Construction of a Geogrid- and Geocomposite-Faced Soil-Nailed Slope Reinforcement Project in Eastern Canada

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A case history of a project involving the installation of soil nails with a connected protective membrane facing as a slope-reinforcement technique is presented. The project involved the reshaping of a naturally stable hillside in Cambridge, Ontario. This slope is formed of very dense glacial till material that was resting at a gradient of 1v:2h, before construction; project requirements necessitated developing the slope at a grade of about 3v:1h. The construction involved the installation of a soil nail reinforcement system to permit the development of an 18-m-high slope. At the highest point, the upper 12 m of this slope is permanent; the lower 6 m was a temporary slope excavated to permit basement construction to proceed. Three slope facing systems were used. In one system a facing of nonwoven geotextile restrained by geogrids tied to the soil nails was applied; a sand, topsoil, and water slurry was injected behind the facing to serve as a void filler and to tension the membrane. In another portion of the slope a geocomposite wall was constructed in front of the soil nail-reinforced slope, and this was tied to the soil nails using geogrid. The third system, which was applied to the temporary slope, consisted of a single membrane of woven geotextile tied to the soil nails with anchor blocks. The reinforced slope has proved to be satisfactory through two winters and two spring thaws. Selection of the soil nail support system for the project was based on economic considerations, and the selection resulted in considerable cost savings for the owner.

A case history of soil nail design and installation is presented for a site in Cambridge, Ontario, that has been developed to permit construction of a highrise condominium. The site is on the side of a relatively steeply sloping river valley, such that the development required considerable encroachment into the slope of the valley side.

The original concept for supporting the excavation was to construct two separate retaining structures: a soldier pile and lagging wall was to have served as temporary support for the basement levels; the upper slope, which extends from the main floor level to the crest of the slope, was to have been supported permanently by a reinforced concrete gravity retaining wall. The maximum supported slope height was to be about 18 m, to be reduced to 12 m after completion of the basement structure. However, it was found that this original earth retention concept could not be constructed without temporary mechanical support because it would have required encroaching into land not owned by the developer, and the adjacent land owners were not willing to permit any construction within their properties. The original concept was therefore impractical.

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At this stage, the author reviewed the geotechnical design of the project, and as a result of this review an alternative was proposed in which a system of soil nailing would be installed to support the reprofiled slope. The soil nails were to be designed to replace both the upper permanent reinforced concrete retaining wall and the lower temporary soldier pile and lagging wall. The principle virtues of the new (soil nail) concept to the developer were that (a) no negotiations were required with the adjacent property owners for excavation easements because the reprofiled slope could be constructed from the top down (starting at the intersection of the cut slope with the crest) rather than from the more conventional bottom up, and (b) there would be a substantial cost saving to the project.

SITE AND SUBSURFACE CONDITIONS

The Cambridge condominium site is located on the north bank of the Eramosa River valley. At this location, the slope has developed naturally to a gradient of about 1v:2h. The subsurface conditions at the site consist of a compact, rapidly becoming very dense till stratum overlying bedrock. Although principally silty in character, the composition of the till is somewhat variable, ranging in gradation from silty clay to silty fine sand. Much of the soil deposit is highly susceptible to frost, and it is also susceptible to piping in the event that seepage from the slope were to be unrestrained. The compact condition of the soil is represented by standard penetration test N-values ranging from 30 to more than 100 and typically more than 60 below a depth of 3 m. The more clayey zones of the deposit are concentrated in lenses, and the soil immediately above most of the clayey lenses was found to be saturated; thus, several perched water tables are present within the slope. Another consideration of the variable moisture condition of the soils exposed in the slope is that free water is available as a source for ice lensing under freezing conditions in the highly frost-susceptible silt soils that predominate in the slope.

Observations of cuts made into the hillside at an early stage in construction indicated that the native soil was able to stand unsupported at a nearly vertical inclination for 1 to 2 weeks before deterioration began to affect the stability of the cut face.

DESIGN CONSIDERATIONS

The project required the design of the slope stabilization system to take into account that the upper portion of the reprofiled slope would be permanent and exposed, whereas the lower portion of the slope was to be temporary, required only for the duration of basement wall construction.

At the design stage, it was expected that the temporary slope would be required only to provide support for a maximum of 2 months in fall and that all the elements of the substructure would be complete before the onset of winter. There was, therefore, no perceived need to design the facing to the temporary slope to withstand freezing forces or heavy lateral pressures resulting from temporary loss of soil strength during the thaw. The (permanent) retained slope carried a trunk water main at its crest that had to remain in service and could not be interrupted or relocated. However, no superimposed loads were expected to be applied to the tableland at the crest.

As noted, the native soils at the site are susceptible to frost. Hence, it was required that the external sheeting membrane to be applied to the permanently exposed slopes be designed to accommodate continuing cyclic soil volume changes and corresponding load variations without rupturing. The membrane would also be required to facilitate drainage from saturated zones in the slope face and prevent the occurrence of piping.

The other consideration at the site was aesthetic: the condominium development was geared to the upper end of the market, so engineering design was to make allowance for the wishes of the project's landscape architect. This was achieved by constructing the slope in a series of terraces, each being about 3 m high, and with a bench about 1.2 m wide. These terraces were designed to extend over the entire height of the permanent slope (see Figure 1). Planters were to be placed on each of the terraces.

SLOPE DESIGN

General

In plan, the excavation entailed removing from the hillside a prism of ground that had a triangular footprint. The height of the excavation was a maximum of about 18 m at the intersection of the two faces of the wedge, and this tapered out with distance away from this intersection. Typical cross sections that illustrate high and low areas of reinforced slope are given in Figure 1, which also shows the relationship of the permanent to temporary slope heights. The west and north excavation faces were approximately 90 and 80 m long, respectively.

Soil Nail Design

The design of the soil nail system was based on the method given in the *NCHRP Report 290* (*1*), and it relied on tensile forces in the soil nails only; no benefit for the shear resistance of the nails across a potential sliding surface was allowed. A straight failure plane was assumed for stability analyses, and only a nominal surcharge loading was allowed on the tableland at the crest of the slope. In the absence of a Canadian code of practice, the design was prepared to conform with safety standards recommended by Stocker et al. (*2*). The specified nail was a "Dywidag" threaded steel bar with a guaranteed minimum yield stress of 415 MPa and diameters of 22 or 25 mm (depending on slope height); they were installed in 100-mm-diameter drilled holes. The bars were grouted into the drill holes with a cement–fly ash grout with a water/cement ratio of 0.4 and a compressive strength of 25 MPa. The design of the nail system was carried out using an angle of internal friction of the very dense till of 38 degrees and a unit weight of 22.5 kN/m^3.

The nails were installed on a 1.8- × 1.5-m grid, in all sections of the slope. The nail lengths for various sections of slope are given in Table 1.

As described, the project requirements dictated two slope conditions: a permanent slope of variable height and a lower temporary slope that increased the total height of retained soil by 6 m. Thus, the stresses in the soil nails in the short term were significantly larger than in the long term. Taking into account the lower long-term stresses, the tensile strength of the installed soil nails is not fully used. Thus, by comparing the forces applied over a reduced area to allow for corrosion, an approach now confirmed by Schlosser and Unterreiner in a paper in this Record, there was found to be no need to encapsulate the steel tendon with a plastic sheath. The outer 2 m of the nail was given two coatings of zinc paint as corrosion protection.

The prepared soil nail design was compared with published case histories of successful installations and design charts before being made final (*3–5*).

Design of Facing Membranes

Considerable attention was given to the selection of the slope face membranes. As noted, it was believed necessary to provide a surface membrane that would retain its strength properties through cyclic loading caused by soil expansion due to seasonal freezing in the permanent section of the slope. Furthermore, in view of the several perched water tables found in the slope and the frost-susceptible character of the native soils, it would be a significant advantage to apply a facing membrane with good drainage characteristics. This would then limit the supply of free water to fuel the growth of ice lenses, in turn reducing tension loads on the soil nails under winter conditions. It was also necessary for the membrane to conform to the terraced slope profile design that was acceptable to the landscape architect. Because of aesthetics, the owner was unwilling to accept a shotcrete facing membrane; a shotcrete facing was also expected to be inefficient at permitting drainage through the membrane as well as to be intolerant to soil movement caused by the formation of ice lenses in the retained soil.

The initially selected membrane system was a combination of geogrid and geotextile. The geotextile was used to permit the exit of groundwater from the slope but at the same time retain soil, thereby eliminating the risk of piping. The grid was used to transfer earth pressures to the soil nails by spanning between levels of soil nails. A system of 65- × 65-mm

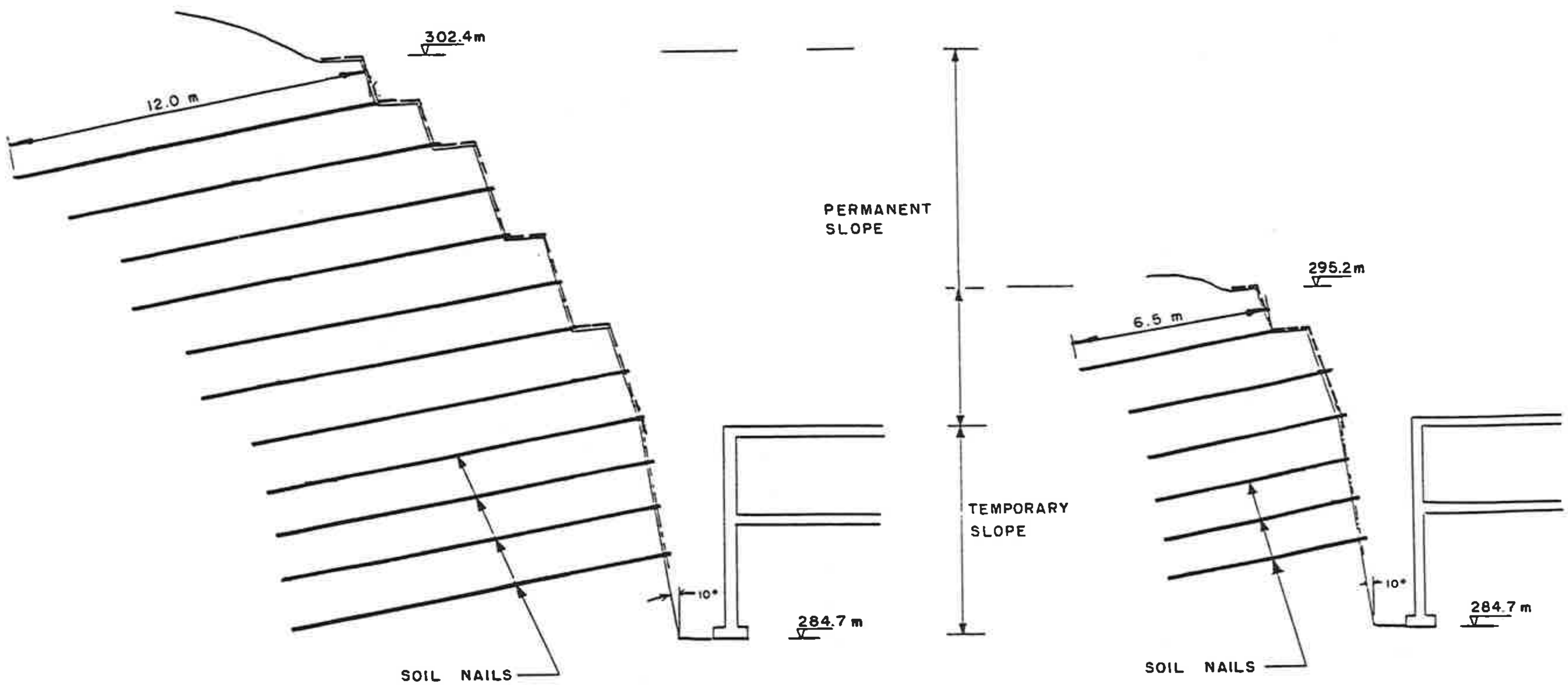

FIGURE 1 Typical cross sections through soil nail reinforced slope.

TABLE 1 SOIL NAIL LENGTHS FOR VARIOUS SLOPE HEIGHTS

Height of Slope		Length of Soil Nail (m)
Permanent (m)	Temporary & Permanent (m)	
4.5	10.5	6.5
8	14	8.8
12	18	12.0

angle bars was used to pick up the earth pressure loads from the geogrid membrane and to span between adjacent soil nails in the horizontal direction (Figure 2). The membrane chosen to provide the strength properties was an unequal bidirectional geogrid in which the prime reinforcement was aligned vertically and the secondary reinforcement horizontally. To accommodate the expected cyclic movement of soil in the face and to protect the reinforcing strands from atmospheric degradation, a grid with multiple polyester strands enclosed in polyolefin with a long-term allowable design load (LTADL) (i.e., design tensile capacity) of 40 kN/m width was selected. Beneath this geogrid, the nonwoven geotextile was used to keep the soil fines from escaping and to prevent surface erosion.

The geotextile was constructed of polyester fibers; it had a mass of 375 g/m^2 and an EOS of 75 μm. The geotextile-geogrid membrane was connected to the soil nails with a system of angle bars that span horizontally between adjacent nails. The angle bars were coated with zinc primer paint and compatible overcoatings of corrosion protection. Application of the membrane consisted of draping the geotextile and then the geogrid over the profiled face of the slope. The angle bars were then placed on top of the membrane and connected to the nails.

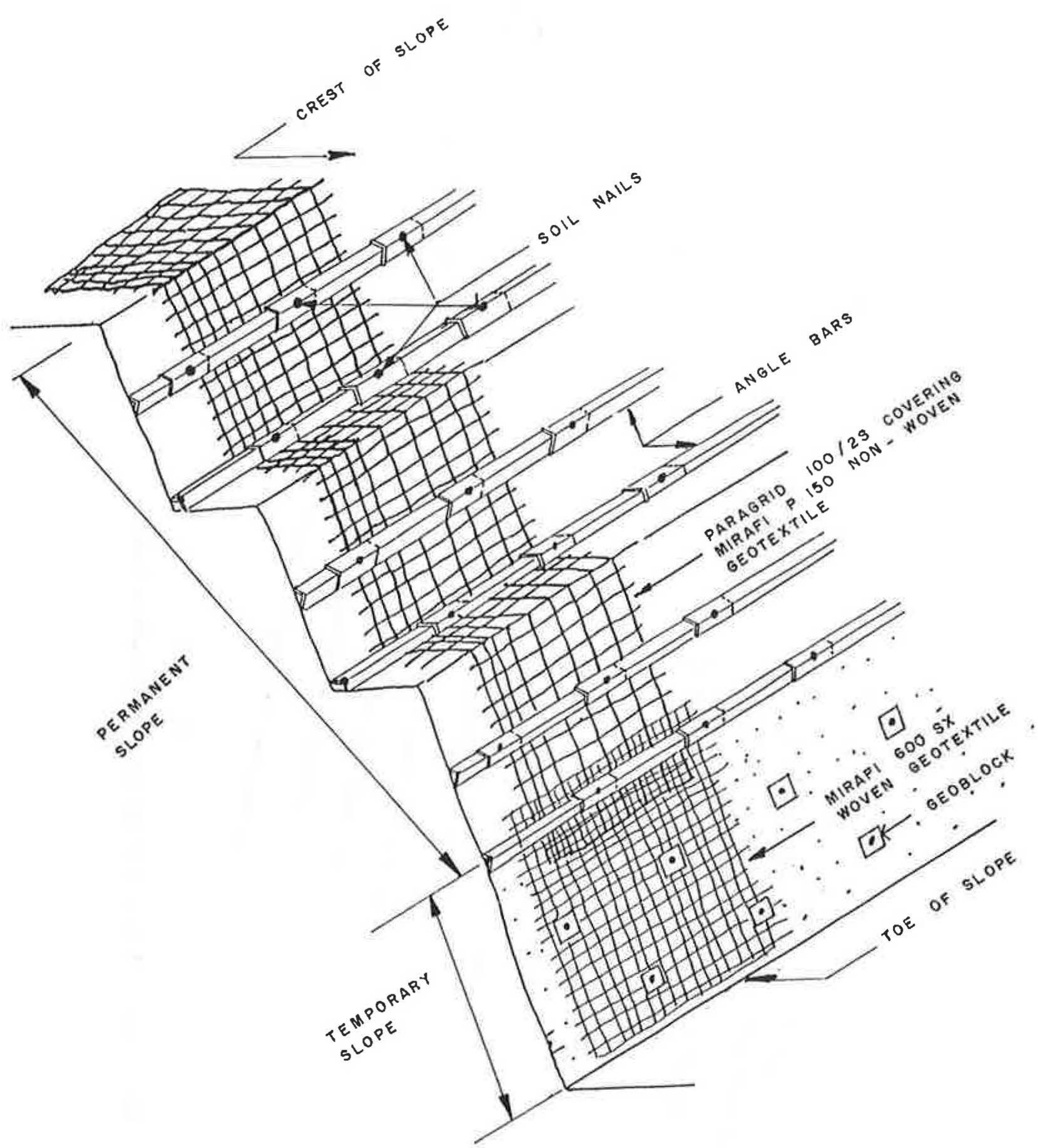

FIGURE 2 Schematic illustration of geotextile-geogrid surface covering of permanent and temporary slopes.

The action of drawing the angle bars to the face of the slope was used to tension the membrane and bring it into contact with the profiled slope. The loads in the membrane and angle bars were determined using the method given in *NCHRP Report 290* (*1*).

After the geogrid-geotextile complex was installed and tensioned, a sand, topsoil, and water slurry mixed in approximately 1.5:1:1 proportions was injected between the geotextile and the native soil as a void filler. The mixture was injected using a gravity-fed funnel and tubing arrangement with pressure being applied with a 1.5-m head of slurry. This method was adopted after trials using a slurry pump to inject the mixture found the pump to be much less effective than was hoped.

Circumstances necessitated a change in facing membrane design over part of the length of the retained slope. A wet area in the slope face (later found to be a consequence of a leak in the water main buried in the crest of the slope) collapsed continually and progressively, and winter came before the area could be stabilized. The face of this portion of the slope remained unrestrained throughout the winter. The winter of 1988–1989 was severe on the incomplete structure in that there were many freeze-thaw cycles and heavy rains alternated with snow. As a consequence, the completely unprotected portion of the "terraced" slope deteriorated and the face slumped to expose a succession of steep scarp slopes. To reduce additional deterioration of the partly slumped slope through the main spring thaw, a restraining membrane of nonwoven geotextile held in place by a timber grillage connected to the soil nails through the angle bars was applied to the soil face. In the spring, a new face had to be constructed and connected to the soil nail reinforcing system.

The face design selected for reconstruction of the deteriorated area of slope featured a facing of geocomposite (Geoweb infilled with sand) with a geogrid-reinforced granular infill placed between the geocomposite and the face of the scarp slope. The design of this section of face was developed on the presumptions that soil nails stabilized the retained slope and that the geocomposite would be required to support the forces developed in the active wedge of soil; these forces were transferred from the face into a reinforced backfill, and to the nails, with horizontally laid sheets of geogrid. The tensile forces in the geogrid and the active pressure were calculated using the method described by Jones for a sloping reinforced structure (*6*).

The Geoweb material was manufactured from 200-mm-wide strips of high-density polyethylene ultransonically welded together to give an open-cell construction having a cell diameter of 200 mm. The concept behind the use of this material in earth works is that when the Geoweb is expanded and infilled with granular material, the cells of the web laterally confine the soil to provide a stable soil-geosynthetic composite. For slope works, the cell structure effectively confines the infill soils so that raveling or sloughing of the slope face is prevented. The Geoweb fascia was also chosen because it is easy to form, it facilitates longitudinal wall contours, and it can be placed rapidly by unskilled labor with a minimum of training; in addition, the exposed cells in the structure can be vegetated (*7*). The geogrid reinforcement for the reconstructed portion of the slope was provided using an uniaxial geogrid, also of multistrand polyester construction, with an LTADL of 15 kN/m. The grid was installed at 0.6-m vertical spacings, and every other grid sheet (or every third, depending on vertical positioning) was tied back to the angle bars, which were in turn attached to the soil nails, thereby connecting the geocomposite facing to the soil nail reinforcement system. Pullout resistance of the geogrid at the geocomposite facing was attained by frictional forces (Figure 3).

In the temporary portion of the slope, the purposes of the surface membrane were to control surface erosion and to retain any minor dislodgements that might occur in the face of the slope. This slope was constructed without any architectural features, at an inclination of 10 degrees to the vertical. For the stated purposes a woven geotextile constructed of polypropylene material was thought to be acceptable for the 2-month design lifespan of the surfacing membrane, and a woven geotextile with a tensile strength of 1350 N was applied (Figure 2). This geotextile was secured to the soil nails with 300-mm^2 blocks ("Geoblocks") attached to the nails by nuts threaded onto the soil nails. Void filler was not injected behind this temporary membrane because of the expected limited design lifetime of this section and the presumption that it would not be exposed to freezing conditions.

CONSTRUCTION OF SOIL NAIL SYSTEM

The threaded steel bar soil nails were installed in 100-mm-diameter predrilled holes at the design locations in the slope. It was found that in the "dry" areas of the face, the drill holes would stay open without collapse for a few hours. This was long enough to allow several holes to be drilled successively, the threaded bars to be installed in the drilled holes (positions maintained with centralizers), and the bars to be grouted into position by pumping the grout through a 25-mm-diameter tube taped to the bars. The tubes fed the grout to the in-ground end of the nail. The grout was pumped in at low pressure and filled the annulus between the bar and the enclosing soil from the in-ground end outward. Grouting of the annulus was judged complete when uncontaminated grout exited freely at the face of the slope; then the tube was withdrawn.

Where the drill holes intercepted one (or more) of the perched water tables, the unsupported hole collapsed when conventional air drilling methods were used, and soil and groundwater would flow out of the face. At these locations, the drill hole would remain temporarily stable when the drilling was carried out using a venturi bit that injected a flushing air and water mixture both backward and forward. These drill holes were grouted immediately on completion, before the next installation was begun. Several times it was necessary to redrill a hole because of collapse, but never more than once.

At the design stage, the adhesion between the soil nail and the enclosing soil was evaluated using an adhesion factor applied to the overburden pressure (*8*). To verify the tensile capacity of the installed nails, cyclic loading tests were carried out on four specially installed test nails. The results for these tests are presented in Figure 4. Tests were carried out more than 7 days after nail installation. The ultimate tensile load capacity of these nails was estimated using the method developed by Chin (*9*) for evaluating the ultimate load-carrying capacity of a piled foundation that has not been taken to

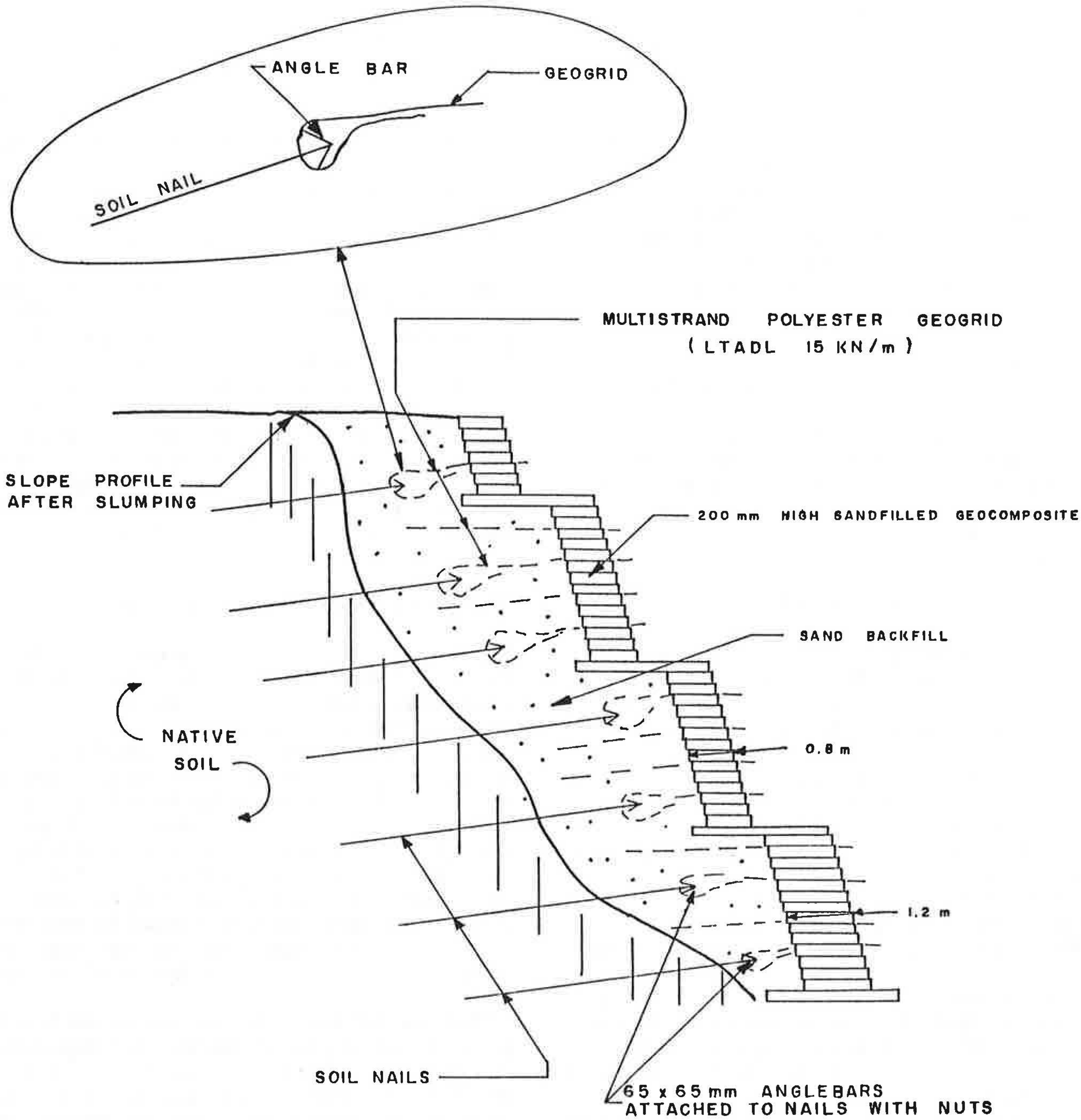

FIGURE 3 Schematic illustration of slope facing construction using geocomposite.

failure. For this analysis, the behavior of the nail was plotted using deflection normalized against tensile load versus deflection of the nail head (Figure 5). The failure load was evaluated from the inverse slope of the plot, and the test results were interpreted to give ultimate adhesions between the grout and the enclosing soil ranging from 225 to 700 kPa. The depth of each of the test nails below the crest and a qualitative estimate of the moisture condition of the soil are given in Table 2. These results illustrate the influence of the moisture condition of the soil on the available adhesion. They also indicate that for the heavily overconsolidated soils found at the subject site, this factor was of greater influence than the overburden pressure.

PERFORMANCE OF STRUCTURES

The performance of the soil nail retaining structure at the site has been generally satisfactory through two winters. The unforeseen major delay that occurred during construction of the building substructure resulted in the temporary slope's having to remain operational well beyond its design lifespan. The

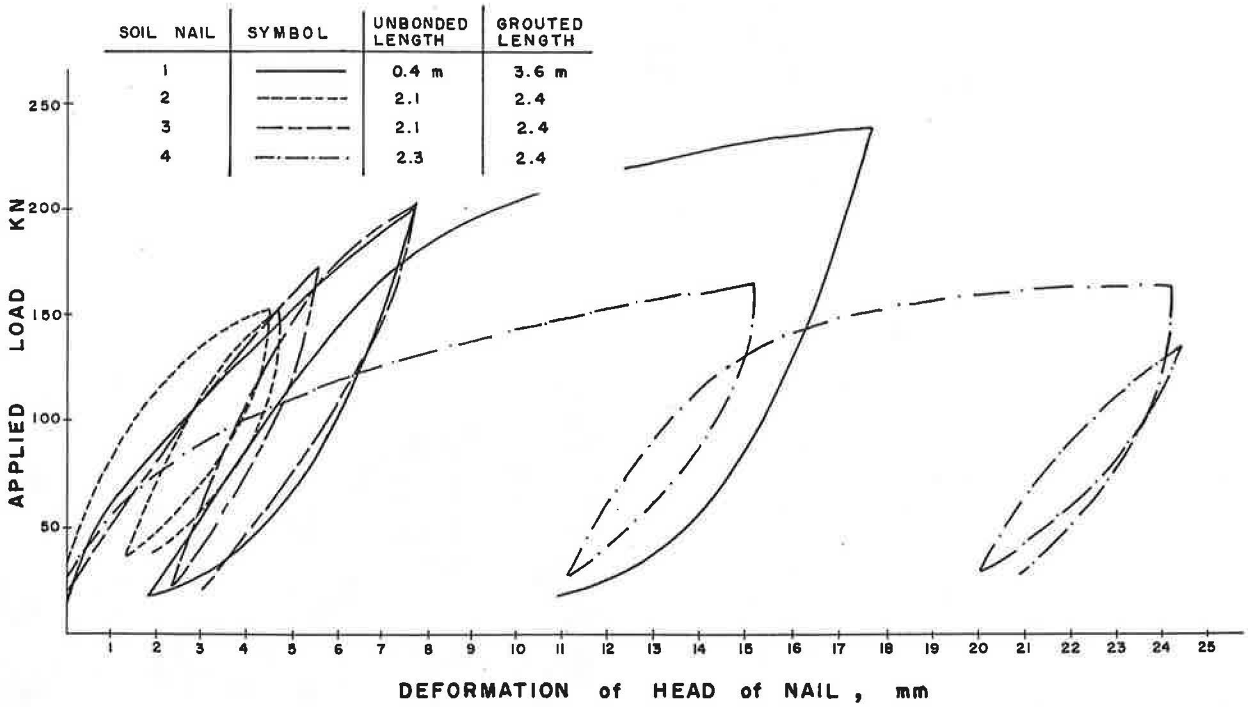

FIGURE 4 Tensile test results for soil nails.

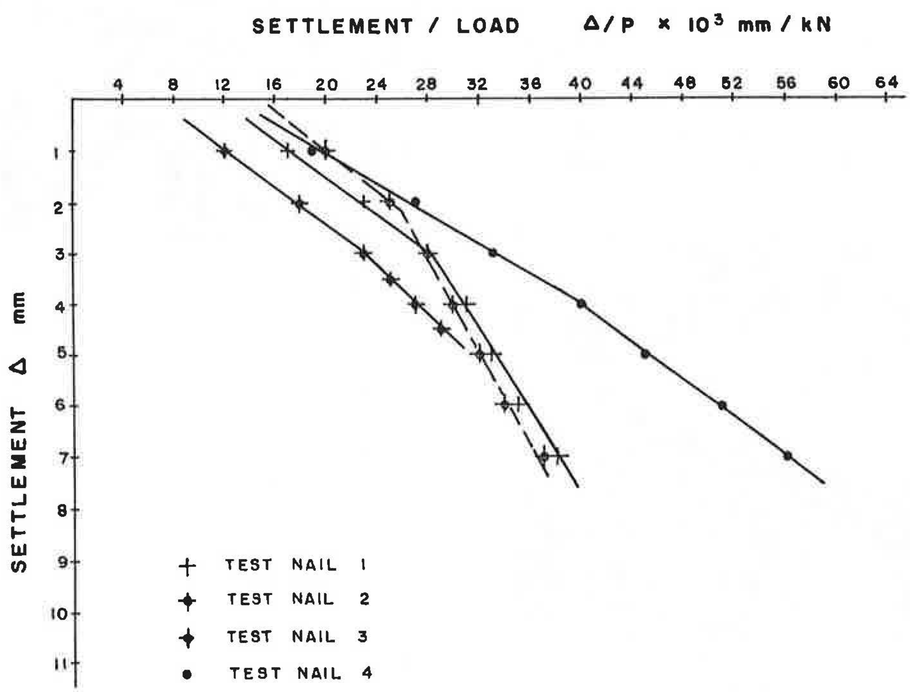

FIGURE 5 Summary of tensile test results for soil nails.

TABLE 2 TENSILE TEST RESULTS FOR TEST NAILS

Test Nail No.	Depth Below Crest of Slope (m)	Moisture Condition of Soil	Representative 'N'-value (blows/0.3 m)	Unbonded Length (m)	Grouted Length (m)	Measured Ultimate Adhesion (kPa)
1	3.0	moist	50	0.4	3.6	700
2	9.0	moist	100	2.1	2.4	475
3	12.0	moist	100	2.1	2.4	600
4	12.0	wet	100	2.3	2.4	255

consequence of this delay was that the temporary slope, including the relatively weak woven geotextile surface membrane, was required to remain in service throughout the entire winter season and spring thaw. During this time, some relatively large slabs of soil dislodged from the soil face, primarily as a consequence of freezing and thawing, and these slabs were too heavy for the woven geotextile surface retention membrane; this resulted in tearing of the geotextile and detachment of the membrane from the restraining blocks that had secured it to the soil nails. Additional support to the temporary slope covering membrane was supplied with a timber grillage in areas where the service life of this slope support system had to be prolonged (Figure 6), and this application was successful.

The unfaced portion of the permanent slope suffered surface deterioration and slumping, and the surface profiling was lost through a succession of minor slumps in the face. This necessitated the rebuilding of the face using the geocomposite wall described previously. There was some loss of the design shape in the portion of the terraced slope that had been covered with geogrid and geotextile, but which had not had the void-filling slurry injected beneath the surfacing membrane before the onset of winter.

The permanent slope at the Cambridge site is now structurally complete; it is being vegetated with creeping vines as visual screening. Photographs of the geogrid-surfaced and geocomposite-faced areas of the slope are presented in Figures 7 and 8. Since the void filler was injected under the geotextile-geogrid surface area of the slope, there has been no further deterioration, and the slope support is satisfactory.

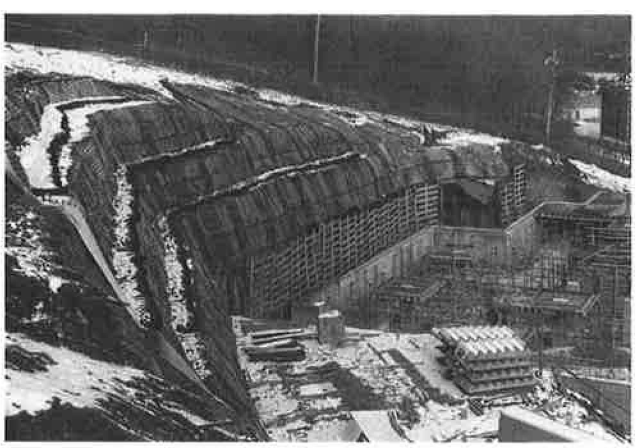

FIGURE 6 Geotextile-geogrid–faced permanent slope and geotextile-timber-grillage–faced temporary slope.

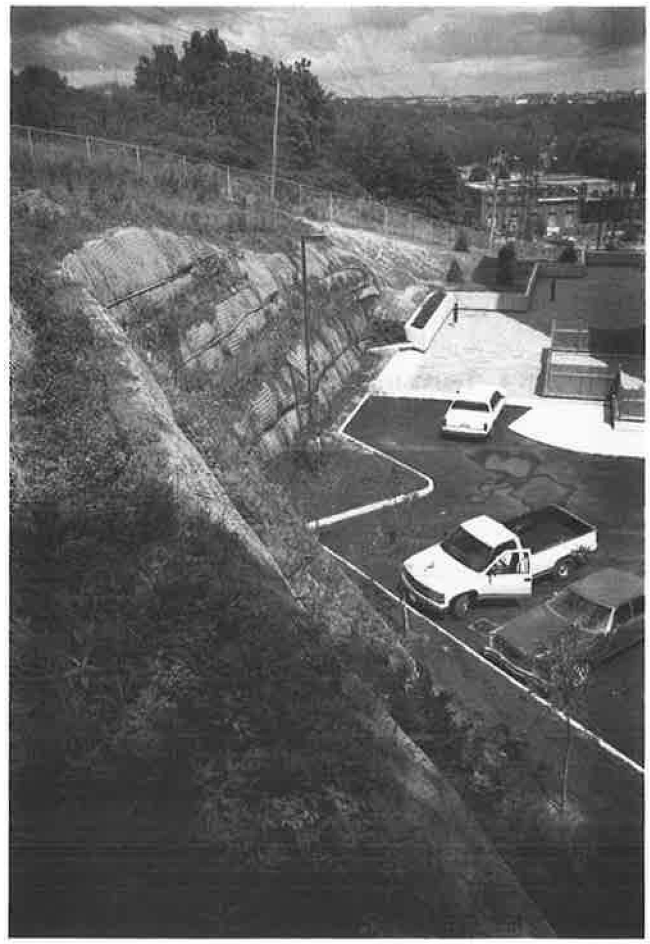

FIGURE 7 Completed slope in area of Figure 6.

The geocomposite facing has not experienced any problems. Both facing systems permit free drainage of wet areas of the face.

The postconstruction monitoring program for the nailed wall consists of two detailed visual inspections of the slope each year: after the spring thaw and before the onset of winter.

CONCLUDING REMARKS

At the subject site, the adoption of a soil nail solution enabled the project to proceed without surface intrusion into neighboring lands. The cost of the soil nail structure was 40 percent

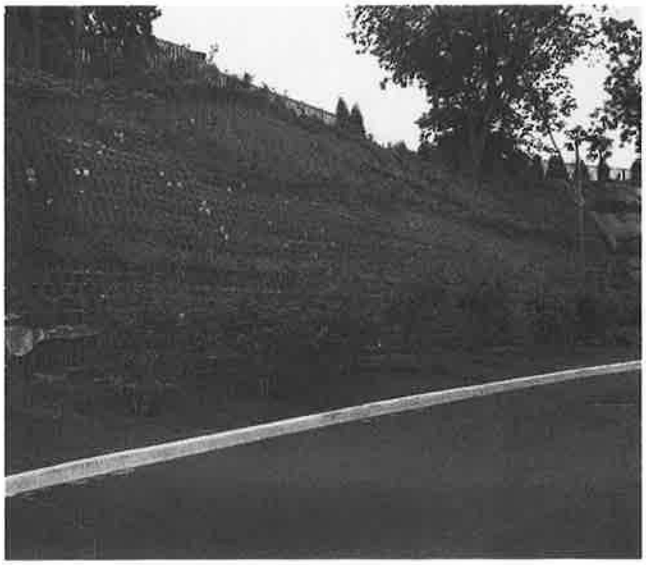

FIGURE 8 Geocomposite-faced slope.

less than the estimated cost of construction of the permanent reinforced concrete and temporary soldier pile and lagging retaining walls that were first envisaged for the site. The adoption of the soil nail solution has thus given the owner a substantial cost saving and a solution less visually obtrusive than the originally proposed retaining wall.

ACKNOWLEDGMENTS

The owner of the Cambridge, Ontario, project is Kressview Springs, Inc. Its project manager, M. Laundon, was very supportive in encouraging the design and implementation of this new slope support system at the site. Its permission to publish details of this project is gratefully acknowledged. The design and supervision of the installation of the soil nail slope support system was carried out by Dominion Soil Investigation, Inc. Detail design and project management were undertaken by Eric Chung of Kitchener-Waterloo. The soil nails at this site were installed by Groundation Engineering Contractors of Georgetown, Ontario. Associated Geotechnical Systems of Milton, Ontario, assisted in the selection of the slope covering materials and supplied and applied the geogrid, geotextile, and geocomposite products. R. Bathurst made many helpful comments during the preparation of this paper, and his input is appreciated.

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