GEOTECHNICAL CHARACTERISTICS OF SEDIMENTS ON THE WESTERN CENTRAL SCOTIA SLOPE

Kevin MacKillop, GSC (Atlantic), Nova Scotia, Canada
Kate Jarrett, GSC (Atlantic), Nova Scotia, Canada
Perry Mitchelmore, Mitchelmore Engineering Ltd., Nova Scotia, Canada
Dave Mosher, GSC (Atlantic), Nova Scotia, Canada
Calvin Campbell, GSC (Atlantic), Nova Scotia, Canada

ABSTRACT
Recent oil and gas exploration in Eastern Canada has included deep water regions of the Scotia Slope. The Geological Survey of Canada in collaboration with non-government agencies, has undertaken a program to understand deep water margin geologic processes and potential geohazards. Short (<15 m) piston cores have been analysed to assess the geology, geotechnicial properties, stress history, and static failure potential. These geotechnical data indicate that the sediment is overconsolidated near the surface to normally to slightly underconsolidated with depth. The stability of the surficial sediments was evaluated using the infinite slope method of analysis. Laboratory shear vane strength data and bulk density measurements were compiled to determine minimum safety factor (FS) and critical slope angles (FS=1). This slope stability analysis shows that surficial sediments are stable (FS>1) under drained and undrained static conditions. Earthquake loading was evaluated by calculating the earthquake-induced horizontal acceleration required to cause failure. The horizontal acceleration required to cause failure varies from 0.013g to 0.046g.

RÉSUMÉ
Quand des sols contaminés sont chauffés, la pression de vapeur des composés chimiques organiques augmente. L'exploration recente du petrole et du gaz en l'est du Canada avait comprendre les regions profonde sous-marine de la Scotian Slope. La Commission Geologique du Canada dans la collaboration avec les agencies non-gouvernementale, avait commencer une programme pour la comprehension des processus geologique de la marge profonde sous-marine et les risques geologiques potential. Les noyaus de piston court (<10m) etaient analyser pour l'assessment de la geologie, les caracteristiques geotechnique, l'histoire de la pression, et potentialle pour la defaillance statique. Ces donnees geotechnique indiquent que le sediment est trop consolider proche de la surface est c'est un peu moins consolider avec la profondeur. La stabilite des sediments proche a la surface etait evaluer avec la methode de la pente infinie analyse. Les donnees de la force transversale et les mesures de la densite en masse etaient evaluer en laboratoire et compiler pour determiner le facteur de securite minimum (FS) et les angles de la pente critique (FS = 1). Cette analyse de la stabilite du versant demontre que les sediments proche de la surface sont stable (FS >1) sous les conditions statique s'ecouler et non s'ecouler. Le chargement en resultant des tremblements de terre etait evaluer par le calcul de l'acceleration necessaire pour causer la defaillance de sediment. L'acceleration horizontale necessaire pour causer la defaillance de'sediment varier de 0.013g a 0.046g..

1. INTRODUCTION

Sediment mass movements resulting from diverse triggering mechanisms have long been known to occur on continential margins with different slopes. The ability to determine potential failures and hazard mapping is becoming more significiant with the development of deep water oil and gas resources and the demand for submarine cable installations. The Scotia Slope (Figure 1) provides many examples of large slumps and debris flows. A comprehensive geohazard assessment requires geotechnicial characeterization of surficial sediments together with identification of features or conditions that represent past and present geologic hazards. This study uses available geotechnicial data including index properties, shear strength, Atterberg limits and consolidation data to characterize the sediments and analyse slope stability in the western central Scotia Slope. The engineering analysis was limited to slope stability using the infinite slope method to determine factor of safety, critical slope inclination, critical sediment thickness and critical earthquake acceleration coefficient using a pseudostatic analysis.

The study area is located on the western central Scotia Slope (43^{o}00' to 42^{o}18' N latitude, 62^{o}30 to 60^{o} 30' W longitude; Figure 2) in water depths from 500 to 2500 metres below sea level (mbsl). The area is bounded on the west by Mohican Channel and on the east by the West Acadia Valley. The shelf break lies between 400-500 mbsl and the transition from slope to continental rise is between 2000-2500 mbsl. The morphology of the central Scotia Slope is smooth relative to regions to the east and west. The seabed, however, is cut by shallow downslope-trending gullies that begin at 500 mbsl. Seafloor slope angles range from 5^{o} on the upper slope to 1.5^{o} on the rise. On the lower slope head scarpes representing slide failure scars are visible (Figure2) that range in height from 10 to 80 m. and are continious for up to 20 km.

Slope instabilities occur just before the shelf break in depths of 400 to 500 m and extend onto the rise in water depths of 2000 m. These flows cover 600 km^2 and extend for 30 km from the source area.

2. GEOLOGIC SETTING

The Scotian Shelf is part of the Mesozoic rifted margin of the central North Atlantic Ocean, with thick Jurassic and Cretaceous strata overlying Triassic salt (Wade and Maclean, 1990). The upper few hundred metres of the sedimentary sequence beneath the Scotian Slope consists of a thick progradational Tertiary succession, overlain both conformably and unconformably by Quaternary sediments (Swift, 1985). There was rapid sedimentation on the Laurentian Fan and Scotian Slope with the onset of terrestrial glaciation in the early

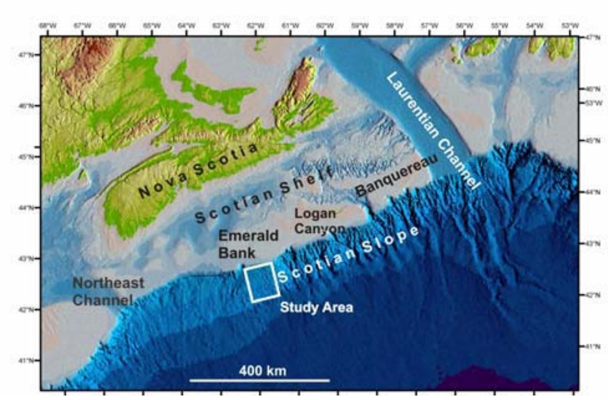

Figure 1 Location of study area on the Scotia Slope

Pleistocene. Widespread gully cutting took place in the early Pleistocene, but the overall style of sedimentation continued to be prodeltaic (Piper and Normark 1989; Newton et al.,2003). The first shelf-crossing glaciation event occurred about 0.5 Ma and since that time, the continental slope has been dominated by proglacial sediment deposition, with little sediment accumulation at high-stands. Ice sheets extended close to the shelf edge at the last glacial maximum (18 ka) and had retreated to the present shoreline by about 12 ka (Stea et al., 1998). Since that time, and with subsequent sea level rise, continental slope sedimentation has been slow, dominated by pelagic and hemipelagic deposition (Mosher et al., 1994). The relative sea level through geological history will have varied from the present level. The Holocene (past 10,000 years) is believed to be a period of transgression and relatively light, hemipelagic deposition. Regression periods during the late and middle Pleistocene, where relative sea levels may have been 110 to 120 m below present, are believed to be periods of more active deposition.

Near-surface sediments in the study area are largely glacial marine turbidites, deposited during glacial/deglacial epochs, interbedded with hemipelagic deposits formed during interglacials. Reworking of this material as mass transport deposits is common. The causes of mass transport are not well understood but earthquakes are thought to be a likely factor (Mosher et al., 1994). In this slope environment, ground-shaking can remobilize sediment and cause massive landslides as it did in the 1929 Grand Banks earthquake event (Piper and Normark, 1982; Piper et al. 1988, 1999a). There is little evidence of sediment failures in the last 10 ka and slope stability analysis has shown the surface sediment to be statically stable, except on steep escarpments and canyon walls. There is, however, abundant evidence of sediment failures that approximately correlate to glacial advances (25-12 ka, ~ 75 ka, ~130 ka) suggesting a potential loading situation as a likely cause (Mosher et al., 1994).

3. METHODS

3.1 Sediment Coring and Physical Properties

There have been a number of expeditions on the Scotian Shelf and Slope to collect geotechnical data and recover samples for laboratory analysis. Synthesis of the data is ongoing for many of the expeditions. The scope of work for this paper pertains to data from 12 cores collected during 2000 (cruise 2000036) on the Scotia Slope in water depths from 532 to 2490 mbsl (Figure 2). Cores were collected from the seabed using the Long Corer Facility (LCF). The device collects a 11 cm-diameter, 7-15 m-long, relatively undisturbed sediment cores. The seabed inclination for the core locations varied from 1.01 degrees to 10.71 degrees. The seabed inclination at each core location was determined from multibeam data (Mosher et al., 2004).

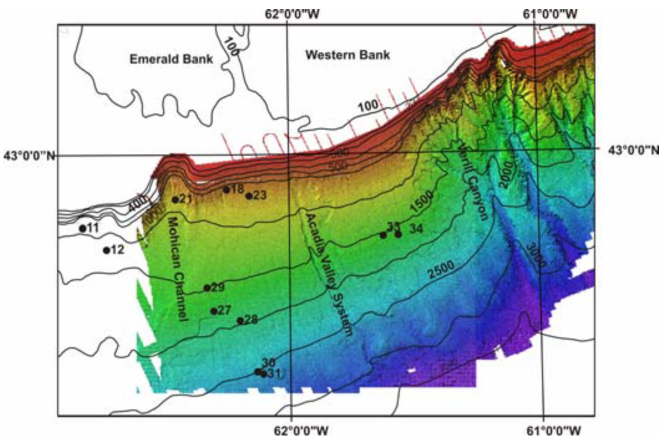

Figure 2. Core location of cores analysed for this study.

Core processing was done on both piston and trigger weight cores at the Bedford Institute of Oceanography core-processing laboratory. The core is initially processed in a GeoTEK multi sensor track logger (MST). The MST measures gamma ray attenuation bulk density, magnetic susceptibility and transverse compressional wave velocity at 1 cm intervals. The core is then split with one half designated as the archive the other as a working half. The archive half is digitally photographed and visually described. Discrete index property samples, shear strength and full waveform transverse and longitudinal acoustic velocity measurements are taken at 10 cm intervals on the working half.

Piston cores from the continental slope have a distinctive Holocene and late Pleistocene sedimentary sequence (Figure 3). Holocene olive-gray muds pass down into early Holocene–late Pleistocene silty muds and thin sand beds that are believed to reflect lower sea level at that time on the outer shelf. Distinctive ice rafted marker horizons date from 12 and 14 ka. Older sediment is proglacial mud with dropstones and related fine-grained turbidites on the continental rise. Distinctive color changes probably related to changing glacial sources are common in these sequences.

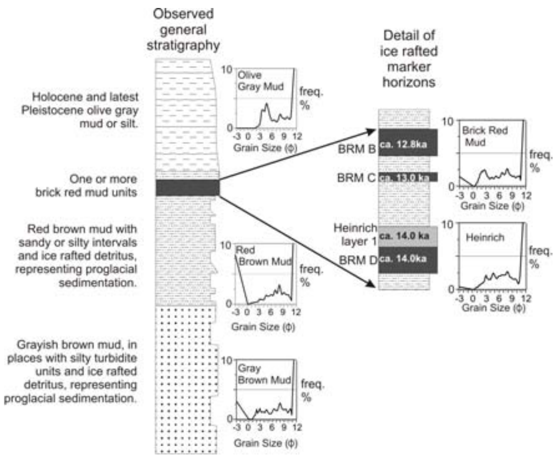

Figure 3. Type lithostratigraphy derived from a composite of shallow piston cores, with representative grain-size-distribution (after Mosher et al. 2004).

3.2 Consolidation testing

Consolidation testing was conducted on four samples from cores 036_11, 036_28 and 036_29. The tests were performed in a back-pressured consolidometer at the GSC(Atlantic) geomechanical lab. The application of back pressure is to ensure 100% saturation. The samples were allowed to adjust to the back pressure for a minimum of twelve hours before incremental loading was started. A load increment ratio of 1 was used for the first two tests and a ratio of 0.5 was used for the last two tests.

3.3 Slope Stability

A successful engineering model for slope stability characterizes the driving forces and the available shear strength. Available shear strength is assessed based on geotechnical characterization of soils, their stress history and the type of loading estimated. The driving forces or loads can be either static or dynamic. Figure 1 illustrates the forces involved in slope stability assessment.

The effects of oversteepening are addressed by altering the seabed slope angle to determine the *critical slope* (β_c) for a given vertical soil profile. Seismic loading is considered using a pseudostatic analysis to develop a critical coefficient due to seismic acceleration (K_h) where the dynamic force caused by the earthquake is represented by an equivalent static force (E) in the horizontal direction. The effects of rapid accumulation and underconsolidation are dealt with by calculating the critical thickness (H_c) for the given vertical soil profile either for static loads or dynamic loads.

This assessment of slope stability uses the infinite slope method of analysis (Figure 3) assuming undrained loading conditions. It is used where the failure surface, or assumed slip surface, is planar and parallel to the ground surface over a significant length, and the depth is small in comparison to the length (Skempton and Delroy, 1957). The infinite slope method will generally give a very conservative assessment of a site and is limited in its application, but may be used for slopes in cohesionless soil and for relatively flat slopes in which stability is governed by the residual shear strength (Skempton and Delroy, 1957). Others have used the infinite slope method to develop relationships for cohesive slopes (e.g. Lambe and Whitman, 1979).

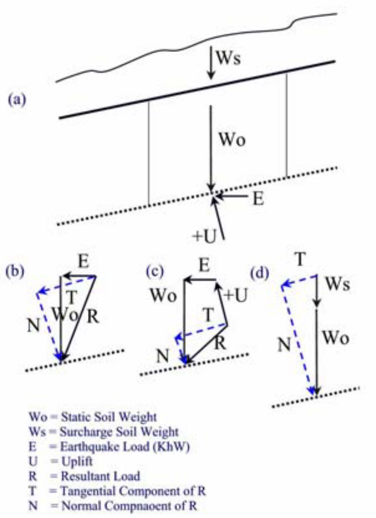

Figure 4. Stress condition in the Infinite slope model. (a) Diagram of slope. Force diagrams (a) for earthquake. (c) Earthquake with uplift, and (d) surcharge load.

The infinite slope method uses force equilibrium theory to evaluate both the resisting and driving forces on an assumed sliding surface. For simplicity of analysis, the end and side restraining conditions of the sliding mass are ignored. The Factor of Safety (FS) for a potential failure plane is defined as

$$FS = \frac{\text{Available Shear strength}}{\text{Driving Force}} \quad [1]$$

A location with a FS value less than one is considered to be unstable. The FS may be determined using either a Total Stress Stability Analysis (TSSA) or an Effective Stress Stability Analysis (ESSA). For TSSA, the available shear strength is equal to the undrained shear strength mobilized when failure occurs before any significant dissipation of shearing induced porewater pressures takes place. For ESSA, the available shear strength is the drained shear strength mobilized at relatively large strains and stresses. For present purposes, a TSSA was performed assuming a completely undrained insitu condition. The FS is determined by

$$FS = \frac{S_u}{\gamma_B H \sin\beta \cos\beta} \quad [2]$$

Where S_u is the undrained shear strength of the soil, measured using laboratory shear vane, β is the slope inclination and γ_B is the buoyant unit weight of the soil above the potential failure plane and H is the vertical distance from the potential failure plane to the surface.

For static loading conditions, the measured value of shear strength was used for the resisting force. For cyclic loading only 80% of the measured shear strength was used for the resisting force. It has been shown that soils subjected to cyclic loads may exhibit progressive accumulation of strain accompanied by softening and subsequent loss of strength. A value of 80% strength loss appears to represent 10 cycles of loading/unloading and was recommended for clayey soils (Madeski and Seed, 1978).

The critical slope inclination is determined as the angle where the factor of safety is unity. To obtain the critical slope inclination, eq. (2) is solved for β_c. The critical slope angle is determined by

$$\beta_c = \left[\frac{1}{2}\left\{\sin^{-1}\left(\frac{2S_u}{\gamma_b H}\right)\right\}\right] \quad [3]$$

The critical thickness is the thickness where the factor of safety is unity. To obtain the critical slope thickness, eq. (2) is solved for H_c, the critical slope thickness. The resulting equation is

$$H_c = \frac{2S_u}{\gamma_b \sin(2\beta)} \quad [4]$$

The critical earthquake acceleration coefficient K_h, is the acceleration coefficient where the factor of safety is unity. To obtain the critical earthquake acceleration coefficient K_h, eq. (2) is solved for K_h, the critical acceleration coefficient. The resulting equation is

$$K_h = \text{Tan}\beta(0.8FS - 1) \quad [5]$$

From equation 5 it is obvious that for a slope with a static FS less than 1.25 will result in an unstable slope under earthquake loads (i.e., FS_E <1).

The conservatism of the assumptions in the infinite slope method was checked using the slope stability package **XStabl™** and block surfaces. The principal difference between the two methods is block stability factors at the end conditions which allows for a bottom failure surface that may not be parallel to the slope.

A wedge analysis was performed in XSTABL™ using ranking wedges at the upslope and downslope ends of the central wedge. The wedge analysis assumes that three blocks contribute to the driving force, an upstream passive wedge, a central wedge and a downstream wedge. The wedges are generated based on the assumption that the limiting surface is a function of the angle of internal friction for the soil as defined by the equation

$$45 \pm \frac{\Phi}{2} \quad [6]$$

For an undrained case, and the angle on the upstream and downstream is 45 degrees. The central segment length was arbitrarily selected as 300 metres, or 30 times the depth. The Xstabl software allows the user to specify search blocks for the end positions of the critical surface. For this analysis, the middle depth was chosen with a search box 50 m long and 8 m high at both ends of the section. Two hundred (200) trial surfaces were generated. The wedge analysis was performed on core 2000_036_027 only.

4. RESULTS

4.1 Sedimentology and index properties

A geotechnical model is proposed consisting of five geotechnical stratigraphic units. The subsurface lithostratigraphy facies for the cores was adopted from Mosher et al., (1994, 2004) and Gauley (2001). Unit 1 is hemiphelagic Holocene soils. Unit 2 corresponds to the Heinrich layers that were deposited between 10 ka and 14 ka. Units 3 and 4 are late Pleistocene soils deposited by ice margin processes during glacials and hemipelagically during inter-glacials (Mosher et al., 1994). Unit 5 consists of undifferentiated mass transport deposit. It shoulld be noted that there is limited data for units 2 and 5. The physical property data for the lithostragraphic units are summarized in Table 1.

4.1.1 Unit 1 – olive grey clay (CL to CH)

The olive grey clay layer of Unit 1 forms a drape like layer up to 2 m (Figure 4) thick over much of the Scotia Slope. The clay is classified as a lean (CL) to fat (CH) clay and consists of 13.2 % sand 59.9 % silt and 28.9 % clay, on average. The Su/Po’ averages 1.76 which is well above Skempton’s (1970) range of 0.2 to 0.5 for normally consolidated soil. This apparent overconsolidation is typicial of the upper 2 m of marine sediments on the Scotian Slope.

4.1.2 Unit 2 – brick red mud (CL)

The mud is described by others as silty with abundant fine and coarse sand, consisting of thin horizontal layers, and interbedded with brown to dark greyish brown clay. It is interpreted as being deposited during the last glaciation (from 12ka to 14 ka) by rapid iceberg discharge from the Gulf of St. Lawrence (Piper and Skene, 1998).

One Atterberg limits test performed from core 036_029 indicates the deposit is a lean clay (CL). The S_u/P_o’ (2 measurements) calculated using laboratory shear vane and MST density data averages 0.22 for unit 2 sediments occuring below 2 mbsf. The measured increase in shear strength with depth is 1.39 kPa per metre.

4.1.3 Unit 3 - red brown mud (CL)

Units 3 is a red brown clay with ice rafted detritus, including sand. It was deposited by ice margin processes during glacials (Mosher et al., 1994). This unit is classified as a lean clay (CL) to lean clay with sand. The grain size varies from 2 to 5.0 % gravel 10 to 21 % sand 31 to 34 % silt and 45.0 to 59.0 % clay. The S_u/P_o’ calculated using laboratory shear vane and MST density data averages 0.36 for unit 3 sediments occuring below 2 mbsf. The measured increase in shear strength with depth is 2.18 kPa per m.

4.1.4 Unit 4 – grey brown mud (CL)

Unit 4 grey brown mud is either massive or, more commonly, contains distinctive interlaminated and interbedded very fine to fine sand and grey brown mud up to 1 m thick. Grey brown mud beds occasionally contain sandy blebs and random granules and is interlayered with Unit 3 (Figure 4). This unit is classified as a lean clay (CL) to lean clay with sand. The grain size varies from 0.4 to 5.0 % gravel 8.0 to 16.8 % sand 31.0 to 35.5 % silt and 44.0 to 59.0 % clay. The S_u/P_o’ calculated using laboratory shear vane and MST density data averages 0.46 for unit 4 sediments occuring below 2 mbsf. The increase in shear strength with depth is is 3.76 kPa per m.

4.1.5 Mass transport deposits (CL)

Failure deposits were recovered in cores 036_023 and 036_029. One Atterberg limits test was conducted on sample from core 036_029 and was classified as a lean clay (CL). The S_u/P_o’ calculated using laboratory shear vane and MST density data averages 0.40 for unit 5 sediments occuring below 2 mbsf. The increase in shear strength with depth is 2.51 kPa per m.

Table 1. Physical properties for the geotechnical units. Density data are MST and discrete values.

Unit	Density (g/cm^3)	Void Ratio	Liquid Limit	Plasticity Index	Liquidity Index
1	1.61	0.96	47.43	23.71	0.98
2	1.89	NA	36.00	21.00	0.92
3	1.72	1.16	38.25	19.00	0.91
4	1.67	1.28	47.43	25.43	1.28
5	1.83	.0.99	37.5	18.0	1.00

4.2 Consolidation testing

Consoliation results are presented in Table 2. The tests were conducted on samples from Units 1 (036_011) and 4 (036_027, 029). The compression indices range from 0.25 to 0.56 (low to moderate compressibility). The high OCR value of 5.5 is from Unit 1 and is within the zone of apparent overconsolidation for marine clays. The results for the two consolidation tests from 036_029 suggest that the sediment is under to normally consolidated. The geotechnicial profile (Figure 4) from core 036_029 indicates that sediment below the MTD are under consolidated as there is a decreasing density profile and natural water contents are greater than the liquid limits.

Table 2. Consolidation test results.

Core No	Depth (cm)	Cc	Cr	Bulk Density (g/cm^3)	OCR
_011	162	0.34	0.041	1.92	5.5
_027	310	0.25	0.035	1.82	1.6
_029	575	0.53	0.047	1.68	0.7
_029	800	0.56	0.074	1.67	1.0

4.3 Slope Stability Analysis

The minimum FS for each core was determined by calculating the factor of safety for each shear strength measurement ($\approx$ 10 cm intervals). The minimum factor of safety was then selected as the actual factor of safety for slope stability at that core location. The effective buoyant weight was calculated from MST bulk density values. The results are presented in Table 3. for the twelve cores comprising the sample set for this study. Five of the cores had a factor of safety less than unity with a minimum of 0.44. A summary of the results are presented in Table 1.

A result suggesting that 5 of the 12 core locations is unstable under static loading conditions is considered very conservative.

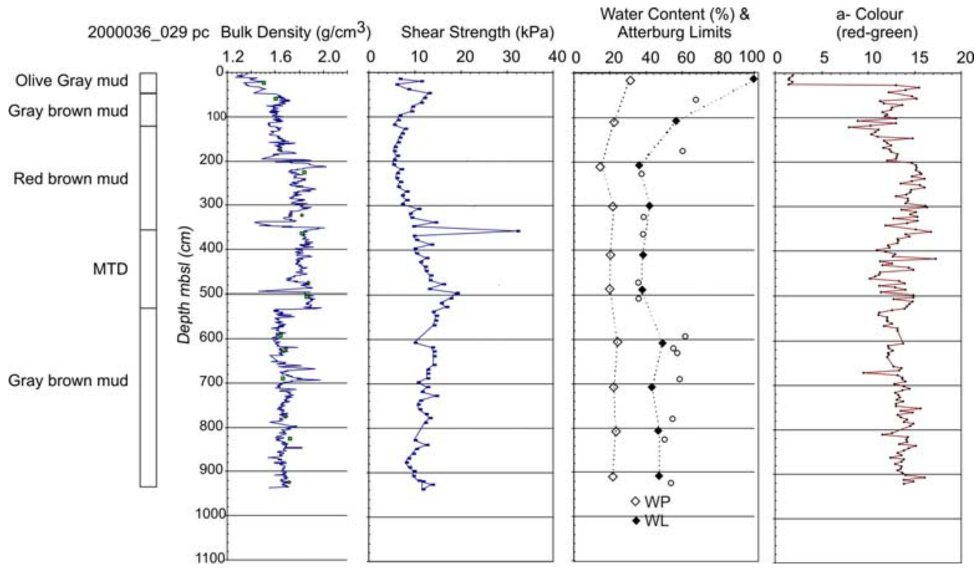

Figure 4. Geotechnicial profile of 2000_036_029PC.

Table 3. Summary of results for FS infinite slope analysis.

	Factor of Safety	Critical slope (deg)	Critical Earthquake Coefficient	Critical Thickness (m)
Minimum	0.44	0.67		1.83
Maximum	7.67	18.71	91.11	64.47
Average	2.17	6.5	9.52	6.94
Median	1.35	5.76	4.16	0.1

In order to remove extreme and/or errorous shear strength measurements, strength data were smoothed by sliding-window averaging. A variety of averaging ranges were attempted but it was found that by simply using the average of the discrete shear strength value with the one shear strength above and below that value produced adequate results. With the exception of core 036_027pc where a 2500 percent increase was noticed, the average increase in the FS was 46 percent and the number of cores with a FS less than unity decreased to two. A summary of the results are presented in Table 4.

Sensitivity of seabed inclination to the calculated factor of safety was assessed for cores 036_ 023, 027, 028 and 029. In general, the FS increases rapidly for slopes less than two degrees and seems to be at unity for slopes between six and nine degrees.

The FS for core 036_027, calculated with the wedge method of analysis and infinite slope method are also presented Figure 5. The FS is improved for the core section using the smoothed data, from less than unity to 1.84. As well, the critical seabed inclination improved from approximately four degrees to greater than nine degrees. The dramatic improvement in FS for a wedge analysis indicated the infinite slope method required laterally homogeneous soils at greater than 300 m extent under static conditions to become unstable.

Table 4. Summary of results for FS infinite slope analysis using averaged shear strength measurements.

	Factor of Safety	Critical slope (deg)	Critical Earthquake Coefficient	Critical Thickness (m)
Minimum	0.77	4.40		4.61
Maximum	12.46	22.96	511.0	99.79
Average	4.14	10.88	110.0	26.48
Median	1.35	8.45	0.17	13.04

The critical acceleration coefficient for a pseudostatic analysis was negative for core 036_027 since the calculated infinite slope FS was less than unity. However, using the wedge analysis and improved FS, the critical acceleration coefficient was still determined to be relatively low at 0.04. Results of the analysis are presented in Figure 3.3. Further, the critical height as calculated in Xstabl was much greater than determined from the infinite slope method.

5. DISCUSSION

Engineering analysis of slope stability was performed using the infinite slope method for twelve locations in the central region of the Scotian Slope. Geotechnical stratigraphic units were established based on depositional history, color differences and litho-stratigraphic interpretation. A geotechnical model is proposed consisting of four geotechnical stratigraphic units

Each of the clay units (Units 3 and 4) below the upper two metres of soil was characterized as lean clays to lean clays with sand, both of low plasticity. A lean clay will normally contain clay mineralogy similar to illite of kaolinite, platy and relatively tightly packed. They also normally contain a component of undrained friction resistance (i.e., N_u>0) in addition to the undrained cohesion (c_u). Even small amounts of friction resistance will greatly improve the FS for each core. Several attempts were made to establish trends in shear strength with depth. Regression analysis of shear strength based on the four geotechnical stratigraphic units and each sub-strata did not provide any pattern. A more general qualitative assessment of the data indicated a shear strength increase with depth of between 2 to 4 kPa per m.

Consolidation tests and physical property data suggest that the sediments are overconsolidated in the upper 2 m and become slightly underconsolidated to slightly overconsolidated with great depth. The near surface apparent overconsolidation is typicial of most marine clays. Deeper underconsolidated sediments may result from increases in pore pressure due to rapid loading by debris flows or other sedimentary processes (Figure 4).

An engineering analysis of slope stability was performed using the infinite slope method. It uses force equilibrium theory to evaluate both the resisting and driving forces on an assumed sliding surface. The FS was calculated for each core using a TSSA and the undrained shear strength, assuming a frictionless soil (i.e., N_u = 0). Sensitivity to unrepresentative values was assessed using a moving average technique for shear strength that gave a 46 percent increase in FS. As a result, only two of the cores had a FS less than unity instead of five using discrete values

The critical seabed inclination was assessed for four cores by calculating the factor of safety for various slopes. Results indicate a critical slope angle of between six to nine degrees for the central Scotian Slope. These values were reasonably consistent for discrete and averaged shear strength profiles.

The lateral homogeneity required for the infinite slope method and the appropriateness of ignoring end forces was assessed preliminarily by using a wedge method of analysis. The FS improved by approximately 300 percent, as did the critical angle, critical acceleration and critical height using the wedge technique. The wedge analysis was performed in XSTABL and assumed a base width of 300 m. The results suggest that the infinite slope method requires more than 300 m lateral distance to develop enough driving force to overcome resistance.

The variation between observations and the calculated FS may be attributed to several factors including sample disturbance, erroneous measurement of available shear strength, analysis methods for earthquake loading and errors in assuming a frictionless soil (i.e., N_u = 0). Pore pressures were not considered in this TSSA. The potential for sample disturbance is greater when the soils are non-cohesive (i.e., sandy) or low plasticity clays. The degree of sample disturbance for sediments recovered using the LCF is not well defined, but correlations suggest the lab shear strength may be as much as 40% lower than insitu shear strength for Lean clays ($10<I_p<20$) (Lee, 1980).

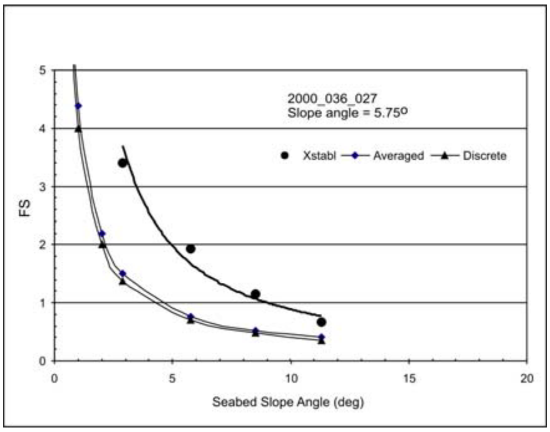

Figure 5. The FS for core 036_027,calculated with the wedge method of analysis.

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