THE UTILITY OF AGGREGATE PROCESSING FINES IN THE REHABILITATION OF DOLOMITE QUARRIES

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ABSTRACT

It is economically and environmentally desirable to use aggregate processing fines (pond fines), a by-product of dolomite crushing/washing operations, in the rehabilitation plans of the aggregate quarry where they are produced. A study was undertaken to determine if the physical and mechanical properties of these dominantly silt-sized materials also made them suitable as embankment material from a geotechnical perspective. The after-use proposed for the dolomite quarry investigated in this study (Milton quarry, Ontario) was to be a naturalized area surrounding a large water reservoir, and pond fines were being used to construct sloped embankments at the base of the vertical perimeter quarry walls for this purpose. The specific objectives were i) to measure and analyze the physical and mechanical properties of the pond fines that are relevant to their usage in site rehabilitation (earthen embankment), and ii) to predict the long term stability of the embankments given current emplacement practices and a scenario of partial submergence by a reservoir. Shear strength, measured by direct shear and drop cone penetrometer methods, increased with higher clay-sized particle content and higher dry bulk density, with lower sand content, and it peaked at soil water contents (4 to 9%kg/kg) well below the measured field capacity of about 16%kg/kg. Factors of safety calculated for slope stability (modified Bishop method of analysis) decreased with decreasing shear strength (cohesion, angle of shearing resistance), with increasing slope gradient, and with the introduction of a water table. Nevertheless, the factors of safety were acceptably high for all scenarios tested (≥ 1.8), including scenarios using the maximum measured slope gradient of 37%. Increasing the dry bulk density by 0.1 g/cm^3 over the mean density measured *in situ* (i.e., to about 87% of the Standard Proctor maximum dry density) did not improve the calculated factors of safety for slope stability enough to warrant increased mechanical compaction of the pond fines during embankment construction.

Introduction

Background

The aggregate industry is an important part of the construction sector of the provincial economy in Ontario, generating 141 million tonnes of aggregate in 1996 (O.M.N.R., 1996). High quality aggregate deposits are located in relatively close proximity to many rapidly growing urban centres in southern Ontario, which require granular material for highway construction, concrete and asphalt production, and fill purposes. Aggregate pits and quarries, however, can be disruptive to surrounding natural areas and to local residents as a result of various nuisances created (Baker, 1993). The duration of these disruptions can be shortened with the implementation of progressive rehabilitation guidelines and regulations (McLellan, 1985; Province of Ontario, 1995). Progressive rehabilitation involves rehabilitating an extracted area of a pit or quarry while continuing extraction in another part of the licensed area. Many decommissioned pits and quarries in Ontario have been converted to after-uses acceptable to local residents, but extraction below the groundwater table usually affects the land use options available (Baker, 1993).

The disposal of aggregate processing fines (pond fines), a by-product from washing crushed rock (dolomite), can represent a major economic burden at many quarries (Brown, 1995; Dukatz, 1995; Wood and Marek, 1995). The relatively high silt- and clay-sized particle content of this material makes it unsuitable as a commercial aggregate product (Dukatz, 1995). Some work has been done to find potential uses for pond fines such as agricultural products, ceramics, industrial minerals, and as fill or embankment material (Brown, 1995; Wood and Marek, 1995). Aggregate companies can potentially lower both rehabilitation and by-product disposal costs by utilizing by-products of their own operations in their rehabilitation plans wherever possible. If it can be demonstrated that pond fines or other by-products of aggregate processing can be used effectively in site rehabilitation, this will benefit both the aggregate producer and the state of the environment.

Site Description

The study site was located at the Milton dolomite quarry (467 ha), which is operated by Dufferin Aggregates and is situated on the Niagara Escarpment about 32 km west of Toronto (Pearson) International Airport (Figure 1). The primary plant is capable of producing up to 2000 tonnes of aggregate per hour and annual production has ranged from 4 to 8 million tonnes, making it the largest aggregate operation in Ontario. The quarry has been in operation since 1962, and Dufferin Aggregates has initiated a progressive rehabilitation plan in the southeastern end, which has been fully excavated. Pond fines from on-site sedimentation ponds have been used to construct sloped embankments about 7 to 8 m high at the base of the vertical quarry walls around the perimeter of the mined area. This medium-textured material has been emplaced by "ramping" it up the slope and compacting it to some degree (Trow, Dames and Moore, 1989). No deliberate mechanical compaction has been carried out, apart from normal wheel/track traffic by earth moving equipment.

The study area was a 650 m length of embankment in a state as described above, but other portions of the quarry embankment were in a more advanced state of progressive rehabilitation (started in the late 1980's) at the time that this study was undertaken. In these areas, loam-textured forest topsoil material (removed from licensed areas being prepared for mining) had been placed over the embankments. Natural vegetation had been allowed to establish and some woody species had been planted (Zimmerman *et al.*, 1998). Timely revegetation of newly emplaced embankments is important to slope maintenance due to the highly erodible nature of the medium-textured pond fines.

The dolomite formation being extracted extends below the level of the regional groundwater table, therefore pumping has been necessary on a continuing basis to dewater the site. Once extraction of the entire licensed area is complete, pumping of groundwater will cease and the quarry will be transformed into a reservoir with a submerged area of about 271 ha (Trow, Dames and Moore, 1989; Zimmerman *et al.*, 1998). The naturalized embankment slopes will provide shallow areas along the reservoir edge (about 14 km of shoreline) capable of supporting both aquatic and terrestrial plant and animal species. Since the quarry is located on the Niagara Escarpment (a UNESCO World Biosphere Reserve), and close to the associated Bruce (hiking) Trail, the creation of both wildlife habitat and dispersed (low intensity) recreational areas are important objectives of the rehabilitation plan, in addition to providing a large open water reservoir.

Study Objectives

It is expected to take almost 20 years for the quarry to fill with water to its final design depth, at which point the perimeter embankment will be submerged to about midway up its slope (Trow, Dames and Moore, 1989). There is little information available in the published literature on the physical and mechanical properties of fine dolomite clastics under different degrees of saturation that would permit the assessment of the long term stability of these embankments at the Milton quarry. The specific objectives of this study were i) to measure and analyze the physical and mechanical properties of the pond fines that are relevant to their

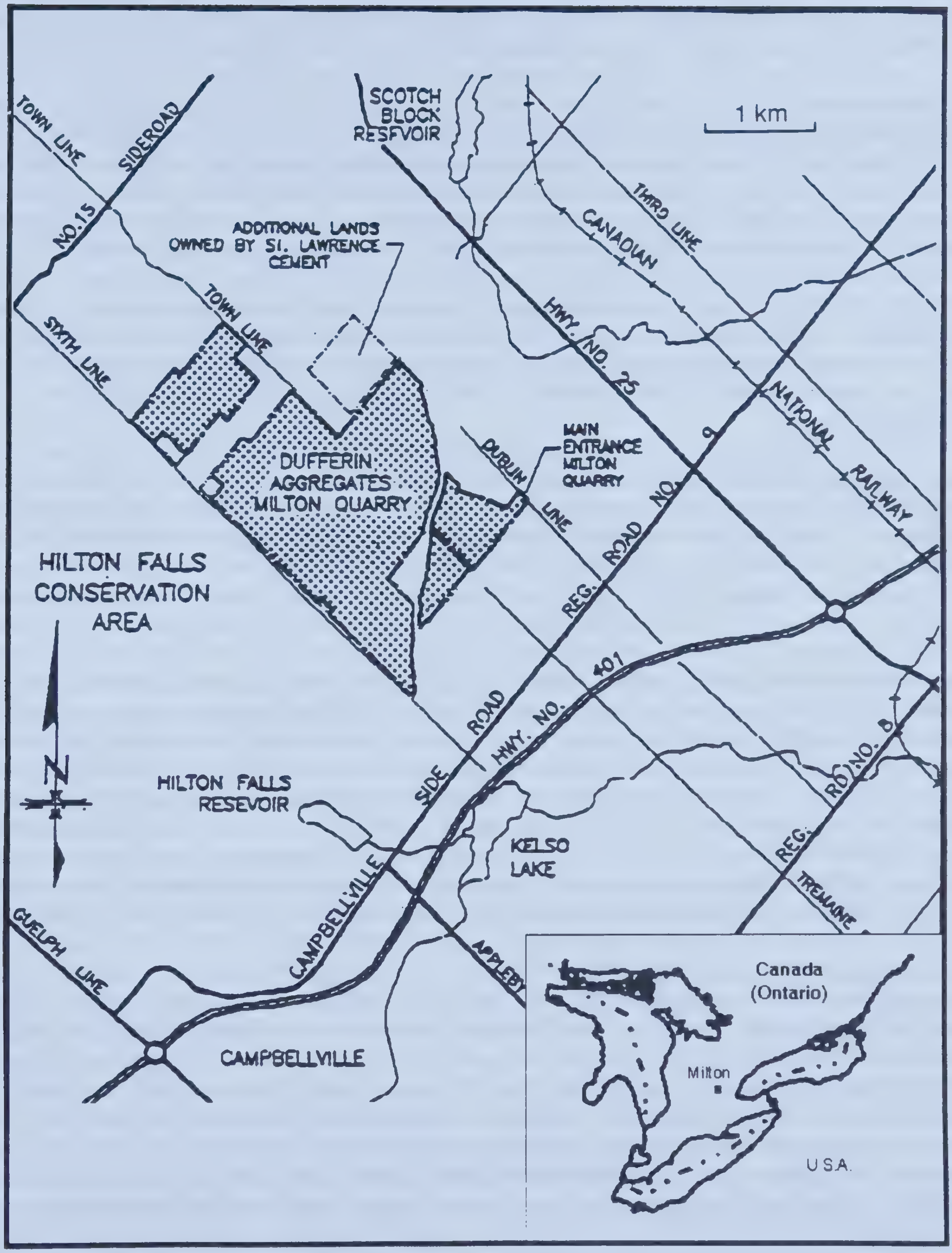

Figure 1 **Locational map of the Milton quarry in southern Ontario (after MacNaughton Hermsen Britton Clarkson Planning Ltd.)**

usage in site rehabilitation (earthen embankment) at the Milton quarry, and ii) to predict the long term stability of the embankments given current emplacement practices and a scenario of partial submergence by a reservoir.

Study Methodology

Theodolite and Terrain Conductivity Surveys of Embankment

A 650 m long section of the embankment located at the southeastern end of the quarry was chosen as the study area, and it was surveyed using an infra-red theodolite (Wild model T1600) to determine the existing slope dimensions. The survey data were downloaded into a mapping program, which was used to create a contour map of the embankment and to determine the range of slope gradients that existed.

A pre-study survey of apparent electrical conductivity was also carried out on the embankment with a Geonics EM31-D non-contacting terrain conductivity meter. Autocorrelation analysis showed that i) there was no spatial dependence in these readings and, therefore, there was unlikely to be any in several other important soil properties such as water content and dry bulk density, and ii) a sampling interval of about 10m was appropriate.

Physical Properties of the Pond Fines (Field)

Sixty-two sampling stations were flagged at 10-metre intervals along a mid-slope transect running perpendicular to the embankment contour. A bore hole (5 cm diameter) about 1.2 metres deep was drilled at each sampling station. The material from each bore hole was placed in a plastic bag to be used as a composite sample for particle-size distribution and shear strength testing. Thin-walled aluminum access tubes (1.3 m long x 5.0 cm i.d.) were placed in the holes with 15 cm protruding from the soil surface.

A CPN density-moisture depth gauge (CPN Model 501DR Depthprobe) was used to measure the *in situ* wet bulk density (ρ_t - g/cm^3) and volumetric soil water content (θ - % m^3/m^3) at each sampling station, for the purpose of generating dry bulk density profiles (ρ_d - g/cm^3). The probe was lowered into the access tube and three repeat measures of gamma and neutron counts were taken at seven different depths (15 cm increments) below the soil surface: 22.5 cm, 37.5 cm, 52.5 cm, 67.5 cm, 82.5 cm, 97.5 cm, and 112.5 cm. Mean count ratios for the three repeat counts at each depth were converted to ρ_t and θ values using calibration equations supplied with the instrument. Dry bulk densities were calculated from the ρ_t and θ values for all measurement depths at each station, and the θ values were also converted to gravimetric soil water content (w - % kg/kg) so that w values from lab tests could be compared directly to field soil water contents.

Physical Properties of the Pond Fines (Lab)

The hydrometer method was used to determine the particle-size distribution for the composite samples collected from the bore holes at each of the 62 sampling stations (Gee and Bauder, 1986). Two test repetitions were completed for each sample. Total sand, clay, and silt contents (% kg/kg) were calculated. Based on the hydrometer testing, samples from four different sampling stations were chosen which represented the full range of particle-size distribution observed in the embankment of pond fines. These four samples were subjected to more detailed analysis (e.g., shear strength, Standard Proctor Density testing). The more precise pipette method (Gee and Bauder, 1986) was also carried out (two repetitions) on these four samples to corroborate the hydrometer particle-size distribution results. Sand fractionation was carried out by wet sieving.

Mechanical Properties of the Pond Fines (Lab)

Standard Proctor Density test. The maximum dry bulk density (MDD) and optimum water content (OWC) of the four samples selected for detailed investigation were determined using the Standard Proctor Density test (A.S.T.M., 1981). The ASTM D698 tests were repeated three times for each sample. A fourth order polynomial equation was fitted to the data to aid in the determination of precise MDD and OWC values.

Determining the effect of sample disturbance and full saturation on shear strength. Due to the cohesionless nature of the material being tested, it was not practicable to extract and transfer structurally intact samples from the field to a direct shear apparatus in the lab for shear strength testing. To study the effect that sample remoulding had on measurable shear strength, a preliminary series of drop cone penetration tests (Houlsby, 1982) was carried out on undisturbed core samples at field water content, which were subsequently remoulded, repacked and retested at the same water content.

Aluminum sampling rings (5 cm high x 4.7 cm i.d.) were used to carefully collect structurally intact soil cores at the surface (16 cores) and at a 30 cm depth (16 cores) at randomly selected locations on the embankment. The core samples were tightly wrapped in plastic bags to maintain them at the field soil water content until penetrometer testing was conducted. The core sample was trimmed carefully using a thin blade to create a flat surface for drop cone penetrometer testing. Up to twelve penetration readings were taken per core.

After being tested in an undisturbed state, the samples were removed from the sampling rings and remoulded in their original sample bags to minimize water loss. The pond fines were then repacked to the same initial ρ_t in the sampling rings in three lifts using an impact compaction apparatus with a 3.0 kg rammer. The drop cone penetrometer tests were repeated as above, and the mean penetration depths (mm) for the undisturbed and corresponding remoulded samples were compared using a Student's t-test to detect significant differences.

An additional 16 undisturbed core samples were taken on the embankment for drop cone penetration testing under fully saturated conditions. This testing would determine the extent of strength loss with saturation of this cohesionless material, and possibly whether there was any evidence of thixotropic behaviour (Maerz and Smalley, 1985). The base of each core was wrapped with porous, acrylic fabric to allow handling when fully saturated. The cores were placed in a tray which was slowly filled with distilled water to the upper level of the sampling rings. Drop cone penetration testing was carried out as above.

Direct shear testing. The shear strength parameters, cohesion (c) and angle of shearing resistance (ϕ), were determined for the four samples (remoulded) using the direct shear method, or ASTM D3080 (A.S.T.M., 1981). Four different normal loads were used for each sample (0.2, 0.4, 0.6 and 0.8 kN), and the test was repeated three times for each normal load. All tests were conducted at a constant horizontal strain rate of 2 mm/min. The samples were first tested at the mean field ρ_d measured along the embankment (1.62 g/cm^3) and at six target water contents (air dry, 1, 2, 3, 5 and 12%kg/kg). Actual w values were calculated by weighing subsamples at both the beginning and end of a given test, drying them for 24 hours at 105°C, and reweighing. The measured w values, rather than the target values, were used in all data analyses.

Testing was also done at a higher ρ_d in order to determine the effects of initial density on the shear strength of the pond fines. It was found that the highest ρ_d that could be attained by applying the maximum achievable static load of 5 kN in the direct shear apparatus was 1.72 g/cm^3. Even this ρ_d value could only be attained at water contents between 2 and 12%kg/kg, as air dry soil could not be compressed to this degree in the shear box. Shear tests were carried out at this higher ρ_d for four target soil water contents (2, 4, 6, and 12%kg/kg). Actual w values were calculated as above.

Horizontal and vertical forces and displacements were recorded by a data logger during each shear test. From these data, the maximum horizontal force and the corresponding vertical force were converted to shear and normal stress values, respectively, by dividing by the area of the sample's shearing plane. These stress values were then plotted and a straight line was fitted to the points by simple regression in order to determine the two shear strength parameters (c, ϕ).

Drop cone penetration testing. For comparison with the direct shear results, the shear strength of the same four samples (remoulded) was also tested using the drop cone penetration method described above. Samples were compacted to two different ρ_d values in large sampling rings (3.6 cm high x 7.6 cm i.d.) using the same impact compaction apparatus mentioned above, but fitted with a larger impact plate. Three replicate samples were created for each combination of ρ_d (1.62 and 1.72 g/cm^3) and target soil water content (air dry, 2, 6, 8, 10 and 12%kg/kg), for a total of 36 samples. Twelve penetration readings (mm) were taken on each core surface. Again, actual w values were calculated by weighing subsamples at both the beginning and end of a test, drying them for 24 hours at 105ºC, and reweighing.

A smaller series of penetration tests was carried out on three of the samples at ρ_d of 1.9 g/cm^3, a density approaching the Standard Proctor MDD for these samples. This density could only be achieved at the high end of the w range used in this study (8, 10 and 12%kg/kg). The fourth sample could not be compacted to this density at any water content, even using the maximum drop height and rammer weight available on the impact compaction apparatus.

Modeling Slope Stability

Factors of safety (FOS) were calculated using a computer program based on the modified Bishop method for slope stability analysis (Bishop, 1955; Craig, 1992). Three different slope angles were considered based on the contour map created from the theodolite survey, representing the maximum, minimum and "typical" slope gradients. Two hydrological scenarios were considered. First, the embankment was modeled in its current unsaturated state (i.e., no groundwater table) by simulating scenarios involving combinations of i) the three slope gradients, ii) the two ρ_d values (1.62 and 1.72 g/cm^3), and iii) the shear strength parameters measured for each of the four texturally-disparate soils at the highest target soil water content (12%kg/kg).

Next, FOS were calculated for the same soil and topographic conditions as above, except that a horizontal groundwater table was simulated at a level 3.5 m above the base of the embankment. This was approximately midway up the 7 to 8 m high embankment, and was the final design depth of water proposed for the reservoir. To model this hydrological scenario, an underlying phreatic (submerged) zone was simulated employing the same ϕ values used for the unsaturated pond fines, but assuming zero cohesion and saturated unit weights.

Results and Discussion

Soil and Site Characteristics

The theodolite survey produced information on the range of slope gradients present on the study area portion (650 m) of the embankment. The minimum measured slope gradient was 15.4% (8.7°, or 6.4:1), the maximum slope was 37.0% (20.1°, or 2.6:1), and the "typical" slope was 25.6% (14.4°, or 3.7:1).

The dry bulk densities (ρ_d) ranged from 1.30 to 1.87 g/cm^3 across all depths at the 62 sampling stations. The mean ρ_d did not vary appreciably with depth (about 1.62 g/cm^3), indicating little natural consolidation of this material with time in the upper 1.2 m. The gravimetric soil water contents (w) ranged from 9.1 to 23.4%kg/kg across all depths and sampling stations. The overall mean of 16.1%kg/kg was also relatively consistent with depth, although the uppermost measurement depth (22.5 cm) showed some evidence of surface drying by early summer. Autocorrelation analysis confirmed the preliminary conclusion from the EM31-D survey that there was no spatial dependency in the ρ_d, w or particle-size distribution data.

The particle-size distribution of the emplaced material (hydrometer method) varied considerably along the embankment, although i) virtually all of the samples belonged to the silt loam textural class, and ii) this textural variation was not reflected in the ρ_d or w profiles. The sand and clay contents showed much higher coefficients of variation than the silt fraction. The 62 sampling stations were organized into four clusters based on the hydrometer data, using a k-means clustering analysis function. This function places samples into a user-defined number of clusters based on the variable in question (i.e., particle-size distribution), with the goal of minimizing the variability within clusters and maximizing that between clusters. A single representative sample was chosen from each of the texturally-disparate clusters for further detailed study. For example, sample #15 had the highest proportion of silt, sample #42 had the highest proportion of sand, and sample #11 had a particle-size distribution that was the closest to the overall mean across the embankment (Table 1). All but sample #42 (sandy loam) belonged to the silt loam textural class (Table 1).

	Effective Diameter Range (mm)	Percentage of Total Sample (%)			
		#11	#15	#42	#51
Very coarse sand	1.0 - 2.0	2.3	2.1	1.2	1.3
Coarse sand	0.5 - 1.0	2.4	1.6	1.4	0.8
Medium sand	0.25 - 0.5	2.8	1.4	1.3	0.6
Fine sand	0.10 - 0.25	6.0	2.5	12.7	4.8
Very fine sand	0.053 - 0.10	13.8	10.8	46.5	33.8
Total Sand	0.053 - 2.0	27.3	18.3	63.0	41.3
Total Silt	0.002 - 0.053	61.8	69.1	33.6	50.5
Total Clay	< 0.002	10.9	12.6	3.4	8.2

Table 1 **Particle-size distribution results for four selected samples (pipette method)**

The results of the Standard Proctor Density tests performed on the four samples are presented in Table 2. Significant differences among the mean test indices were more pronounced in the MDD values compared to the OWC values. Particle shape and grading are known to influence both test indices (Panayiotopoulos, 1989). Coarse-grained soils will normally compact to higher densities than fine-grained soils with the same compactive effort, and will also have correspondingly lower optimum water contents. Sample #42, which had the highest sand content of the four soils tested (sandy loam), did yield the highest MDD and the OWC was also significantly lower than two of the other three samples. Sample #11, which was most representative of the mean particle-size distribution along the embankment, also had a statistically intermediate MDD of 1.98 g/cm^3 (Table 2). If we take this MDD to be representative of the mean textural conditions on the embankment, then the mean field ρ_d of 1.62 g/cm^3 corresponded to about 82% of this reference Standard Proctor MDD. This confirmed that the embankment was not overly compacted or consolidated.

<u>Shear Strength</u>

General strength characteristics. The drop cone penetrometer was used to determine whether structural disturbance of this cohesionless material would cause a significant change in shear strength. The depths of drop cone penetration generally increased after samples were repacked, as compared to the penetration

Sample #	†Mean MDD (g/cm^3)	Coefficient of Variation (%)	†Mean OWC (% kg/kg)	Coefficient of Variation (%)
11	1.98*b*	0.001	10.9*a*	0.58
15	2.01*b*	0.008	10.7*ab*	1.16
42	2.06*a*	0.001	9.4*b*	4.70
51	1.92*c*	0.007	11.0*a*	3.38

† means of Maximum Dry Density (MDD) or Optimum Water Content (OWC) that are denoted with the same lower case letter are not significantly different at $\alpha = 0.05$.

Table 2 **Standard Proctor density test results for four selected samples**

depths measured on structurally intact cores at the same field water content. Cone penetration was found to increase significantly ($\alpha = 0.05$) after remoulding for 13 of the 16 surface core samples taken (Student's t-test). All 16 samples taken at the 30 cm depth had significantly ($\alpha = 0.05$) greater cone penetration depths after remoulding. Since the drop cone penetration test is a rudimentary measurement of a soil's shear strength, this test indicates that there are interparticle bonds present in this material that significantly contribute to soil strength and that will be ruptured by remoulding. Hence, shear strength measured on remoulded samples (e.g., direct shear) is likely to be lower than that actually present in the field. It can be concluded that slope stability predictions made on the basis of the shear strength measures on remoulded samples will be somewhat conservative (i.e., true FOS will be understated).

The drop cone was observed to freely penetrate the full length of the core under fully saturated conditions. The cone penetration depth was well in excess of 20 mm, the depth of penetration considered to coincide with the liquid limit of plastic soils (i.e., viscous fluid consistency). Even though the pond fines are non-plastic, this result indicates that there is negligible cohesive strength when saturated. These saturated core samples did not appear to exhibit true thixotropic behaviour, despite clay-sized particle contents of up to 19%kg/kg. Clay-rich soils exhibiting quick or sensitive behaviour seem to require a mixture of non-expanding phyllosilicates and primary minerals in their clay-sized fraction (Maerz and Smalley, 1985).

Direct shear. Direct shear testing was carried out on the four samples at the mean field ρ_d (1.62 g/cm^3) and at the highest density that could be attained with static loading in the direct shear apparatus (1.72 g/cm^3). Tests were run over a range of gravimetric soil water contents from air dry up to about 12%kg/kg. It was found that 12%kg/kg was approximately the highest soil water content yielding a distinct maximum horizontal stress, which was necessary in order to obtain results for shear strength at failure. Plots of maximum horizontal shear stress (or shear strength [τ_f]) versus vertical stress (or total normal stress [σ_f]) were created, and a simple regression line was fitted to the data in order to define an equation (Coulomb failure criterion) for each sample and combination of initial w and ρ_d conditions as follows:

$$\tau_f = c + \sigma_f \tan \phi \quad (1)$$

The responses of the shear strength parameters with changing w and ρ_d are summarized in Figures 2 and 3 (1.62 g/cm^3). Both linear and second order polynomial functions were needed to adequately fit these relationships. A density as high as 1.72 g/cm^3 could not be obtained, using a maximum achievable static load of 5 kN, for soils drier than 1.7%kg/kg or for sample #42.

Figure 2 shows the effect of w on cohesion for samples prepared at a ρ_d of 1.62 g/cm^3. The regressions of

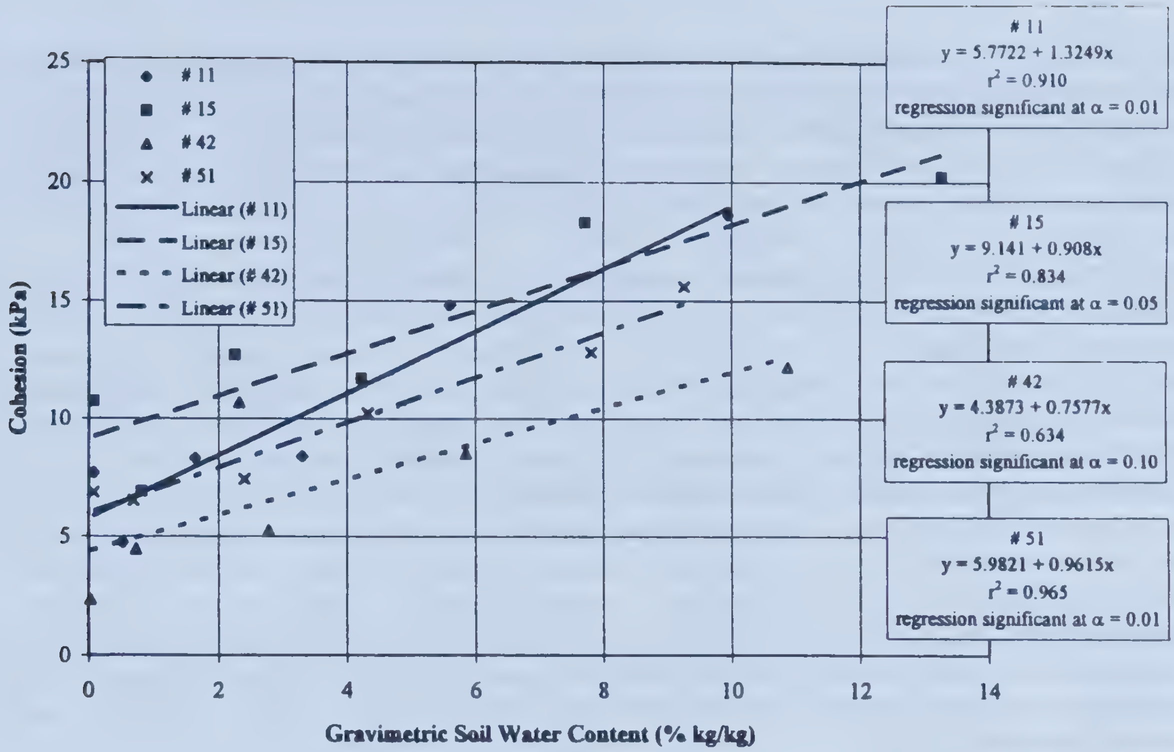

Figure 2 Soil cohesion vs. water content at a dry bulk density of 1.62 g/cm³

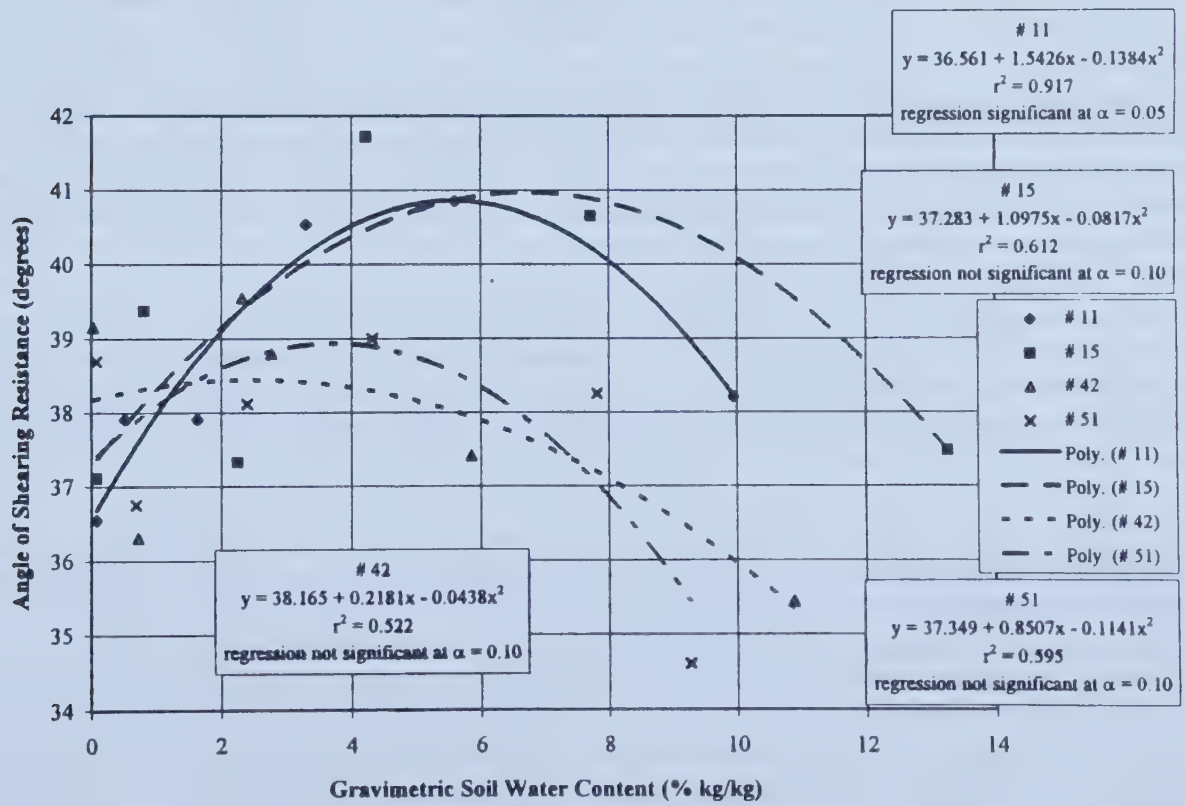

Figure 3 Angle of shearing resistance vs. water content at a dry bulk density of 1.62 g/cm³

cohesion versus w for samples #15 and #51 at a ρ_d of 1.72 g/cm^3 were not significant at $\alpha = 0.10$ and therefore no statistical contrasts were done on these results. An increase in ρ_d, however, seemed to result in a corresponding increase in cohesion at an equivalent w for three of the samples. As ρ_d increases, the measurable cohesion normally rises due to increased interparticle contact (Panayiotopoulos, 1989). The cohesion values for sample #51 at a w of about 10%kg/kg, however, were very similar for both densities.

For samples at a ρ_d of 1.62 g/cm^3, cohesion increased linearly over the range of w tested (Figure 2). The relationship between c and w in plastic soils is frequently found to be curvilinear, similar to that found for these non-plastic pond fines at a ρ_d of 1.72 g/cm^3. At this higher ρ_d, cohesion seemed to increase up to a w of about 6 to 8%kg/kg and then decrease, however the quadratic regressions were not significant at α = 0.05 (the quadratic regression for sample #11 was significant only at α = 0.10). It has been widely observed that a plastic soil's cohesion increases with w until it reaches a maximum close to the plastic limit, and then decreases (Nichols, 1932; Wells and Treesuwan,1978). It will be shown from the results of the drop cone testing (Figure 4) that the w at minimum cone penetration coincides with that at maximum cohesion for samples prepared at the higher ρ_d (1.72 g/cm^3). The point where the plot of drop cone penetration depth versus w reaches a minimum is a good estimator of a cohesive soil's plastic limit (Campbell, 1976). Although this material is clearly non-plastic and therefore has no measurable plastic limit, some parallels can be drawn. It is reasonable to assume that cohesion would eventually reach a maximum and then begin to decrease in the samples prepared at the lower ρ_d (1.62 g/cm^3) at more elevated water contents (Figure 2).

Cohesion seemed to be influenced also by the particle-size distribution of the samples tested. Two of the three regressions for the samples prepared at the higher ρ_d (1.72 g/cm^3) were not significant at α = 0.10, and therefore no further statistical contrasts were made. The linear regressions for the lower density samples were all significant at α = 0.10 (Figure 2) and contrasts were carried out using the General Linear Models (GLM) procedure in PC-SAS. Sample #42 (sandy loam), with the highest sand and lowest clay-sized particle contents, was found to have significantly lower cohesion than samples #11 and #15 (α = 0.05) over the range of soil water contents tested. Therefore, a decrease in sand and an increase in clay-sized particle contents gave rise to an increase in cohesion. Greater clay-sized particle contents will result in a greater proportion of the total porosity comprised of micropores, which can retain water at considerable negative pore water pressures contributing to soil cohesiveness.

Figure 3 shows the effect of w on ϕ for samples prepared at a dry bulk density of 1.62 g/cm^3. There was some evidence of a curvilinear relationship in most samples, but the only significant regression (α = 0.05) was for sample #11 at the lower ρ_d (1.62 g/cm^3). As a result, no statistical contrasts were made. It was concluded from the lack of statistical significance that ϕ is not sensitive to the degree of saturation of the pond fines, which is normally the case for granular materials (Craig, 1992).

An increase in ρ_d did not seem to cause an appreciable increase in the ϕ shear strength parameter. For example, the ϕ of sample #11 over a w range of 1.5 to 10%kg/kg varied from 37.9 to 40.9° at the lower ρ_d (1.62 g/cm^3), and from 37.3 to 38.0° at the higher ρ_d (1.72 g/cm^3). Since an increase in ρ_d results in an increase in interparticle contact, this should bring about an increase in friction and thus in the angle of shearing resistance. This effect was observed only very slightly in samples #15 and #51 at higher w values. There was a tendency, however, for the overall range of ϕ values measured at the higher density to be skewed into the upper end of the range observed at the lower density (Figure 3). The relatively small difference in ρ_d of 0.1 g/cm^3 between the two density treatments was probably insufficient for clear distinctions in ϕ to be observed in this material.

The distribution of size fractions between 0.002 and 0.25 mm (fine sands, very fine sands and silts) in the pond fines appears to have the most influence on the angle of shearing resistance. A shift to smaller particle-sizes in this range will increase the ratio of surface area to mass of the particles which will tend to increase interparticle friction and ϕ (Panayiotopoulos, 1989). The maximum ϕ observed for samples #11 and #15 at the lower ρ_d (1.62 g/cm^3) was about 41° (Figure 3). These two samples have relatively similar particle-size distributions and contain the highest silt and lowest fine/very fine sand contents of the four samples tested (Table 1). The maximum ϕ values for samples #51 and #42 were about 39° and 38.5°, respectively. More than half of the 0.002 to 0.25 mm component of sample #42 is fine and very fine sand, and would therefore be expected to have the lowest ϕ of the four samples.

Drop cone penetration. The drop cone penetrometer was used to further study the shear strength of the four samples over a wider range of w and ρ_d than was possible with the direct shear box apparatus. Figure 4 shows the relationship between the depth of drop cone penetration and w for a ρ_d of 1.62 g/cm^3. It was found that the maximum rammer mass and drop height available with the impact compaction apparatus was not sufficient to achieve a ρ_d of 1.90 g/cm^3 (i.e., 96% of the reference MDD) in samples drier than 6%kg/kg or for sample #42.

The depth of cone penetration showed a strong quadratic relationship with w (Figure 4) and significant regressions ($\alpha = 0.05$) were attained for all but sample #51 at the two lowest ρ_d values (1.62 and 1.72 g/cm^3). The GLM procedure in PC-SAS was used to perform statistical contrasts (second order polynomial model) between cone penetration depths for the two lowest densities. An increase in ρ_d resulted in a decrease in drop cone penetration depth, and this shear strength increase was significant $\alpha = 0.01$. For samples #15, #51 and #42, the depth of cone penetration decreased (indicating an increase in shear strength) as the sand content decreased and clay-sized particle content increased. The direct shear testing revealed similar results. Sample #11 appeared to exhibit the lowest cone penetration depths at its minimum, however the curve is the steepest of the four samples and it intersects the other curves.

The depth of cone penetration decreased from air dry to a w of about 6 to 9%kg/kg, then increased with increasing w (Figure 4). It is known that the plastic limit of cohesive soils corresponds to the w at which cohesion is at a maximum (Nichols, 1932), and at which cone penetration is at a minimum (Campbell, 1976). Although this applies only to plastic soils, where the drop cone test measures the undrained cohesion, a parallel can be drawn to the w of the pond fines where the maximum cohesion values and minimum drop cone penetration depth are reached. This was best illustrated in the higher density treatment (1.72 g/cm^3).

The shear strength of pond fines reaches a maximum at soil water contents considerably lower than the field capacity, and this strength can be increased by increasing ρ_d. This observation is consistent with the results of the direct shear box testing. Again, samples of higher clay-sized particle content and lower sand content seemed to exhibit higher shear strengths.

Slope Stability Analysis

Factors of safety (FOS) for rotational (circular) slope failure were calculated using a computer model based on the modified Bishop method of slope stability analysis (Bishop, 1955). The maximum (37.0%), minimum (15.4%), and "typical" (25.6%) slope gradients were used in these simulations. Input data on the shear strength parameters for unsaturated conditions were taken from the direct shear test results for the four samples at the mean field ρ_d of 1.62 g/cm^3. Values used for c and ϕ were taken at the w closest to the measured field capacity of 16%kg/kg (i.e., the highest target w that could be used in the direct shear apparatus was 12%kg/kg [Figures 2 and 3]). An analysis was also done at the higher ρ_d (1.72 g/cm^3) to determine if increased effort by the aggregate company in mechanically compacting the embankment was warranted.

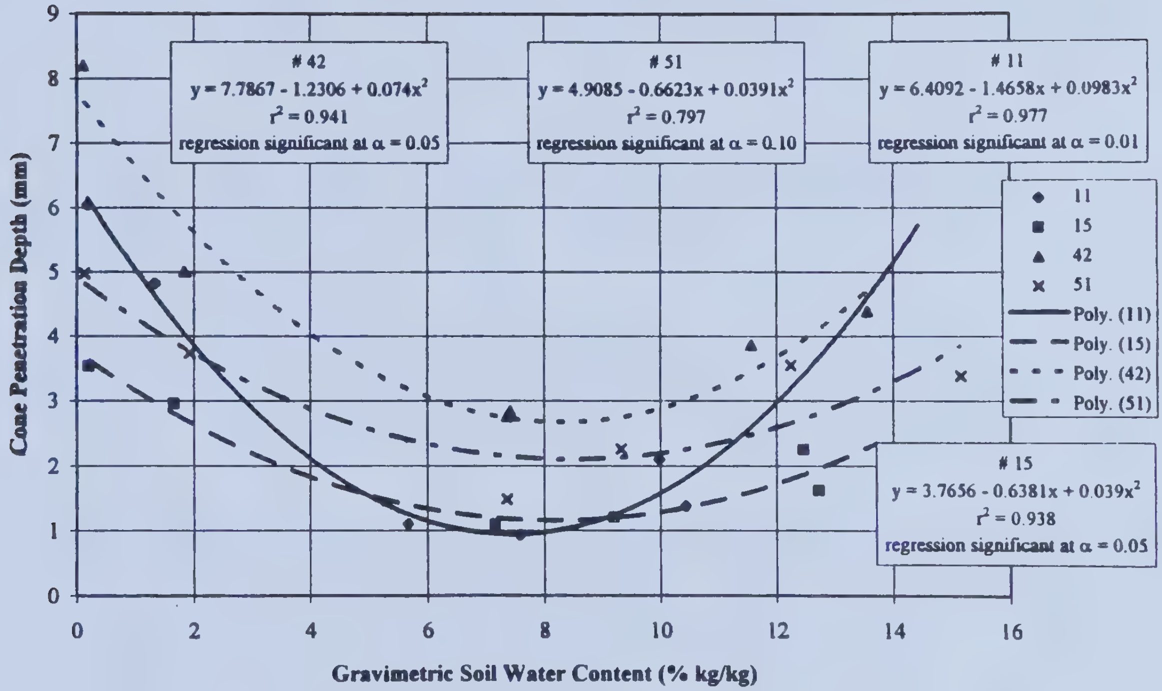

Figure 4 **Depth of cone penetration vs. water content at a dry bulk density of 1.62 g/cm³**

The first hydrological scenario represented the current condition with an unsaturated embankment (i.e., no groundwater table). The second hydrological scenario included a horizontal groundwater table at the 3.5 m level above the base of the embankment, thus producing two zones; an unsaturated (vadose) zone with the same shear strength parameters as the first hydrological scenario, and a saturated (phreatic) zone below the groundwater table with the same angle of shearing resistance as for the unsaturated zone, but with saturated unit weights and an assumed cohesion of zero. The zero cohesion assumption is amply supported in the literature (Craig, 1992), as well as by the observation from this study that there was virtually no resistance to drop cone penetration in saturated core samples of the pond fines. The shear strength of this material at saturation, therefore, is assumed to be a solely a function of the friction component of Eqn. [1]. Table 3 shows the input parameters used in the slope stability analysis for saturated and unsaturated pond fines.

Unsaturated (Field Soil Water Content)						
	ρ_d = 1.62 g/cm^3			ρ_d = 1.72 g/cm^3		
Sample #	ϕ (°)	c (kPa)	γ (kN/m^3)	ϕ (°)	c (kPa)	γ (kN/m^3)
11	38.2	18.6	17.45	38.0	21.9	18.24
15	37.5	20.2	17.98	40.5	21.3	18.64
42	35.4	12.2	17.60	---	---	---
51	34.6	15.6	17.35	37.6	14.6	18.60
Saturated						
	ρ_d = 1.62 g/cm^3			ρ_d = 1.72 g/cm^3		
Sample #	ϕ (°)	c (kPa)	γ (kN/m^3)	ϕ (°)	c (kPa)	γ (kN/m^3)
11	38.2	0	19.69	38.0	0	20.29
15	37.5	0	19.69	40.5	0	20.29
42	35.4	0	19.69	---	---	---
51	34.6	0	19.69	37.6	0	20.29

Table 3 **Input parameters used in the slope stability analysis**

The failure surface yielding the lowest FOS was determined iteratively by the computer model and these values are reported in Table 4. In this study, a FOS of 1.2 or higher was considered acceptable for earthen embankments (Craig, 1992). Table 4 shows that the FOS using the modified Bishop's method was 1.8 or higher for all of the scenarios tested. The steepest slope angle on the embankment (37.0%) yielded FOS values ranging from 1.8 to 4.7, whereas the lowest slope (15.4%) yielded FOS values ranging 4.7 to 9.1. Dry bulk density, however, had a more complex influence on the FOS. For example, an increase in unit weight would normally have the effect of decreasing the FOS, however this was often negated by an increase in the shear strength parameters at higher ρ_d values. The direct shear box testing showed that sample #42 at the lower ρ_d possessed the lowest cohesion of the four samples, and this resulted in the lowest FOS for the vadose scenario.

Slope Gradient	Water Table	Dry Bulk Density (g/cm^3)	Sample #			
			11	15	42†	51
Minimum (15.4%)	no	1.62	8.8	8.8	7.8	8.2
		1.72	9.0	9.1	---	8.1
	yes	1.62	5.2	5.0	4.7	4.5
		1.72	5.1	5.6	---	5.1
Typical (25.6%)	no	1.62	5.4	5.4	4.5	4.8
		1.72	5.2	5.2	---	4.6
	yes	1.62	2.9	2.9	2.7	2.7
		1.72	2.9	3.2	---	2.9
Maximum (37.0%)	no	1.62	4.4	4.4	3.5	3.8
		1.72	4.6	4.7	---	3.8
	yes	1.62	2.0	2.0	1.8	1.8
		1.72	2.0	2.2	---	2.0

† direct shear testing could not be performed on sample #42 at 1.72 g/cm^3, and as a result, its shear strength parameters are unknown.

Table 4 **Minimum factors of safety against rotational failure calculated using the modified Bishop method of analysis**

There was a dramatic decrease in FOS when a horizontal groundwater table was introduced in the scenario testing (Table 4). Nevertheless, the same trends observed in the vadose scenario can be seen in the effects of ρ_d and particle-size distribution on the FOS for the vadose/phreatic scenario.

Conclusions

The shear strength parameters of the pond fines were found to be adequate to maintain a stable embankment with slope gradients of up to 37%, whether the embankment was unsaturated or partially submerged. Material with higher clay-sized particle content and lower sand content possessed higher measured shear strength. The measured angles of shearing resistance, some in excess of 40°, are somewhat higher than for other granular materials with similar particle-size characteristics, possibly due to the angularity of the clastics from the dolomite crushing operations. Even though the pond fines material is classified as cohesionless, it does have measurable cohesion at water contents below field capacity which is attributable to effective stresses caused by the presence of water menisci in micropores. This material has virtually no measurable cohesion when saturated, however the angle of shearing resistance is likely to be adequate to maintain the embankment's stability within the range of slope gradients found on the embankment studied. Any overestimation of ϕ (and hence the FOS) in the phreatic zone due to the inability to conduct direct shear tests on saturated pond fines is likely to be compensated for by the underestimation of FOS caused by using remoulded samples for direct shear testing. Modification in the current methods used to construct the embankment might achieve a higher degree of compaction (i.e., >82% of reference Standard Proctor MDD [1.98 g/cm^3]), but this did not appear to be warranted from the slope stability analysis results. Timely revegetation of newly emplaced embankments, and possibly the installation of biotechnical slope stabilization structures, will be important to the long-term maintenance of acceptable slope gradients by preventing sheet/rill erosion and undercutting of the embankment by reservoir wave action.

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REFERENCES

American Society for Testing and Materials. 1981.
Annual Book of A.S.T.M. Standards, Part 19: Natural Building Stones; Soil and Rock, A.S.T.M., Philadelphia, PA.

Baker, D. 1993.
Land use conflict in the rural-urban fringe: planning for aggregate extraction in Puslinch Township. Plan Canada 33(2):24-30.

Bishop, A.W. 1955.
The use of the slip circle in the stability analysis of slopes. Geotechnique 5:7-17.

Brown, R.H. 1995.
Mineral fines: today's opportunity, tomorrow's success. In: Aggregates: concrete, bases, and fines, 3rd Annual Symposium Proceedings, Center for Aggregate Research (CAR), University of Texas and Texas A & M University, March 1-4, 1995.

Campbell, D.J. 1976.
Plastic limit determination using a drop-cone penetrometer. Journal of Soil Science 27:295-300.

Craig, R.F. 1992.
Soil Mechanics, 5th Edition, Chapman and Hall, London.

Dukatz, E.L. 1995.
Effective use of aggregate fines. In: Aggregates: concrete, bases, and fines, 3rd Annual Symposium Proceedings, Center for Aggregate Research (CAR), University of Texas and Texas A & M University, March 1-4, 1995.

Gee, G.W. and Bauder, J.W. 1986.
Particle-size analysis. In: A. Klute (ed.) Methods of Soil Analysis, Part 1. Physical and Mineralogical Methods, 2nd edn., Agronomy Monographs No. 9, ASA and SSSA, Madison, WI. pp. 383-411.

Houlsby, G.T. 1982.
Theoretical analysis of the fall cone test. Geotechnique 32:111-118.

Maerz, N.H. and Smalley, I.J. 1985.
The Nature and Properties of Very Sensitive Clays: A Descriptive Bibliography, Department of Earth Sciences, University of Waterloo, Waterloo, ON. 135 pp.

McLellan, A.G. 1985.
Government regulatory control of surface mining operations: new performance guideline models for progressive rehabilitation. Landscape Planning 12:15-28.

Nichols, M.L. 1932.
The dynamic properties of soil. III. Shear values of uncemented soils. Agricultural Engineering 13:201-204.

Ontario Ministry of Natural Resources. 1996.
Mineral Aggregates in Ontario: Overview and Statistical Update. Lands and Natural Heritage Branch, Toronto, ON.

Panayiotopoulos, K.P. 1989.
Packing of sands - a review. Soil and Tillage Research 13:101-121.

Province of Ontario. 1995.
Aggregate Resources Act, Queen's Printer for Ontario, Toronto, ON.

Trow, Dames and Moore. 1989.
Rehabilitation Plan for the Milton Quarry (final report), Mississauga, ON.

Wells, L.G. and Treesuwan, O. 1978.
The response of various soil strength indices to changing water content and bulk density. Transactions of the American Society of Agricultural Engineers 21:854-861.

Wood, S.A. and Marek, C.R. 1995.
Recovery and utilization of quarry by-products for use in highway construction. In: Aggregates: concrete, bases, and fines, 3rd Annual Symposium Proceedings. Center for Aggregate Research (CAR), University of Texas and Texas A & M University, March 1-4, 1995.

Zimmerman, K., Lowe, S.B. and Carbone, S. 1998.
Planning for the Future: Rehabilitation of Dufferin Aggregates' Milton Quarry. In: Proceedings from the Leading Edge Conference, October, 1997, Burlington, ON.

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