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Building Science

Noise and Vibration
Studies

Geotechnical Investigation
Proposed Residential Building
1185 Beaverwood Road
Ottawa, Ontario

Prepared For

ARK Construction Ltd.

June 8, 2023

Report: PG6160-1
Revision 6

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by ARK Construction Ltd. to conduct a geotechnical investigation for the proposed residential building to be located at 1185 Beaverwood Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- Determine the subsoil and groundwater conditions at this site by means of test holes.

- Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating for the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey residential building with a partial below-grade level which will daylight to the east. At finished grades, the proposed building will be surrounded by landscaped areas and asphalt-paved access lanes and parking areas. It is also understood that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on March 1, 2022 and consisted of advancing a total of 4 boreholes to a maximum depth of 4.5 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG6160-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low clearance 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. The testing procedure consisted of augering and excavating to the required depth at the selected location and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The 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.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Borehole BH 4-22 was fitted with a 51 mm diameter PVC groundwater monitoring well. The other boreholes were fitted with flexible piezometers to allow for groundwater level monitoring. The groundwater observations are discussed in Section 4.3 and are presented on the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the boreholes, and the ground surface elevation at each borehole location, are presented on Drawing PG6160 - 1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 2 grain size distribution analyses and 1 Atterberg limit test were completed on selected soil samples. The results of the testing are presented in Section 4.2 and on the Grain Size Distribution and Hydrometer Testing Results, and Atterberg Limits Testing Results sheets presented 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 samples were submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a residential dwelling and detached shed, which are located on the western portion of the site. The remainder of the site generally consists of landscaped areas. The site is bordered by Beaverwood Road to the south, Scharfield Road to the east, and residential properties to the north and west. The existing ground surface across the site slopes downward moderately from west to east, from approximate geodetic elevation 94 m at the west property line, down to approximate geodetic elevation 90 m at the east property line.

4.2 Subsurface Profile

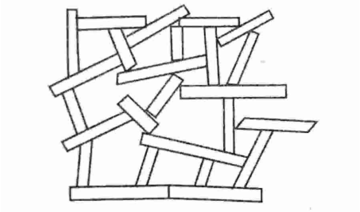
Generally, the subsurface profile at the test hole locations consists of a thin layer of topsoil or asphalt overlying a layer of fill extending to depths ranging from 0.2 to 1.2 m below the existing ground surface. The fill was generally observed to consist of a silty sand to silty clay with trace gravel.

With the exception of borehole BH 2-22, a hard to stiff silty clay was encountered underlying the fill, extending to approximate depths of 1.5 to 3.4 m below the existing ground surface. Based on an Atterberg Limits test at borehole BH 4-22 from approximate depths of 2.2 to 2.9 m, the in-situ moisture content of the clay (45.2%) at this location and depth exceeds the measured liquid limit of 40%. The results of the Atterberg Limits test is provided in Table 1 on the next page.

These silty clay soils have a moisture content above the liquid limit while remaining in a solid state due to the flocculated structure of the silty clay particles within this deposit. A flocculated structure occurs when the thin but elongated, plate-like clay particles align in an end-to-end orientation due to attractive electrical forces at the time of deposition.

This is illustrated on Figure 1, below:

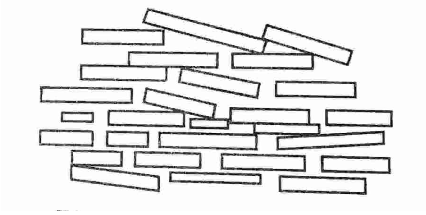
Figure 1 – Flocculated Clay Structure



The flocculated structure allows for significant water volume between the clay particles, while maintaining a solid structure.

When the clay is remoulded, it develops a dispersed structure, where the plate-like clay particles are oriented in a face-to-face configuration, which can hold far less water while remaining in a solid state, as compared to the undisturbed, flocculated configuration of the undisturbed silty clay. A dispersed structure is shown in Figure 2, below:

Figure 2 – Dispersed Clay Structure



With a liquidity index of 1.3, based on the results of the Atterberg Limits testing, the silty clay deposit at the subject site is considered to be very sensitive.

A glacial till deposit was generally encountered underlying the silty clay, consisting of a compact, brown silty sand to sandy silt with gravel, cobbles, and boulders.

Practical refusal to augering was encountered at approximate depths ranging from 0.2 m at the west end of the site, to 4.5 m at the east end of the site. Where auger refusal was encountered at depths of less than 2.5 m, a second borehole was drilled (BH 1A-22, BH 2A-22 and BH 3A-22) in the vicinity of the initial borehole, in order to confirm the refusal depth.

The shallow refusals (less than 2.5 m depth) are considered to be indicative of boulders in the glacial till deposit. The deeper refusal at borehole BH 4-22 at a depth of 4.5 m approximately coincides with the bedrock depths of 5 to 10 m in the available geological mapping, and is considered to be indicative of the bedrock surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic Dolomite of the Oxford formation, with an overburden drift thickness of 5 to 10 m depth.

Atterberg Limits Testing

Atterberg limits testing was completed on a recovered silty clay sample from borehole BH 4-22. The result of the Atterberg limits test is presented in Table 1 and on the Atterberg Limits Testing Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	w (%)	Classification
BH 4-22 SS4	2.2-2.9	40	23	17	45.2	CL

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

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on 2 selected soil samples. The results of the grain size analysis are summarized in Table 2 and are presented on the Grain-Size Distribution and Hydrometer Testing Results sheet in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 3-22	SS3	0.4	17.1	82.5	
BH 4-22	SS3	0.0	12.8	87.2	

4.3 Groundwater

Groundwater levels were measured in the monitoring wells and piezometers installed at the borehole locations on March 9 and November 25, 2022, and March 21, 2023. The measured groundwater levels noted at that time are presented in Table 3 below.

Table 3 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	March 9, 2022		November 25, 2022		March 21, 2023		May 4, 2023	
		Depth (m)	EL. (m)	Depth (m)	EL. (m)	Depth (m)	EL. (m)	Depth (m)	EL. (m)
BH 1-22	94.24	1.23	93.01	1.54	92.70	Not Found	-	1.36	92.88
BH 3-22	91.65	Dry	-	Not Found	-	Not Found	-	1.30	90.35
BH 4-22	90.67	3.14	87.53	3.95	86.72	2.10	88.57	0.98	89.69

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

Based on the site observations, the long-term groundwater table can be expected at approximate geodetic elevation 87 to 93 m. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed residential building. It is recommended that the proposed building be supported on conventional spread footings bearing on the undisturbed, compact to dense glacial till.

Where silty clay is encountered at the underside of footing (USF) elevation, it should be sub-excavated to the undisturbed, compact to dense glacial till and backfilled with engineered fill or lean concrete up to the USF elevation. The lateral limits of the engineered fill or lean concrete placement should be in accordance with our lateral support recommendations provided herein. Silty clay is expected to be encountered at the USF elevation in the southeast portion of the proposed building. Elsewhere, the undisturbed, compact to dense glacial till deposit is anticipated to be encountered at the USF elevation.

Based on the results of the geotechnical investigation, boulder removal is anticipated to be required to complete the basement levels and/or site servicing works. All contractors should be prepared for oversized boulder removal.

Due to the presence of the silty clay layer, the subject site will have a permissible grade raise restriction where the silty clay was observed. The permissible grade raise recommendations are discussed in Section 5.3.

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

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, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Boulder Removal

Boulder removal may be required at the subject site and can be accomplished by hoe ramming the boulders into smaller fragments, which then can be excavated and handled the same as other soils.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by others construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today’s construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill placed 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. The imported fill material should be tested and approved prior to delivery. The fill, where required, should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Lean Concrete Filled Trenches

As an alternative to placing engineered fill, where silty clay is encountered at the proposed building USF elevation, consideration should be given to excavating vertical trenches to the undisturbed, compact to dense glacial till, and backfilling with lean concrete to the founding elevation (minimum 17 MPa 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying undisturbed, compact to dense glacial till. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Conventional spread footings, placed on an undisturbed, compact to dense glacial, or on engineered fill or lean concrete which is placed directly over the undisturbed, compact to dense glacial till, can be designed using a bearing resistance value at serviceability limit states (SLS) of **170 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **255 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of a surface 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 the placement of concrete for footings.

Lateral Support

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 the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edges 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 that of the bearing medium.

Permissible Grade Raise Recommendations

Due to the presence of a silty clay deposit at the subject site, permissible grade raise restriction of **2.5 m** is recommended for development.

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

The site class for seismic site response can be taken as **Class D**. If a higher seismic site class is required (Class C), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

In order to mitigate potential impacts on the proposed building foundations due to strength loss or consolidation associated with the sensitive silty clay deposit, it has been recommended that any silty clay encountered at the proposed building USF elevation be sub-excavated to the undisturbed, compact to dense glacial till and replaced with engineered fill or lean concrete up to the USF elevation.

Accordingly, soils underlying the subject site are not susceptible to liquefaction. The coarse-grained soils below the USF elevation (glacial till deposit) at the subject site have been evaluated for liquefaction potential in accordance with the “Liquefaction Resistance of Soils” document prepared by Youd et al. (2001), and were determined to have suitable factors of safety against liquefaction, ranging from 1.6 to 3.3, which are greater than the required factor of safety of 1.1 against liquefaction potential. This study is provided in Appendix 3.

5.5 Basement Slab / Slab-on-Grade Construction

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

As the proposed below-grade level will mostly consist of vehicle parking, the recommended pavement structure noted in Table 5 in Section 5.7 below will be applicable for the parking level of the proposed building.

However, when storage or other uses of the lower level will involve the construction of a concrete floor slab, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. It is also recommended to install an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, below lowest level floor. This is discussed further in Section 6.1.

5.6 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the underground parking level should 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 Areas

Thickness (mm)	Material Description
150	**Exposure Class C2 – 32 MPa Concrete** (5 to 8 % Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Top of Raft Foundation	

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 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

The following flexible pavement structures presented in Tables 5 and 6 should be used for exterior, at-grade parking areas and access lanes, respectively.

Table 5 – Recommended Pavement Structure – Driveways and at-grade car 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 Residential Roadways and Access Lanes

Thickness (mm)	Material Description
40	**Wear Course –** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course –** HL-8 or 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 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 for this project. For residential driveways and car only parking areas, an Ontario Traffic Category A will be used. For local roadways, an Ontario traffic Category B should be used for design purposes. 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 compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

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

Underslab Drainage

Underslab drainage will be required to control water infiltration below the lowest level floor slab. For preliminary design purposes, we recommend that 150 or 100 mm diameter perforated pipes be placed at approximate 6 m centres. The spacing of the underslab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining, non-frost susceptible granular materials. The site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Miradrain G100N or Delta Drain 6000) connected to a drainage system is provided. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material should otherwise be used for this purpose.

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.

Exterior unheated footings, such as for isolated piers, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover, or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or retained by shoring systems from the start of the excavation until the structure is backfilled.

Due to the proposed depth of excavation below existing site grades and the proximity to the property line, a temporary shoring system is anticipated to be required along the western boundary of the site, and the western portion of the northern boundary of the site. For the remainder of the site, due to the proposed depth of excavation below the existing site grades and the setback from the property lines, it is anticipated that the excavation can be sloped as per the recommendations below.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of approximately 3 m should be stable cut back at 1H:1V. Flatter slopes could be required for deeper excavations or for excavations below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring systems should be used.

The subsoil at this site is considered to be mainly a Type 2 or 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.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. Efforts should also be made to maintain dry surfaces at the bottom of the excavation footprints and along the bottom of side slopes. Additional measures may be recommended at the time of construction by the geotechnical consultant.

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.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations at the western boundary of the site, and the western portion of the northern boundary of the site. The shoring requirements will depend on the depth of the excavation and the proximity of the adjacent structures.

If a temporary shoring system is considered, the design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures, and include dewatering control measures.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

The temporary shoring system, where required, may generally consist of a soldier pile and lagging system which could be cantilevered, anchored or braced.

The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the temporary shoring system may be calculated using the following parameters.

Table 7 – Soil Parameters for Shoring System Design

Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-rest Earth Pressure Coefficient (K_o)	0.5
Total Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible.

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

Based on the subsurface soil conditions observed, the groundwater conditions, and the proposed depth of excavation for the proposed residential building, basal heaving is not considered an issue at the subject site.

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 Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

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 reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate 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.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four 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. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and 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.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

Impacts on Neighbouring Structures

The proposed structure is not anticipated to extend significantly below the groundwater level. Therefore, no adverse effects from short term and/or long term dewatering are expected for the surrounding structures.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is 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 low to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a soils review of the site 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 a recovered silty clay sample. Sieve analysis testing was also completed on a selected soil sample. The above-noted testing was completed on a sample taken at a depth between the anticipated underside of footing elevation and a 3 m depth below finished grade. The results of the testing are presented in Tables 1 and 2 in Section 4.2 and in Appendix 1.

Based on the results of our review, the plasticity index was found to be less than 40%. Therefore, the following tree planting setbacks are recommended for the silty clay deposit. Large trees (mature height over 14 m) can be planted within the silty clay areas 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 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 un-compacted 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.

6.9 Slope Stability Assessment

Due to the slope across the site, it is understood that a slope stability assessment is required in accordance with the City of Ottawa guidelines. Accordingly, a slope stability assessment of the proposed site conditions was conducted using SLIDE, a computer program which permits a two-dimensional stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. A horizontal acceleration of 0.16 g (50% of PGA = 0.32g) was utilized for the seismic analysis.

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. A minimum F.o.S. of 1.5 is generally recommended for static analysis conditions and a mimimum F.o.S. of 1.1 is generally recommended for seismic analysis conditions, where the failure of the slope would endanger permanent structures.

The cross-section A-A (location indicated on Drawing PG6160-1 in Appendix 2) was analyzed based on the proposed site conditions and a review of the available topographic mapping.

The effective strength soil parameters used for static analysis were chosen based on the subsoil information recovered during the geotechnical investigation and in general accordance with the typical ranges of values provided in the City of Ottawa's "Slope Stability Guidelines for Development Applications". The effective strength soil parameters used for static analysis are presented in Table 8 below.

Table 8 - Effective Strength Soil and Material Parameters (Static Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	18	33	0
Brown Silty Clay	17	33	7
Glacial Till	20	36	0

The total strength soil parameters used for seismic analysis were also chosen based on the subsoil information recovered during the geotechnical investigation and in general accordance with the typical ranges of values provided in the City of Ottawa's "Slope Stability Guidelines for Development Applications".

The strength parameters used for seismic analysis at the slope cross-sections are presented in Table 9 below:

Table 9 - Total Strength Soil and Material Parameters (Seismic Analysis)

Soil Layer	Unit Weight (kN/m^3)	Friction Angle (degrees)	Cohesion (kPa)
Fill	18	33	0
Brown Silty Clay	17	0	100
Glacial Till	20	36	0

The results for the slope stability analyses under static and seismic conditions at cross-section A-A are shown on Figures 2 and 3, which are provided in Appendix 2. The results of the slope stability analyses indicate that the factor of safety exceeds 1.5 and 1.1 under static and seismic analysis conditions, respectively.

Therefore, the slope stability for the proposed site conditions is considered acceptable, from a geotechnical perspective.

Global Stability Analysis

Retaining walls with heights greater than 1 m were noted on the available grading plan at the northwest corner of the site (cross-section B-B), along the north side of the parking lot (cross-section C-C), and on the south side of the building (cross-section D-D). In accordance with City of Ottawa guidelines, global stability analyses are required for all retaining walls greater than 1 m in height.

The global stability analyses of the retaining walls were conducted using SLIDE. A horizontal acceleration of 0.16 g (50% of PGA = 0.32g) was utilized for the seismic analysis.

The results for the global stability analyses under static conditions for cross-sections B-B, C-C, and D-D are shown in Figures 4, 6 and 8, which are attached in Appendix 2. The results of the global stability analyses indicate that the factor of safety exceeds 1.5 under static conditions.

The results for the global stability analyses under seismic conditions at cross-sections B-B, C-C, and D-D are shown on Figure 5, 7 and 9, which are also attached in Appendix 2. The results of this analyses indicate that the factor of safety exceeds 1.1 under seismic conditions.

Therefore, the proposed retaining walls at the subject site are considered stable, from a global stability perspective.

In accordance with the “Slope Stability Guidelines for Development Applications in the City of Ottawa”, retaining wall failure modes such as toppling, forward sliding, structural failure, and bearing capacity are to be addressed at the detailed design stage of the project.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- Review detailed 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 material.
- Sampling and testing of the concrete and fill materials.
- Observation of clay seal placement at specified locations.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

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 immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 ARK Construction Ltd. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Otillia McLaughlin B.Eng.

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
June 8, 2023
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. **PG6160**

HOLE NO. **BH 1-22**

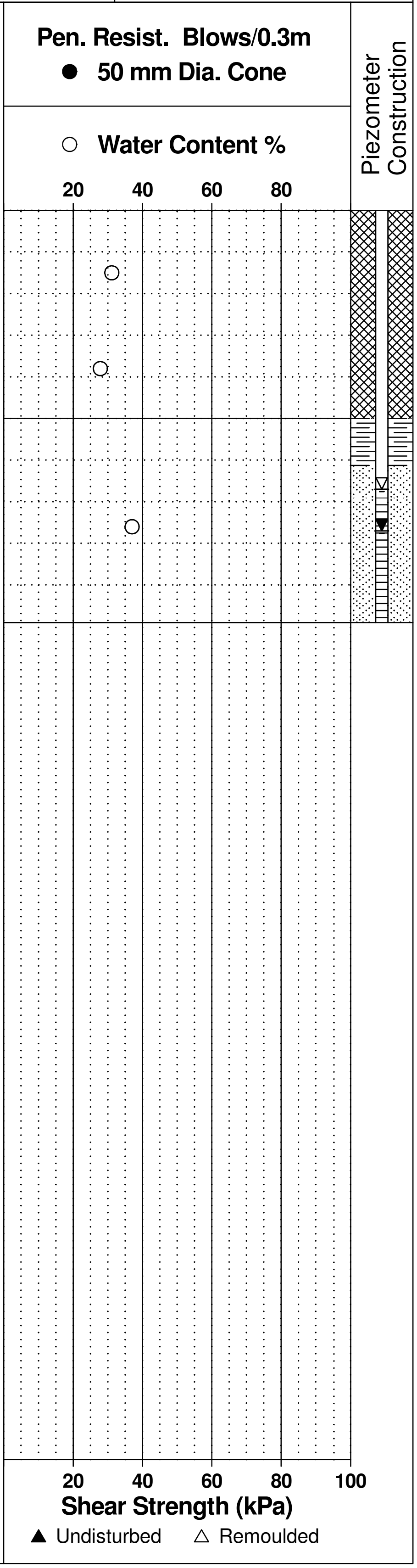
SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE							
Asphaltic concrete 0.05						0	94.24
FILL: Brown silty sand trace gravel and topsoil 0.69		AU	1				
Very stiff, brown **SILTY CLAY,** trace sand and gravel 1.52		SS	2	50	6	1	93.24
GLACIAL TILL: Compact, brown silty sand to sandy silt with gravel, cobbles, trace clay and boulders 1.98		SS	3	56	50		

End of Borehole

Practical refusal to augering at 1.98m depth.

(GWL @ 1.34m - March 9, 2022)

(GWL at 1.54m - Nov. 25, 2022)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS Moved east approx 1 m from BH 1-22 location

BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. PG6160

HOLE NO. BH 1A-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	94.24		
Asphaltic concrete 0.05									
OVERBURDEN						1	93.24		
2.13						2	92.24		
End of Borehole Practical refusal to augering at 2.13m depth									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

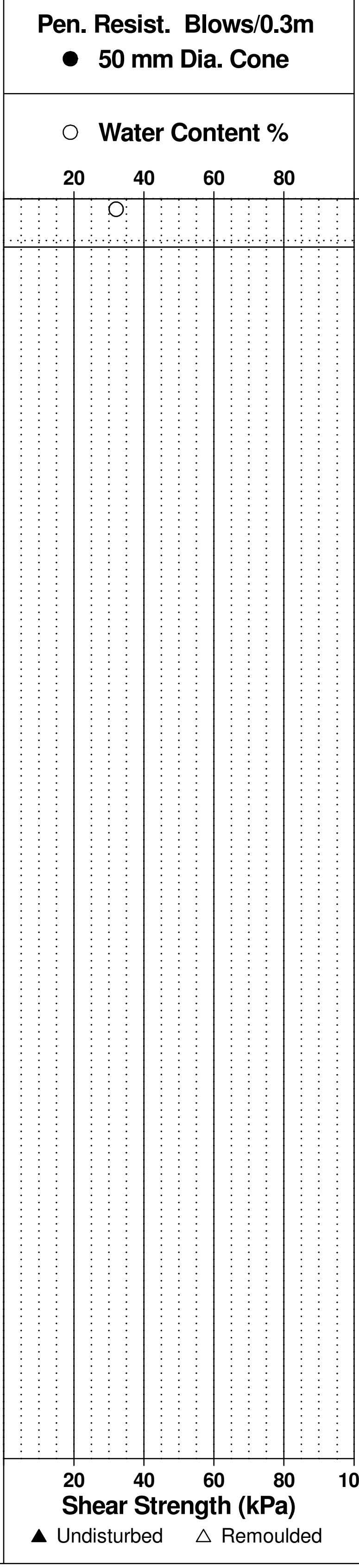
BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. PG6160

HOLE NO. BH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	94.46	
Asphaltic concrete 0.05								
FILL: Brown silty sand with gravel and crushed stone 0.23		AU	1					
End of Borehole Practical refusal to augering at 0.23m depth								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS Moved north approx 1 m from BH 2-22 location

BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. PG6160

HOLE NO. BH 2A-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE									
Asphaltic concrete 0.05						0	94.46		
OVERBURDEN 0.23									
End of Borehole Practical refusal to augering at 0.23m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. PG6160

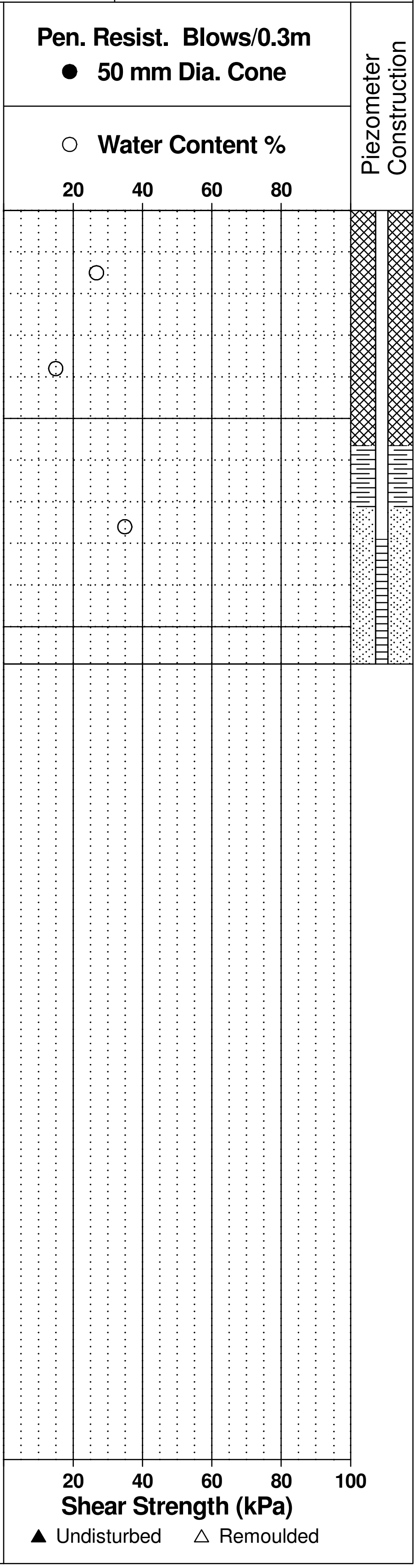
HOLE NO. BH 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	91.65
TOPSOIL 0.10							
FILL: Brown silty clay trace sand and gravel		AU	1				
1.22		SS	2	33	11	1	90.65
Very stiff, brown **SILTY CLAY** 1.68							
GLACIAL TILL: Compact, brown silty sand to sandy silt with gravel, trace clay, cobbles and boulders 2.18		SS	3	90	12	2	89.65

End of Borehole

Practical refusal to augering at 2.18m depth

(BH dry - March 9, 2022)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS Moved north approx 1 m from BH 3-22 location

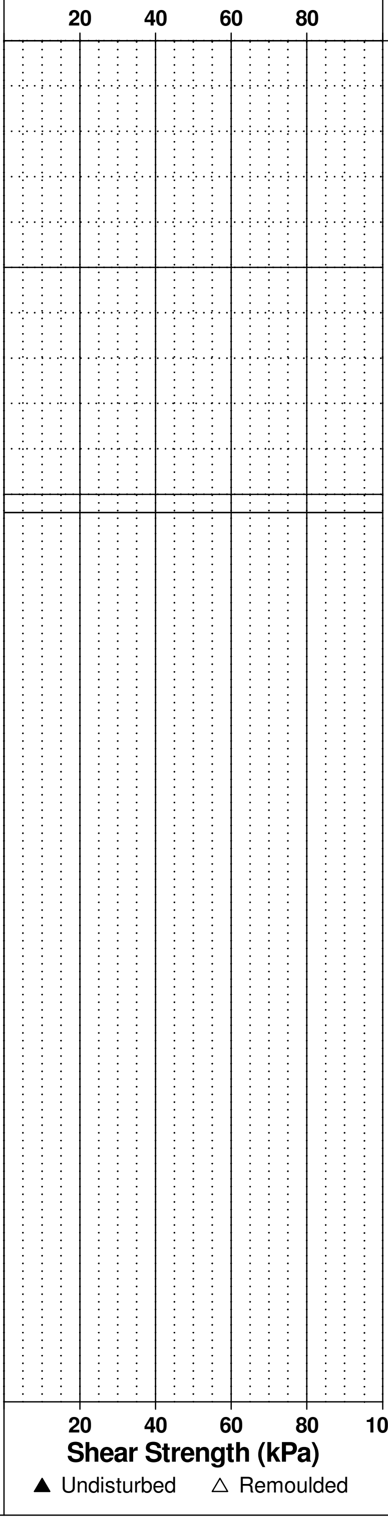
BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. PG6160

HOLE NO. BH 3A-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	91.65		
TOPSOIL 0.10	■								
						1	90.65		
OVERBURDEN									
2.18						2	89.65		
End of Borehole Practical refusal to augering at 2.18m depth									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 1185 Beaverwood Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 1, 2022

FILE NO. PG6160

HOLE NO. BH 4-22

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.67
TOPSOIL 0.10						
FILL: Brown silty clay trace sand 0.69	AU	1				
Brown **SILTY CLAY**	SS	2	75	10	1	89.67
	SS	3	42	7	2	88.67
	SS	4	42	4		
3.35					3	87.67
	SS	5	50	2		
GLACIAL TILL: Compact, brown silty sand to sandy silt with gravel, trace clay, cobbles and boulders	SS	6	50	30	4	86.67
4.52						

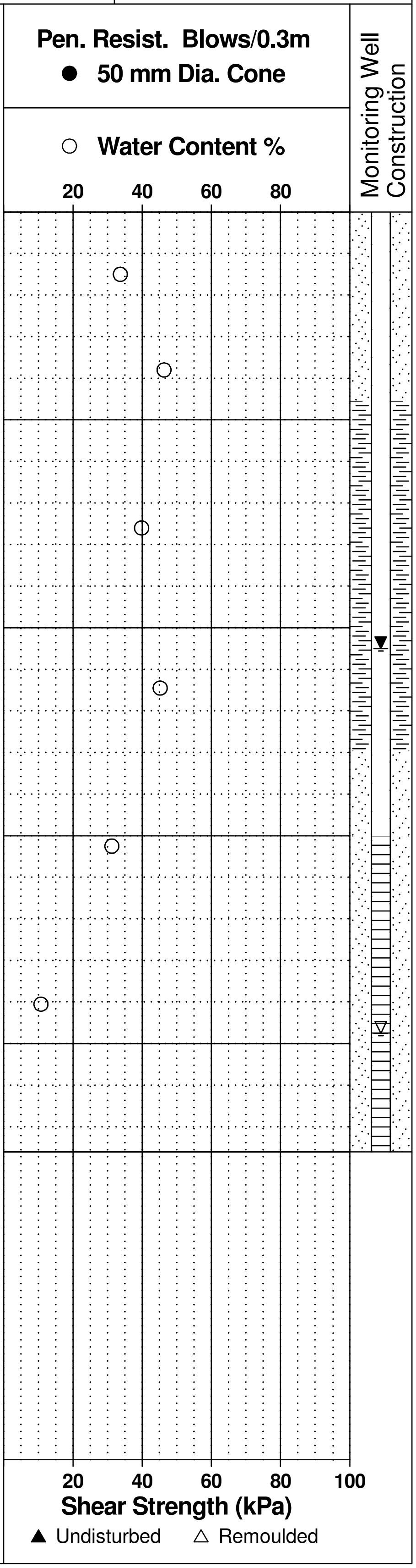
End of Borehole

Practical refusal to augering at 4.52m depth

(GWL @ 3.14m - March 9, 2022)

(GWL @ 3.95m - Nov. 25, 2022)

(GWL @ 2.10m - March 21, 2023)



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)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

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)

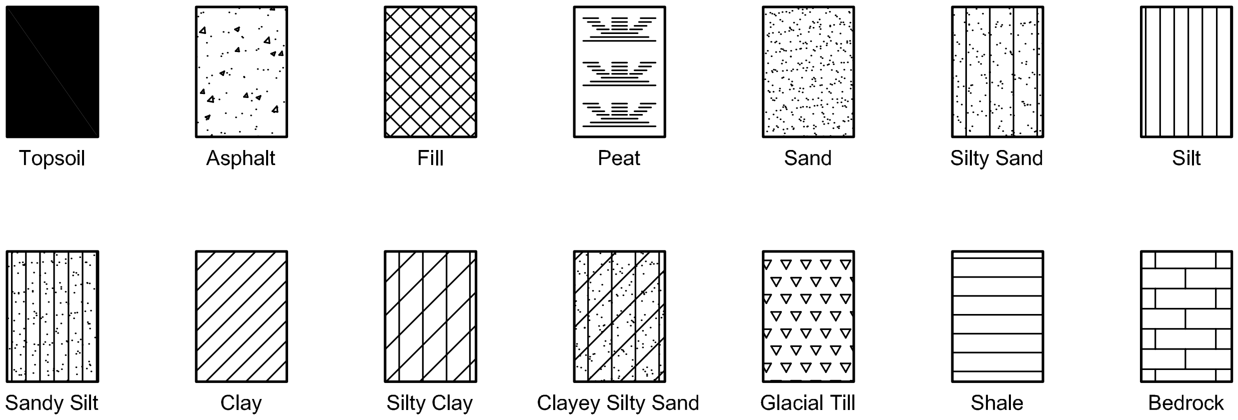
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

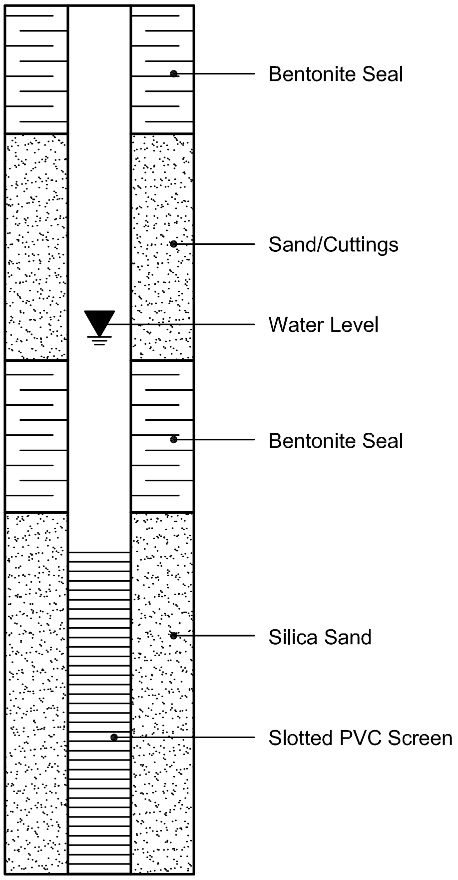
SYMBOLS AND TERMS (continued)

STRATA PLOT

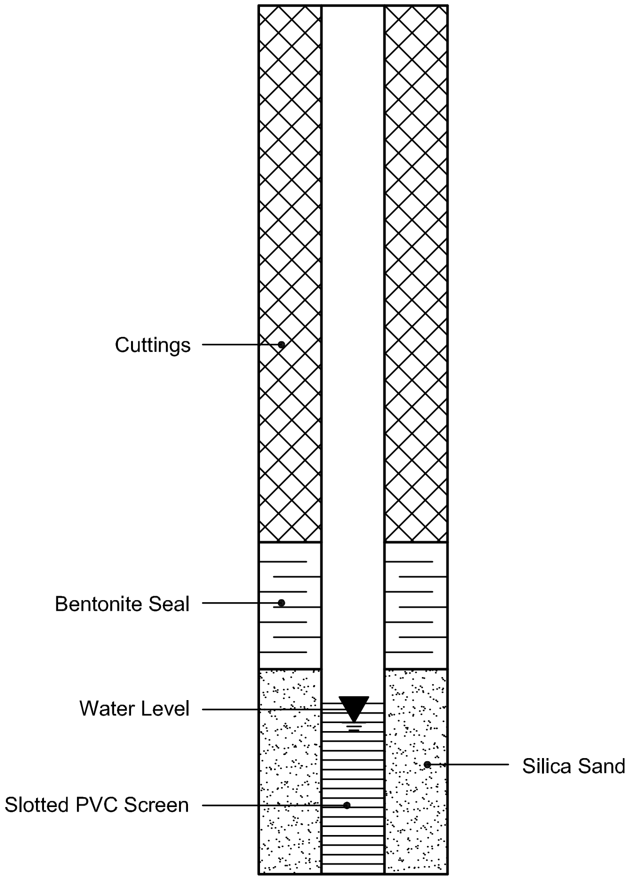


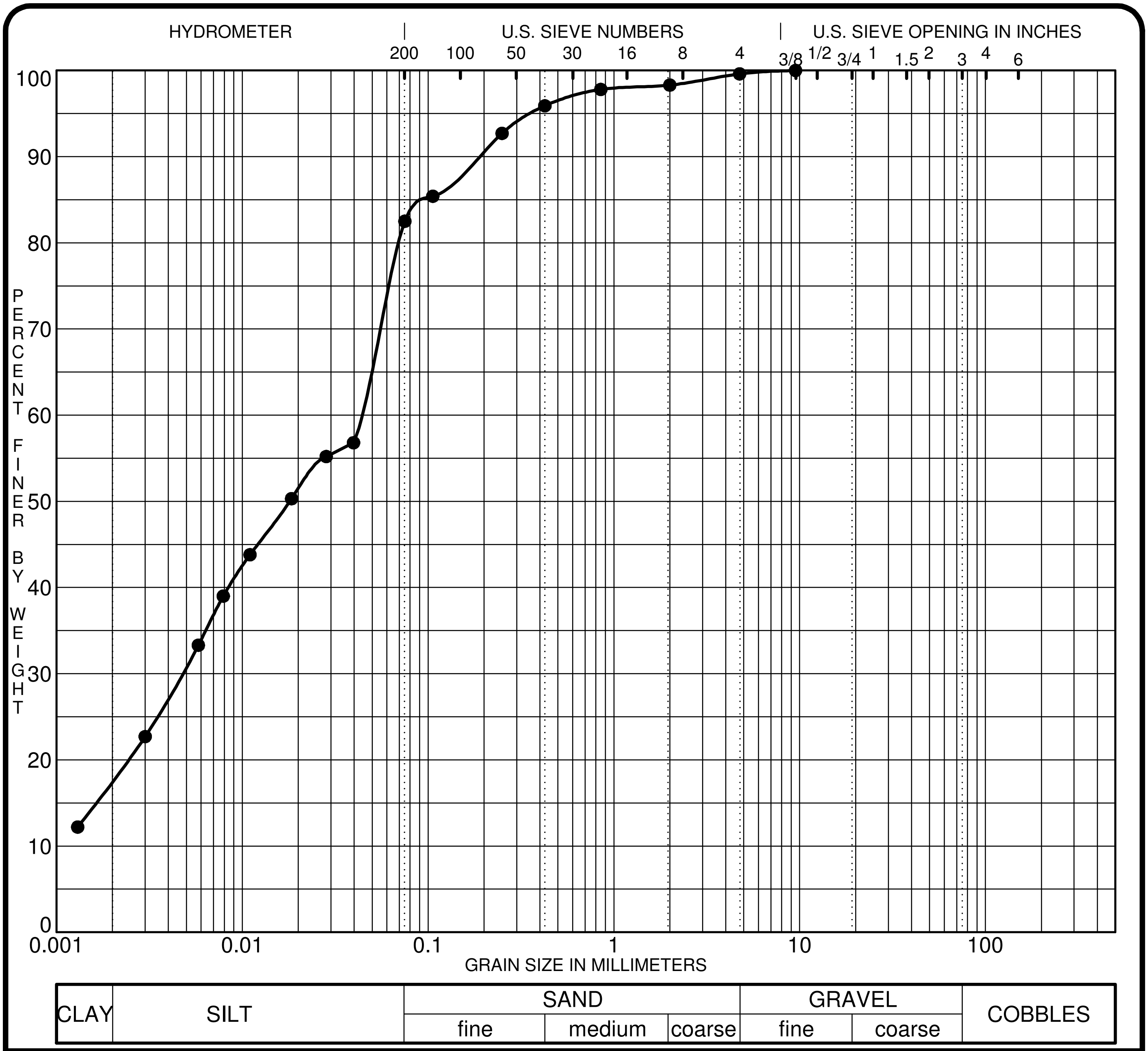
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**BH 3-22 SS3**		**30.1**					
☒								
▲								
★								

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH 3-22 SS3**	**9.50**	**0.04**	**0.005**		**0.4**	**17.1**	**64.0**	**18.5**
☒									
▲									
★									

CLIENT **Ark Construction**

PROJECT **Geotechnical Investigation - Proposed Development**

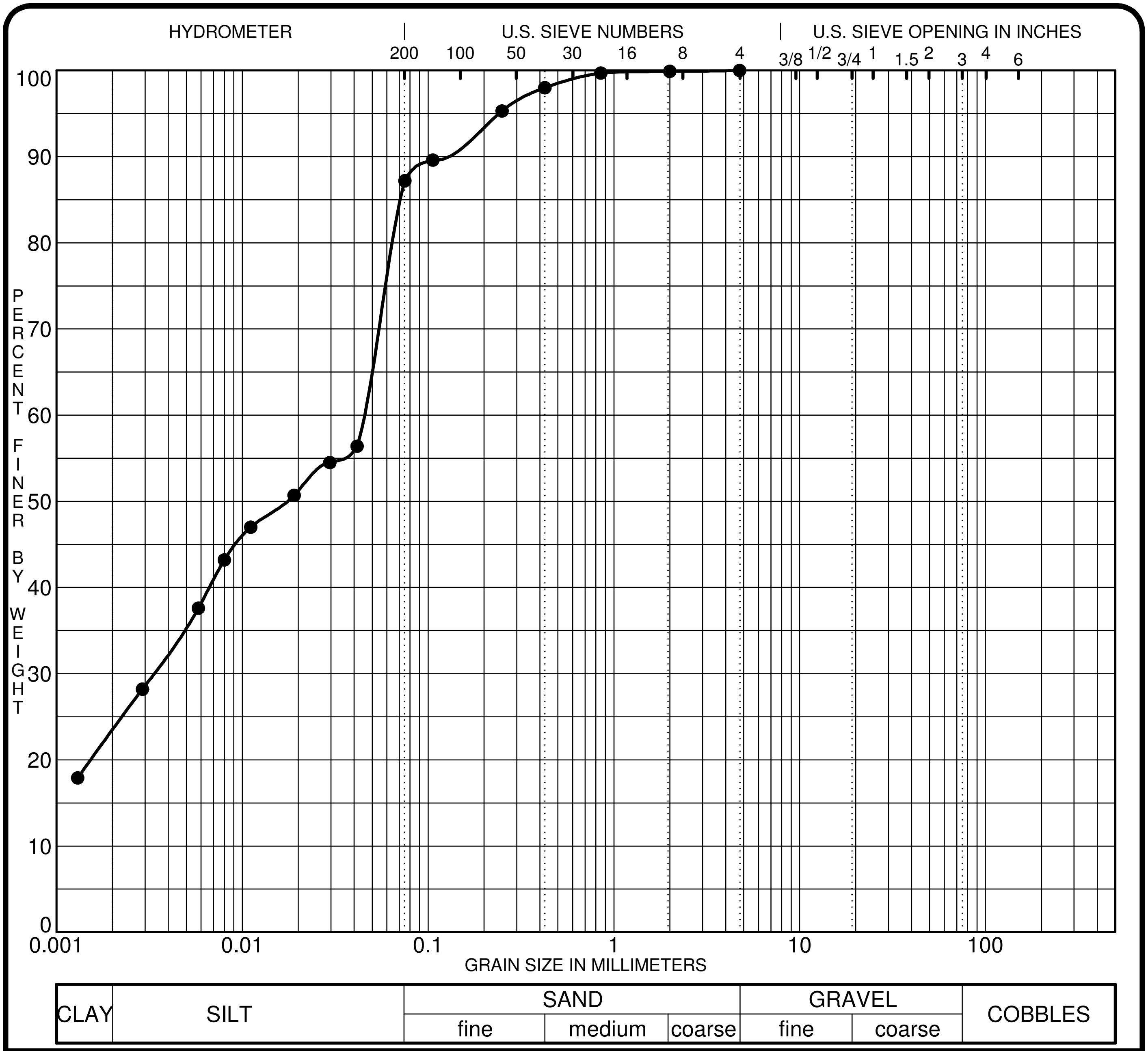
FILE NO. **PG6160**

DATE **1 Mar 22**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
● **BH 4-22 SS3**		**33.6**					
☒							
▲							
★							

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● **BH 4-22 SS3**	**4.75**	**0.04**	**0.003**		**0.0**	**12.8**	**64.2**	**23.0**
☒								
▲								
★								

CLIENT **Ark Construction**

PROJECT **Geotechnical Investigation - Proposed Development**

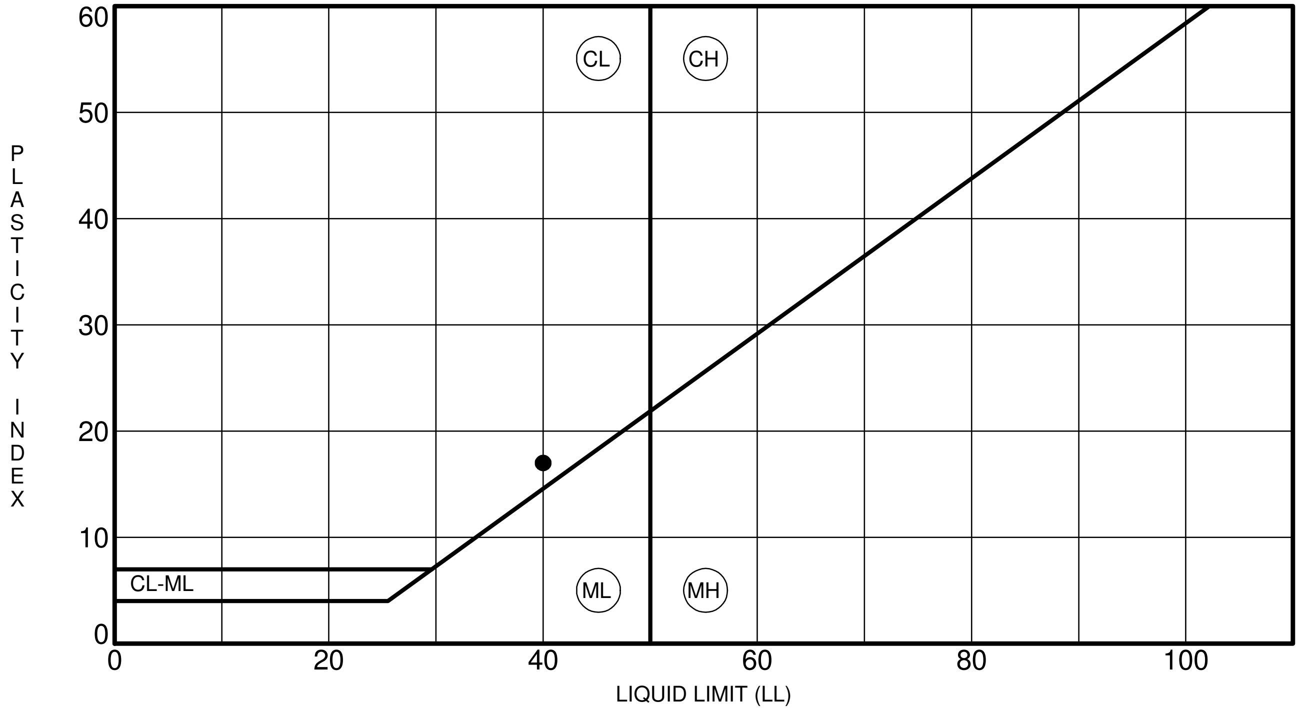
FILE NO. **PG6160**

DATE **1 Mar 22**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 4-22 SS4**	**40**	**23**	**17**		**CL = Inorganic Clays of Low Plasticity**

CLIENT **Ark Construction**

PROJECT **Geotechnical Investigation - Proposed Development**

FILE NO. **PG6160**

DATE **1 Mar 22**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 33999

Report Date: 04-Mar-2022

Order Date: 2-Mar-2022

Project Description: PG6160

	Client ID:	BH4-22 (SS2)	-	-	-
	Sample Date:	01-Mar-22 09:00	-	-	-
	Sample ID:	2210363-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	69.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.43	-	-	-
Resistivity	0.10 Ohm.m	83.4	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	<5	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SLOPE STABILITY ANALYSIS CROSS-SECTIONS

FIGURES 4 TO 9 – GLOBAL STABILITY ANALYSIS CROSS-SECTIONS

DRAWING PG6160-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

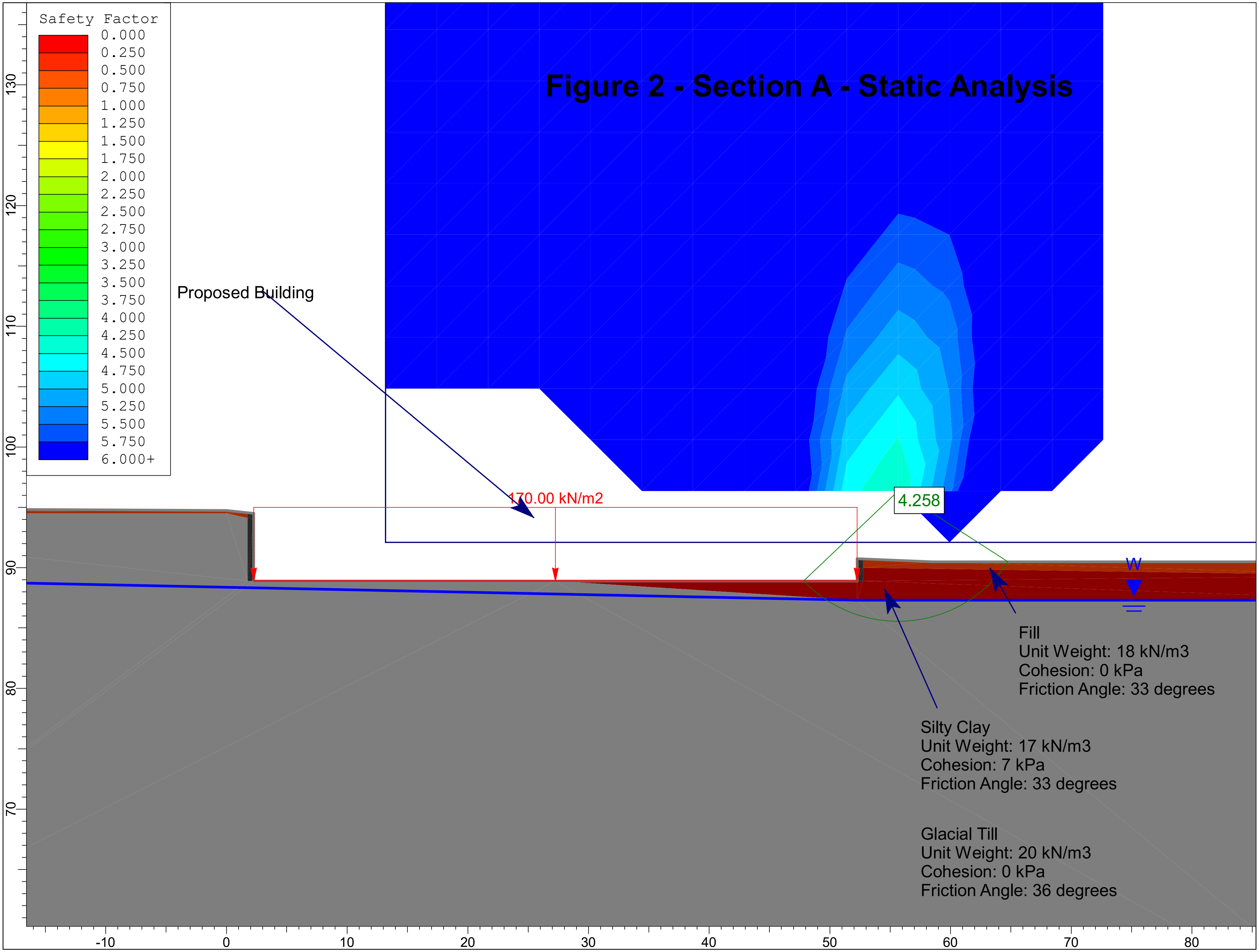


Figure 2 - Section A - Static Analysis

Fill
Unit Weight: 18 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees

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

Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 36 degrees

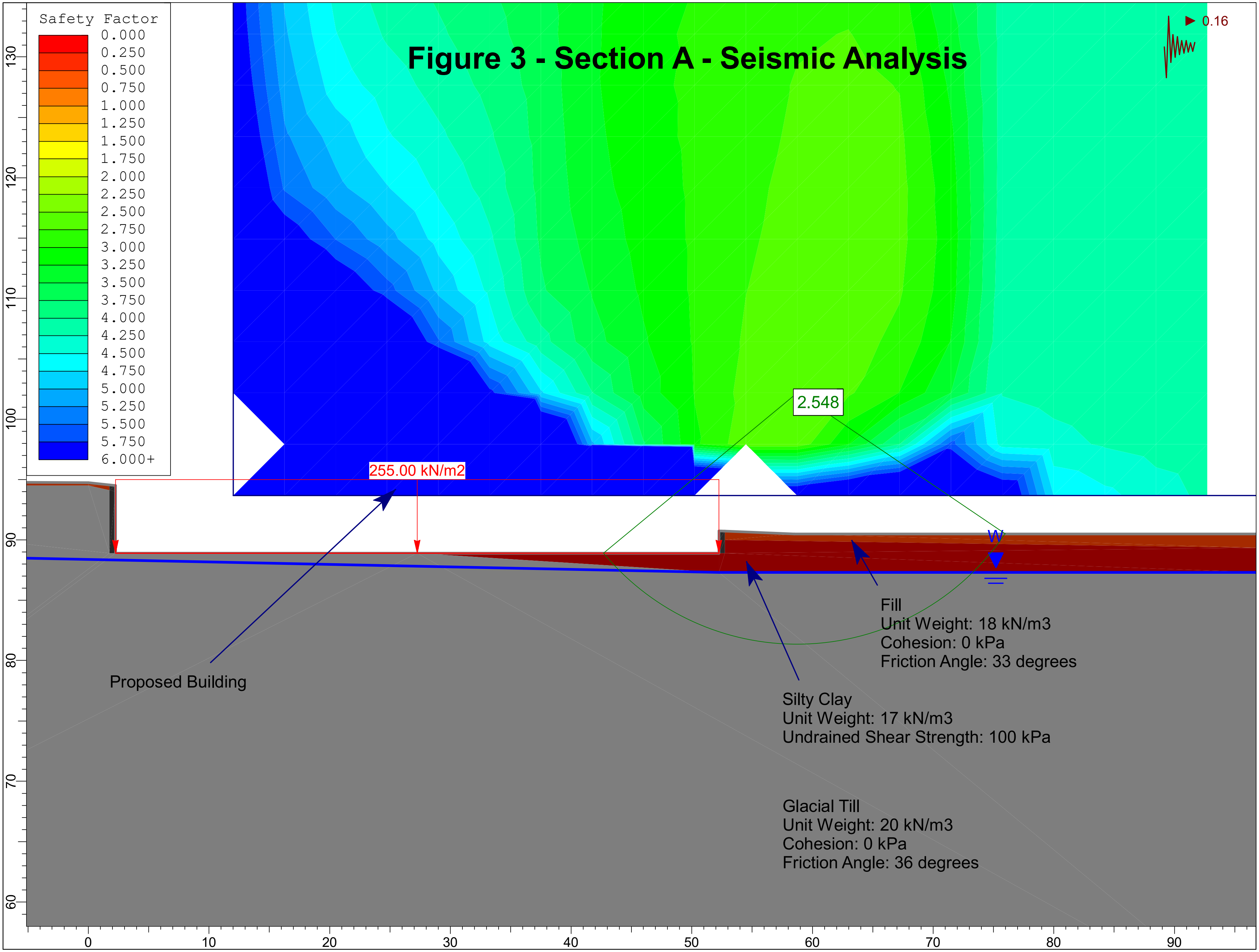


Figure 3 - Section A - Seismic Analysis

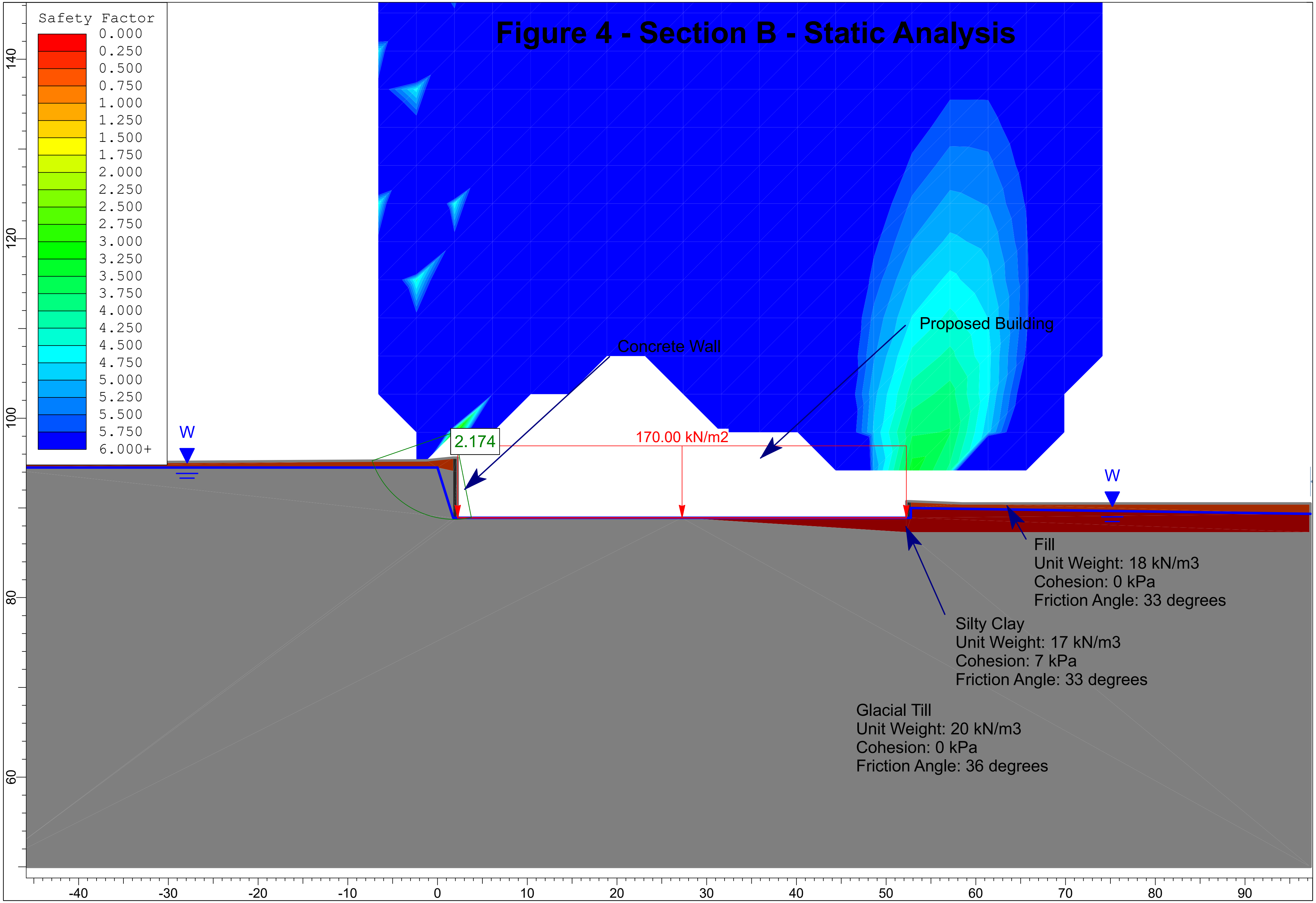
Proposed Building

Fill
Unit Weight: 18 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees

Silty Clay
Unit Weight: 17 kN/m3
Undrained Shear Strength: 100 kPa

Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 36 degrees

Figure 4 - Section B - Static Analysis



Fill
Unit Weight: 18 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees

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

Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 36 degrees

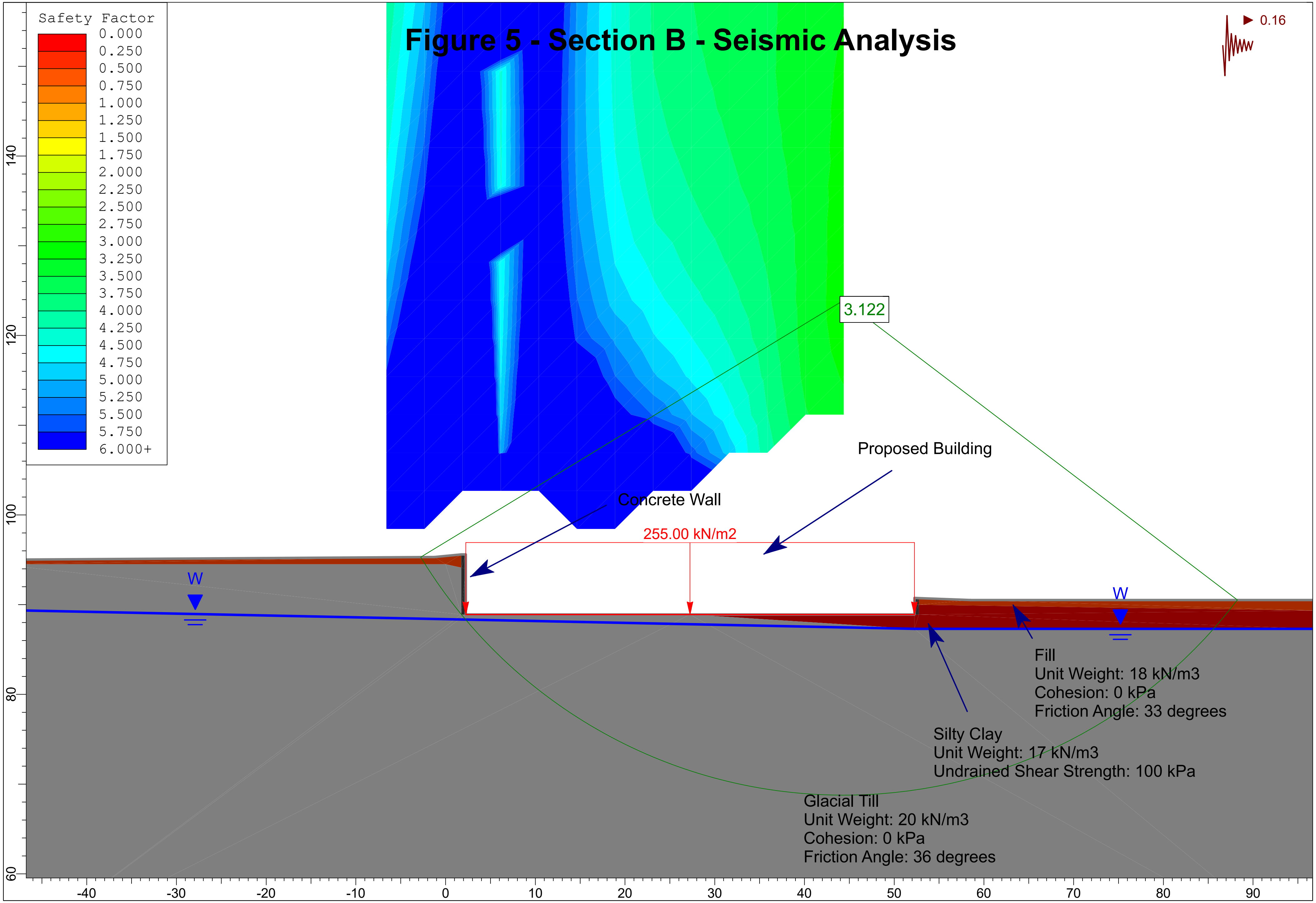


Figure 5 - Section B - Seismic Analysis

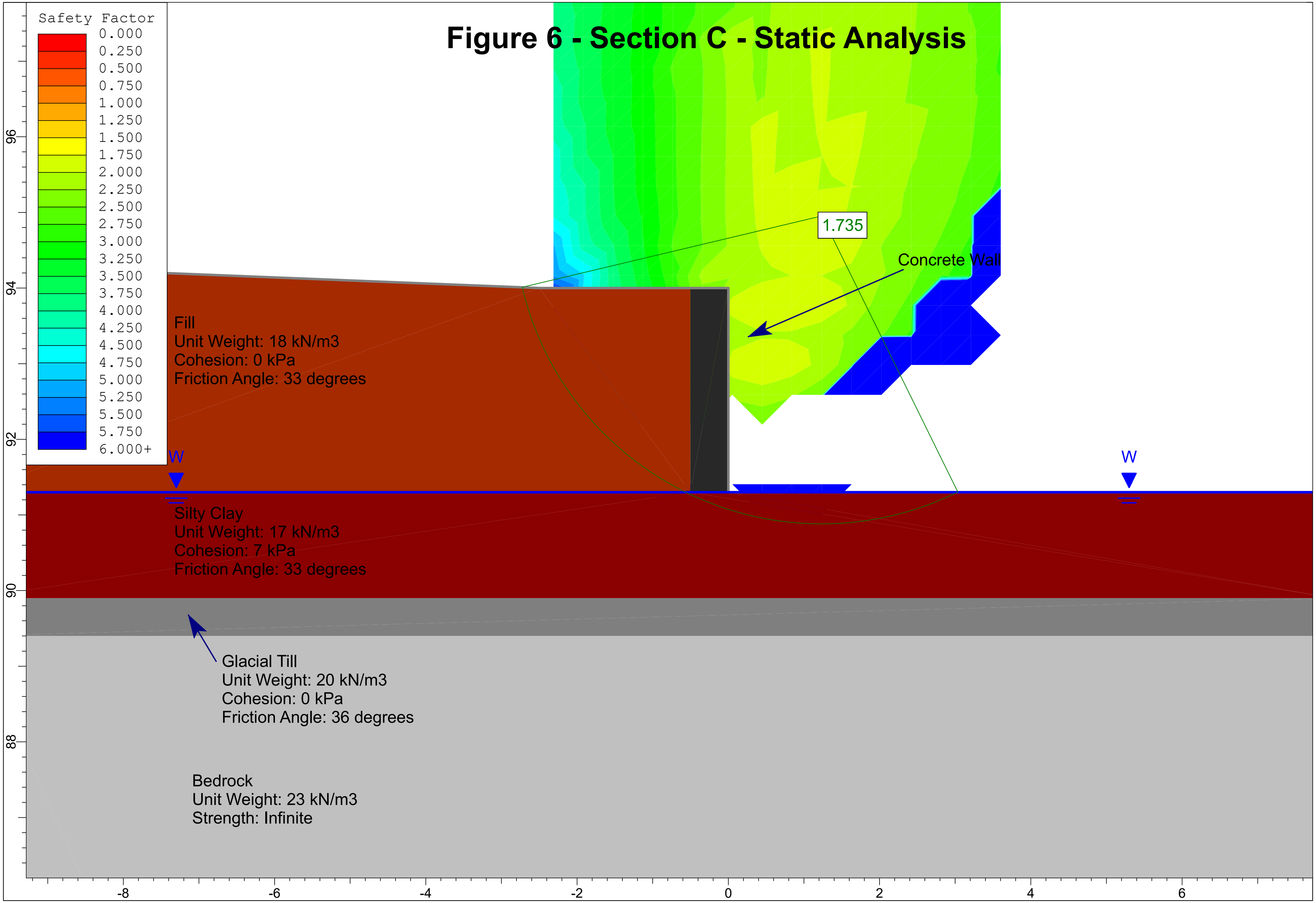


Figure 6 - Section C - Static Analysis

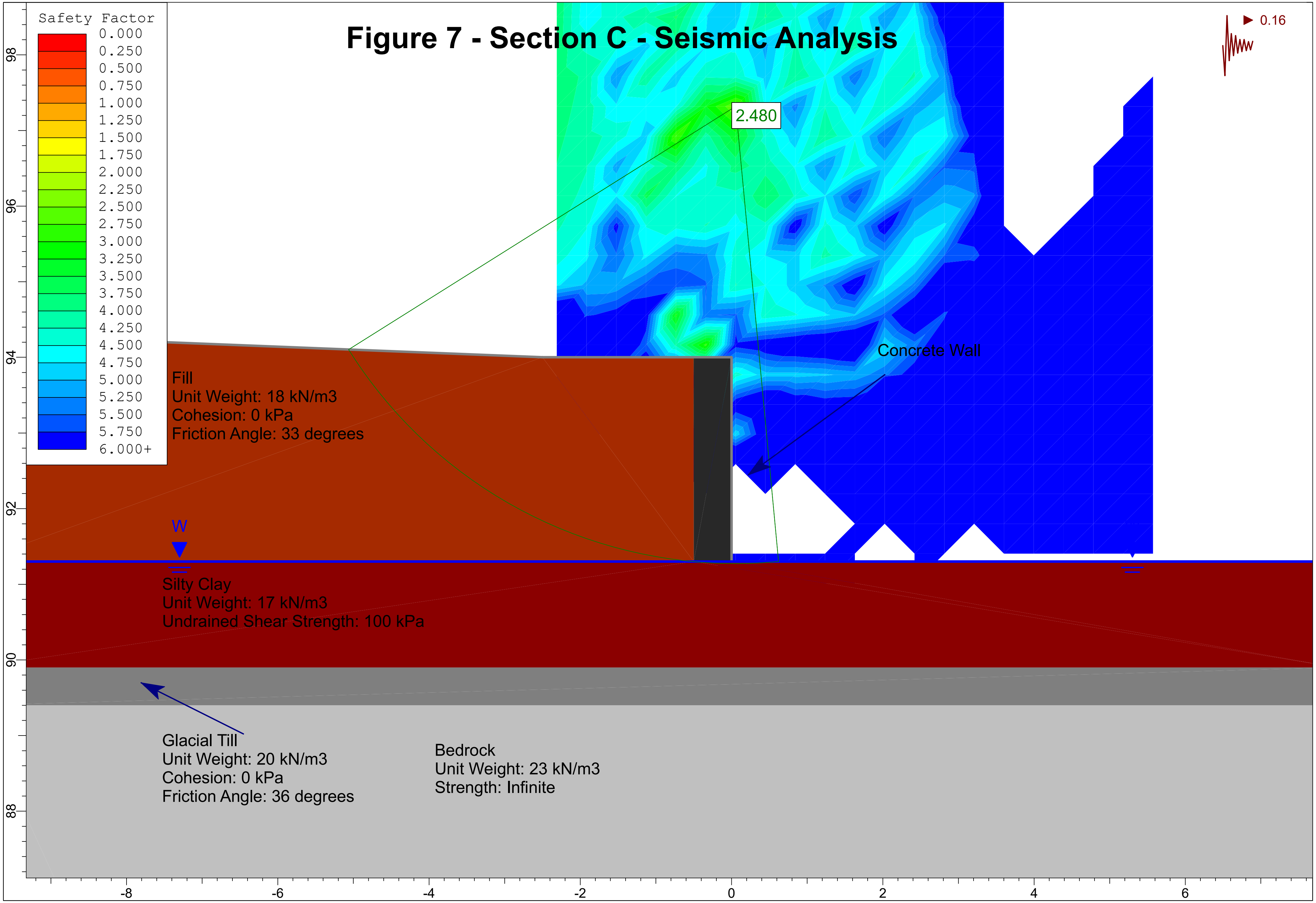


Figure 7 - Section C - Seismic Analysis

Fill
Unit Weight: 18 kN/m3
Cohesion: 0 kPa
Friction Angle: 33 degrees

Silty Clay
Unit Weight: 17 kN/m3
Undrained Shear Strength: 100 kPa

Glacial Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 36 degrees

Bedrock
Unit Weight: 23 kN/m3
Strength: Infinite

Concrete Wall

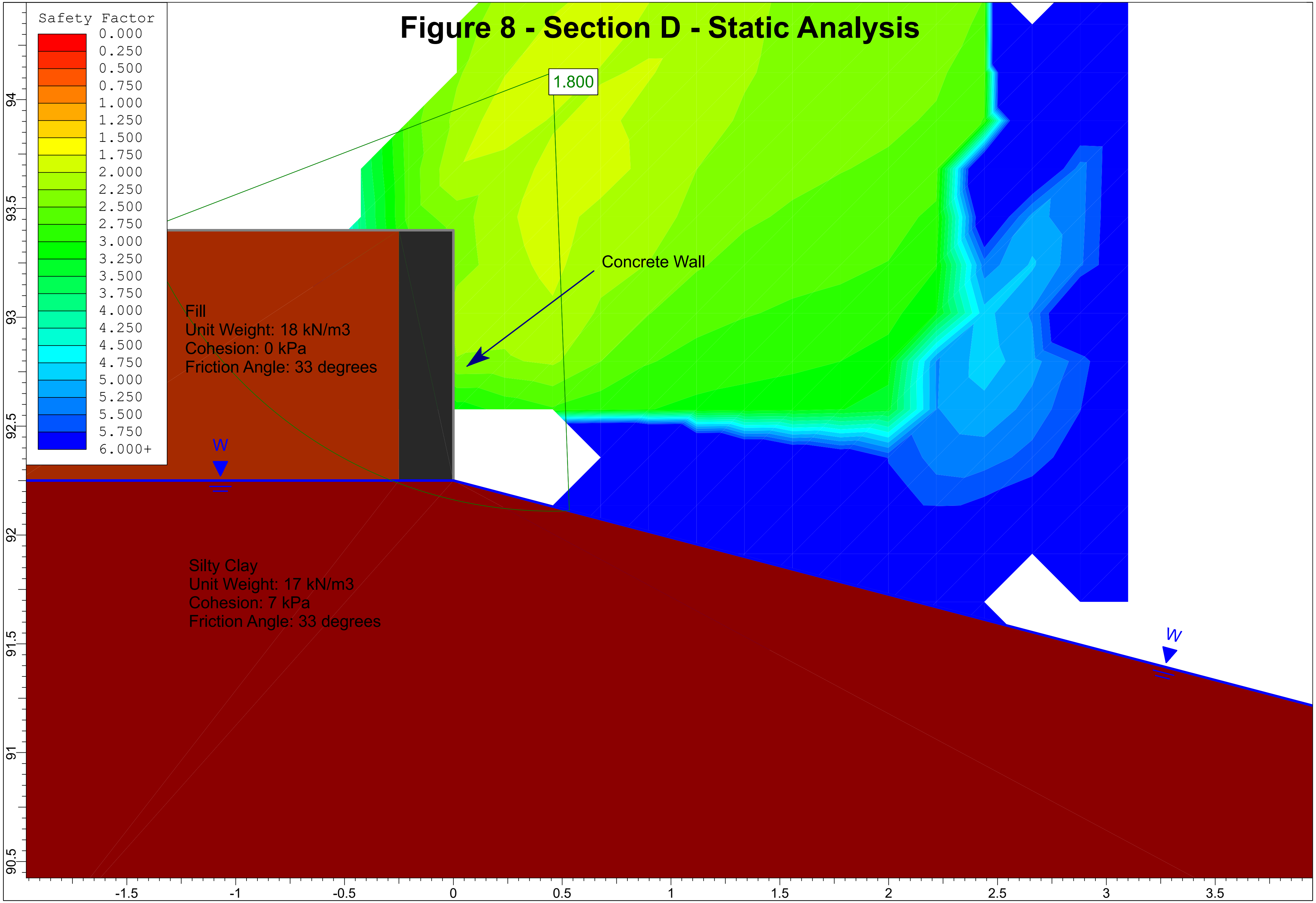


Figure 8 - Section D - Static Analysis

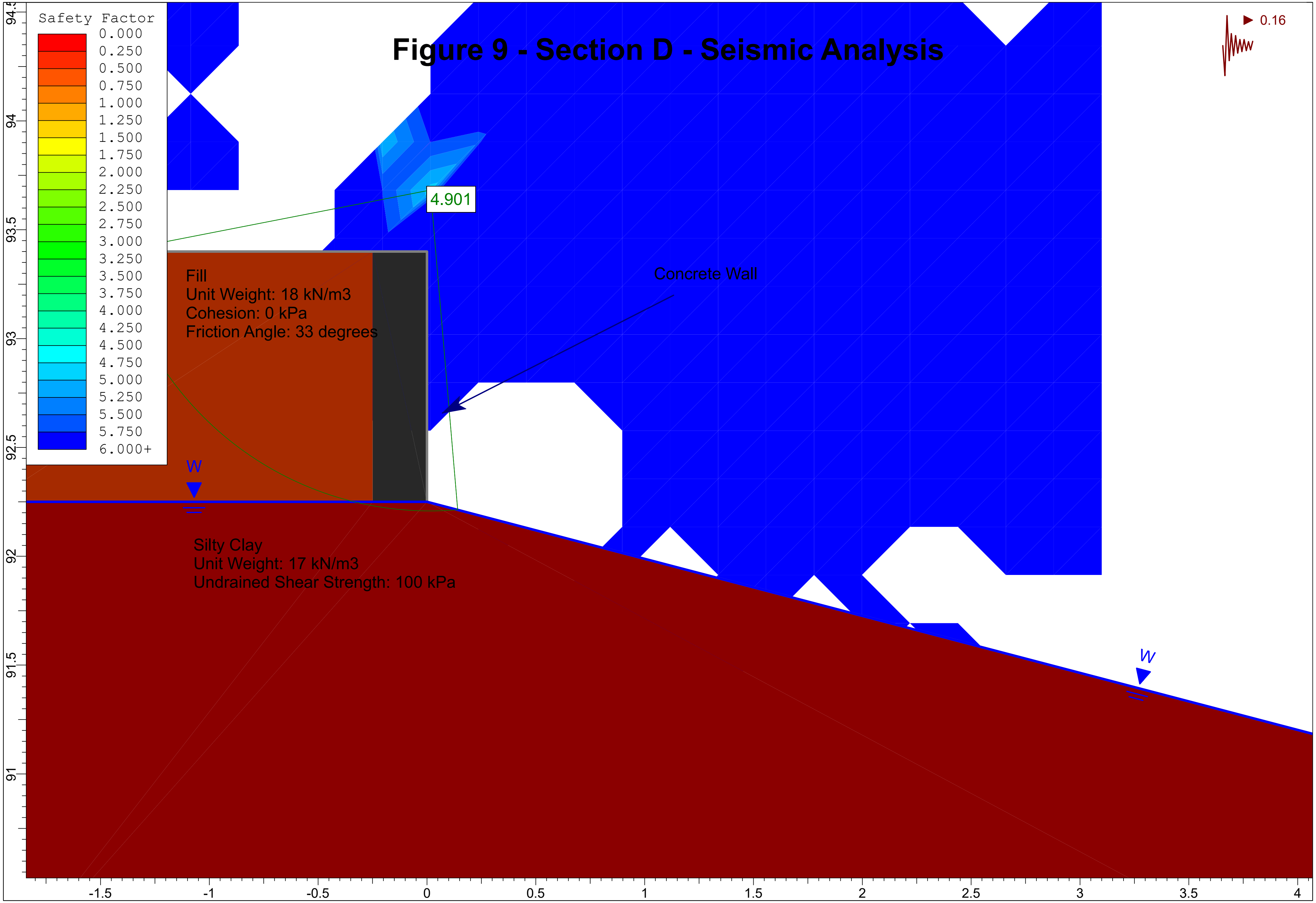
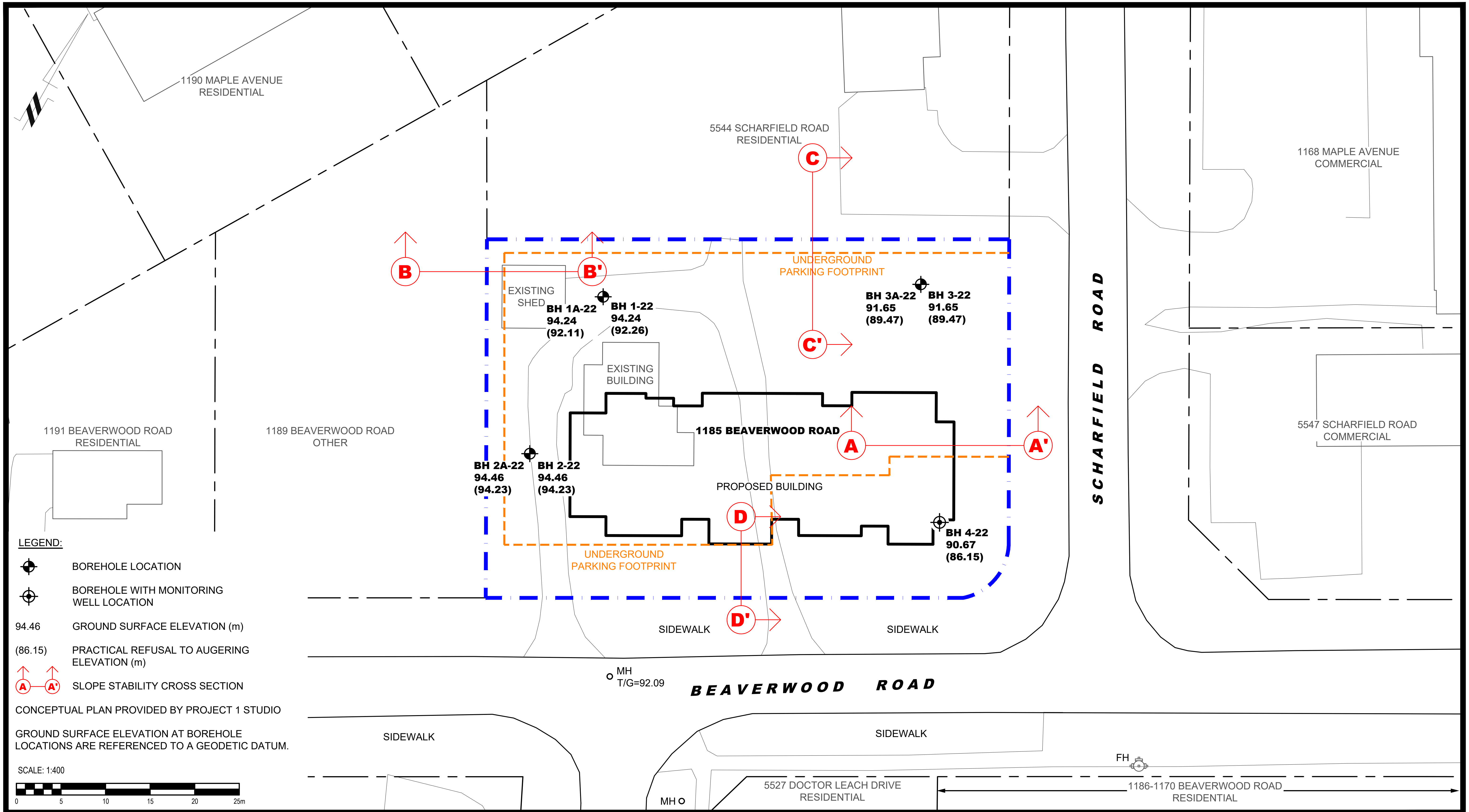


Figure 9 - Section D - Seismic Analysis



LEGEND:

⌖	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION
94.46	GROUND SURFACE ELEVATION (m)
(86.15)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
Ⓐ Ⓐ'	SLOPE STABILITY CROSS SECTION

CONCEPTUAL PLAN PROVIDED BY PROJECT 1 STUDIO

GROUND SURFACE ELEVATION AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:400

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED BH 1A, BH 2A, AND BH 3A TO PLAN	15/12/2022	OM

ARK CONSTRUCTION
GLOBAL STABILITY ANALYSIS
PROPOSED DEVELOPMENT
1185 BEAVERWOOD ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 09/2022
Drawn by: NFRV
Report No.: PG6160-1
Checked by: OM
Dwg. No.: PG6160-2
Approved by: SD
Revision No.: 1

p:\autocad drawings\geotechnical\pg61xx\pg6160\pg6160-1-test hole location plan (rev.01).dwg

APPENDIX 3

LIQUEFACTION RESISTANCE OF SOILS
BY YOUD ET AL. (2001)

Preface

Evaluation of soil liquefaction resistance is an important aspect of geotechnical engineering practice. To update and enhance criteria that are routinely applied in practice, workshops were convened in 1996 and 1998 to gain consensus from 20 experts on updates and augmentations that should be made to standard procedures that have evolved over the past 30 years. At the outset, the goal was to develop this state-of-the-art summary of consensus recommendations. A commitment was also made to those who participated in the workshops that all would be listed as co-authors. Unfortunately, the previous publication of this summary paper (April 2001) listed only the co-chairs of the workshop, Profs. Youd and Idriss, as authors; the remaining workshop participants were acknowledged in a footnote. In order to correct this error and to fully acknowledge and credit those who significantly contributed to the work, this paper is being republished in its entirety, at the request of the journal's editors, with all the participants named as co-authors. All further reference to this paper should be to this republication. The previous publication should no longer be cited. Also, several minor errors are corrected in this republication.

Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils[a]

By T. L. Youd,[1] Chair, Member, ASCE, I. M. Idriss,[2] Co-Chair, Fellow, ASCE,
Ronald D. Andrus,[3] Ignacio Arango,[4] Gonzalo Castro,[5] John T. Christian,[6]
Richardo Dobry,[7] W. D. Liam Finn,[8] Leslie F. Harder Jr.,[9] Mary Ellen Hynes,[10]
Kenji Ishihara,[11] Joseph P. Koester,[12] Sam S. C. Liao,[13] William F. Marcuson III,[14]
Geoffrey R. Martin,[15] James K. Mitchell,[16] Yoshiharu Moriwaki,[17] Maurice S. Power,[18]
Peter K. Robertson,[19] Raymond B. Seed,[20] and Kenneth H. Stokoe II[21]

Abstract: Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Professors H. B. Seed and I. M. Idriss developed and published a methodology termed the "simplified procedure" for evaluating liquefaction resistance of soils. This procedure has become a standard of practice throughout North America and much of the world. The methodology which is largely empirical, has evolved over years, primarily through summary papers by H. B. Seed and his colleagues. No general review or update of the procedure has occurred, however, since 1985, the time of the last major paper by Professor Seed and a report from a National Research Council workshop on liquefaction of soils. In 1996 a workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) was convened by Professors T. L. Youd and I. M. Idriss with 20 experts to review developments over the previous 10 years. The purpose was to gain consensus on updates and augmentations to the simplified procedure. The following topics were reviewed and recommendations developed: (1) criteria based on standard penetration tests; (2) criteria based on cone penetration tests; (3) criteria based on shear-wave velocity measurements; (4) use of the Becker penetration test for gravelly soil; (4) magnitude scaling factors; (5) correction factors for overburden pressures and sloping ground; and (6) input values for earthquake magnitude and peak acceleration. Probabilistic and seismic energy analyses were reviewed but no recommendations were formulated.

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[a] This Summary Report, originally published in April 2001, is being republished so that the contribution of all workshop participants as authors can be officially recognized. The original version listed only two authors, plus a list of 19 workshop participants. This was incorrect; all 21 individuals should have been identified as authors. ASCE deeply regrets the error.

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Note. Discussion open until March 1, 2002. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on January 18, 2000; revised November 14, 2000. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 127, No. 10, October, 2001. ©ASCE, ISSN 1090-0241/01/0010-0817–0833/$8.00 + $.50 per page. Paper No. 22223.

INTRODUCTION

Over the past 25 years a methodology termed the "simplified procedure" has evolved as a standard of practice for evaluating the liquefaction resistance of soils. Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Seed and Idriss (1971) developed and published the basic "simplified procedure." That procedure has been modified and improved periodically since that time, primarily through landmark papers by Seed (1979), Seed and Idriss (1982), and Seed et al. (1985). In 1985, Professor Robert V. Whitman convened a workshop on behalf of the National Research Council (NRC) in which 36 experts and observers thoroughly reviewed the state-of-knowledge and the state-of-the-art for assessing liquefaction hazard. That workshop produced a report (NRC 1985) that has become a widely used standard and reference for liquefaction hazard assessment. In January 1996, T. L. Youd and I. M. Idriss convened a workshop of 20 experts to update the simplified procedure and incorporate research findings from the previous decade. This paper summarizes recommendations from that workshop (Youd and Idriss 1997).

To keep the workshop focused, the scope of the workshop was limited to procedures for evaluating liquefaction resistance of soils under level to gently sloping ground. In this context, liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soils. Important postliquefaction phenomena, such as residual shear strength, soil deformation, and ground failure, were beyond the scope of the workshop.

The simplified procedure was developed from empirical evaluations of field observations and field and laboratory test data. Field evidence of liquefaction generally consisted of surficial observations of sand boils, ground fissures, or lateral spreads. Data were collected mostly from sites on level to gently sloping terrain, underlain by Holocene alluvial or fluvial sediment at shallow depths (<15 m). The original procedure was verified for, and is applicable only to, these site conditions. Similar restrictions apply to the implementation of the updated procedures recommended in this report.

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson 1978). Increased pore-water pressure is induced by the tendency of granular materials to compact when subjected to cyclic shear deformations. The change of state occurs most readily in loose to moderately dense granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment. As liquefaction occurs, the soil stratum softens, allowing large cyclic deformations to occur. In loose materials, the softening is also accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground. In moderately dense to dense materials, liquefaction leads to transient softening and increased cyclic shear strains, but a tendency to dilate during shear inhibits major strength loss and large ground deformations. A condition of cyclic mobility or cyclic liquefaction may develop following liquefaction of moderately dense granular materials. Beneath gently sloping to flat ground, liquefaction may lead to ground oscillation or lateral spread as a consequence of either flow deformation or cyclic mobility. Loose soils also compact during liquefaction and reconsolidation, leading to ground settlement. Sand boils may also erupt as excess pore water pressures dissipate.

CYCLIC STRESS RATIO (CSR) AND CYCLIC RESISTANCE RATIO (CRR)

Calculation, or estimation, of two variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on a soil layer, expressed in terms of CSR; and (2) the capacity of the soil to resist liquefaction, expressed in terms of CRR. The latter variable has been termed the cyclic stress ratio or the cyclic stress ratio required to generate liquefaction, and has been given different symbols by different writers. For example, Seed and Harder (1990) used the symbol CSRℓ, Youd (1993) used the symbol CSRL, and Kramer (1996) used the symbol CSR_L to denote this ratio. To reduce confusion and to better distinguish induced cyclic shear stresses from mobilized liquefaction resistance, the capacity of a soil to resist liquefaction is termed the CRR in this report. This term is recommended for engineering practice.

EVALUATION OF CSR

Seed and Idriss (1971) formulated the following equation for calculation of the cyclic stress ratio:

$$\mathrm{CSR} = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{\max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (1)$$

where $a_{\max}$ = peak horizontal acceleration at the ground surface generated by the earthquake (discussed later); g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively; and r_d = stress reduction coefficient. The latter coefficient accounts for flexibility of the soil profile. The workshop participants recommend the following minor modification to the procedure for calculation of CSR.

For routine practice and noncritical projects, the following equations may be used to estimate average values of r_d (Liao and Whitman 1986b):

$$r_d = 1.0 - 0.00765z \quad \text{for } z \le 9.15 \text{ m} \qquad (2a)$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ m} < z \le 23 \text{ m} \qquad (2b)$$

where z = depth below ground surface in meters. Some investigators have suggested additional equations for estimating r_d at greater depths (Robertson and Wride 1998), but evaluation of liquefaction at these greater depths is beyond the depths where the simplified procedure is verified and where routine applications should be applied. Mean values of r_d calculated from (2) are plotted in Fig. 1, along with the mean and range of values proposed by Seed and Idriss (1971). The workshop participants agreed that for convenience in programming spreadsheets and other electronic aids, and to be consistent with past practice, r_d values determined from (2) are suitable for use in routine engineering practice. The user should understand, however, that there is considerable variability in the

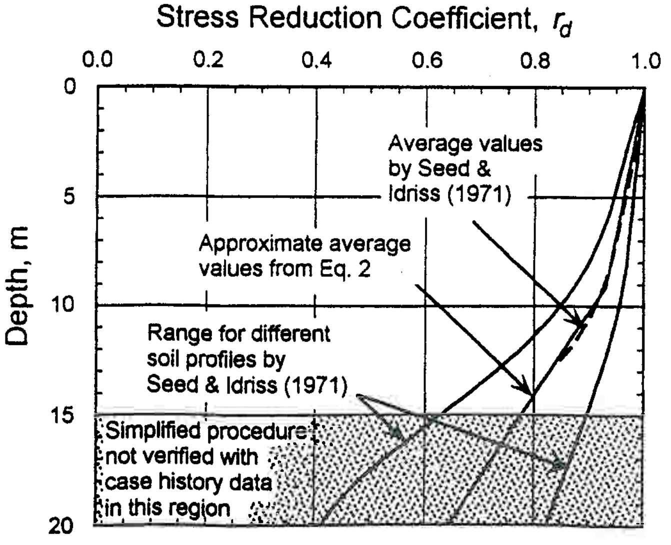


FIG. 1. r_d versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean-Value Lines Plotted from Eq. (2)

flexibility and thus r_d at field sites, that r_d calculated from (2) are the mean of a wide range of possible r_d, and that the range of r_d increases with depth (Golesorkhi 1989).

For ease of computation, T. F. Blake (personal communication, 1996) approximated the mean curve plotted in Fig. 1 by the following equation:

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^{2})} \tag{3}$$

where z = depth beneath ground surface in meters. Eq. (3) yields essentially the same values for r_d as (2), but is easier to program and may be used in routine engineering practice.

I. M. Idriss [Transportation Research Board (TRB) (1999)] suggested a new procedure for determining magnitude-dependent values of r_d. Application of these r_d require use of a corresponding set of magnitude scaling factors that are compatible with the new r_d. Because these r_d were developed after the workshop and have not been independently evaluated by other experts, the workshop participants chose not to recommend the new factors at this time.

EVALUATION OF LIQUEFACTION RESISTANCE (CRR)

A major focus of the workshop was on procedures for evaluating liquefaction resistance. A plausible method for evaluating CRR is to retrieve and test undisturbed soil specimens in the laboratory. Unfortunately, in situ stress states generally cannot be reestablished in the laboratory, and specimens of granular soils retrieved with typical drilling and sampling techniques are too disturbed to yield meaningful results. Only through specialized sampling techniques, such as ground freezing, can sufficiently undisturbed specimens be obtained. The cost of such procedures is generally prohibitive for all but the most critical projects. To avoid the difficulties associated with sampling and laboratory testing, field tests have become the state-of-practice for routine liquefaction investigations.

Several field tests have gained common usage for evaluation of liquefaction resistance, including the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements (V_s), and the Becker penetration test (BPT). These tests were discussed at the workshop, along with associated criteria for evaluating liquefaction resistance. The participants made a conscientious attempt to correlate liquefaction resistance criteria from each of the various field tests to provide generally consistent results, no matter which test is applied. SPTs and CPTs are generally preferred because of the more extensive databases and past experience, but the other tests may be applied at sites underlain by gravelly sediment or where access by large equipment is limited. Primary advantages and disadvantages of each test are listed in Table 1.

SPT

Criteria for evaluation of liquefaction resistance based on the SPT have been rather robust over the years. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ plot reproduced in Fig. 2. $(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure of approximately 100 kPa (1 ton/sq ft) and a hammer energy ratio or hammer efficiency of 60%. The normalization factors for these corrections are discussed in the section entitled Other Corrections. Fig. 2 is a graph of calculated CSR and corresponding $(N_1)_{60}$ data from sites where liquefaction effects were or were not observed following past earthquakes with magnitudes of approximately 7.5. CRR curves on this graph were conservatively positioned to separate regions with data indicative of liquefaction from regions with data indicative of nonliquefaction. Curves were developed for granular soils with the fines contents of 5% or less, 15%, and 35% as shown on the plot. The CRR curve for fines contents <5% is the basic penetration criterion for the simplified procedure and is referred to hereafter as the "SPT clean-sand base curve." The CRR curves in Fig. 2 are valid only for magnitude 7.5 earthquakes. Scaling factors to adjust CRR curves to other magnitudes are addressed in a later section of this report.

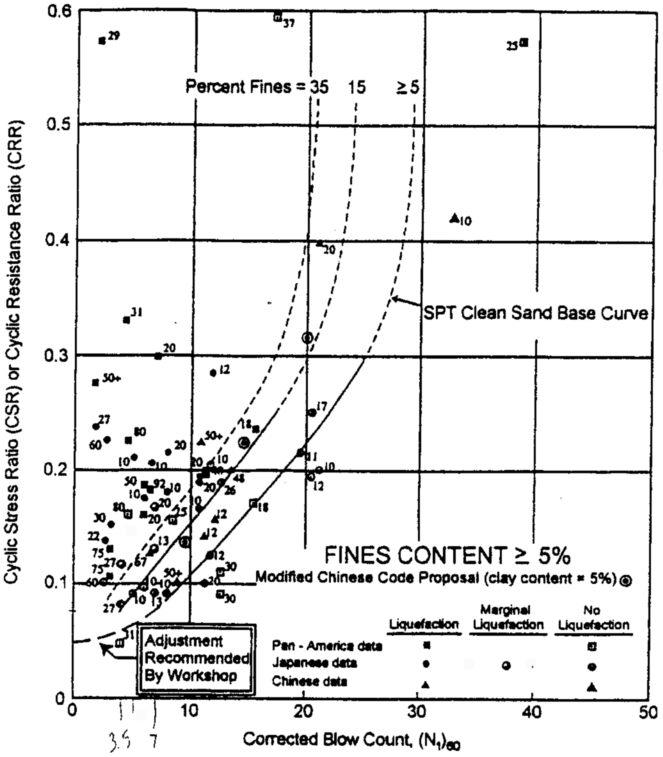


FIG. 2. SPT Clean-Sand Base Curve for Magnitude 7.5 Earthquakes with Data from Liquefaction Case Histories (Modified from Seed et al. 1985)

SPT Clean-Sand Base Curve

Several changes to the SPT criteria are recommended by the workshop participants. The first change is to curve the trajec-

TABLE 1. Comparison of Advantages and Disadvantages of Various Field Tests for Assessment of Liquefaction Resistance

Feature	Test Type			
	SPT	CPT	V_s	BPT
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced tests	Very good	Fair	Fair
Soil types in which test is recommended	Nongravel	Nongravel	All	Primarily gravel
Soil sample retrieved	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering	Index

tory of the clean-sand base curve at low $(N_1)_{60}$ to a projected intercept of about 0.05 (Fig. 2). This adjustment reshapes the clean-sand base curve to achieve greater consistency with CRR curves developed for the CPT and shear-wave velocity procedures. Seed and Idriss (1982) projected the original curve through the origin, but there were few data to constrain the curve in the lower part of the plot. A better fit to the present empirical data is to bow the lower end of the base curve as indicated in Fig. 2.

At the University of Texas, A. F. Rauch (personal communication, 1998), approximated the clean-sand base curve plotted in Fig. 2 by the following equation:

$$\mathrm{CRR}_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10 \cdot (N_1)_{60} + 45]^2} - \frac{1}{200} \qquad (4)$$

This equation is valid for $(N_1)_{60} < 30$. For $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as nonliquefiable. This equation may be used in spreadsheets and other analytical techniques to approximate the clean-sand base curve for routine engineering calculations.

Influence of Fines Content

In the original development, Seed et al. (1985) noted an apparent increase of CRR with increased fines content. Whether this increase is caused by an increase of liquefaction resistance or a decrease of penetration resistance is not clear. Based on the empirical data available, Seed et al. developed CRR curves for various fines contents reproduced in Fig. 2. A revised correction for fines content was developed by workshop attendees to better fit the empirical database and to better support computations with spreadsheets and other electronic computational aids.

The workshop participants recommend (5) and (6) as approximate corrections for the influence of fines content (FC) on CRR. Other grain characteristics, such as soil plasticity, may affect liquefaction resistance as well as fines content, but widely accepted corrections for these factors have not been developed. Hence corrections based solely on fines content should be used with engineering judgment and caution. The following equations were developed by I. M. Idriss with the assistance of R. B. Seed for correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \qquad (5)$$

where α and β = coefficients determined from the following relationships:

$$\alpha = 0 \quad \text{for FC} \leq 5\% \qquad (6a)$$

$$\alpha = \exp[1.76 - (190/\mathrm{FC}^2)] \quad \text{for } 5\% < \text{FC} < 35\% \qquad (6b)$$

$$\alpha = 5.0 \quad \text{for FC} \geq 35\% \qquad (6c)$$

$$\beta = 1.0 \quad \text{for FC} \leq 5\% \qquad (7a)$$

$$\beta = [0.99 + (\mathrm{FC}^{1.5}/1{,}000)] \quad \text{for } 5\% < \text{FC} < 35\% \qquad (7b)$$

$$\beta = 1.2 \quad \text{for FC} \geq 35\% \qquad (7c)$$

These equations may be used for routine liquefaction resistance calculations. A back-calculated curve for a fines content of 35% is essentially congruent with the 35% curve plotted in Fig. 2. The back-calculated curve for a fines contents of 15% plots to the right of the original 15% curve.

Other Corrections

TABLE 2. Corrections to SPT (Modified from Skempton 1986) as Listed by Robertson and Wride (1998)

Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

Several factors in addition to fines content and grain characteristics influence SPT results, as noted in Table 2. Eq. (8) incorporates these corrections

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \qquad (8)$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; and C_S = correction for samplers with or without liners.

Because SPT N-values increase with increasing effective overburden stress, an overburden stress correction factor is applied (Seed and Idriss 1982). This factor is commonly calculated from the following equation (Liao and Whitman 1986a):

$$C_N = (P_a/\sigma'_{vo})^{0.5} \qquad (9)$$

where C_N normalizes N_m to an effective overburden pressure σ'_{vo} of approximately 100 kPa (1 atm) P_a. C_N should not exceed a value of 1.7 [A maximum value of 2.0 was published in the National Center for Earthquake Engineering Research (NCEER) workshop proceedings (Youd and Idriss 1997), but later was reduced to 1.7 by consensus of the workshop participants] Kayen et al. (1992) suggested the following equation, which limits the maximum C_N value to 1.7, and in these writers' opinion, provides a better fit to the original curve specified by Seed and Idriss (1982):

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a) \qquad (10)$$

Either equation may be used for routine engineering applications.

The effective overburden pressure σ'_{vo} applied in (9) and (10) should be the overburden pressure at the time of drilling and testing. Although a higher ground-water level might be used for conservatism in the liquefaction resistance calculations, the C_N factor must be based on the stresses present at the time of the testing.

The C_N correction factor was derived from SPT performed in test bins with large sand specimens subjected to various confining pressures (Gibbs and Holtz 1957; Marcuson and Bieganousky 1997a,b). The results of several of these tests are reproduced in Fig. 3 in the form of C_N curves versus effective overburden stress (Castro 1995). These curves indicate considerable scatter of results with no apparent correlation of C_N with soil type or gradation. The curves from looser sands, however, lie in the lower part of the C_N range and are reasonably approximated by (9) and (10) for low effective overburden pressures [200 kPa (<2 tsf)]. The workshop participants endorsed the use of (9) for calculation of C_N, but acknowledged that for overburden pressures >200 kPa (2 tsf) the results are uncertain. Eq. (10) provides a better fit for overburden

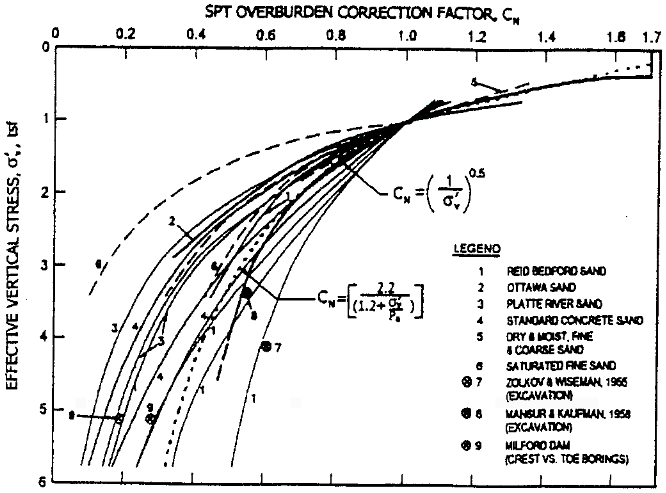


FIG. 3. C_N Curves for Various Sands Based on Field and Laboratory Test Data along with Suggested C_N Curve Determined from Eqs. (9) and (10) (Modified from Castro 1995)

pressures up to 300 kPa (3 tsf). For pressures >300 kPa (3 tsf), the uncertainty is so great that (9) should not be applied. At these high pressures, which are generally below the depth for which the simplified procedure has been verified, C_N should be estimated by other means.

Another important factor is the energy transferred from the falling hammer to the SPT sampler. An ER of 60% is generally accepted as the approximate average for U.S. testing practice and as a reference value for energy corrections. The ER delivered to the sampler depends on the type of hammer, anvil, lifting mechanism, and the method of hammer release. Approximate correction factors (C_E = ER/60) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 2. Because of variations in drilling and testing equipment and differences in testing procedures, a rather wide range in the energy correction factor C_E has been observed as noted in the table. Even when procedures are carefully monitored to conform to established standards, such as ASTM D 1586-99, some variation in C_E may occur because of minor variations in testing procedures. Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%. The workshop participants recommend measurement of the hammer energy frequently at each site where the SPT is used. Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate a C_E value for use in liquefaction resistance calculations. Use of good-quality testing equipment and carefully controlled testing procedures conforming to ASTM D 1586-99 will generally yield more consistent energy ratios and C_E with values from the upper parts of the ranges listed in Table 2.

Skempton (1986) suggested and Robertson and Wride (1998) updated correction factors for rod lengths <10 m, borehole diameters outside the recommended interval (65–125 mm), and sampling tubes without liners. Range for these correction factors are listed in Table 2. For liquefaction resistance calculations and rod lengths <3 m, a C_R of 0.75 should be applied as was done by Seed et al. (1985) in formulating the simplified procedure. Although application of rod-length correction factors listed in Table 2 will give more precise $(N_1)_{60}$ values, these corrections may be neglected for liquefaction resistance calculations for rod lengths between 3 and 10 m because rod-length corrections were not applied to SPT test data from these depths in compiling the original liquefaction case history databases. Thus rod-length corrections are implicitly incorporated into the empirical SPT procedure.

A final change recommended by workshop participants is the use of revised magnitude scaling factors rather than the original Seed and Idriss (1982) factors to adjust CRR for earthquake magnitudes other than 7.5. Magnitude scaling factors are addressed later in this report.

CPT

A primary advantage of the CPT is that a nearly continuous profile of penetration resistance is developed for stratigraphic interpretation. The CPT results are generally more consistent and repeatable than results from other penetration tests listed in Table 1. The continuous profile also allows a more detailed definition of soil layers than the other tools listed in the table. This stratigraphic capability makes the CPT particularly advantageous for developing liquefaction-resistance profiles. Interpretations based on the CPT, however, must be verified with a few well-placed boreholes preferably with standard penetration tests, to confirm soil types and further verify liquefaction-resistance interpretations.

Fig. 4 provides curves prepared by Robertson and Wride (1998) for direct determination of CRR for clean sands (FC $\leq$ 5%) from CPT data. This figure was developed from CPT case history data compiled from several investigations, including those by Stark and Olson (1995) and Suzuki et al. (1995). The chart, valid for magnitude 7.5 earthquakes only, shows calculated cyclic resistance ratio plotted as a function of dimensionless, corrected, and normalized CPT resistance q_{c1N} from sites where surface effects of liquefaction were or were not observed following past earthquakes. The CRR curve conservatively separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction.

Based on a few misclassified case histories from the 1989 Loma Prieta earthquake, I. M. Idriss suggested that the clean sand curve in Fig. 4 should be shifted to the right by 10–15%. However, a majority of workshop participants supported a curve in its present position, for three reasons. First, a purpose of the workshop was to recommend criteria that yield roughly equivalent CRR for the field tests listed in Table 1. Shifting the base curve to the right makes the CPT criteria generally more conservative. For example, for $(N_1)_{60} > 5$, q_{c1N}:$(N_1)_{60}$ ra-

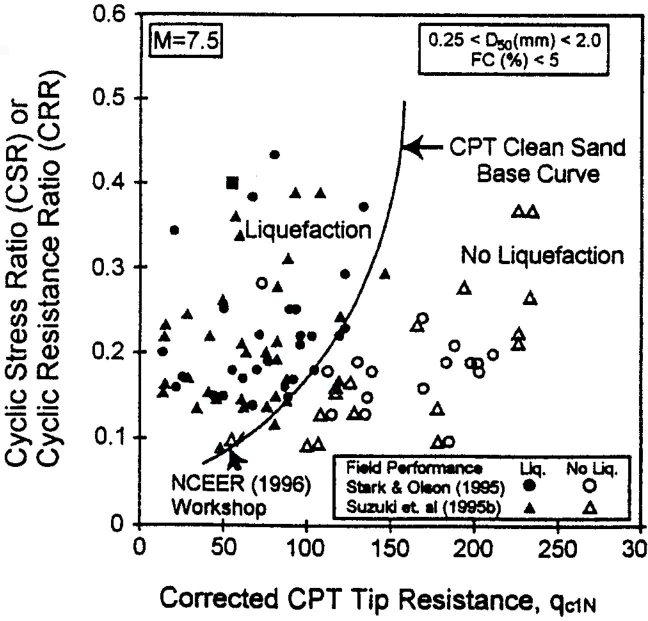


FIG. 4. Curve Recommended for Calculation of CRR from CPT Data along with Empirical Liquefaction Data from Compiled Case Histories (Reproduced from Robertson and Wride 1998)

tios between the two clean-sand base curves, plotted in Figs. 4 and 2, respectively, range from 5 to 8—values that are slightly higher than those expected for clean sands. Shifting the CPT base curve to the right by 10 to 15% would increase those ratios to unusually high values ranging from 6 to 9. Second, base curves, such as those plotted in Figs. 2 and 4, were intended to be conservative, but not necessarily to encompass every data point on the plot. Thus the presence of a few points beyond the base curve should be allowable. Finally, several studies have confirmed that the CPT criteria in Fig. 4 are generally conservative. Robertson and Wride (1998) verified these criteria against SPT and other data from sites they investigated. Gilstrap and Youd (1998) compared calculated liquefaction resistances against field performance at 19 sites and concluded that the CPT criteria correctly predicted the occurrence or nonoccurrence of liquefaction with >85% reliability.

The clean-sand base curve in Fig. 4 may be approximated by the following equation (Robertson and Wride 1998):

$$\text{If } (q_{c1N})_{cs} < 50 \quad \text{CRR}_{7.5} = 0.833[(q_{c1N})_{cs}/1{,}000] + 0.05 \qquad (11a)$$

$$\text{If } 50 \leq (q_{c1N})_{cs} < 160 \quad \text{CRR}_{7.5} = 93[(q_{c1N})_{cs}/1{,}000]^3 + 0.08 \qquad (11b)$$

where $(q_{c1N})_{cs}$ = clean-sand cone penetration resistance normalized to approximately 100 kPa (1 atm).

Normalization of Cone Penetration Resistance

The CPT procedure requires normalization of tip resistance using (12) and (13). This transformation yields normalized, dimensionless cone pentration resistance q_{c1N}

$$q_{c1N} = C_Q(q_c/P_a) \qquad (12)$$

where

$$C_Q = (P_a/\sigma'_{vo})^n \qquad (13)$$

and where C_Q = normalizing factor for cone penetration resistance; P_a = 1 atm of pressure in the same units used for σ'_{vo}; n = exponent that varies with soil type; and q_c = field cone penetration resistance measured at the tip. At shallow depths C_Q becomes large because of low overburden pressure; however, values >1.7 should not be applied. As noted in the following paragraphs, the value of the exponent n varies from 0.5 to 1.0, depending on the grain characteristics of the soil (Olsen 1997).

The CPT friction ratio (sleeve resistance f_s divided by cone tip resistance q_c) generally increases with increasing fines content and soil plasticity, allowing rough estimates of soil type and fines content to be determined from CPT data. Robertson and Wride (1998) constructed the chart reproduced in Fig. 5 for estimation of soil type. The boundaries between soil types 2–7 can be approximated by concentric circles and can be used to account for effects of soil characteristics on q_{c1N} and CRR. The radius of these circles, termed the soil behavior type index I_c is calculated from the following equation:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \qquad (14)$$

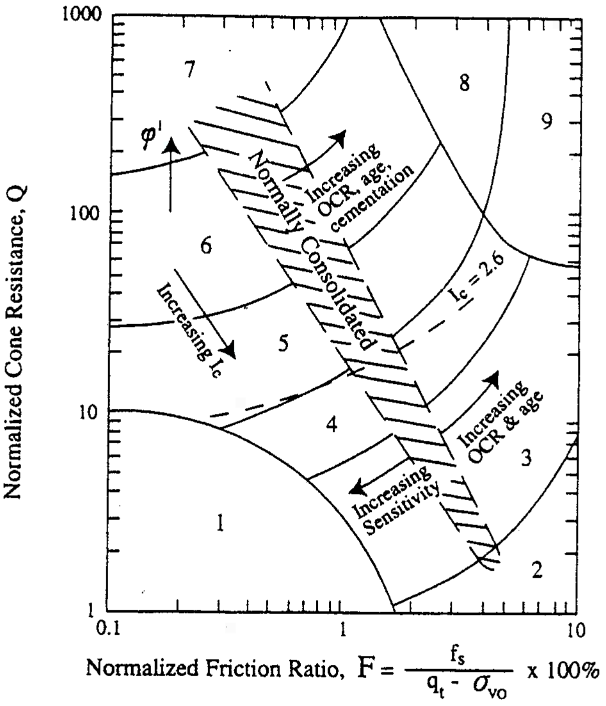
where

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \qquad (15)$$

and

$$F = [f_s/(q_c - \sigma_{vo})] \times 100\% \qquad (16)$$

The soil behavior chart in Fig. 5 was developed using an exponent n of 1.0, which is the appropriate value for clayey soil types. For clean sands, however, an exponent value of 0.5 is more appropriate, and a value intermediate between 0.5 and 1.0 would be appropriate for silts and sandy silts. Robertson and Wride recommended the following procedure for calculating the soil behavior type index I_c. The first step is to differentiate soil types characterized as clays from soil types characterized as sands and silts. This differentiation is performed by assuming an exponent n of 1.0 (characteristic of clays) and calculating the dimensionless CPT tip resistance Q from the following equation:



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

**Heavily overconsolidated or cemented*

FIG. 5. CPT-Based Soil Behavior-Type Chart Proposed by Robertson (1990)

$$Q = [(q_c - \sigma_{vo})/P_a][P_a/\sigma'_{vo}]^{1.0} = [(q_c - \sigma_{vo})/\sigma'_{vo}] \qquad (17)$$

If the I_c calculated with an exponent of 1.0 is >2.6, the soil is classified as clayey and is considered too clay-rich to liquefy, and the analysis is complete. However, soil samples should be retrieved and tested to confirm the soil type and liquefaction resistance. Criteria such as the Chinese criteria might be applied to confirm that the soil is nonliquefiable. The so-called Chinese criteria, as defined by Seed and Idriss (1982), specify that liquefaction can only occur if all three of the following conditions are met:

1. The clay content (particles smaller than 5 μ) is <15% by weight.
2. The liquid limit is <35%.
3. The natural moisture content is >0.9 times the liquid limit.

If the calculated I_c is <2.6, the soil is most likely granular in nature, and therefore C_q and Q should be recalculated using an exponent n of 0.5. I_c should then be recalculated using (14). If the recalculated I_c is <2.6, the soil is classed as nonplastic and granular. This I_c is used to estimate liquefaction resistance, as noted in the next section. However, if the recalculated I_c is

>2.6, the soil is likely to be very silty and possibly plastic. In this instance, q_{c1N} should be recalculated from (12) using an intermediate exponent n of 0.7 in (13). I_c is then recalculated from (14) using the recalculated value for q_{c1N}. This intermediate I_c is then used to calculate liquefaction resistance. In this instance, a soil sample should be retrieved and tested to verify the soil type and whether the soil is liquefiable by other criteria, such as the Chinese criteria.

Because the relationship between I_c and soil type is approximate, the consensus of the workshop participants is that all soils with an I_c of 2.4 or greater should be sampled and tested to confirm the soil type and to test the liquefiability with other criteria. Also, soil layers characterized by an $I_c > 2.6$, but with a normalized friction ratio $F < 1.0\%$ (region 1 of Fig. 5) may be very sensitive and should be sampled and tested. Although not technically liquefiable according to the Chinese criteria, such sensitive soils may suffer softening and strength loss during earthquake shaking.

Calculation of Clean-Sand Equivalent Normalized Cone Penetration Resistance $(q_{c1N})_{cs}$

The normalized penetration resistance (q_{c1N}) for silty sands is corrected to an equivalent clean sand value $(q_{c1N})_{cs}$, by the following relationship:

$$(q_{c1N})_{cs} = K_c q_{c1N} \tag{18}$$

where K_c, the correction factor for grain characteristics, is defined by the following equation (Robertson and Wride 1998):

$$\text{for } I_c \leq 1.64 \quad K_c = 1.0 \tag{19a}$$

$$\text{for } I_c > 1.64 \quad K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \tag{19b}$$

The K_c curve defined by (19) is plotted in Fig. 6. For $I_c > 2.6$, the curve is shown as a dashed line, indicating that soils in this range of I_c are most likely too clay-rich or plastic to liquefy.

With an appropriate I_c and K_c, (11) and (19) can be used to calculate $\mathrm{CRR}_{7.5}$. To adjust CRR to magnitudes other than 7.5, the calculated $\mathrm{CRR}_{7.5}$ is multiplied by an appropriate magnitude scaling factor. The same magnitude scaling factors are used with CPT data as with SPT data. Magnitude scaling factors are discussed in a later section of this report.

Olsen (1997) and Suzuki et al. (1995) Procedures

Olsen (1997), who pioneered many of the techniques for assessing liquefaction resistance from CPT soundings, suggested a somewhat different procedure for calculating CRR from CPT data. Reasons for recommending the Robertson and Wride (1998) procedure over that of Olsen are the ease of application and the ease with which relationships can be quantified for computer-aided calculations. Results from Olsen's procedure, however, are consistent with results from the procedure proposed here for shallow (<15 m deep) sediment beneath level to gently sloping terrain. Olsen (1997) noted that almost any CPT normalization technique will give results consistent with his normalization procedure for soil layers in the 3–15 m depth range. For deeper layers, significant differences may develop between the two procedures. Those depths are also beyond the depth for which the simplified procedure has been verified. Hence any procedure based on the simplified procedure yields rather uncertain results at depths >15 m.

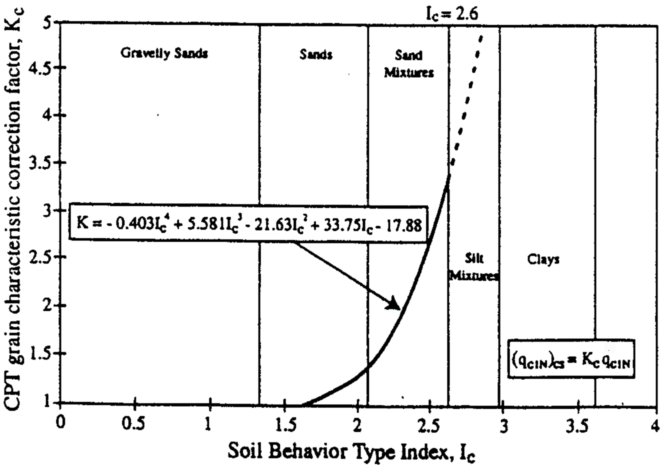


FIG. 6. Grain-Characteristic Correction Factor K_c for Determination of Clean-Sand Equivalent CPT Resistance (Reproduced from Robertson and Wride 1998)

Suzuki et al. (1995) also developed criteria for evaluating CRR from CPT data. Those criteria are slightly more conservative than those of Robertson and Wride (1998) and were considered by the latter investigators in developing the criteria recommended herein.

Correction of Cone Penetration Resistance for Thin Soil Layers

Theoretical as well as laboratory studies indicate that CPT tip resistance is influenced by softer soil layers above or below the cone tip. As a result, measured CPT tip resistance is smaller in thin layers of granular soils sandwiched between softer layers than in thicker layers of the same granular soil. The amount of the reduction of penetration resistance in soft layers is a function of the thickness of the softer layer and the stiffness of the stiffer layers.

Using a simplified elastic solution, Vreugdenhil et al. (1994) developed a procedure for estimating the thick-layer equivalent cone penetration resistance of thin stiff layers lying within softer strata. The correction applies only to thin stiff layers embedded within thick soft layers. Because the corrections have a reasonable trend, but appear rather large, Robertson and Fear (1995) recommended conservative corrections from the $q_{cA}/q_{cB} = 2$ curve sketched in Fig. 7.

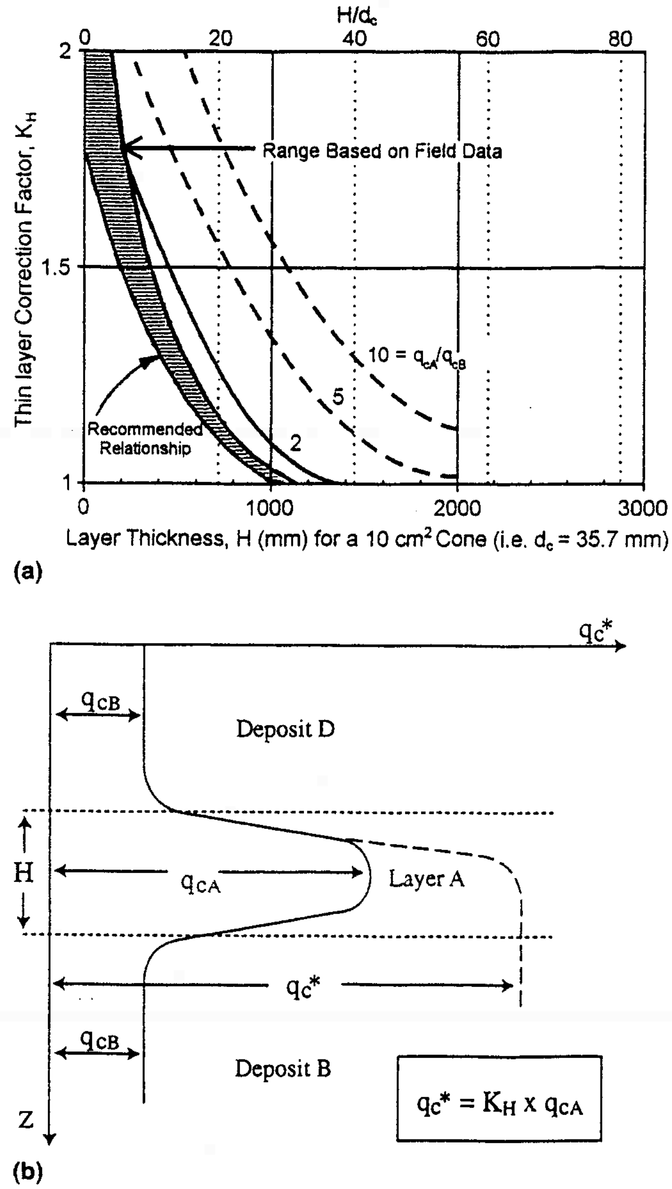
Further analysis of field data by Gonzalo Castro and Peter Robertson for the NCEER workshop indicates that corrections based on the $q_{cA}/q_{cB} = 2$ curve may still be too large and not adequately conservative. They suggested, and the workshop participants agreed, that the lower bound of the range of field data plotted by G. Castro in Fig. 7 provides more conservative K_H values that should be used until further field studies and analyses indicate that higher values are viable. The equation for the lower bound of the field curve is

$$K_H = 0.25[((H/d_c)/17) - 1.77]^2 + 1.0 \tag{20}$$

where H = thickness of the interbedded layer in mm; q_{cA} and q_{cB} = cone resistances of the stiff and soft layers, respectively; and d_c = diameter of the cone in mm (Fig. 7).

V_s

Andrus and Stokoe (1997, 2000) developed liquefaction resistance criteria from field measurements of shear wave velocity V_s. The use of V_s as a field index of liquefaction resistance is soundly based because both V_s and CRR are similarly, but not proportionally, influenced by void ratio, effective confining stresses, stress history, and geologic age. The advantages of using V_s include the following: (1) V_s measurements are possible in soils that are difficult to penetrate with CPT and SPT or to extract undisturbed samples, such as gravelly soils, and at sites where borings or soundings may not be permitted; (2) V_s is a basic mechanical property of soil materials, directly related to small-strain shear modulus; and (3) the



$q_c^* = K_H \times q_{cA}$

FIG. 7. Thin-Layer Correction Factor K_H for Determination of Equivalent Thick-Layer CPT Resistance (Modified from Robertson and Fear 1995)

small-strain shear modulus is a parameter required in analytical procedures for estimating dynamic soil response and soil-structure interaction analyses.

Three concerns arise when using V_s for liquefaction-resistance evaluations: (1) seismic wave velocity measurements are made at small strains, whereas pore-water pressure buildup and the onset of liquefaction are medium- to high-strain phenomena; (2) seismic testing does not provide samples for classification of soils and identification of nonliquefiable soft clay-rich soils; and (3) thin, low V_s strata may not be detected if the measurement interval is too large. Therefore the preferred practice is to drill sufficient boreholes and conduct in situ tests to detect and delineate thin liquefiable strata, nonliquefiable clay-rich soils, and silty soils above the ground-water table that might become liquefiable should the water table rise. Other tests, such as the SPT or CPT, are needed to detect liquefiable weakly cemented soils that may have high V_s values.

V_s Criteria for Evaluating Liquefaction Resistance

Following the traditional procedures for correcting penetration resistance to account for overburden stress, V_s is also corrected to a reference overburden stress using the following equation (Sykora 1987; Kayen et al. 1992; Robertson et al. 1992):

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25} \qquad (21)$$

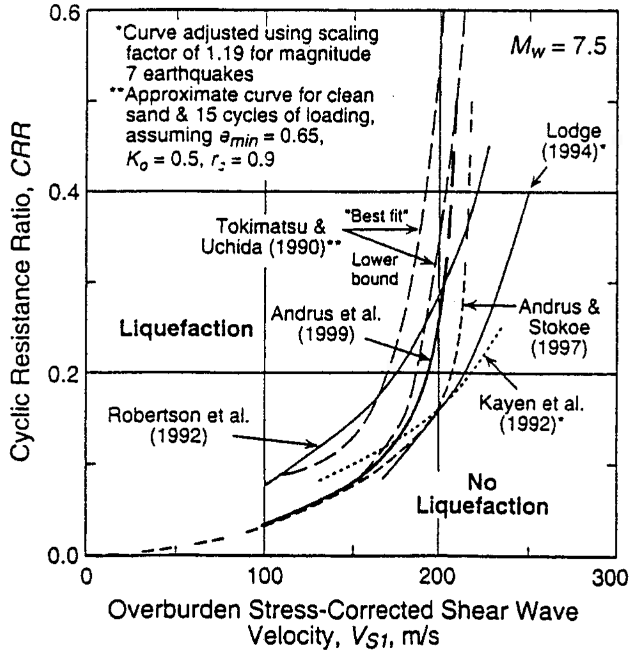
where V_{s1} = overburden-stress corrected shear wave velocity; P_a = atmospheric pressure approximated by 100 kPa (1 TSF); and σ'_{vo} = initial effective vertical stress in the same units as P_a. Eq. (21) implicitly assumes a constant coefficient of earth pressure K'_o which is approximately 0.5 for sites susceptible to liquefaction. Application of (21) also implicitly assumes that V_s is measured with both the directions of particle motion and wave propagation polarized along principal stress directions and that one of those directions is vertical (Stokoe et al. 1985).

Fig. 8 compares seven CRR-V_{s1} curves. The "best fit" curve by Tokimatsu and Uchida (1990) was determined from laboratory cyclic triaxial test results for various sands with <10% fines and 15 cycles of loading. The more conservative "lower bound" curve for Tokimatsu and Uchida's laboratory test results is also shown as a lower bound for liquefaction occurrences. The bounding curve by Robertson et al. (1992) was developed using field performance data from sites in Imperial Valley, Calif., along with data from four other sites. The curves by Kayen et al. (1992) and Lodge (1994) are from sites that did and did not liquefy during the 1989 Loma Prieta earthquake. Andrus and Stokoe's (1997) curve was developed for uncemented, Holocene-age soils with 5% or less fines using field performance data from 20 earthquakes and over 50 measurement sites. Andrus and Stokoe (2000) revised this curve based on new information and an expanded database that includes 26 earthquakes and more than 70 measurement sites.

Andrus and Stokoe (1997) proposed the following relationship between CRR and V_{s1}:

$$\mathrm{CRR} = a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V^*_{s1} - V_{s1}} - \frac{1}{V^*_{s1}}\right) \qquad (22)$$

where V^*_{s1} = limiting upper value of V_{s1} for liquefaction occurrence; and a and b are curve fitting parameters. The first parenthetical term of (22) is based on a modified relationship



*Curve adjusted using scaling factor of 1.19 for magnitude 7 earthquakes

**Approximate curve for clean sand & 15 cycles of loading, assuming e_{min} = 0.65, K_o = 0.5, r_c = 0.9

FIG. 8. Comparison of Seven Relationships between Liquefaction Resistance and Overburden Stress-Corrected Shear Wave Velocity for Granular Soils

between V_{s1} and CSR for constant average cyclic shear strain suggested by R. Dobry (personal communication to R. D. Andrus, 1996). The second parenthetical term is a hyperbola with a small value at low V_{s1}, and a very large value as V_{s1} approaches V^*_{s1}, a constant limiting velocity for liquefaction of soils.

CRR versus V_{s1} curves recommended for engineering practice by Andrus and Stokoe (2000) for magnitude 7.5 earthquakes and uncemented Holocene-age soils with various fines contents are reproduced in Fig. 9. Also plotted and presented in Fig. 9 are points calculated from liquefaction case history information for magnitude 5.9–8.3 earthquakes. The three curves shown were determined through an iterative process of varying the values of a and b until nearly all the points indicative of liquefaction were bounded by the curves with the least number of nonliquefaction points plotted in the liquefaction region. The final values of a and b used to draw the curves were 0.022 and 2.8, respectively. Values of V^*_{s1} were assumed to vary linearly from 200 m/s for soils with fines content of 35% to 215 m/s for soils with fines content of 5% or less.

The recommended curves shown in Fig. 9 are dashed above CRR of 0.35 to indicate that field-performance data are limited in that range. Also, they do not extend much below 100 m/s, because there are no field data to support extending them to the origin. The calculated CRR is 0.033 for a V_{s1} of 100 m/s. This minimal CRR value is generally consistent with intercept CRR values assumed for the CPT and SPT procedures. Eq. (22) can be scaled to other magnitude values through use of magnitude scaling factors. These factors are discussed in a later section of this paper.

BPT

Liquefaction resistance of nongravelly soils has been evaluated primarily through CPT and SPT, with occasional V_s measurements. CPT and SPT measurements, however, are not generally reliable in gravelly soils. Large gravel particles may interfere with the normal deformation of soil materials around the penetrometer and misleadingly increase penetration resistance. Several investigators have employed large-diameter penetrometers to surmount these difficulties; the Becker penetration test (BPT) in particular has become one of the more effectively and widely used larger tools. The BPT was developed in Canada in the late 1950s and consists of a 168-mm diameter, 3-m-long double-walled casing driven into the ground with a double-acting diesel-driven pile hammer. The hammer impacts are applied at the top of the casing and peneration is continuous. The Becker penetration resistance is defined as the number of blows required to drive the casing through an increment of 300 mm.

The BPT has not been standardized, and several different types of equipment and procedures have been used. There are currently very few liquefaction sites from which BPT data have been obtained. Thus the BPT cannot be directly correlated with field behavior, but rather through estimating equivalent SPT N-values from BPT data and then applying evaluation procedures based on the SPT. This indirect method introduces substantial additional uncertainty into the calculated CRR.

To provide uniformity, Harder and Seed (1986) recommended newer AP-1000 drill rigs equipped with supercharged diesel hammers, 168-mm outside diameter casing, and a plugged bit. From several sites where both BPT and SPT tests were conducted in parallel soundings, Harder and Seed (1986) developed a preliminary correlation between Becker and standard penetration resistance [Fig. 10(a)]. Additional comparative data compiled since 1986 are plotted in Fig. 10(b). The original Harder and Seed correlation curve (solid line) is drawn in Fig. 10(b) along with dashed curves representing 20% over- and underpredictions of SPT blow counts. These plots indicate that SPT blow counts can be roughly estimated

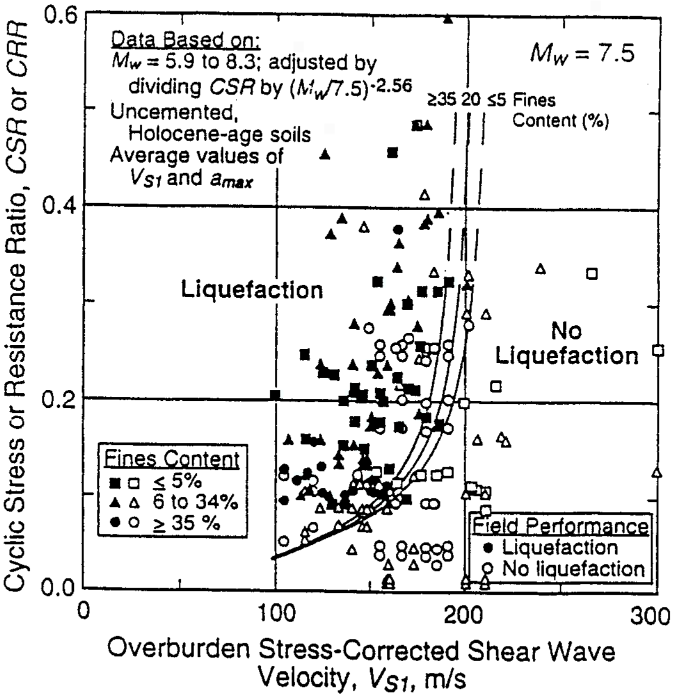


FIG. 9. Liquefaction Relationship Recommended for Clean, Uncemented Soils with Liquefaction Data from Compiled Case Histories (Reproduced from Andrus and Stokoe 2000)

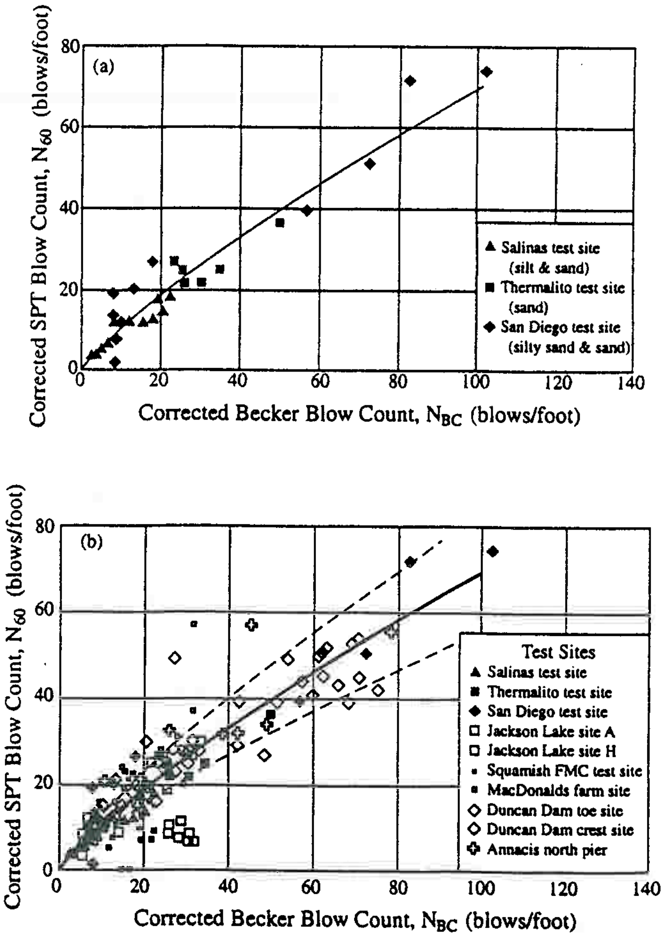


FIG. 10. Correlation between Corrected Becker Penetration Resistance N_{BC} and Corrected SPT Resistance N_{60}: (a) Harder and Seed (1986); (b) Data from Additional Sites (Reproduced from Harder 1997)

from BPT measurements. These plots indicate that although SPT blow counts can be roughly estimated from BPT measurements, there can be considerable uncertainty for calculating liquefaction resistance because the data scatter is greatest in the range of greatest importance [*N*-values of 0–30 blows/300 mm (ft)].

A major source of variation in BPT blow counts is deviations in hammer energy. Rather than measuring hammer energy directly, Harder and Seed (1986) monitored bounce-chamber pressures and found that uniform combustion conditions (e.g., full throttle with a supercharger) correlated rather well with variations in Becker blow count. From this information, Harder and Seed developed an energy correction procedure based on measured bounce-chamber pressure.

Direct measurement of transmitted hammer energy could provide a more theoretically rigorous correction for Becker hammer efficiency. Sy and Campanella (1994) and Sy et al. (1995) instrumented a small length of Becker casing with strain gauges and accelerometers to measure transferred energy. They analyzed the recorded data with a pile-driving analyzer to determine strain, force, acceleration, and velocity. The transferred energy was determined by time integration of force times velocity. They were able to verify many of the variations in hammer energy previously identified by Harder and Seed (1986), including effects of variable throttle settings and energy transmission efficiencies of various drill rigs. However, they were unable to reduce the amount of scatter and uncertainty in converting BPT blow counts to SPT blow counts. Because the Sy and Campanella procedure requires considerably more effort than monitoring of bounce-chamber pressure without producing greatly improved results, the workshop participants agreed that the bounce-chamber technique is adequate for routine practice.

Friction along the driven casing also influences penetration resistance. Harder and Seed (1986) did not directly evaluate the effect of casing friction; hence, the correlation in Fig. 10(b) intrinsically incorporates an unknown amount of casing friction. However, casing friction remains a concern for depths >30 m and for measurement of penetration resistance in soft soils underlying thick deposits of dense soil. Either of these circumstances could lead to greater casing friction than is intrinsically incorporated in the Harder and Seed correlation.

The following procedures are recommended for routine practice: (1) the BPT should be conducted with newer AP-1000 drill rigs equipped with supercharged diesel hammers to drive plugged 168-mm outside diameter casing; (2) bounce-chamber pressures should be monitored and adjustments made to measured BPT blow counts to account for variations in diesel hammer combustion efficiency—for most routine applications, correlations developed by Harder and Seed (1986) may be used for these adjustments; and (3) the influence of some casing friction is indirectly accounted for in the Harder and Seed BPT-SPT correlation. This correlation, however, has not been verified and should not be used for depths >30 m or for sites with thick dense deposits overlying loose sands or gravels. For these conditions, mudded boreholes may be needed to reduce casing friction, or specially developed local correlations or sophisticated wave-equation analyses may be applied to quantify frictional effects.

MAGNITUDE SCALING FACTORS (MSFs)

The clean-sand base or CRR curves in Figs. 2 (SPT), 4 (CPT), and 10 (V_{s1}) apply only to magnitude 7.5 earthquakes. To adjust the clean-sand curves to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors termed "magnitude scaling factors (MSFs)." These factors are used to scale the CRR base curves upward or downward on CRR versus $(N_1)_{60}$, q_{c1N}, or V_{s1} plots. Conversely, magnitude weighting factors, which are the inverse of magnitude scaling factors, may be applied to correct CSR for magnitude. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. Because the original papers by Seed and Idriss were written in terms of magnitude scaling factors, the use of magnitude scaling factors is continued in this report.

To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (FS) against liquefaction is written in terms of CRR, CSR, and MSF as follows:

$$FS = (CRR_{7.5}/CSR)MSF \qquad (23)$$

where CSR = calculated cyclic stress ratio generated by the earthquake shaking; and $CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquakes. $CRR_{7.5}$ is determined from Fig. 2 or (4) for SPT data, Fig. 4 or (11) for CPT data, or Fig. 9 or (22) for V_{s1} data.

Seed and Idriss (1982) Scaling Factors

Because of the limited amount of field liquefaction data available in the 1970s, Seed and Idriss (1982) were unable to adequately constrain bounds between liquefaction and nonliquefaction regions on CRR plots for magnitudes other than 7.5. Consequently, they developed a set of MSF from average numbers of loading cycles for various earthquake magnitudes and laboratory test results. A representative curve developed by these investigators, showing the number of loading cycles required to generate liquefaction for a given CSR, is reproduced in Fig. 11. The average number of loading cycles for various magnitudes of earthquakes are also noted on the plot. The initial set of magnitude scaling factors was derived by dividing CSR values on the representative curve for the number of loading cycles corresponding to a given earthquake magnitude by the CSR for 15 loading cycles (equivalent to a magnitude 7.5 earthquake). These scaling factors are listed in column 2 of Table 3 and are plotted in Fig. 12. These MSFs have been routinely applied in engineering practice since their introduction in 1982.

Revised Idriss Scaling Factors

In preparing his H. B. Seed Memorial Lecture, I. M. Idriss reevaluated the data that he and the late Professor Seed used to calculate the original (1982) magnitude scaling factors. In so doing, Idriss replotted the data on a log-log plot and suggested that the data should plot as a straight line. He noted, however, that one outlying point had strongly influenced the original analysis, causing the original plot to be nonlinear and characterized by unduly low MSF values for magnitudes <7.5. Based on this reevaluation, Idriss defined a revised set of magnitude scaling factors listed in column 3 of Table 3 and plotted

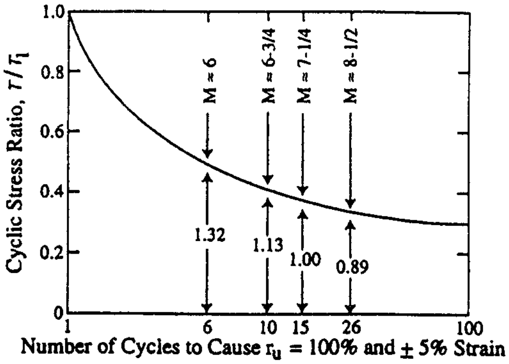


FIG. 11. Representative Relationship between CSR and Number of Cycles to Cause Liquefaction (Reproduced from Seed and Idriss 1982)

TABLE 3. Magnitude Scaling Factor Values Defined by Various Investigators (Youd and Noble 1997a)

Magnitude, M	Seed and Idriss (1982)	Idriss*	Ambraseys (1988)	Arango (1996)		Andrus and Stokoe (1997)	Youd and Noble (1997b)		
				Distance based	Energy based		$P_L < 20\%$	$P_L < 32\%$	$P_L < 50\%$
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.6	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00	—	—	1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.8?	—	—	0.73?
8.5	0.89	0.72	0.44	—	—	0.65?	—	—	0.56?

Note: ? = Very uncertain values.

*1995 Seed Memorial Lecture, University of California at Berkeley (I. M. Idriss, personal communication to T. L. Youd, 1997).

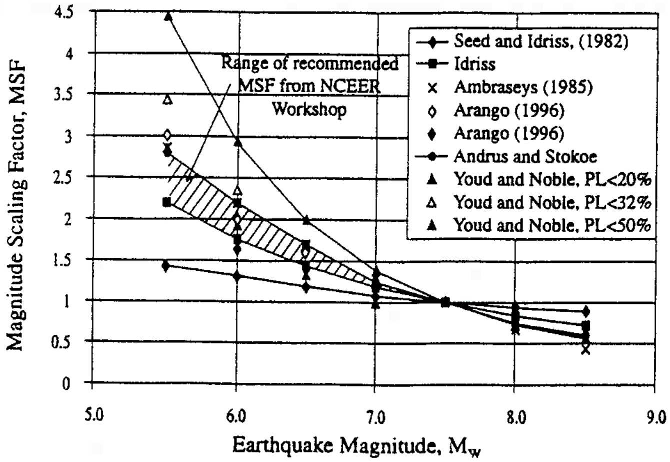


FIG. 12. Magnitude Scaling Factors Derived by Various Investigators (Reproduced from Youd and Noble 1997a)

in Fig. 12. The revised MSFs are defined by the following equation:

$$\text{MSF} = 10^{2.24}/M_w^{2.56} \tag{24}$$

The workshop participants recommend these revised scaling factors as a lower bound for MSF values.

The revised scaling factors are significantly higher than the original scaling factors for magnitudes <7.5 and somewhat lower than the original factors for magnitudes >7.5. Relative to the original scaling factors, the revised factors lead to a reduced calculated liquefaction hazard for magnitudes <7.5, but increase calculated hazard for magnitudes >7.5.

Ambraseys (1988) Scaling Factors

Field performance data collected since the 1970s for magnitudes <7.5 indicate that the original Seed and Idriss (1982) scaling factors are overly conservative. For example, Ambraseys (1988) analyzed liquefaction data compiled through the mid-1980s and plotted calculated cyclic stress ratios for sites that did or did not liquefy versus $(N_1)_{60}$. From these plots, Ambraseys developed empirical exponential equations that define CRR as a function of $(N_1)_{60}$ and moment magnitude M_w. By holding the value of $(N_1)_{60}$ constant in the equations and taking the ratio of CRR determined for various magnitudes of earthquakes to the CRR for magnitude 7.5 earthquakes, Ambraseys derived the magnitude scaling factors listed in column 4 of Table 3 and plotted in Fig. 12. For magnitudes <7.5, the MSFs suggested by Ambraseys are significantly larger than both the original factors developed by Seed and Idriss (column 2, Table 3) and the revised factors suggested by Idriss (column 3). Because they are based on observational data, these factors have validity for estimating liquefaction hazard; however, they have not been widely used in engineering practice.

For magnitudes >7.5, Ambraseys factors are significantly lower and much more conservative than the original (Seed and Idriss 1982) and Idriss's revised scaling factors. Because there are few data to constrain Ambraseys' scaling factors for magnitudes >7.5, they are not recommended for hazard evaluation for large earthquakes.

Arango (1996) Scaling Factors

Arango (1996) developed two sets of magnitude scaling factors. The first set (column 5, Table 3) is based on furthest observed liquefaction effects from the seismic energy source, the estimated average peak accelerations at those distant sites, and the seismic energy required to cause liquefaction. The second set (column 6, Table 3) was developed from energy concepts and the relationship derived by Seed and Idriss (1982) between numbers of significant stress cycles and earthquake magnitude. The MSFs listed in column 5 are similar in value (within about 10%) to the MSFs of Ambraseys (column 4), and the MSFs listed in column 6 are similar in value (within about 10%) to the revised MSFs proposed by Idriss (column 3).

Andrus and Stokoe (1997) Scaling Factors

From their studies of liquefaction resistance as a function of shear wave velocity V_s Andrus and Stokoe (1997) drew bounding curves and developed (22) for calculating CRR from V_s for magnitude 7.5 earthquakes. These investigators drew similar bounding curves for sites where surface effects of liquefaction were or were not observed for earthquakes with magnitudes of 6, 6.5, and 7. The positions of the CRR curves were visually adjusted on each graph until a best-fit bound was obtained. Magnitude scaling factors were then estimated by taking the ratio of CRR for a given magnitude to the CRR for magnitude 7.5 earthquakes. These MSFs are quantified by the following equation:

$$\text{MSF} = (M_w/7.5)^{-2.56} \tag{25}$$

MSFs for magnitudes <6 and >7.5 were extrapolated from this equation. The derived MSFs are listed in column 7 of Table 3, and plotted in Fig. 12. For magnitudes <7.5, the MSFs proposed by Andrus and Stokoe are rather close in value (within about 5%) to the MSFs proposed by Ambraseys. For magnitudes >7.5, the Andrus and Stokoe MSFs are slightly smaller than the revised MSFs proposed by Idriss.

Youd and Noble (1997a) Scaling Factors

Youd and Noble (1997a) used a probabilistic or logistic analysis to analyze case history data from sites where effects of liquefaction were or were not reported following past earthquakes. This analysis yielded the following equation, which was updated after publication of the NCEER proceedings (Youd and Idriss 1997):

$$\mathrm{Logit}(P_L) = \ln(P_L/(1 - P_L)) = -7.0351 + 2.1738M_w - 0.2678(N_1)_{60cs} + 3.0265 \ln \mathrm{CRR} \quad (26)$$

where P_L = probability that liquefaction occurred; $1 - P_L$ = probability that liquefaction did not occur; and $(N_1)_{60cs}$ = corrected equivalent clean-sand blow count. For magnitudes <7.5, Youd and Noble recommended direct application of this equation to calculate the CRR for a given probability of liquefaction. In lieu of direct application, Youd and Noble defined three sets of MSFs for use with the simplified procedure. These MSFs are for probabilities of liquefaction occurrence <20, 32, and 50%, respectively, and are defined by the following equations:

$$\text{Probability } P_L < 20\% \quad \mathrm{MSF} = 10^{3.81}/M^{4.53} \text{ for } M_w < 7 \quad (27)$$

$$\text{Probability } P_L < 32\% \quad \mathrm{MSF} = 10^{3.74}/M^{4.33} \text{ for } M_w < 7 \quad (28)$$

$$\text{Probability } P_L < 50\% \quad \mathrm{MSF} = 10^{4.21}/M^{4.81} \text{ for } M_w < 7.75 \quad (29)$$

New Recommendation by Idriss

I. M. Idriss (TRB 1999) proposed a new set of MSFs that are compatible with, and are only to be used with, the magnitude-dependent r_d that he also proposed. These new MSFs have lower values than the revised MSFs listed in Table 3, but slightly higher values than the original Seed and Idriss (1982) MSFs. Because the proposed r_d and associated MSFs have not been published and the factors have not been independently verified, the workshop participants chose not to recommend the new r_d or MSFs at this time.

Recommendations for Engineering Practice

The workshop participants reviewed the MSFs listed in Table 3, and all but one (S. S. C. Liao) agree that the original factors were too conservative and that increased MSFs are warranted for engineering practice for magnitudes <7.5. Rather than recommending a single set of factors, the workshop participants suggest a range of MSFs from which the engineer is allowed to choose factors that are requisite with the acceptable risk for any given application. For magnitudes <7.5, the lower bound for the recommended range is the new MSF proposed by Idriss [column 3 in Table 3, or (23)]. The suggested upper bound is the MSF proposed by Andrus and Stokoe [column 7 in Table 3, or (26)]. The upper-bound values are consistent with MSFs suggested by Ambraseys (1988), Arango (1996), and Youd and Noble (1997a) for $P_L < 20\%$.

For magnitudes >7.5, the new factors recommended by Idriss [column 3 in Table 3; (25)] should be used for engineering practice. These new factors are smaller than the original Seed and Idriss (1982) factors, hence their application leads to increased calculated liquefaction hazard compared to the original factors. Because there are only a few well-documented liquefaction case histories for earthquakes with magnitudes >8, MSFs in that range are poorly constrained by field data. Thus the workshop participants agreed that the greater conservatism embodied in the revised MSF by Idriss (column 3, Table 3) should be recommended for engineering practice.

CORRECTIONS FOR HIGH OVERBURDEN STRESSES, STATIC SHEAR STRESSES, AND AGE OF DEPOSIT

Correction factors K_σ and K_α were developed by Seed (1983) to extrapolate the simplified procedure to larger overburden pressure and static shear stress conditions than those embodied in the case history data set from which the simplified procedure was derived. As noted previously, the simplified procedure was developed and validated only for level to gently sloping sites (low static shear stress) and depths less than about 15 m (low overburden pressures). Thus applications using K_σ and K_α are beyond routine practice and require specialized expertise. Because these factors were discussed at the workshop and some new information was developed, recommendations from those discussions are included here. These recommendations, however, apply mostly to liquefaction hazard analyses of embankment dams and other large structures. These factors are applied by extending (23) to include K_σ and K_α as follows:

$$\mathrm{FS} = (\mathrm{CRR}_{7.5}/\mathrm{CSR}) \cdot \mathrm{MSF} \cdot K_\sigma \cdot K_\alpha \quad (30)$$

K_σ Correction Factor

Cyclically loaded laboratory test data indicate that liquefaction resistance increases with increasing confining stress. The rate of increase, however, is nonlinear. To account for the nonlinearity between CRR and effective overburden pressure, Seed (1983) introduced the correction factor K_σ to extrapolate the simplified procedure to soil layers with overburden pressures >100 kPa. Cyclically loaded, isotropically consolidated triaxial compression tests on sand specimens were used to measure CRR for high-stress conditions and develop K_σ values. By taking the ratio of CRR for various confining pressures to the CRR determined for approximately 100 kPa (1 atm) Seed (1983) developed the original K_σ correction curve. Other investigators have added data and suggested modifications to better define K_σ for engineering practice. For example, Seed and Harder (1990) developed the clean-sand curve reproduced in Fig. 13. Hynes and Olsen (1999) compiled and analyzed an enlarged data set to provide guidance and formulate equations for selecting K_σ values (Fig. 14). The equation they derived for calculating K_σ is

$$K_\sigma = (\sigma'_{vo}/P_a)^{(f-1)} \quad (31)$$

where σ'_{vo}, effective overburden pressure; and P_a, atmospheric pressure, are measured in the same units; and f is an exponent that is a function of site conditions, including relative density, stress history, aging, and overconsolidation ratio. The workshop participants considered the work of previous investigators and recommend the following values for f (Fig. 15). For relative densities between 40 and 60%, f = 0.7–0.8; for relative densities between 60 and 80%, f = 0.6–0.7. Hynes and Olsen recommended these values as minimal or conservative esti-

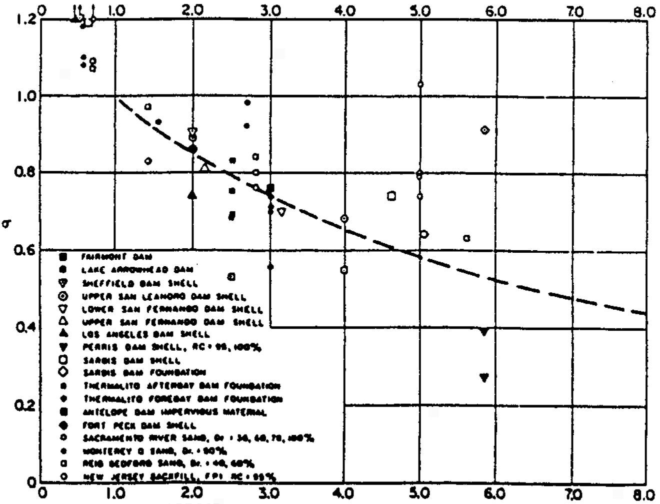


FIG. 13. K_σ-Values Determined by Various Investigators (Reproduced from Seed and Harder 1990)

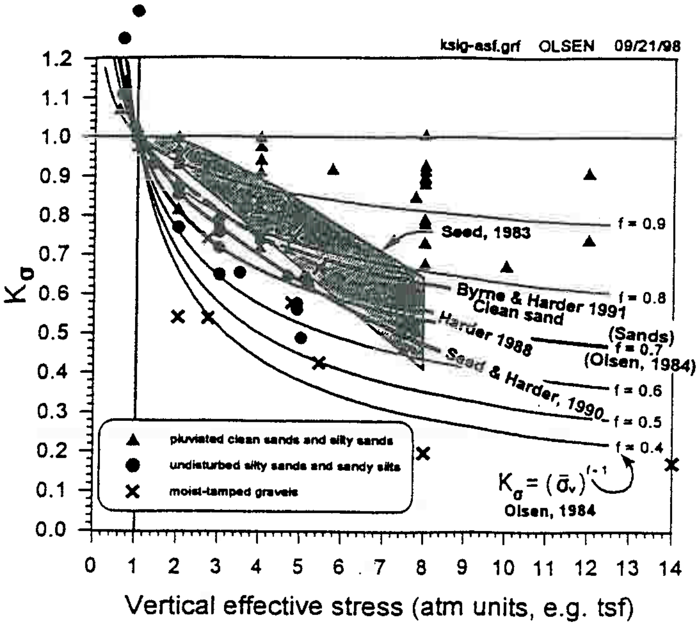


FIG. 14. Laboratory Data and Compiled K_σ Curves (Reproduced from Hynes and Olsen 1999)

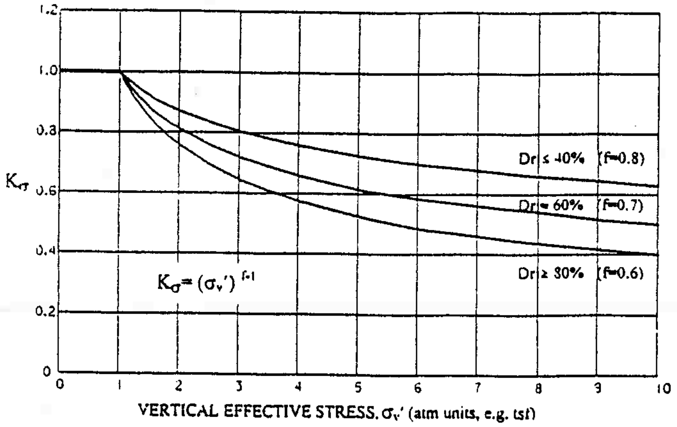


FIG. 15. Recommended Curves for Estimating K_σ for Engineering Practice

mates of K_σ for use in engineering practice for both clean and silty sands, and for gravels. The workshop participants concurred with this recommendation.

K_α Correction Factor for Sloping Ground

The liquefaction resistance of dilative soils (moderately dense to dense granular materials under low confining stress) increases with increased static shear stress. Conversely, the liquefaction resistance of contractive soils (loose soils and moderately dense soils under high confining stress) decreases with increased static shear stresses. To incorporate the effect of static shear stresses on liquefaction resistance, Seed (1983) introduced a correction factor K_α. To generate values for this factor, Seed normalized the static shear stress τ_{st} acting on a plane with respect to the effective vertical stress σ'_{vo} yielding a parameter α, where

$$\alpha = \tau_{st}/\sigma'_{vo} \tag{32}$$

Cyclically loaded triaxial compression tests were then used to empirically determine values of the correction factor K_α as a function of α.

For the NCEER workshop, Harder and Boulanger (1997) reviewed past publications, test results, and analyses of K_α. They noted that a wide range of K_α values have been proposed, indicating a lack of convergence and a need for continued research. The workshop participants agreed with this assessment. Although curves relating K_α to α have been published (Harder and Boulanger 1997), these curves should not be used by nonspecialists in geotechnical earthquake engineering or in routine engineering practice.

Influence of Age of Deposit

Several investigators have noted that liquefaction resistance of soils increases with age. For example, Seed (1979) observed significant increases in liquefaction resistance with aging of reconstituted sand specimens tested in the laboratory. Increases of as much as 25% in cyclic resistance ratio were noted between freshly constituted and 100-day-old specimens. Youd and Hoose (1977) and Youd and Perkins (1978) noted that liquefaction resistance increases markedly with geologic age. Sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and pre-Pleistocene sediments are generally immune to liquefaction. Although qualitative time-dependent increases have been documented as noted above, few quantitative data have been collected. In addition, the factors causing increased liquefaction resistance with age are poorly understood. Consequently, verified correction factors for age have not been developed.

In the absence of quantitative correction factors, engineering judgment is required to estimate the liquefaction resistance of sediments more than a few thousand years old. For deeply buried sediments dated as more than a few thousand years old, some knowledgeable engineers have omitted application of the K_σ factor as partial compensation for the unquantified, but substantial increase of liquefaction resistance with age. For man-made structures, such as thick fills and embankment dams, aging effects are minimal, and corrections for age should not be applied in calculating liquefaction resistance.

SEISMIC FACTORS

Application of the simplified procedure for evaluating liquefaction resistance requires estimates of two ground motion parameters—earthquake magnitude and peak horizontal ground acceleration. These factors characterize duration and intensity of ground shaking, respectively. The workshop addressed the following questions with respect to selection of magnitude and peak acceleration values for liquefaction resistance analyses.

Earthquake Magnitude

Records from recent earthquakes, such as 1979 Imperial Valley, 1988 Armenia, 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe, indicate that the relationship between duration and magnitude is rather uncertain and that factors other than magnitude also influence duration. For example, unilateral faulting, in which rupture begins at one end of the fault and propagates to the other, usually produces longer shaking duration for a given magnitude than bilateral faulting, in which slip begins near the midpoint on the fault and propagates in both directions simultaneously. Duration also generally increases with distance from the seismic energy source and may vary with tectonic province, site conditions, and bedrock topography (basin effects).

Question: Should correction factors be developed to adjust duration of shaking to account for the influence of earthquake source mechanism, fault rupture mode, distance from the energy source, basin effects, etc.?

Answer: Faulting characteristics and variations in shaking duration are difficult to predict in advance of an earthquake event. The influence of distance generally is of secondary importance within the range of distances to which damaging liquefaction effects commonly develop. Basin effects are not yet sufficiently predictable to be adequately accounted for in engineering practice. Thus the workshop participants recommend continued use of the generally conservative relationship between magnitude and duration that is embodied in the simplified procedure.

Question: An important difference between eastern U.S. earthquakes and western U.S. earthquakes is that eastern ground motions are generally richer in high-frequency energy and thus could generate more significant stress cycles and equivalently longer durations than western earthquakes of the same magnitude. Is a correction needed to account for higher frequencies of motions generated by eastern U.S. earthquakes?

Answer: The high-frequency motions of eastern earthquakes are generally limited to near-field rock sites. High-frequency motions attenuate or are damped out rather quickly as they propagate through soil layers. This filtering action reduces the high-frequency energy at soil sites and thus reduces differences in numbers of significant loading cycles. Because liquefaction occurs only within soil strata, duration differences on soil sites between eastern and western earthquakes are not likely to be great. Without more instrumentally recorded data from which differences in ground motion characteristics can be quantified, there is little basis for the development of additional correction factors for eastern localities.

Another difference between eastern and western U.S. earthquakes is that strong ground motions generally propagate to greater distances in the east than in the west. By applying present state-of-the-art procedures for estimating peak ground acceleration at eastern sites, differences in amplitudes of ground motions between western and eastern earthquakes are properly taken into account.

Question: Which magnitude scale should be used for selection of earthquake magnitudes for liquefaction resistance analyses?

Answer: Seismologists commonly calculate earthquake magnitudes using five different scales: (1) local or Richter magnitude M_L; (2) surface-wave magnitude M_s; (3) short-period body-wave magnitude m_b; (4) long-period body-wave magnitude m_B; and (5) moment magnitude M_w. Moment magnitude, the scale most commonly used for engineering applications, is the scale preferred for calculation of liquefaction resistance. As Fig. 16 shows, magnitudes from other scales may be substituted directly for M_w within the following limitations—$M_L < 6$, $m_B < 7.5$, and $6 < M_s < 8$—m_b, a scale commonly used for eastern U.S. earthquakes, may be used for magnitudes between 5 and 6, provided m_b values are corrected to equivalent M_w values. The curves plotted in Fig. 16 may be used for this adjustment (Idriss 1985).

Peak Acceleration

In the simplified procedure, peak horizontal acceleration a_{max} is used to characterize the intensity of ground shaking. To provide guidance for estimation of a_{max}, the workshop addressed the following questions.

Question: What procedures are preferred for estimating a_{max} at potentially liquefiable sites?

Answer: The following methods, in order of preference, may be used for estimating a_{max}:

1) The preferred method for estimating a_{max} is through empirical correlations of a_{max} with earthquake magnitude, distance from the seismic energy source, and local site conditions. Several correlations have been published for estimating a_{max} for sites on bedrock or stiff to moderately stiff soils. Preliminary attenuation relationships have also been developed for a limited range of soft soil sites (Idriss 1991). Selection of an attenuation relationship should be based on such factors as region of the country, type of faulting, and site condition.

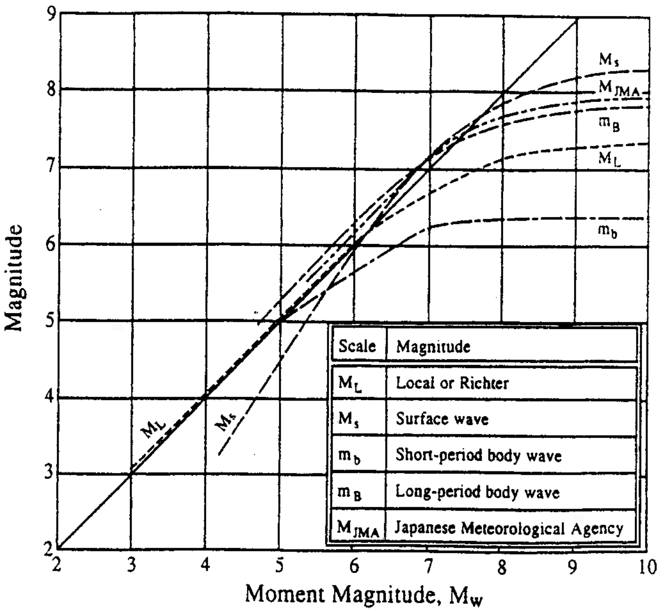


FIG. 16. Relationship between Moment M_w and Other Magnitude Scales (Reproduced from Heaton et al., Unpublished Report, 1982)

2) For soft sites and other soil profiles that are not compatible with available attenuation relationships, a_{max} may be estimated from local site response analyses. Computer programs such as SHAKE and DESRA may be used for these calculations (Schnabel et al. 1972; Finn et al. 1977). Input ground motions in the form of recorded accelerograms are preferable to synthetic records. Accelerograms derived from white noise should be avoided. A suite of plausible earthquake records should be used in the analysis, including as many as feasible from earthquakes with similar magnitudes, source distances, etc.

3) The third and least desirable method for estimating peak ground acceleration is through amplification ratios, such as those developed by Idriss (1990, 1991) and Seed et al. (1994). These factors use a multiplier or ratio by which bedrock outcrop motions are amplified to estimate surface motions at soil sites. Because amplification ratios are influenced by strain level, earthquake magnitude, and frequency content, caution and considerable engineering judgment are required in the application of these relationships.

Question: Which peak acceleration should be used: (1) the largest horizontal acceleration recorded on a three-component accelerogram; (2) the geometric mean (square root of the product) of the two maximum horizontal components; or (3) a vectorial combination of horizontal accelerations?

Answer: According to I. M. Idriss (oral discussion at NCEER workshop, 1996), where recorded motions were available, the larger of the two horizontal peak components of acceleration was used in the compilation of data used to derive the original simplified procedure. Where recorded values were not available, which was the circumstance for most sites, peak acceleration values were estimated from attenuation relationships based on the geometric mean of the two orthogonal peak horizontal accelerations. In nearly all instances where recorded motions were used, the peaks from the two horizontal records were approximately equal. Thus where a single peak was used, the peak and the geometric mean of the two peaks were about the same value. Based on this information, the workshop participants concurred that use of the geometric mean is consistent with the development of the procedure and is preferred for use in engineering practice. However, use of the larger of the two orthogonal peak accelerations yields a larger estimate of a_{max}, is conservative, and is allowable. Vectorial accelerations are seldom calculated and should not be used. Peak vertical accelerations are generally much smaller than peak horizontal accelerations and are ignored for calculation of liquefaction resistance.

Question: Liquefaction usually develops at soil sites where ground motion amplification may occur and where sediment may soften, reducing motions as excess pore pressure develop. How should investigators account for these factors in estimating peak acceleration?

Answer: The recommended procedure is to calculate or estimate the a_{max} that would occur at the site in the absence of increased pore pressure or the onset of liquefaction. That peak acceleration incorporates the influence of site amplification, but neglects the influence of excess pore-water pressure.

Question: Should high-frequency spikes (periods <0.1 s) in acceleration records be considered or ignored?

Answer: In general, short-duration, high-frequency acceleration spikes are too short in duration to generate significant instability or deformation of granular structures, and should be ignored. By using attenuation relationships for estimation of peak acceleration, as noted above, high-frequency spikes are essentially ignored because few high-frequency peaks are incorporated in databases from which attenuation the relationships were derived. Similarly, ground response analyses programs such as SHAKE and DESRA generally attenuate or filter out high-frequency spikes, reducing their influence. Where amplification ratios are used, engineering judgment should be used to determine which bedrock acceleration is to be amplified.

ENERGY-BASED CRITERIA AND PROBABILISTIC ANALYSES

The workshop considered two additional topics: (1) liquefaction resistance criteria based on seismic energy passing through a liquefiable layer (Kayen and Mitchell 1997; Youd et al. 1997), and probabilistic analyses of case history data (Liao et al. 1988; Youd and Noble 1997b). Although probabilistic or risk analyses have been made for some localities and critical facilities, the workshop participants concluded that probabilistic procedures are still under development and not sufficiently formulated for routine engineering practice. Similarly, new energy-based criteria need to be independently tested before recommendations can be made for general practice. The workshop participants recommend that research and development continue on both of these relatively new and potentially useful procedures.

CONCLUSIONS

The participants in the NCEER workshop reviewed the state-of-the-art for evaluating liquefaction resistance and recommend several augmentations to that procedure. Specific recommendations, including procedures and equations, are listed in each section of this summary paper. Consensus conclusions from the workshop are:

1. Four field tests are recommended for routine evaluation of liquefaction resistance—the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity (V_s) measurements, and for gravelly sites the Becker penetration test (BPT). Criteria for each test were reviewed and revised to incorporate recent developments and to achieve consistency between resistances calculated from the various tests. Each test has its advantages and limitations (Table 1). the CPT provides the most detailed soil stratigraphy and robust field-data based liquefaction resistance curves now available. CPT testing should always be accompanied by soil sampling for validation of soil type identification. The SPT has a longer record of application and provides disturbed soil samples from which fines content and other grain characteristics can be determined. Measured shear-wave velocities provide fundamental information on small-strain soil behavior that is useful beyond analyses of liquefaction resistance. V_s is also applicable at sites, such as landfills and gravelly sediments, where CPT and SPT soundings may not be possible or reliable. The BPT test is recommended only for gravelly sites and requires use of rough correlations between BPT and SPT, making the results less certain than other tests. Where possible, two or more test procedures should be applied to assure adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance.
2. The magnitude scaling factors originally derived by Seed and Idriss (1982) are overly conservative for earthquakes with magnitudes <7.5. A range of scaling factors is recommended for engineering practice, the lower end of the range being the new MSF recommended by Idriss (column 3, Table 3), and the upper end of the range being the MSF suggested by Andrus and Stokoe (column 7, Table 3). These MSFs are defined by (25) and (26), respectively. For magnitudes >7.5, the new factors by Idriss (column 3, Table 3) should be used. These factors, which are more conservative than the original Seed and Idriss (1982) factors, should be applied.
3. The K_σ factors suggested by Seed and Harder (1990) appear to be overly conservative for some soils and field conditions. The workshop participants recommend K_σ values defined by the curves in Fig. 14 or (31). Because K_σ values are usually applied to depths greater than those verified for the simplified procedure, special expertise is generally required for their application.
4. Procedures for evaluation of liquefaction resistance beneath sloping ground or embankments (slopes greater than about 6%) have not been developed to a level allowable for routine use. Special expertise is required for evaluation of liquefaction resistance beneath sloping ground.
5. Moment magnitude M_w should be used for liquefaction resistance calculations. Magnitude, as used in the simplified procedure, is a measure of the duration of strong ground shaking. The present magnitude criteria are conservative and should not be corrected for source mechanism, style of faulting, distance from the energy source, subsurface bedrock topography (basin effect), or tectonic region (eastern versus western U.S. earthquakes).
6. The peak acceleration a_{max} applied in the procedure is the peak horizontal acceleration that would occur at ground surface in the absence of pore pressure increases or liquefaction. Attenuation relationships compatible with soil conditions at a site should be applied in estimating a_{max}. Relationships based on the geometric mean of the peak horizontal accelerations are preferred, but use of relationships based on peak horizontal acceleration is allowable and conservative. Where site conditions are incompatible with existing attenuation relationships, site-specific response calculations, using programs such as SHAKE or DESRA, should be used. The least preferable technique is application of amplification factors.

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NOTATION

The following symbols are used in this paper:

a, b = curve fitting parameters for use with V_s criteria for evaluating liquefaction resistance;
$a_{\max}$ = peak horizontal acceleration at ground surface;
C_B = correction factor for borehole diameter;
C_E = correction factor for hammer energy;
C_N = correction factor for overburden pressure applied to SPT;
C_Q = correction factor for overburden pressure applied to CPT;
C_R = correction factor for drilling rod length;
C_S = correction factor for split spoon sampler without liners;
$\mathrm{CRR}_{7.5}$ = cyclic resistance ratio for $M_w = 7.5$ earthquakes;
d_c = diameter of CPT tip;
F = normalized friction ratio;
f = exponent estimated from site conditions used in calculation of K_σ;
f_s = sleeve friction measured with CPT;
g = acceleration of gravity;
H = thickness of thin granular layer between softer sediment layers;
I_c = soil behavior type index for use with CPT liquefaction criteria;
K_c = correction factor for grain characteristics applied to CPT;
K_H = thin-layer correction factor for use with CPT;
K_α = correction factor for soil layers subjected to large static shear stresses;
K_σ = correction factor for soil layers subjected to large static normal stresses;
M_L = local or Richter magnitude of earthquake;
M_s = surface-wave magnitude of earthquake;
M_w = moment magnitude of earthquake;
m_B = long period body-wave magnitude of earthquake;
m_b = short period body-wave magnitude of earthquake;
N_m = measured standard penetration resistance;
$(N_1)_{60}$ = corrected standard penetration resistance;
$(N_1)_{60cs}$ = $(N_1)_{60}$ adjusted to equivalent clean-sand value;
n = exponent used in normalizing CPT resistance for overburden stress;
P_a = atmospheric pressure, approximately 100 kPa;
P_L = probability of liquefaction;
Q = normalized and dimensionless cone penetration resistance;
q_{c1N} = normalized cone penetration resistance;
$(q_{c1N})_{cs}$ = normalized cone penetration resistance adjusted to equivalent clean-sand value;
r_d = stress reduction coefficient to account for flexibility in soil profile;
V_s = measured shear-wave velocity;
V_{s1} = overburden-stress corrected shear-wave velocity;
V_{s1}^* = limiting upper value of V_{s1} for liquefaction occurrences;
z = depth below ground surface (m);
α, β = coefficients, that are functions of fines content, used to correct $(N_1)_{60}$ to $(N_1)_{60cs}$;
σ'_{vo} = effective overburden pressure;
τ_{av} = average horizontal shear stress acting on soil layer during shaking generated by given earthquake; and
τ_{st} = static shear stress acting on soil element due to gravitational forces.