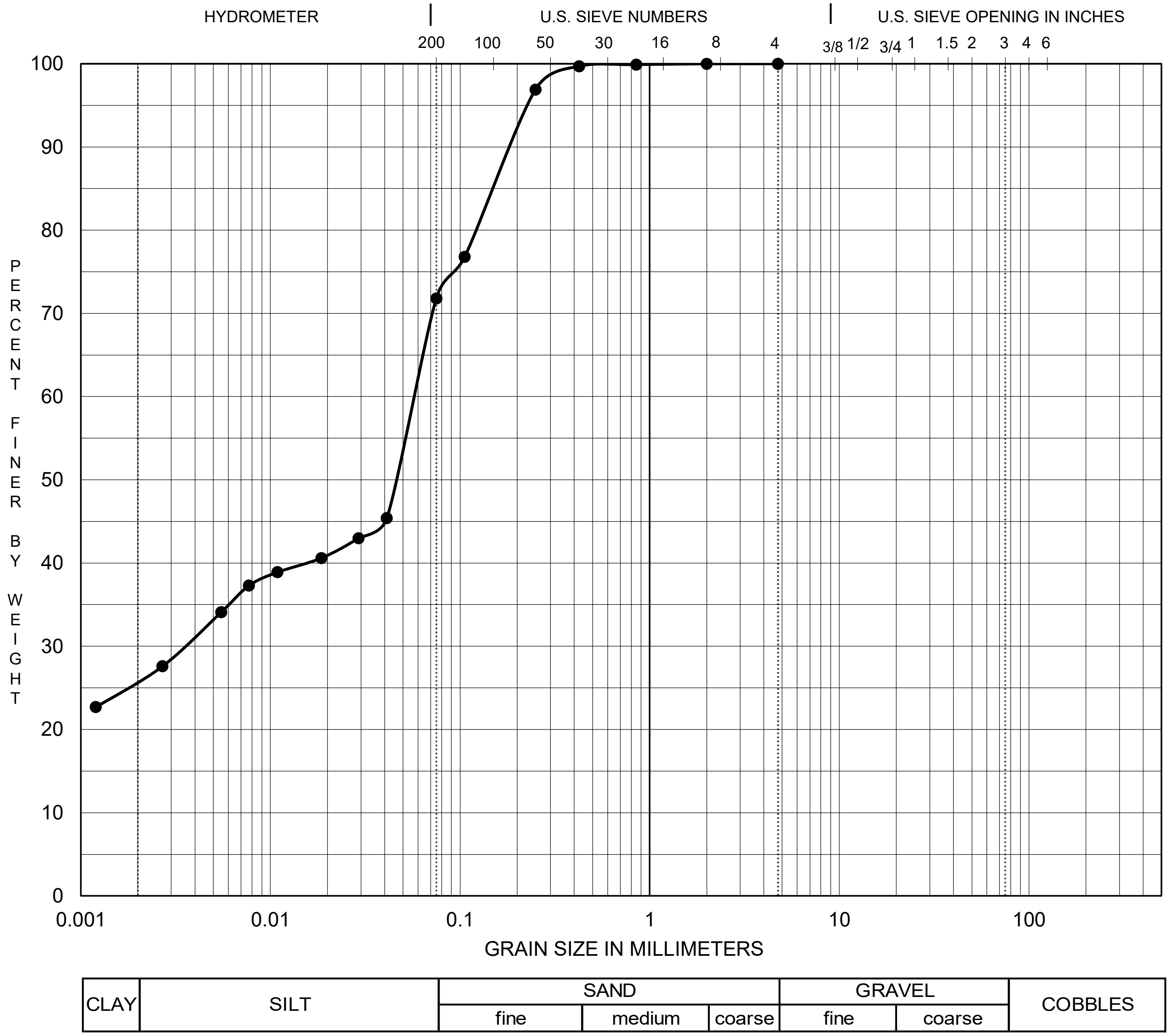
PROJECT **Geotechnical Investigation - Phase 17-2 Riverside South**
Residential Development - Spratt Road

DATE **14-Sep-23**

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

GRAIN SIZE DISTRIBUTION



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 4-23 SS4**	**CH - Inorganic clays of high plasticity**	**36.2**	**50**	**22**	**28**		
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 4-23 SS4**					**0.0**	**28.2**	**46.8**	**25.0**
☒									
▲									
★									

CLIENT **Riverside South Developments Corporation**
FILE NO. **PG5131**
PROJECT **Geotechnical Investigation - Phase 17-2 Riverside South**
Residential Development - Spratt Road
DATE **14-Sep-23**

PATERSON GROUP

LINEAR SHRINKAGE
ASTM D4943-02

CLIENT:	Inverness Homes	DEPTH:	2'6" - 4'6"	FILE NO.:	PG4918
PROJECT:	147 Langstaff Drive	BH OR TP No:	BH2-26 SS2	DATE SAMPLED:	27-Jan-26
LAB No.:	65123	TESTED BY:	CP	DATE RECEIVED:	29-Jan-26
SAMPLED BY:	-	DATE REPORTED:	04-Feb-26	DATE TESTED:	29-Jan-26

LABORATORY INFORMATION & TEST RESULTS

Moisture No. of Blows(8)		**Calibration (Two Trials) Tin NO.(A1)**		
Tare	4.98	Tin	4.45	4.46
Soil Pat Wet + Tare	64.17	Tin + Grease	4.98	4.98
Soil Pat Wet	59.19	Glass	43.2	43.2
Soil Pat Dry + Tare	39.44	Tin + Glass + Water	85.05	85.04
Soil Pat Dry	34.46	Volume	36.87	36.86
Moisture	**71.76**	**Average Volume**	**36.87**	

Soil Pat + String	34.87
Soil Pat + Wax + String in Air	39.59
Soil Pat + Wax + String in Water	14.86
Volume Of Pat (Vdx)	24.73

RESULTS:

Shrinkage Limit	**21.16**
Shrinkage Ratio	**1.774**
Volumetric Shrinkage	**89.765**
Linear Shrinkage	**19.226**

COMMENTS:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

Linear Shrinkage
ASTM D4943-02

CLIENT:	Inverness Homes	DEPTH	5'-7'	FILE NO.:	PG4918
PROJECT:	147 Langstaff Drive	BH OR TP No:	BH3-23 SS3	DATE SAMPLED	23-Aug-23
LAB No:	47375	TESTED BY:	CP	DATE RECEIVED	1-Sep-23
SAMPLED BY:	JP	DATE REPORTED:	14-Sep-23	DATE TESTED	8-Sep-23

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(8)	Calibration (Two Trials)		Tin NO.(x22)
Tare	5	Tin	4.84	4.85
Soil Pat Wet + Tare	66.15	Tin + Grease	5.01	5.01
Soil Pat Wet	61.15	Glass	48.97	48.97
Soil Pat Dry + Tare	42.13	Tin + Glass + Water	91.53	91.46
Soil Pat Dry	37.13	Volume	37.55	37.48
Moisture	**64.69**	**Average Volume**	**37.52**	

Soil Pat + String	37.28
Soil Pat + Wax + String in Air	40.25
Soil Pat + Wax + String in Water	17.22
Volume Of Pat (Vdx)	23.03

RESULTS:

Shrinkage Limit	**16.69**
Shrinkage Ratio	**1.885**
Volumetric Shrinkage	**90.500**
Linear Shrinkage	**19.330**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

Client ID:		BH1-26-SS3	-	-	-		
Sample Date:		27-Jan-26 09:00	-	-	-	-	-
Sample ID:		2605398-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	75.8	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	6.80	-	-	-	-	-
Resistivity	0.1 Ohm.m	145	-	-	-	-	-
Anions							
Chloride	10 ug/g	<10	-	-	-	-	-
Sulphate	10 ug/g	<10	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 10-13 &46-49 & 56-58 - SLOPE STABILITY ANALYSIS SECTIONS

DRAWING PG4918-6 - TEST HOLE LOCATION PLAN

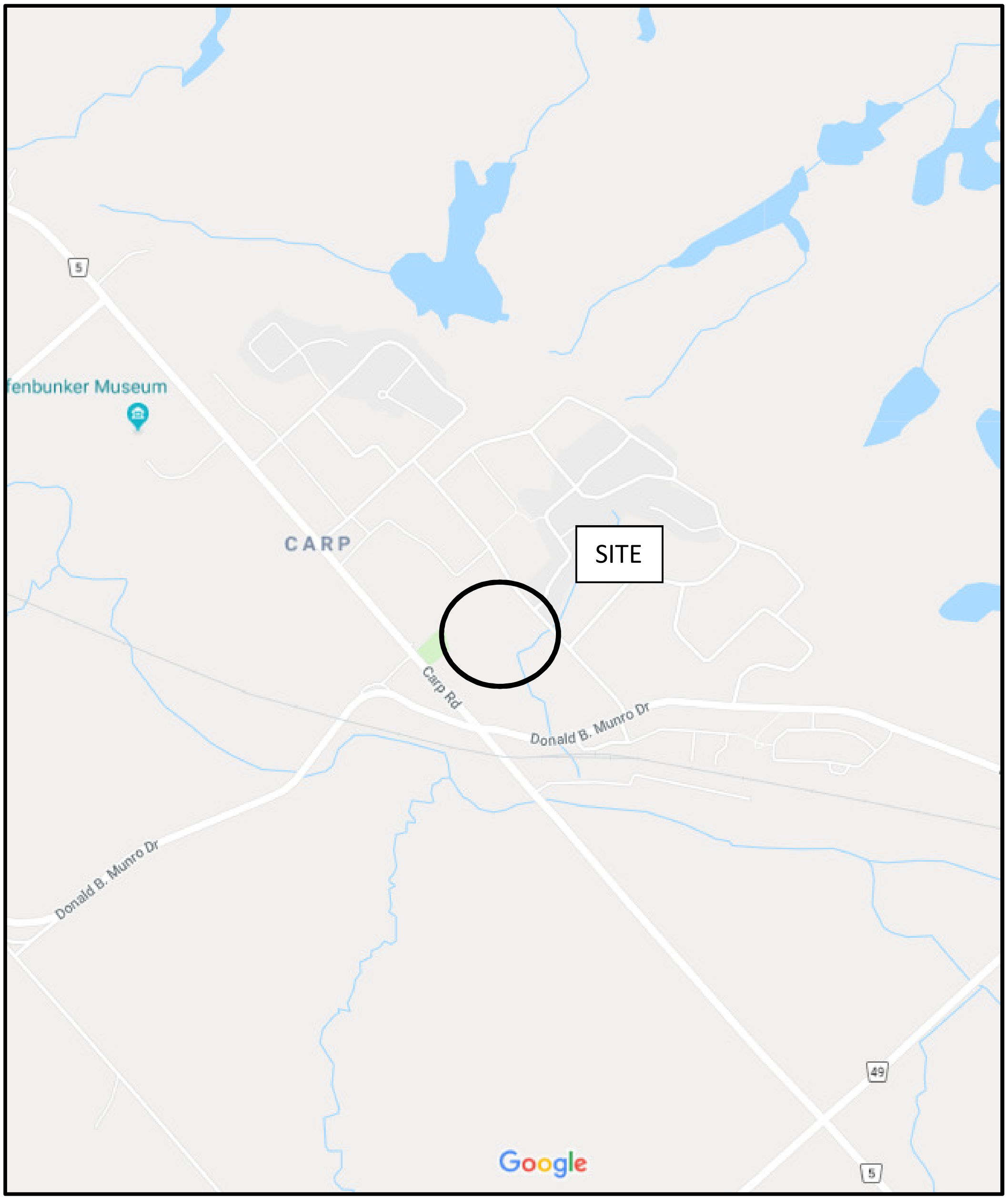


FIGURE 1

KEY PLAN

patersongroup

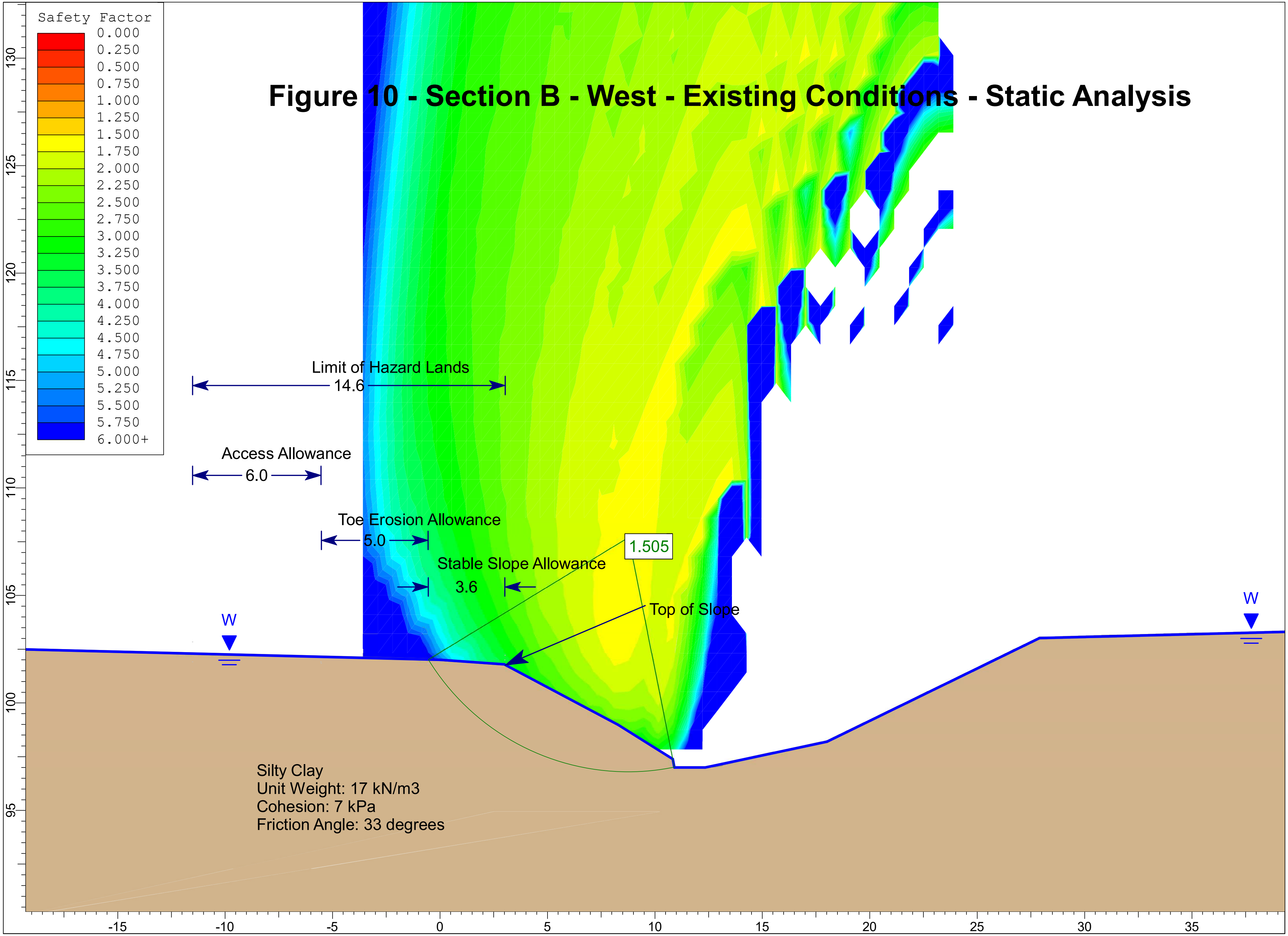


Figure 10 - Section B - West - Existing Conditions - Static Analysis

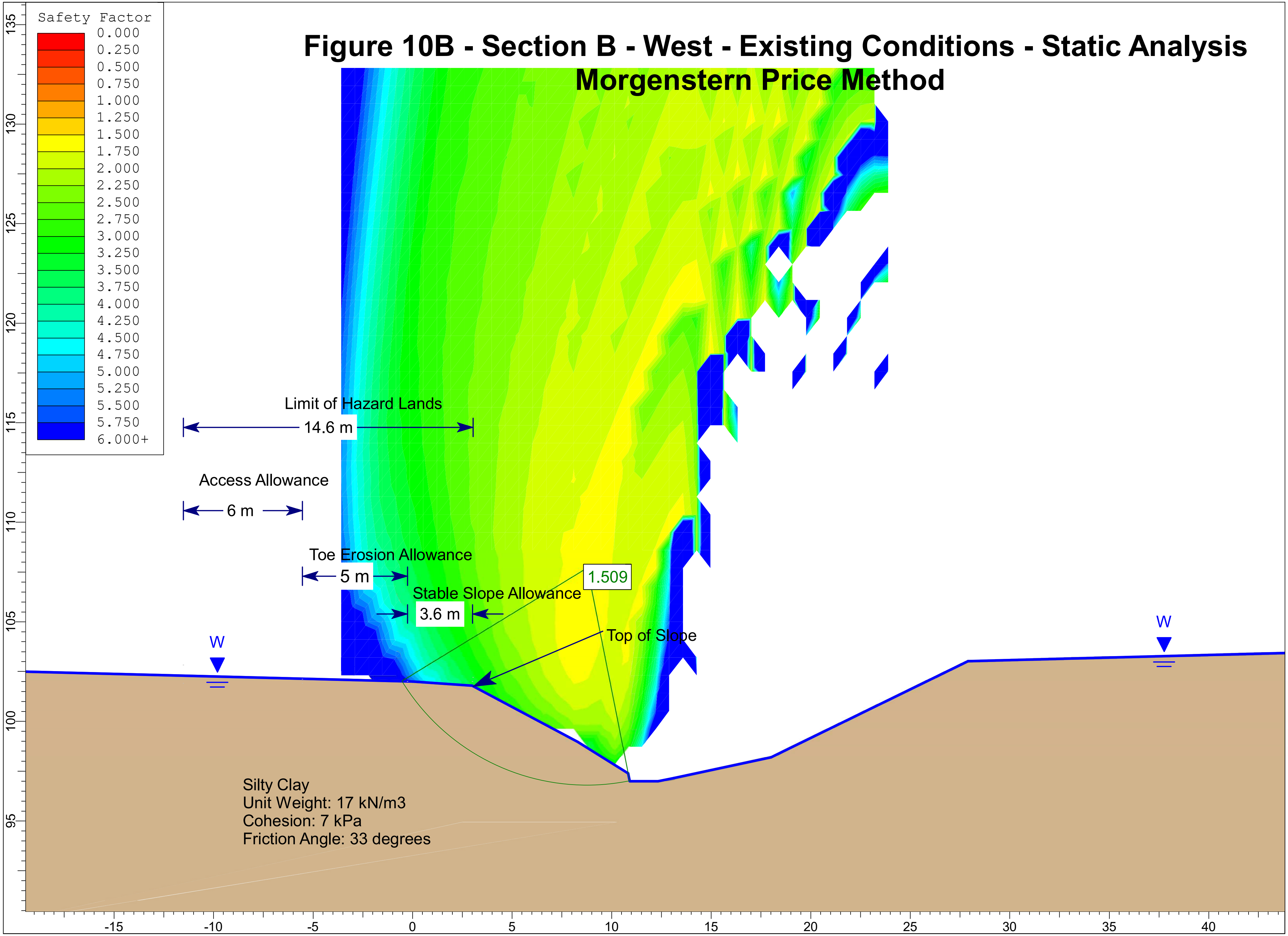
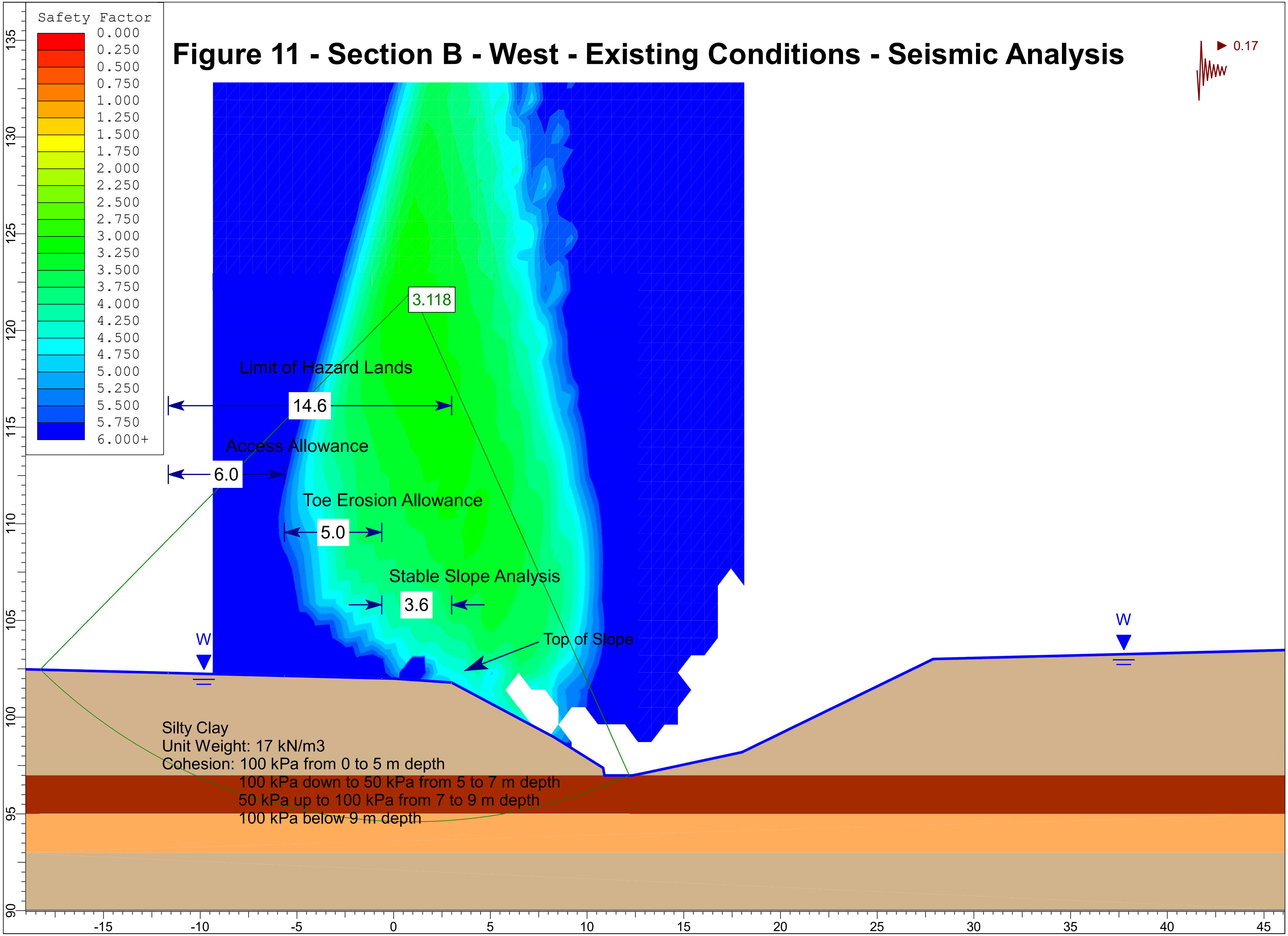


Figure 10B - Section B - West - Existing Conditions - Static Analysis
Morgenstern Price Method

Figure 11 - Section B - West - Existing Conditions - Seismic Analysis



Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 100 kPa from 0 to 5 m depth
100 kPa down to 50 kPa from 5 to 7 m depth
50 kPa up to 100 kPa from 7 to 9 m depth
100 kPa below 9 m depth

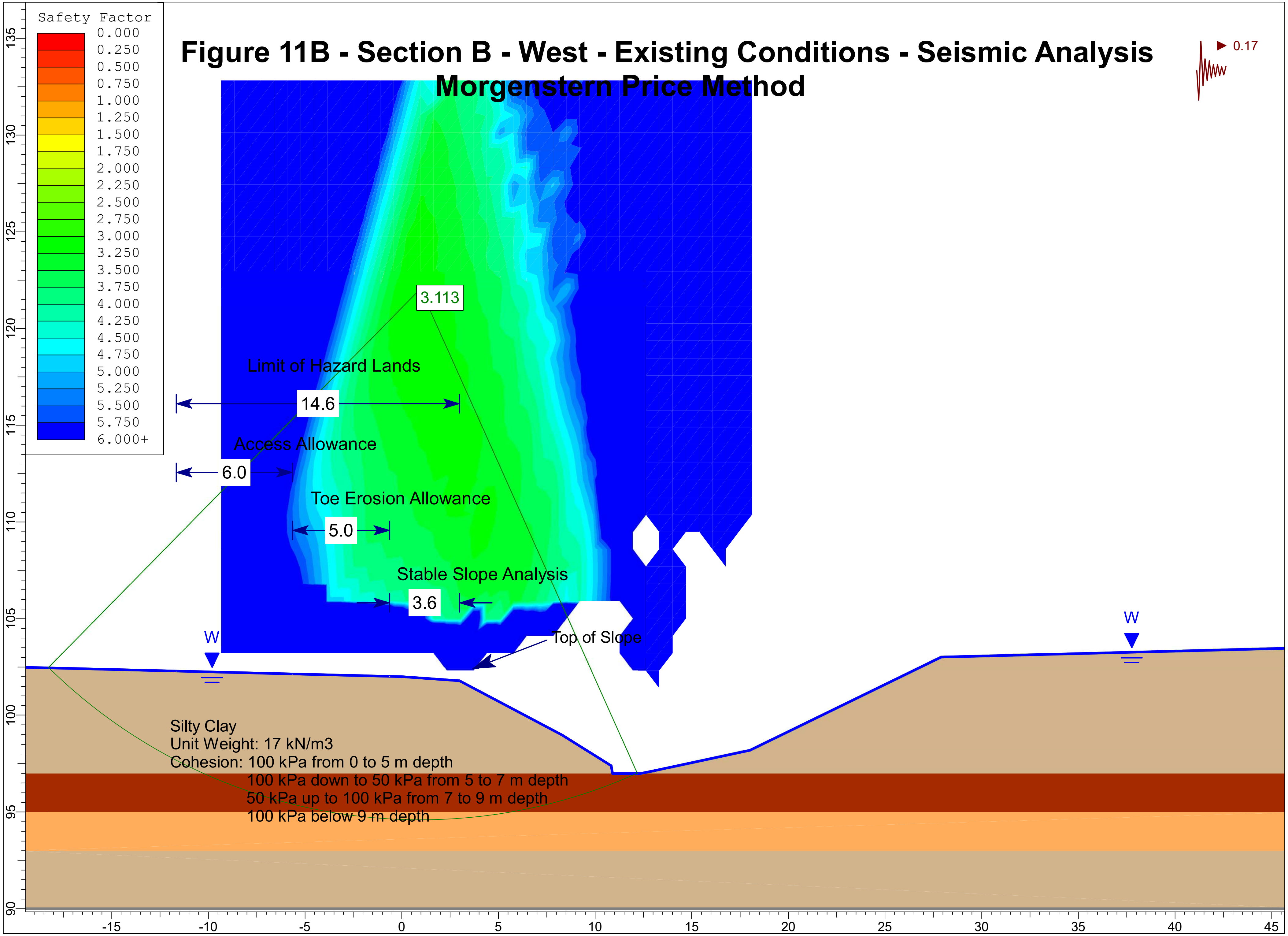


Figure 11B - Section B - West - Existing Conditions - Seismic Analysis
Morgenstern Price Method

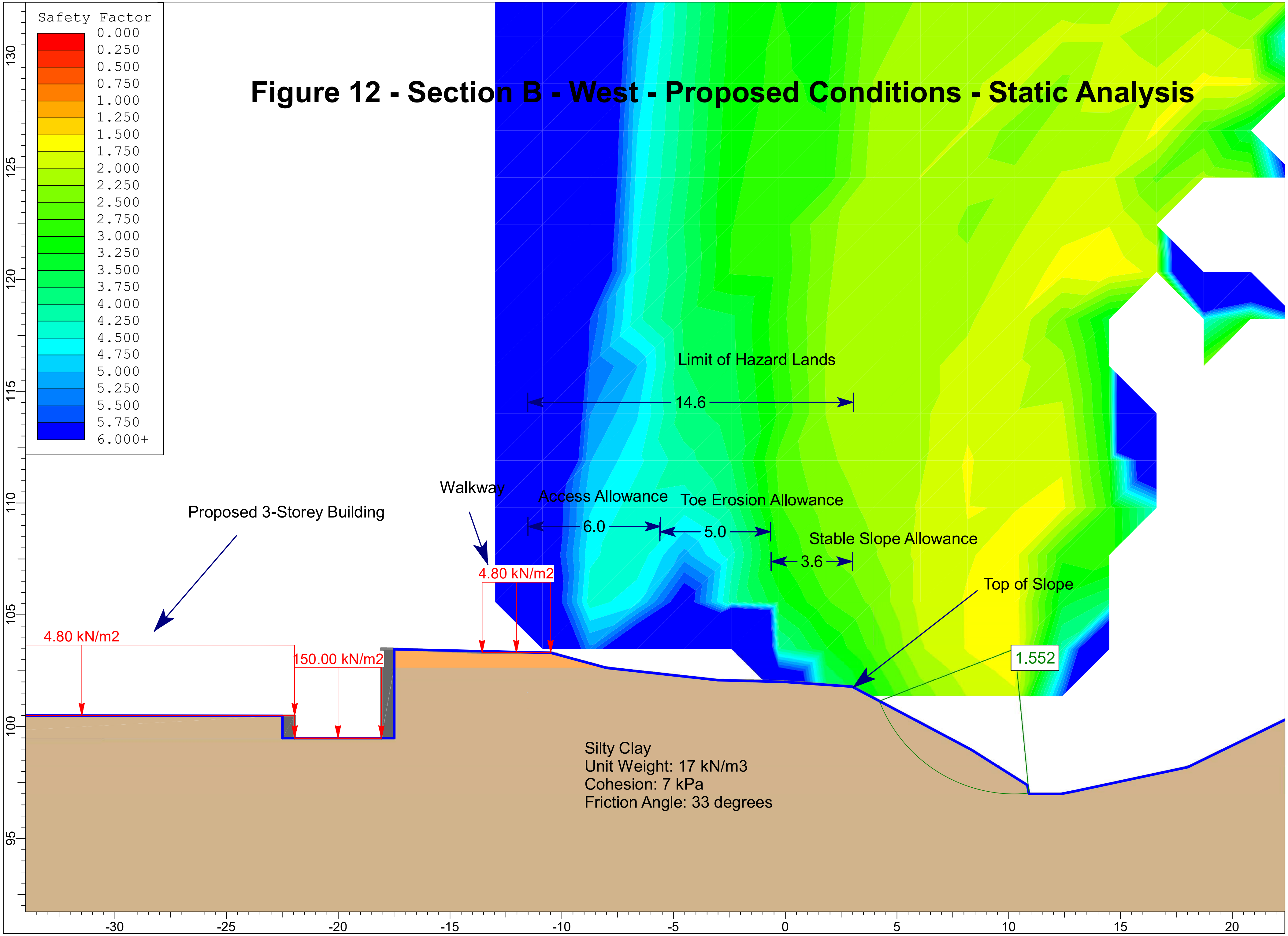
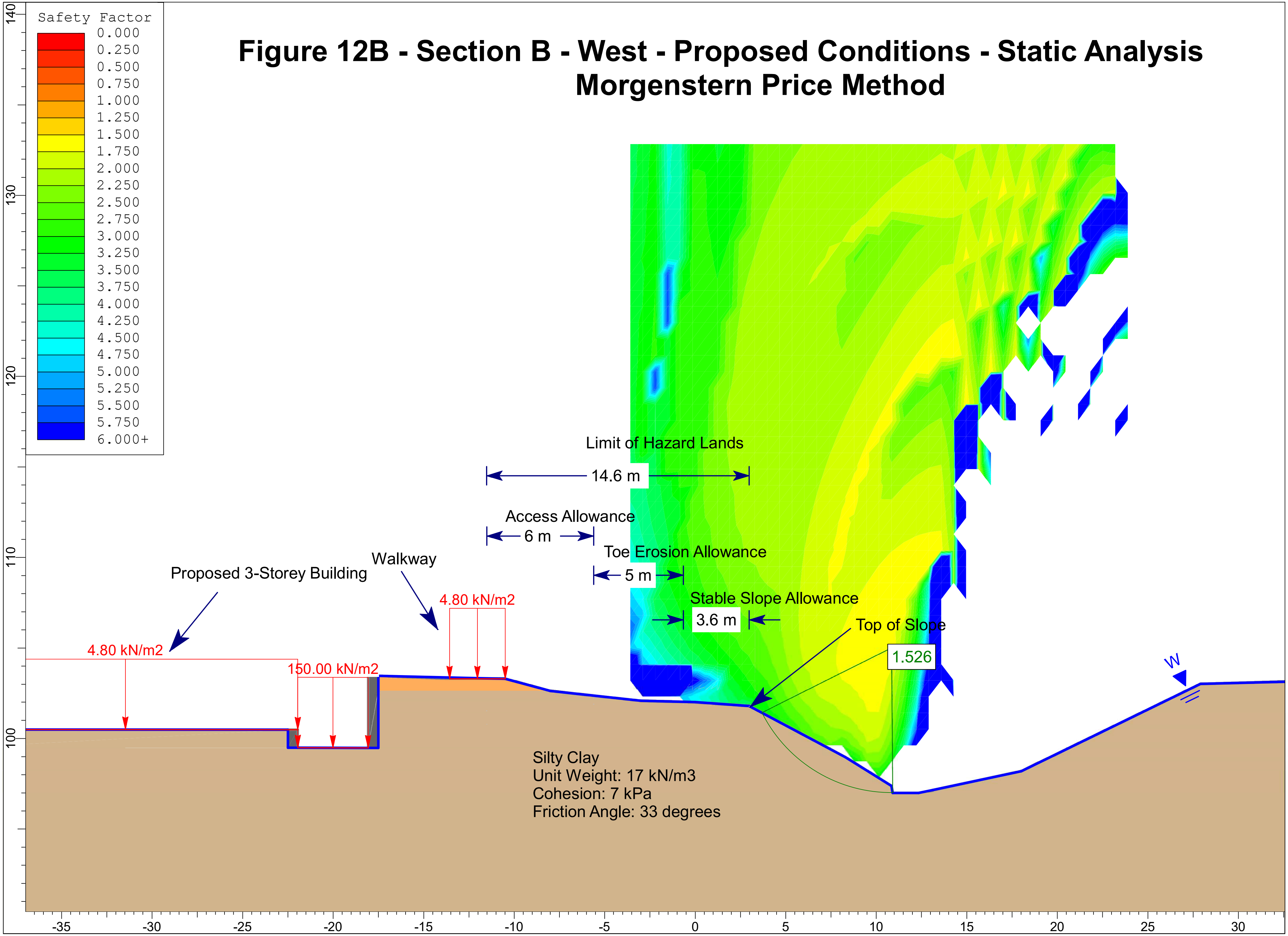


Figure 12 - Section B - West - Proposed Conditions - Static Analysis

Figure 12B - Section B - West - Proposed Conditions - Static Analysis
Morgenstern Price Method



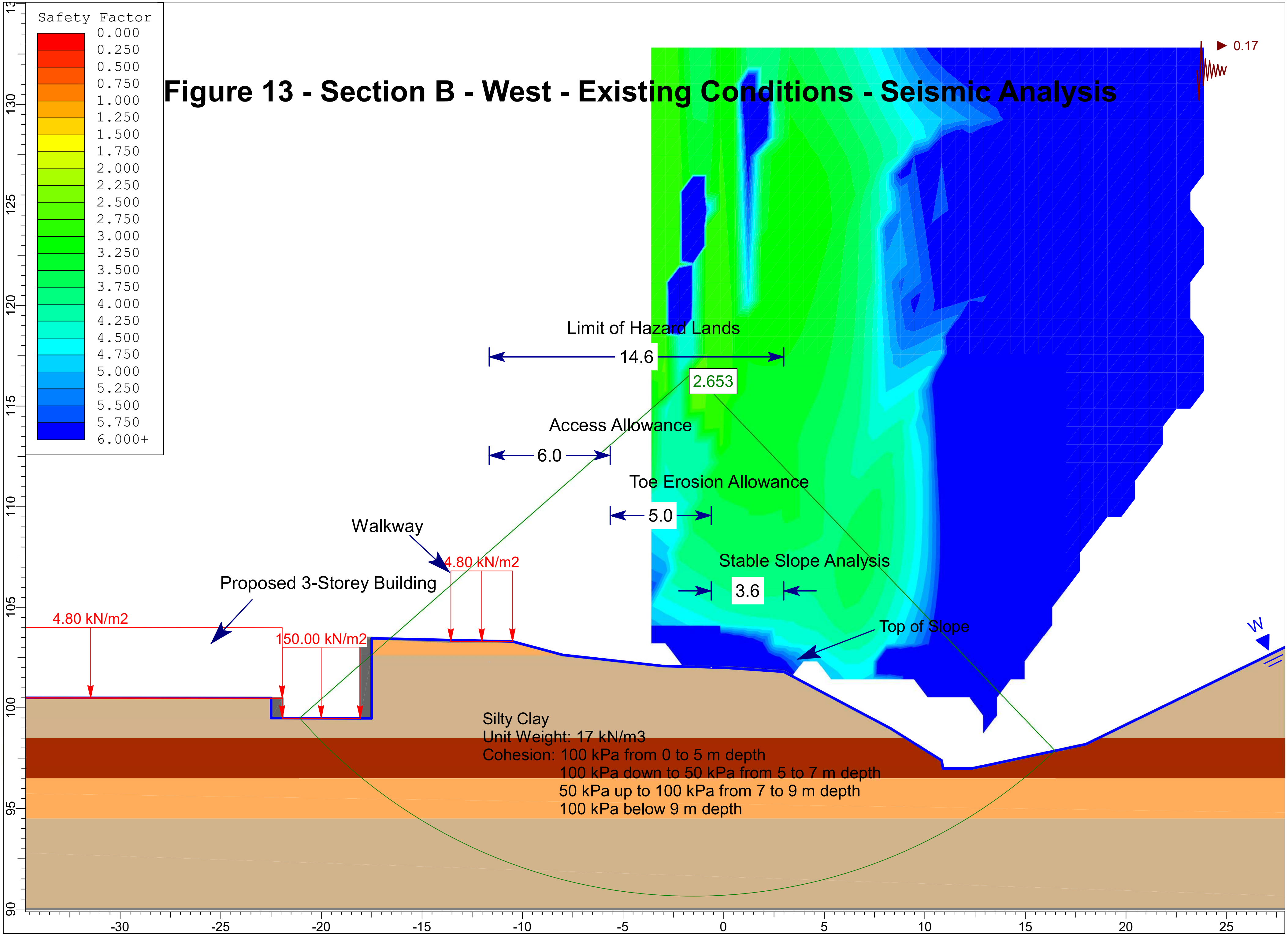
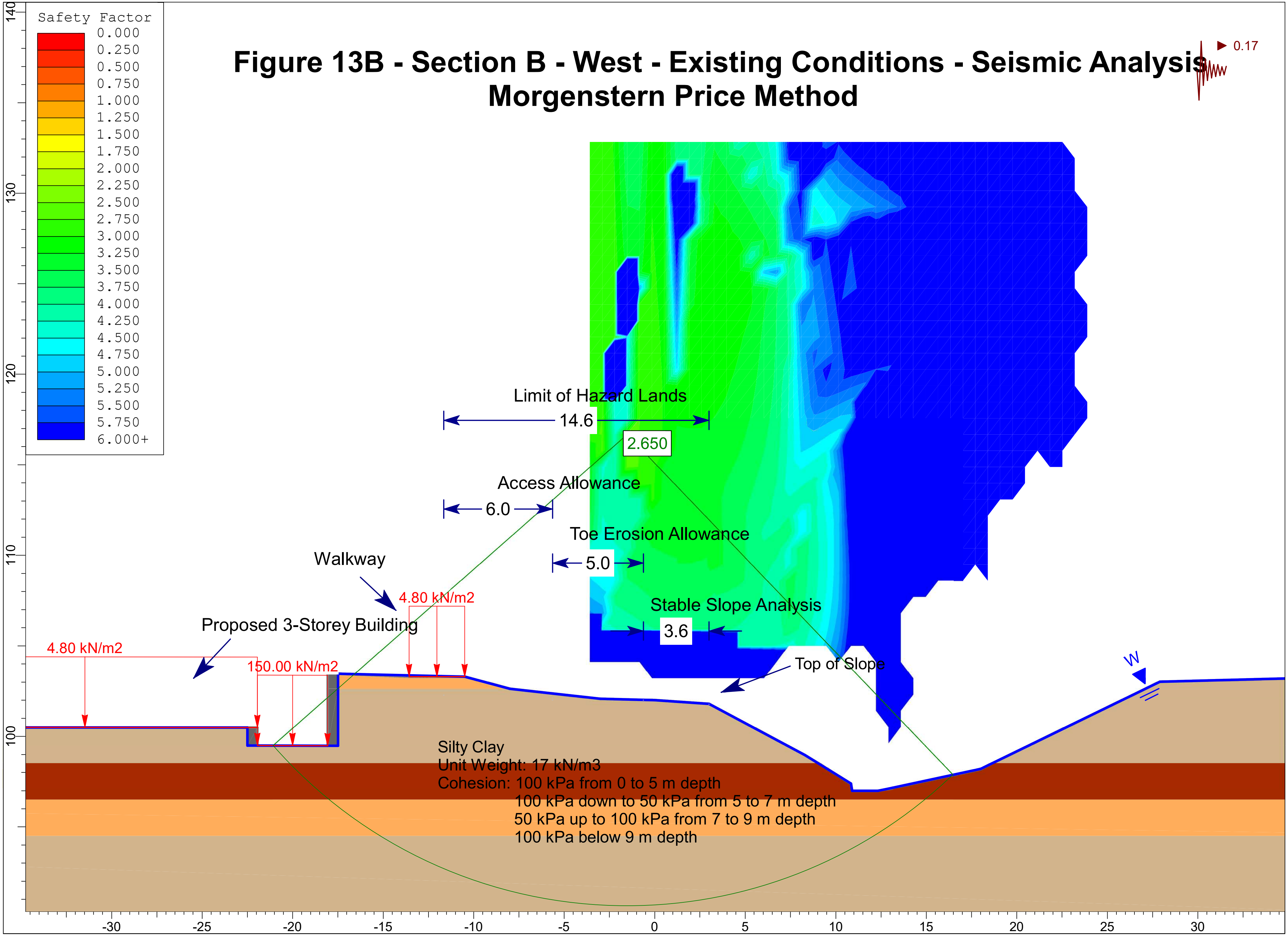


Figure 13 - Section B - West - Existing Conditions - Seismic Analysis

Figure 13B - Section B - West - Existing Conditions - Seismic Analysis
Morgenstern Price Method



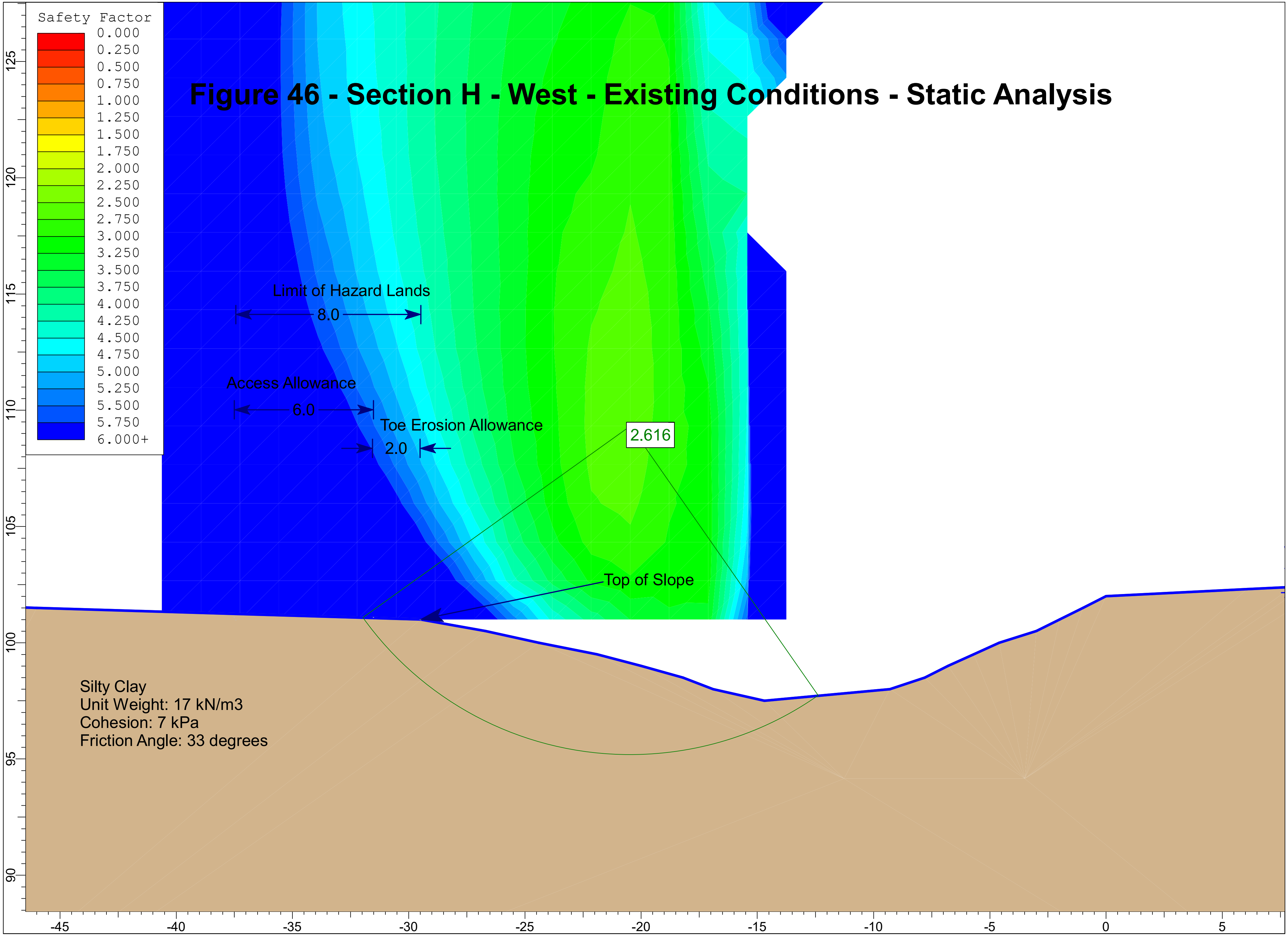


Figure 46 - Section H - West - Existing Conditions - Static Analysis

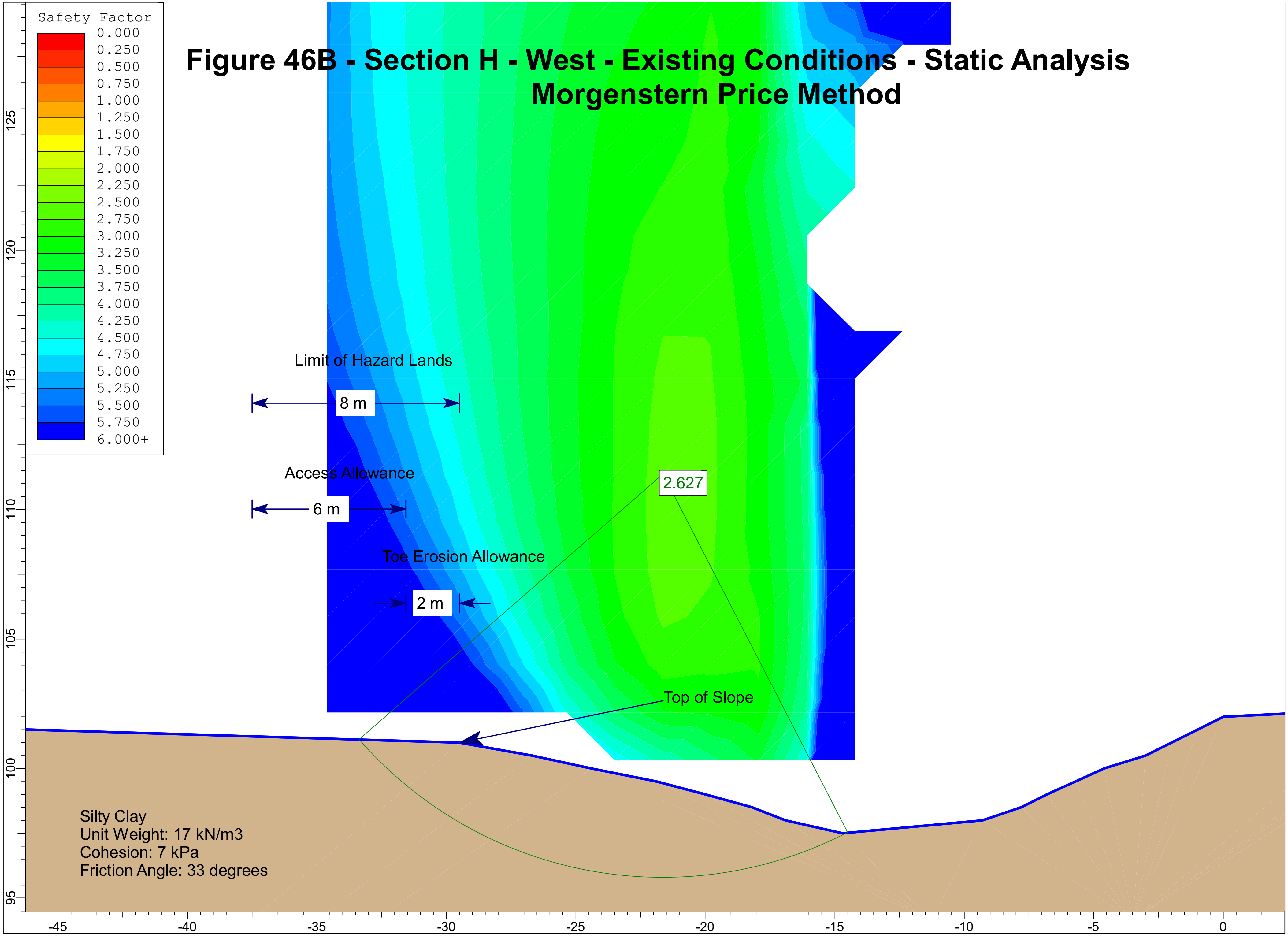


Figure 46B - Section H - West - Existing Conditions - Static Analysis
Morgenstern Price Method

Figure 47 - Section H - West - Existing Conditions - Seismic Analysis

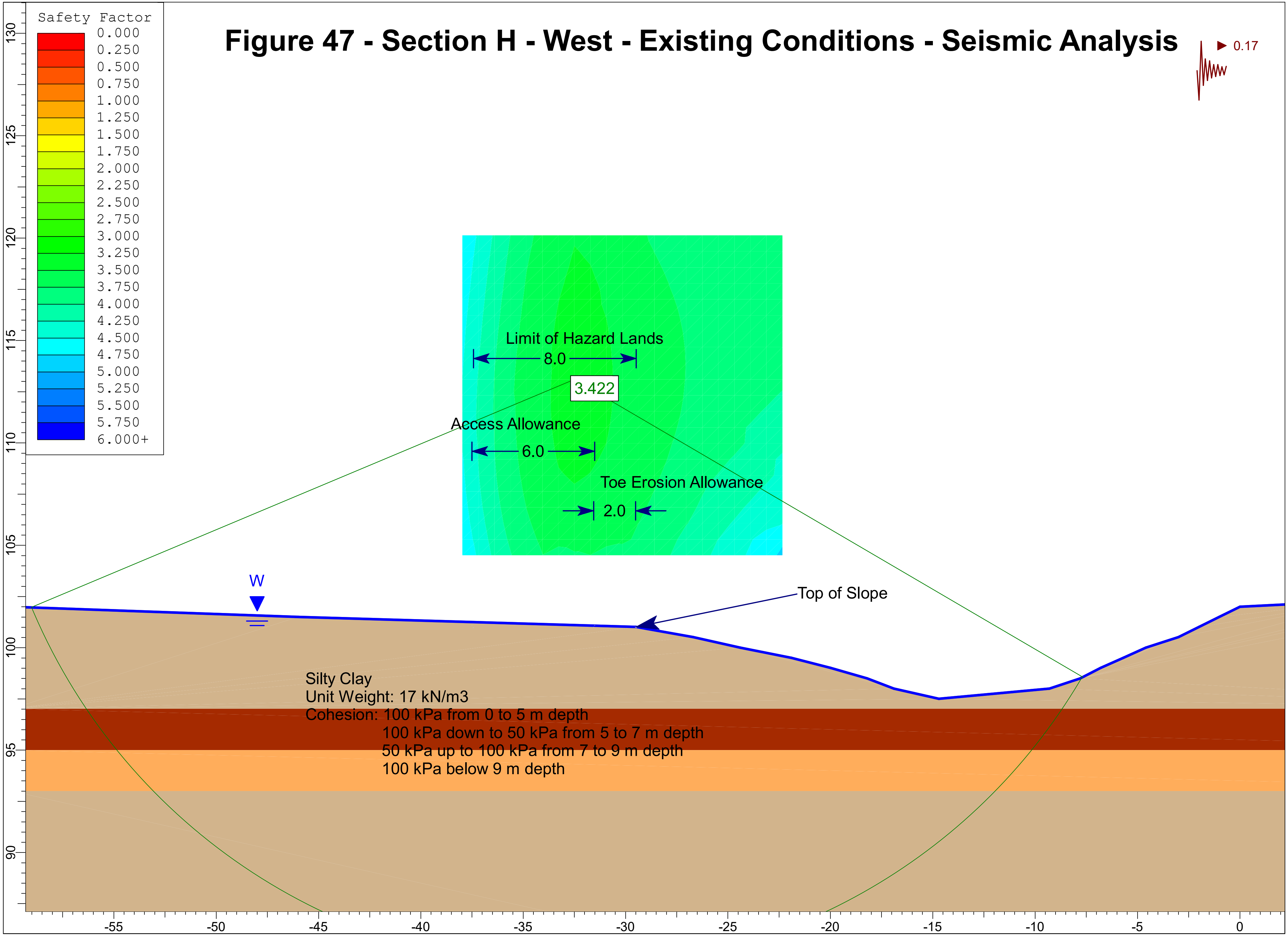


Figure 47B - Section H - West - Existing Conditions - Seismic Analysis
Morgenstern Price Method

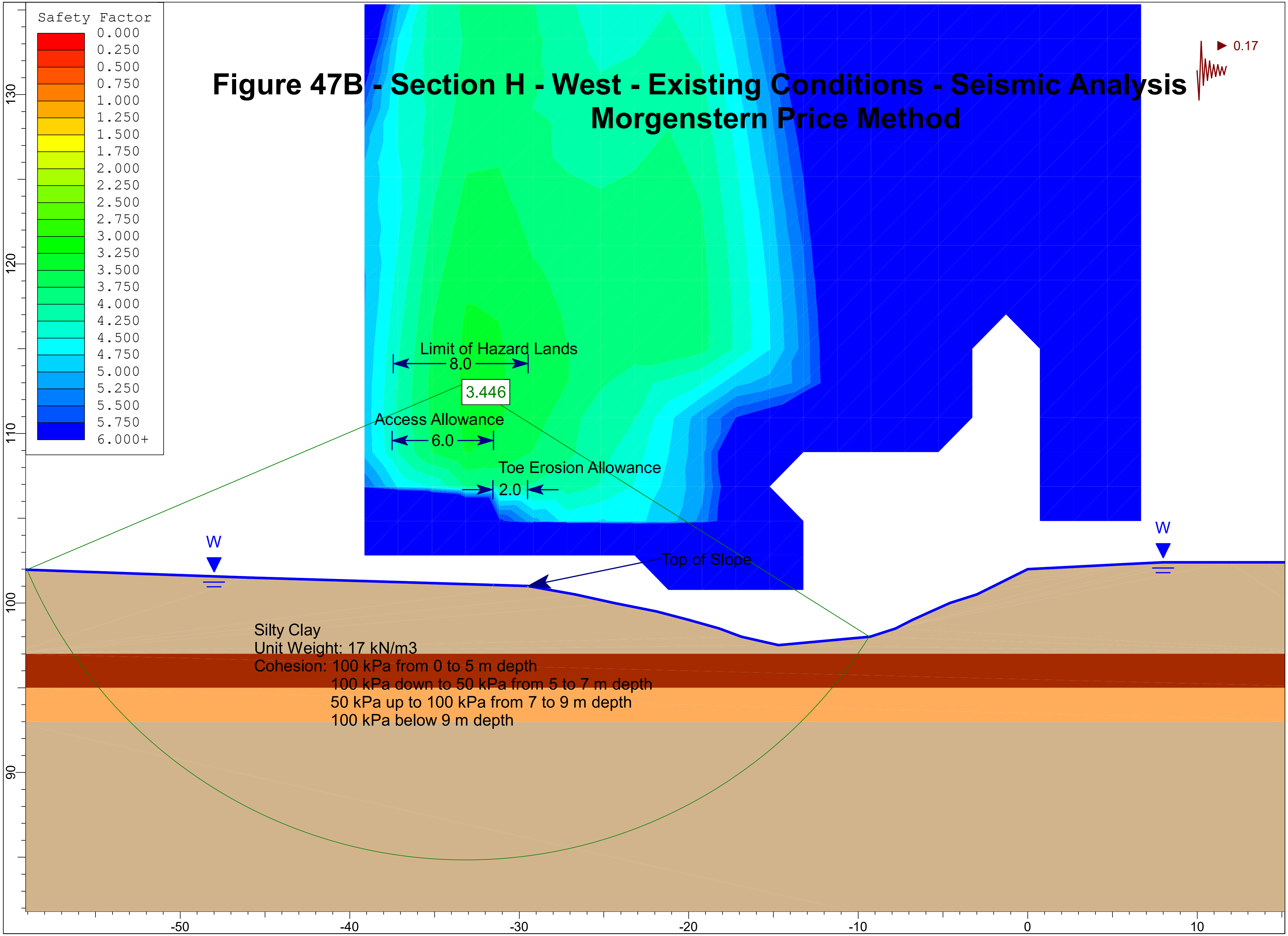
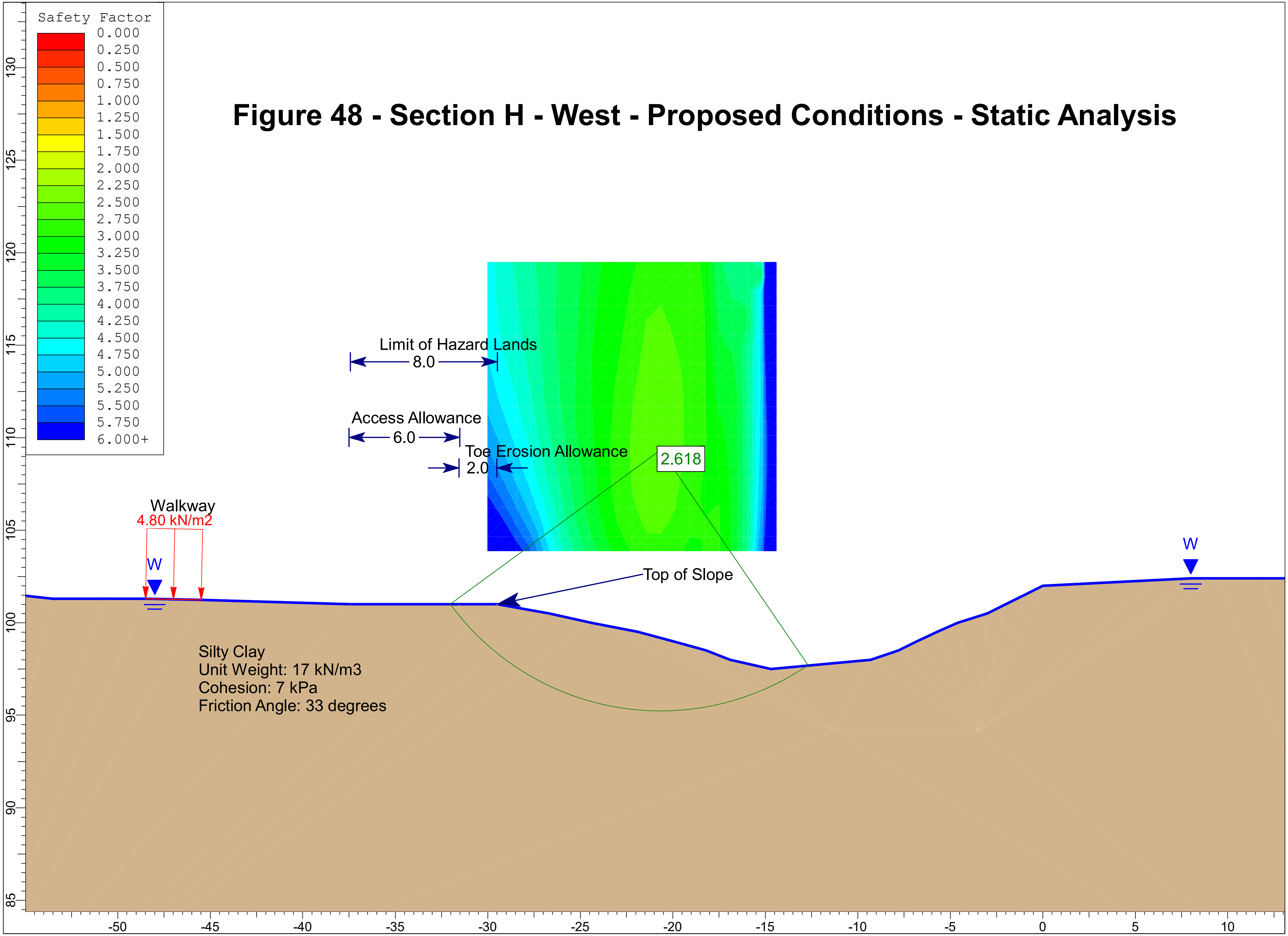


Figure 48 - Section H - West - Proposed Conditions - Static Analysis



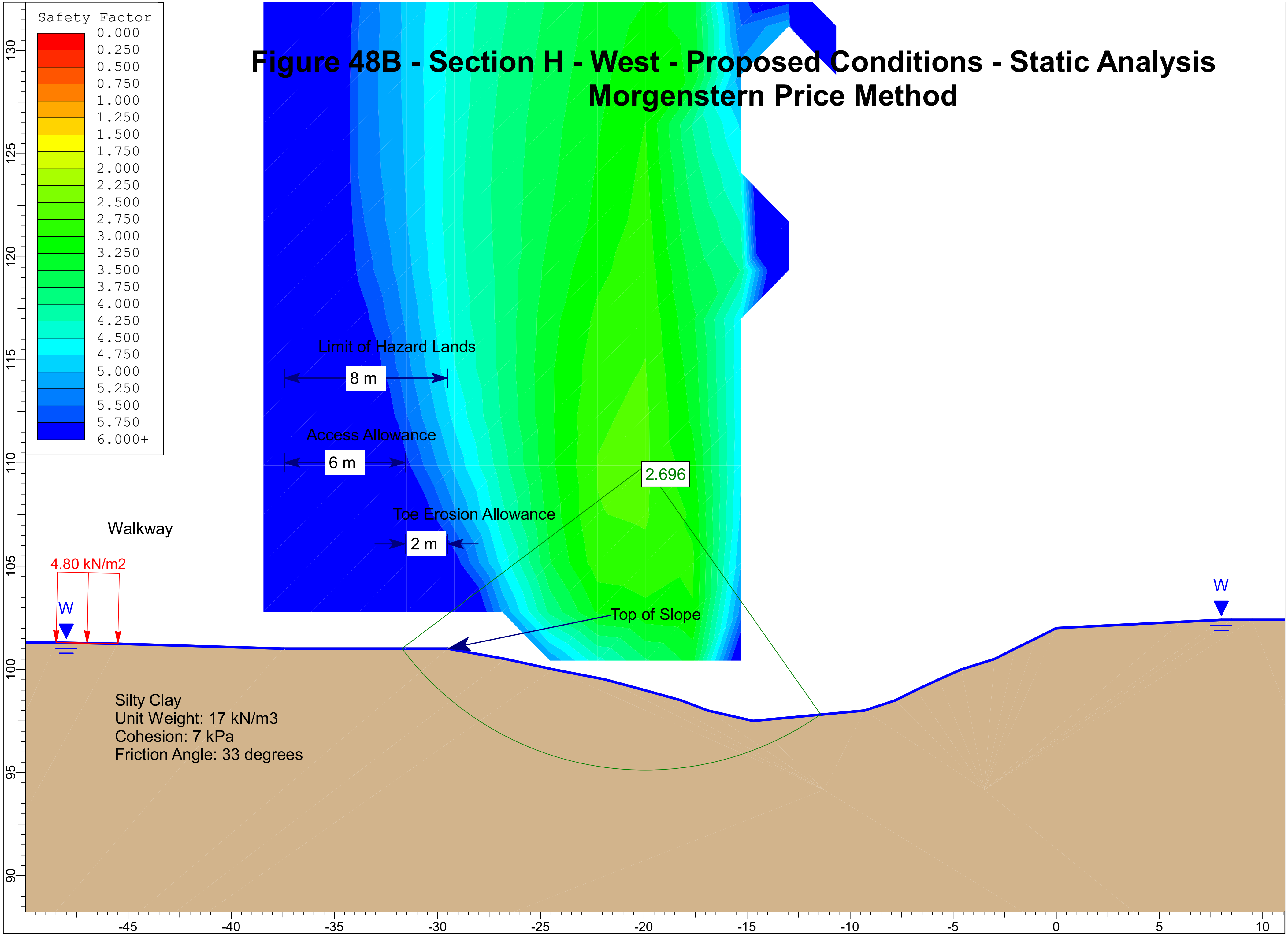


Figure 48B - Section H - West - Proposed Conditions - Static Analysis
Morgenstern Price Method

Figure 49 - Section H - West - Proposed Conditions - Seismic Analysis

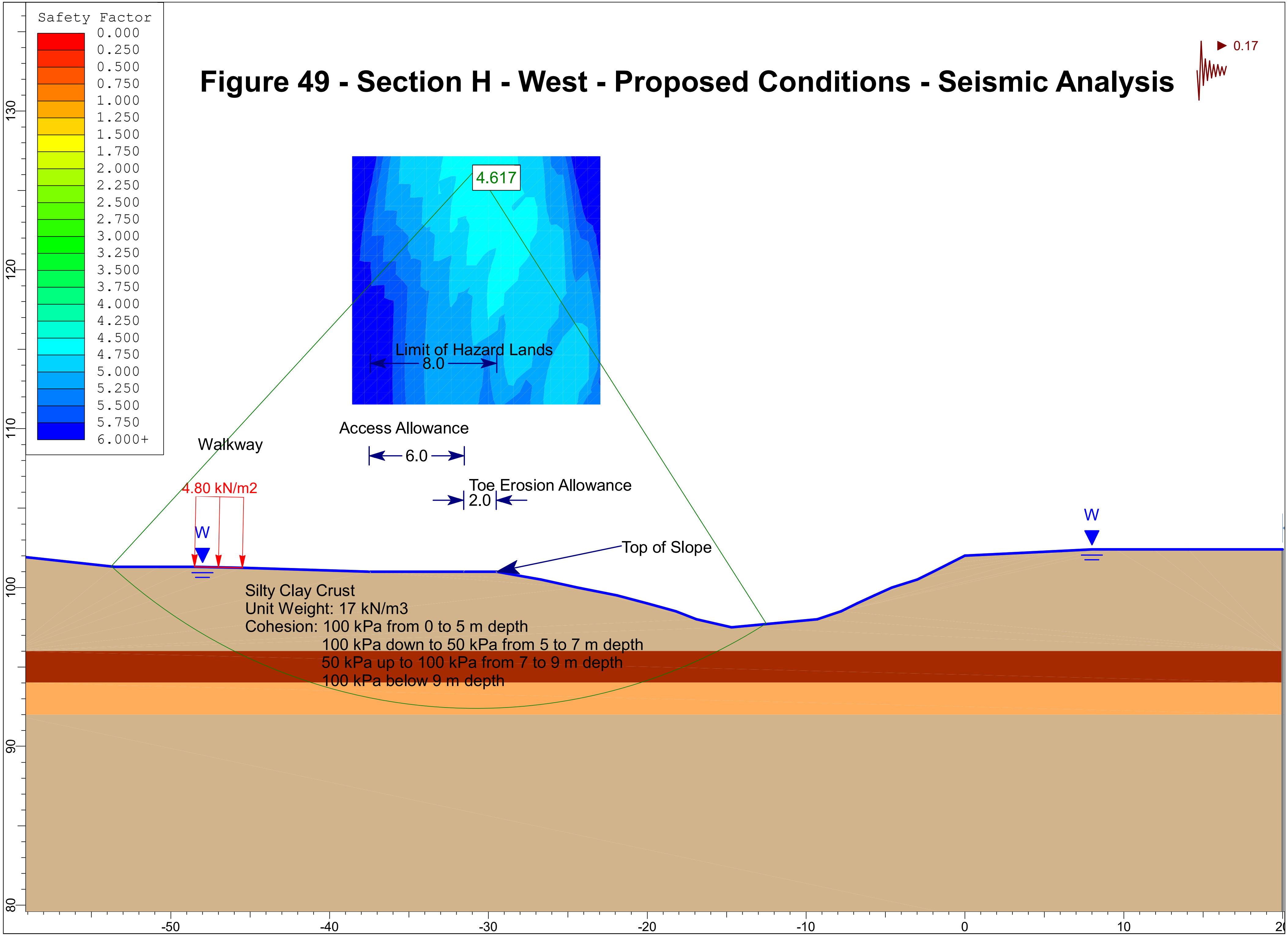
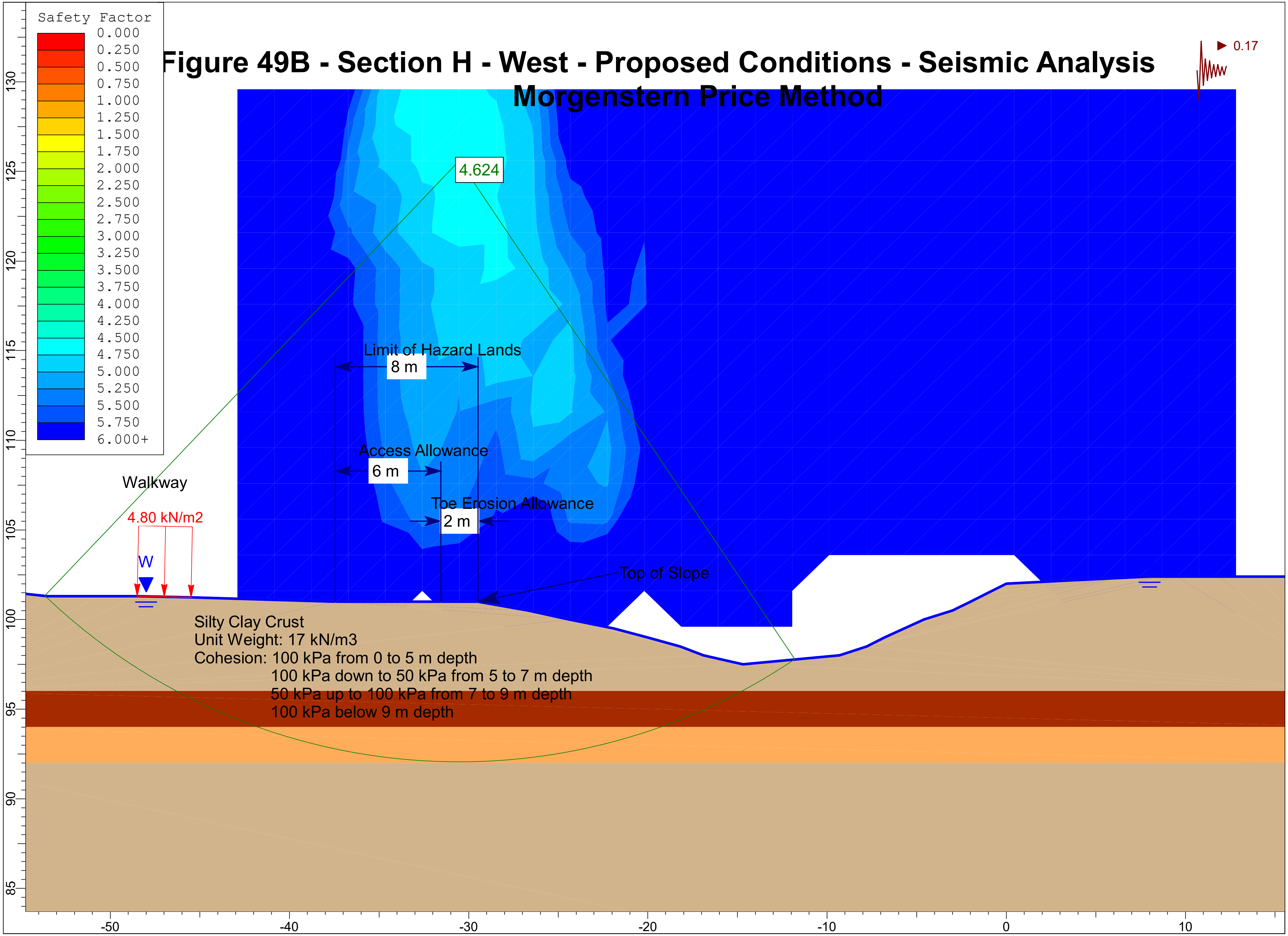


Figure 49B - Section H - West - Proposed Conditions - Seismic Analysis
Morgenstern Price Method



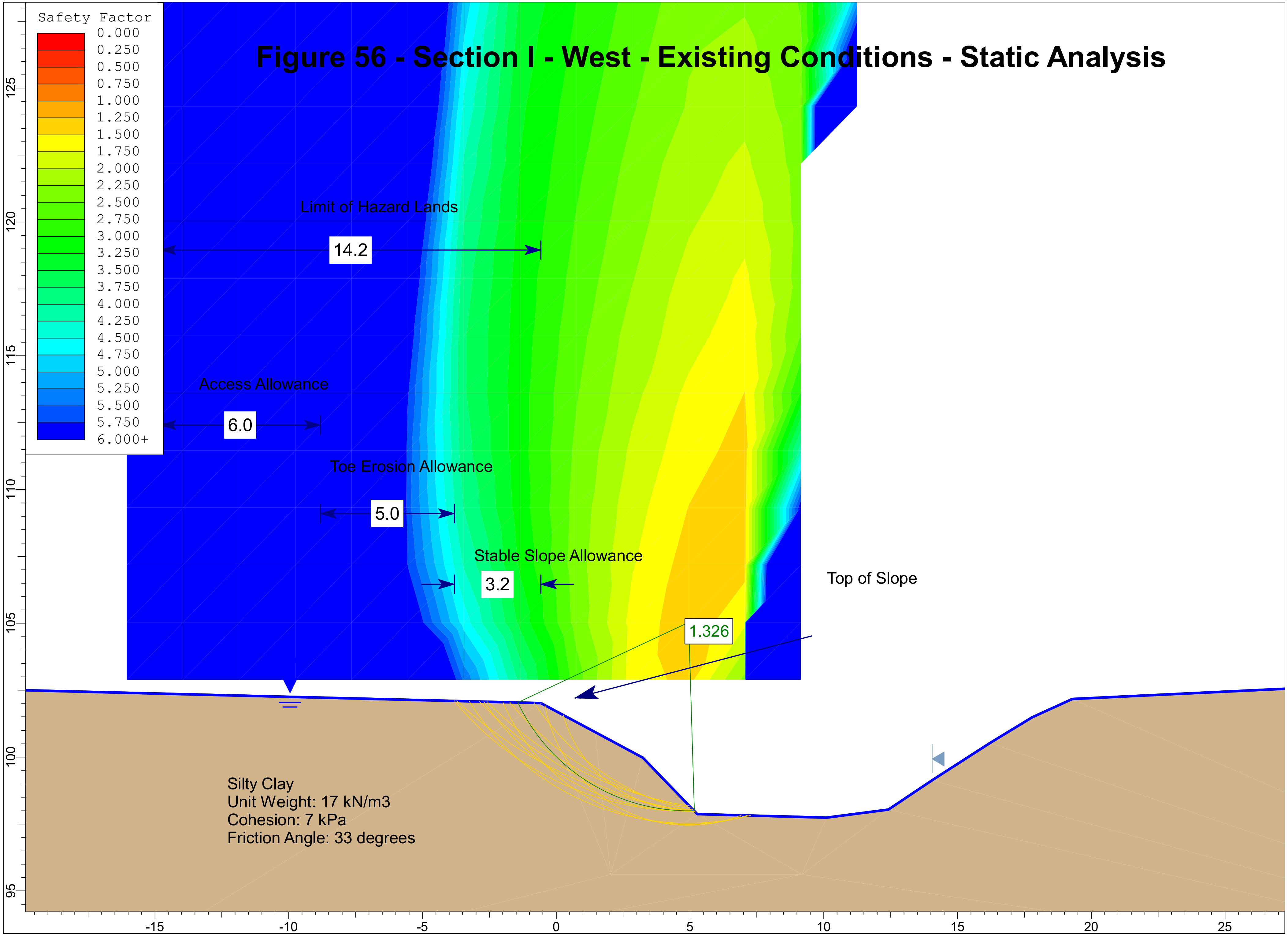
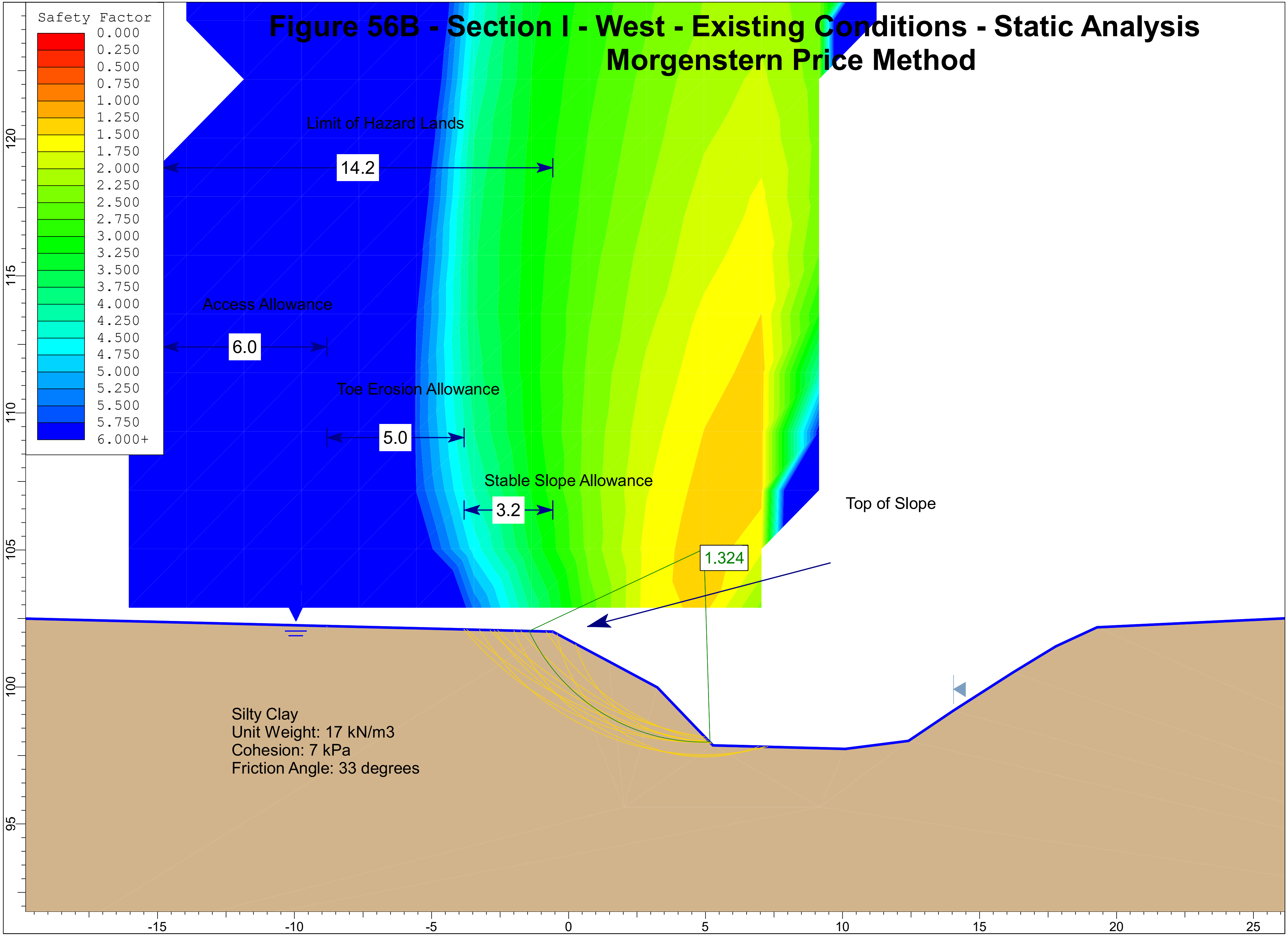


Figure 56 - Section I - West - Existing Conditions - Static Analysis

Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 7 kPa
Friction Angle: 33 degrees

Figure 56B - Section I - West - Existing Conditions - Static Analysis
Morgenstern Price Method



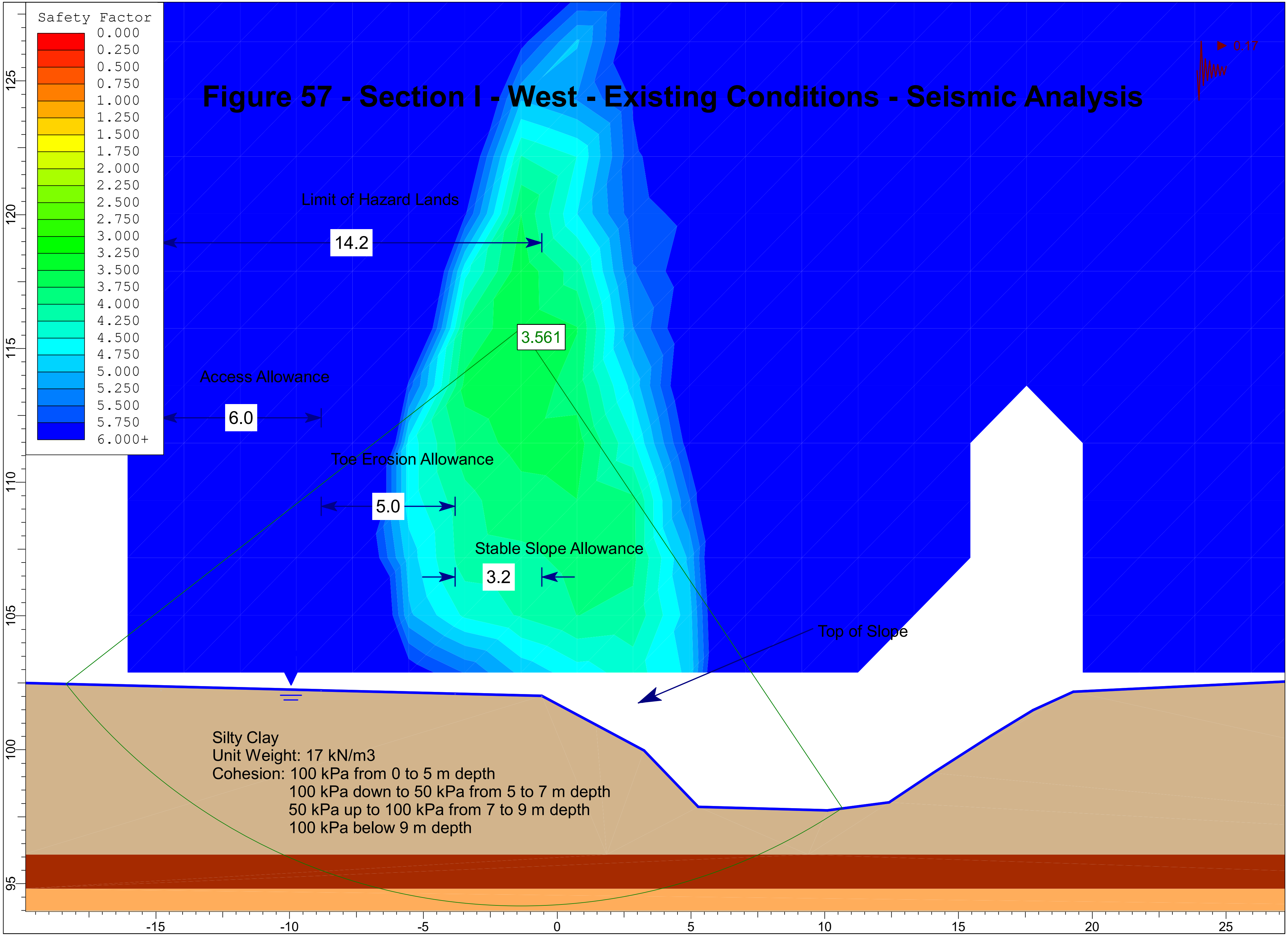
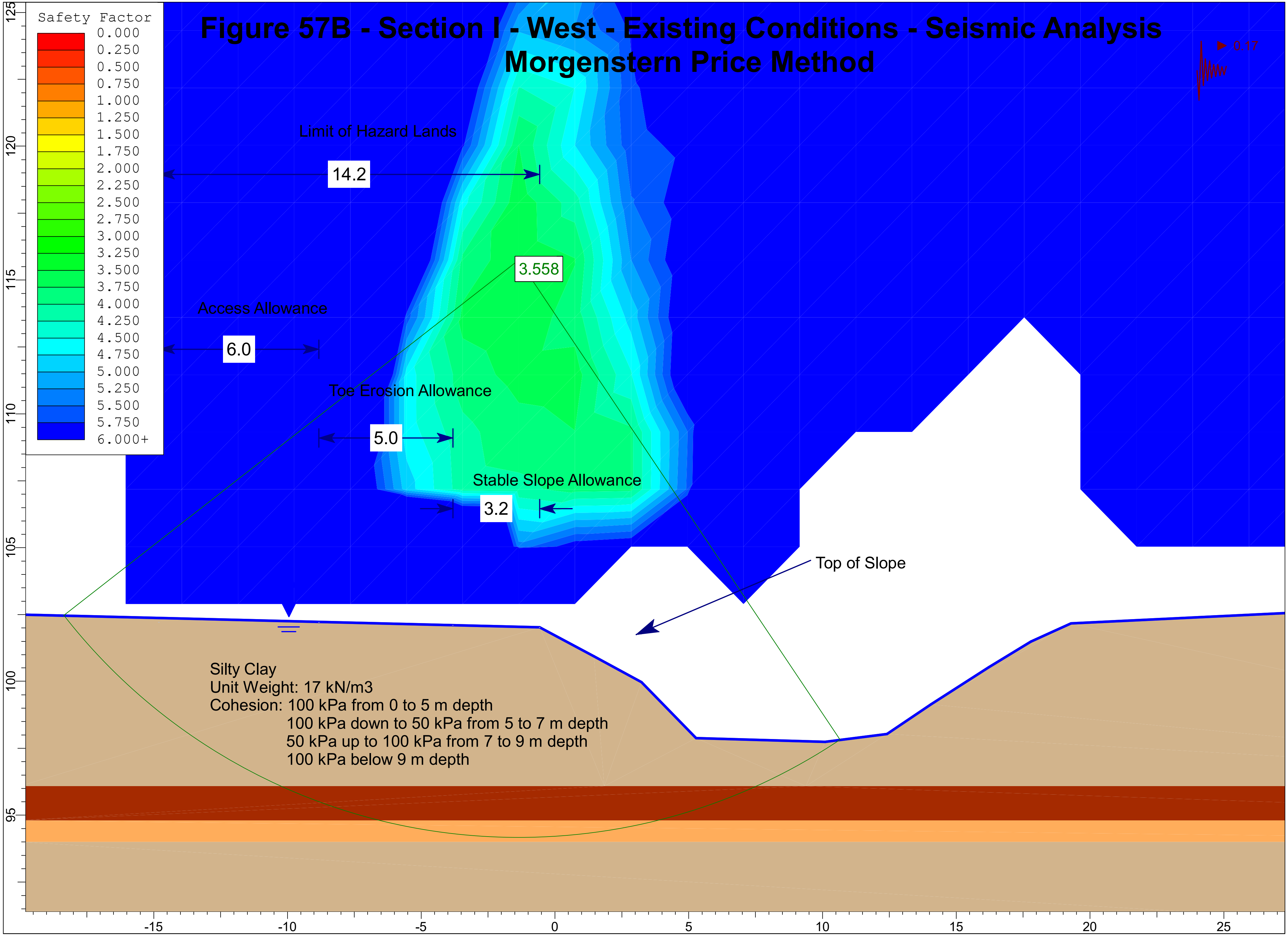


Figure 57 - Section I - West - Existing Conditions - Seismic Analysis

Figure 57B - Section I - West - Existing Conditions - Seismic Analysis
Morgenstern Price Method



Silty Clay
Unit Weight: 17 kN/m3
Cohesion: 100 kPa from 0 to 5 m depth
100 kPa down to 50 kPa from 5 to 7 m depth
50 kPa up to 100 kPa from 7 to 9 m depth
100 kPa below 9 m depth

Figure 58 - Section I - West - Proposed Conditions - Static Analysis

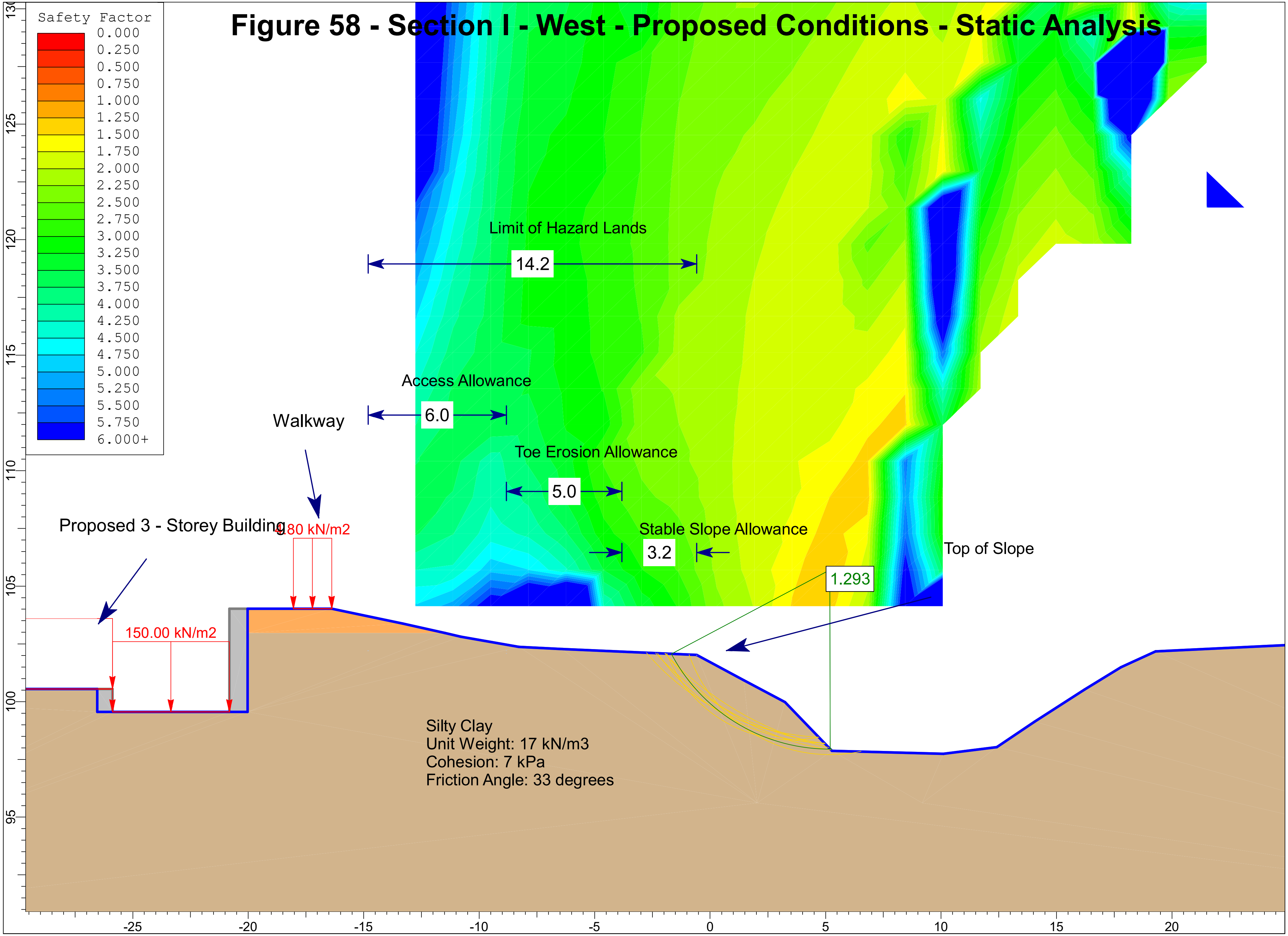
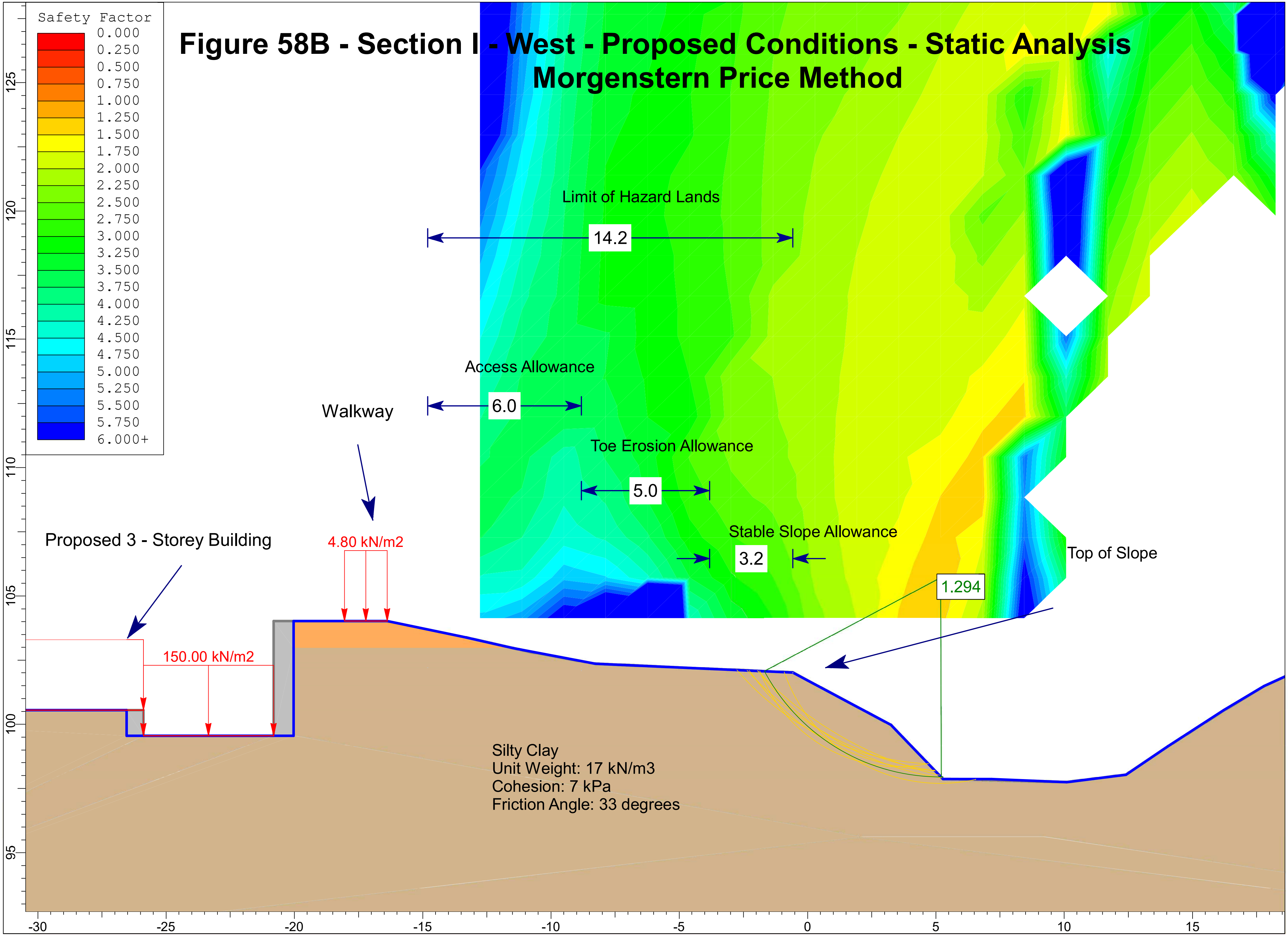


Figure 58B - Section I - West - Proposed Conditions - Static Analysis
Morgenstern Price Method



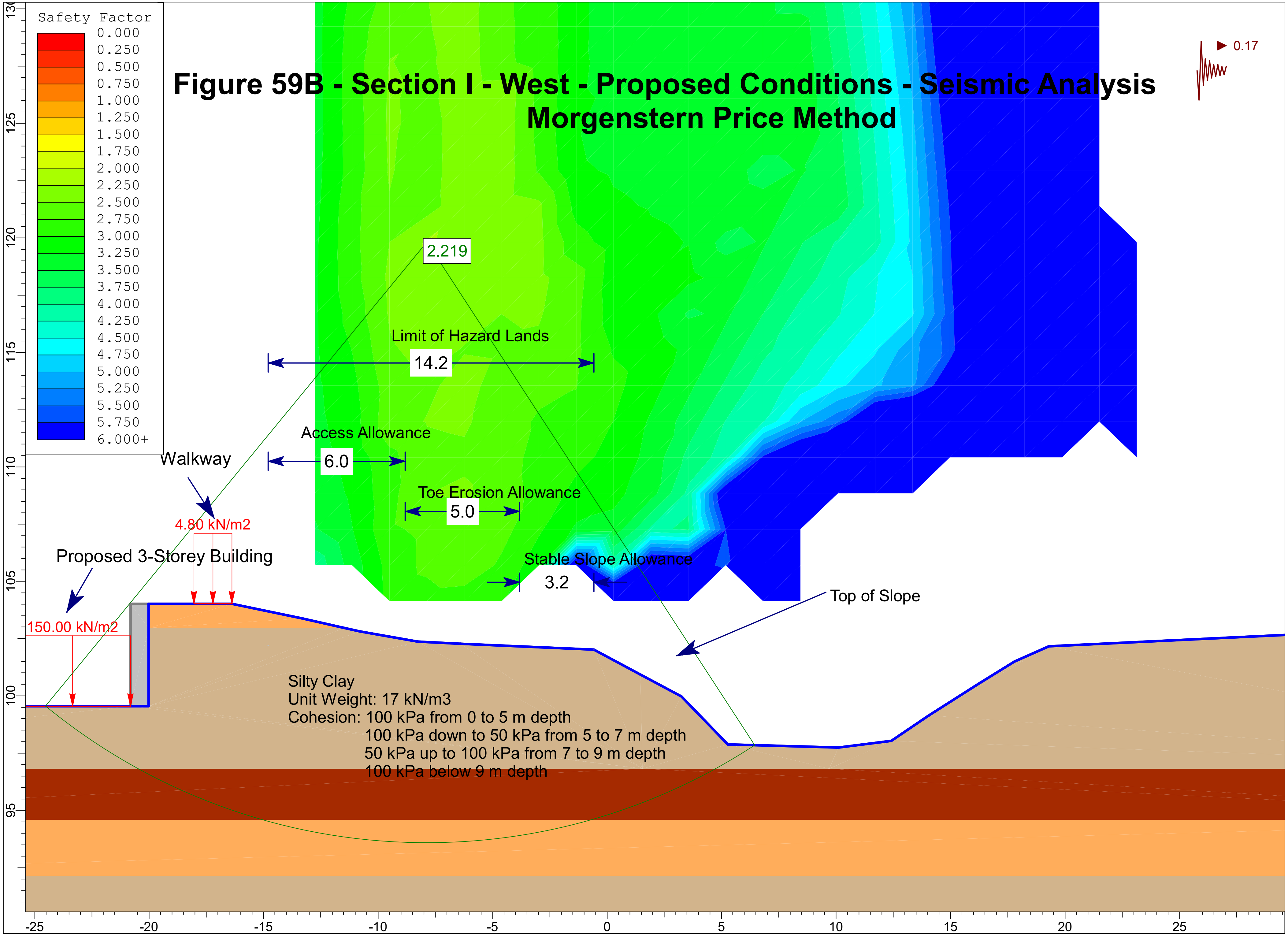
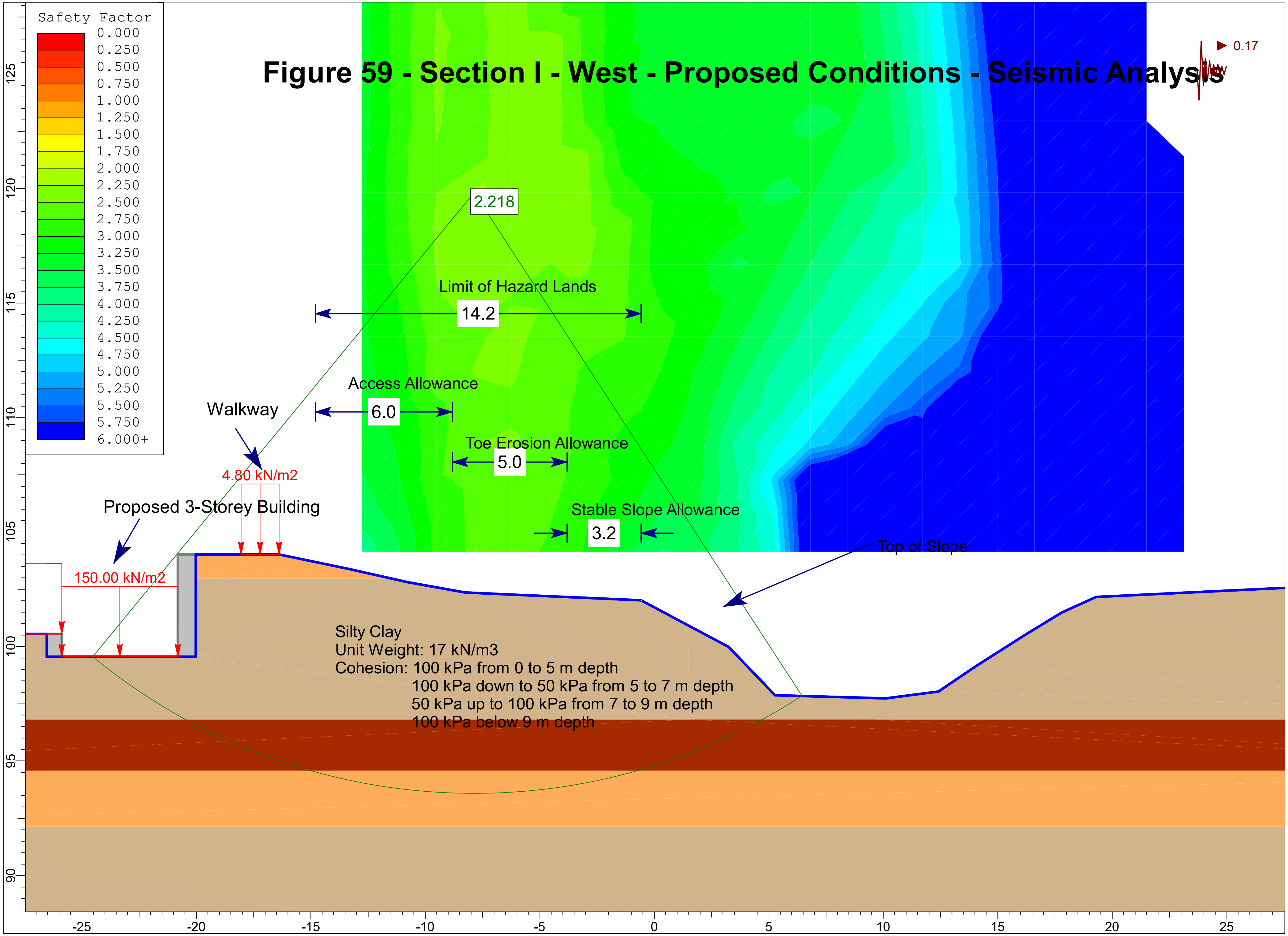
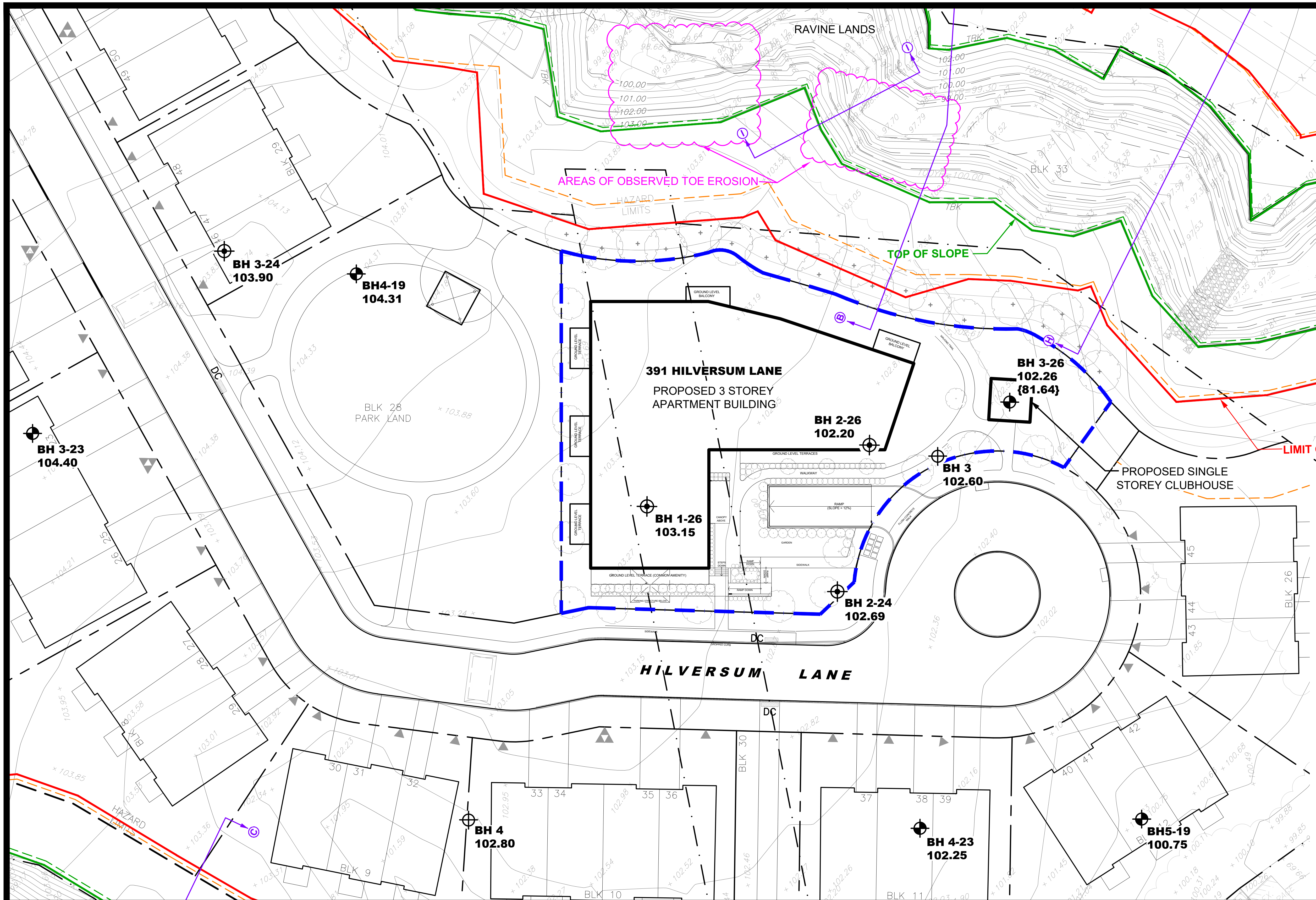


Figure 59B - Section I - West - Proposed Conditions - Seismic Analysis
Morgenstern Price Method

Figure 59 - Section I - West - Proposed Conditions - Seismic Analysis





LEGEND:

⊕	BOREHOLE WITH MONITORING WELL LOCATION
●	BOREHOLE LOCATION
⊕	BOREHOLE LOCATION, PATERSON GROUP REPORT PG1773, 2008
102.69	GROUND SURFACE ELEVATION (m)
{81.64}	PRACTICAL REFUSAL TO DCPT ELEVATION (m)
Ⓑ Ⓑ	CROSS SECTION LOCATION

BASE PLAN PROVIDED BY ROBINSON CONSULTANTS INC.

GROUND SURFACE ELEVATIONS AT 2023 AND 2024 BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

2019 TEST HOLE LOCATIONS AND GROUND SURFACE ELEVATIONS AT TEST HOLES PROVIDED BY ROBINSON LAND DEVELOPMENT

SCALE: 1:600

0 5 10 15 20 25 30m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

INVERNESS HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - PHASE 1 APARTMENT BUILDING (BLOCK 27)
391 HILVERSUM LANE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 02/2026
Drawn by: ZS
Report No.: PG4918-2
Checked by: OM
Dwg. No.: PG4918-6
Approved by: SD
Revision No.:

p:\autocad drawings\geotechnical\pg49xx\pg4918\pg4918-6-test hole location plan.dwg

11x17