Design and Construction of Two Low Retaining Wall Systems Restrained by Soil Nail Anchors

COLIN ALSTON AND R. E. (ERNIE) CROWE

Case histories relating to the design and construction of two low retaining wall systems are presented. Both cases involve a wall that retains a relatively steep slope at the top of the wall, use near-horizontal soil anchors as lateral restraint, and feature a modular face connected to the anchors with geogrids. At one of the sites, the retaining system was installed to secure a failing wall, and installation of the soil nail anchors was effected with small power tools. At the second site, the slope profile had previously been trimmed to a vertical face. Horizontal restraint was provided by screw plate anchors to accommodate the close-by property boundary. Both walls have proved satisfactory in service.

Two case histories that describe the use of near-horizontal reinforcement to stabilize a soil mass are presented. Both of the case histories refer to relatively low retaining wall systems topped by slopes that rest at a relatively steep gradient. In both cases, the adoption of either conventional gravity retaining walls or geosynthetic membrane–reinforced walls was precluded by site constraints. Both sites are situated in the metro Toronto area.

At the Old Mill Drive site, a naturally formed valley slope was benched to provide the rail track bed for the (defunct) Toronto Belt Line railway at the turn of the century; the bench is about 10 m wide and is situated at about mid-height of the slope. Both the upper and lower slopes (above and below the bench) had rested at a steep angle of 1V:1.4H for several decades; tree growth patterns indicated that the slopes experience creep movements of the near-surface soil horizons.

At this site, as part of the development of a large custom home, cuts had been made into the upper slope of the steep hillside to accommodate the construction of a swimming pool and adjacent patio at the level of the bench. These steeply inclined cuts had been developed as a landscaping feature with a series of three terraces, as shown in Figure 1. The vertical face of each terrace was finished with a low (1.2-m-high) dry stone retaining wall. The horizontal restraint provided to these dry stone retaining walls consisted of a few stone headers that projected beyond the rear face of each terrace wall by about 0.5 m. Not surprisingly, the dry stone walls showed considerable movement and, at the time of inspection by the authors, appeared to be at incipient toppling failure. Had these walls failed, the stability of the entire hillside would have been compromised.

C. Alston, Dames & Moore Canada, 7560 Airport Road, Mississauga, Ontario L4T 2H5 Canada; current affiliation: Alston Associates, Inc., 7725 Birchmount Road, Unit 5, Markham, Ontario, L3R 9X3 Canada. R. E. Crowe, AGS Canada, 375 Bronte Street North, Milton, Ontario L9T 3N7 Canada.

Because of constraints such as the height and gradient of the lower slope, tree cover, and property boundaries, access of equipment to the rear yard was difficult and could be made only by manual means (i.e., dragging equipment up the slope) or by using a very large crane at the extent of its reach. The use of conventional mechanized equipment to install horizontal reinforcement was, therefore, effectively precluded by the site access situation and the need to effect construction from the patio adjacent to the vinyl-lined swimming pool. As a consequence, the stabilizing system for this set of retaining walls had to be designed such that it could be installed by light equipment and hand-operated power tools. In addition to the constraints that apply to the design of conventional retaining structures were factors such as the presence of a swimming pool near the base of the lowest terrace wall, the presence of a substantial dry stone retaining wall at the crest of the slope (on neighboring uphill property), and difficult and restricted access to the site area, which had to be accommodated in the design.

The second case history (Dufferin Street site) describes the design and construction of a low retaining wall that was required to retain ground on an adjacent, higher property. At the time of the authors' first involvement with the site, the slope to be retained had been profiled to a near-vertical unsupported slope 3 to 4 m high that was topped by a 1:1 slope that was up to 3 m high above the near-vertical slope (total slope height was 7 m): the property line with the adjoining property was located near the crest of the cut slope. The authors became involved in this project when the geotechnical engineer who had authorized the initial profiling of the slope refused to extend certification of the stability (safety) of the slope after the cut had been exposed to the elements for 2 weeks. Thus, a design that was appropriate to the site conditions had to be prepared and implemented within a few days. Economic consideration (i.e., the high cost of obtaining a temporary property easement on the adjoining property) prohibited the extension of excavations beyond the property boundary. A profile through this portion of the site that shows the physical constraints of the property boundary and the cut slope, as well as the position of the wall and reprofiled slope, is shown in Figure 2.

At neither site was it safe to make cuts into the then existing slope profiles in order to accommodate sheet reinforcing. Similarly, it would not have been safe to install a conventional gravity retaining structure at either site without the installation of very extensive temporary earth support works. Hence, a restraint system that derived support from near-horizontal

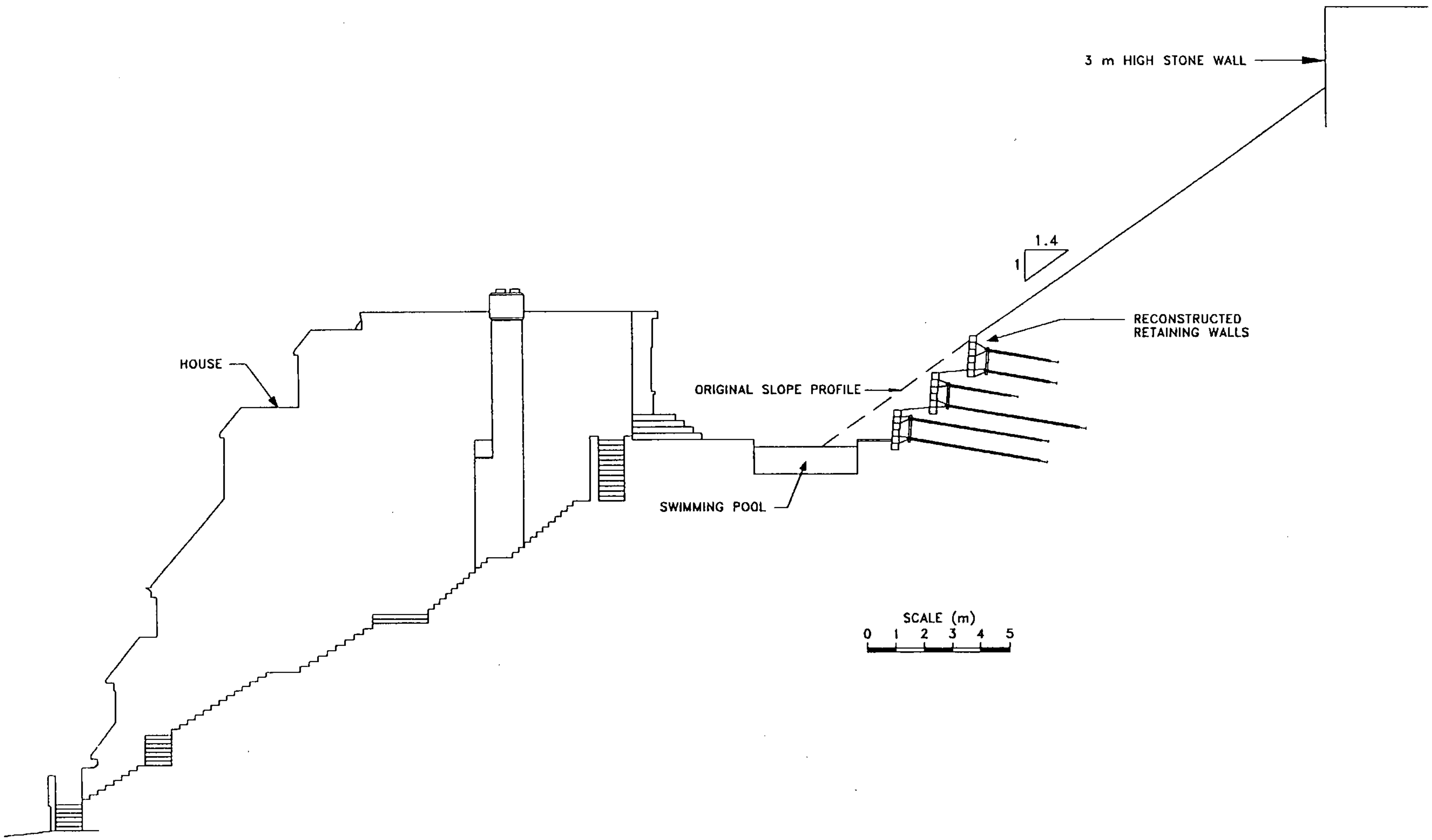

FIGURE 1 Section through hillside, Old Mill Drive site.

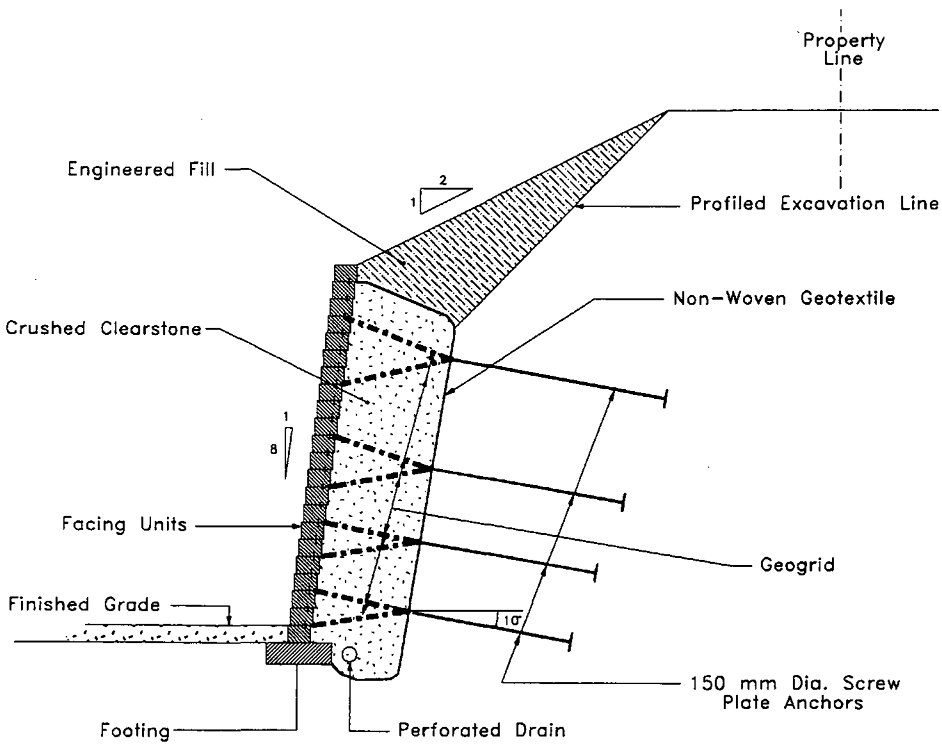

FIGURE 2 Section through reprofiled slope, Dufferin Street site.

reinforcing tendons installed in the body of the slopes from the face of the exposed steeply inclined, unsupported soil faces would be appropriate to both situations.

OLD MILL DRIVE SITE

The subsurface conditions at the Old Mill Drive site consist of dense to very dense sandy silt. The groundwater table is located several meters below the base of the retaining wall system. A representative borehole log sheet for this site is shown in Figure 3, and a typical grain size distribution envelope is provided in Figure 4. Standard penetration tests carried out in the native silt and fine and measured *N*-values greater than 70 blows per 300 mm. On the basis of the *N*-values, the inferred angle of internal friction of the soil used in analysis and for design was taken to be 42 degrees.

Design considerations for the retaining structure that had to be taken into account were as follows:

- The existing walls that retained the profiled terraces in the soil bank had displaced laterally by several centimeters, were leaning outward, and were regarded as being in a state of incipient failure.
- At the crest of the upper slope on the neighboring property, there is a substantial (up to 3 m high) dry stone retaining wall that had been constructed to provide a level backyard area for those neighbors.
- Situated in the midslope bench, the edge of a vinyl-lined swimming pool is about 2 m from the base of the lower terrace wall.
- Reconstruction of the (failed) retaining wall system would have to be carried out in such a way that the potentially unstable terraces would not be further destabilized, causing a large and dangerous earth movement to take place.
- Construction equipment would have to be sufficiently light to be transported to the retaining wall site and to have minimal destabilizing effect on the potentially unstable slope and nearby unreinforced swimming pool wall, and sufficiently powerful to be able to install wall reinforcement tendons in very dense soil.
- The dry stone wall appearance had to be maintained.

The developed solution to wall design was to use a system of soil nail reinforcement that would be connected through an intermediate system to the dry stone facia. Before the remedial design for the slope was prepared, the method of construction that would meet site constraints was first developed. This involved researching various pieces of installation equipment to determine which items would meet the site handling criteria and would have enough power to install ground reinforcement. Thus, the construction scheme envisaged that the soil nails would be installed by advancing a 75-mm-diameter steel casing into the ground by a percussive air hammer and that the advancement would be facilitated by air-flushing the soil entering the tip of the casing at appropriate embedment increments; the casing was to be left in place and regarded as a nonstructural element.

The long-term tensile stresses were to be taken by a stainless steel cable that would in turn be grouted inside the steel casing. The in-ground end of the tensile tendon was then to be attached to a Duckbill 88 earth anchor (Figure 5), which was to be embedded in the soil about 0.5 m beyond the tip of the casing. Because the degree of difficulty of installation would increase with height above the base of the slope, the lower soil nails would have to be designed to provide sufficient

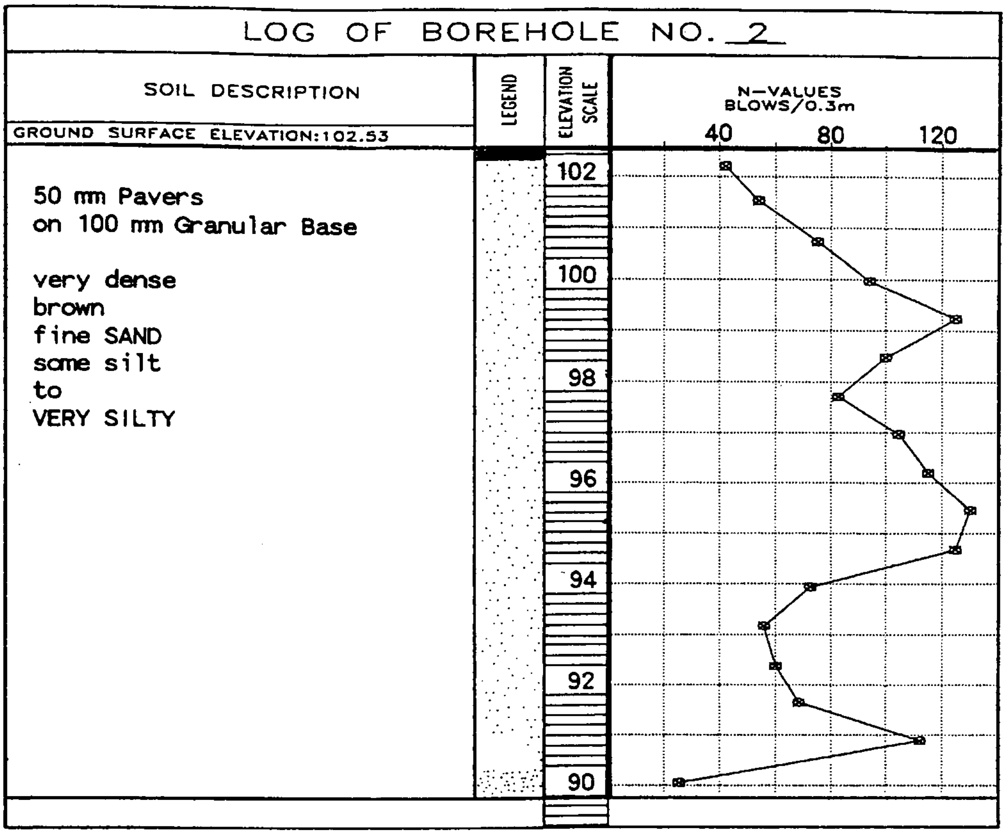

FIGURE 3 Typical soil profile, Old Mill Drive site.

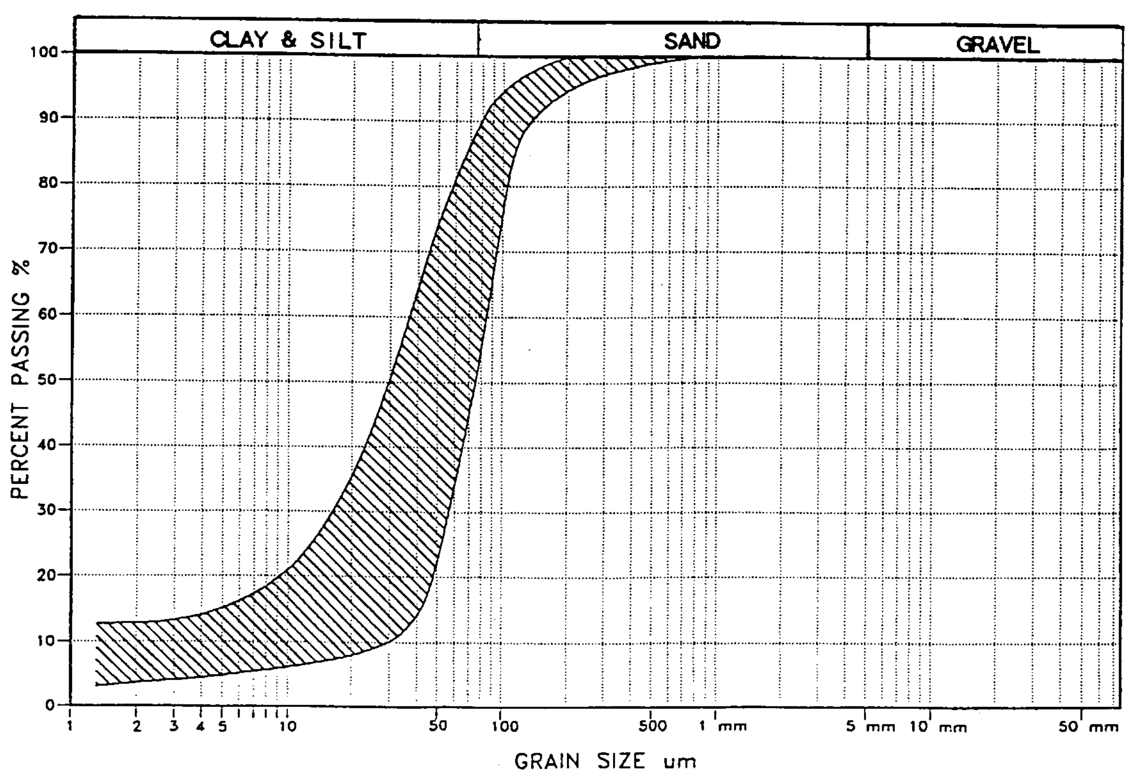

FIGURE 4 Envelope of grain size distribution, native silt and fine sand, Old Mill Drive site.

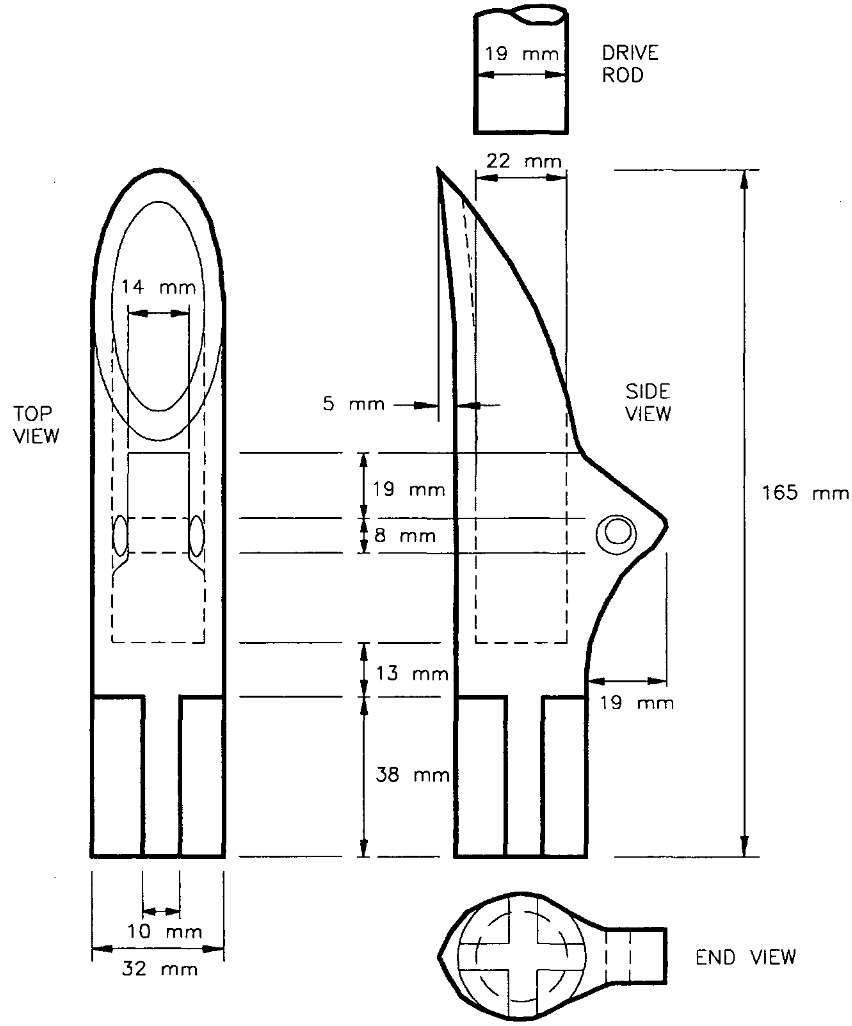

FIGURE 5 Schematic of Duckbill 88 anchor.

tensile reinforcement to support the slope with respect to rotational failure into the swimming pool. Thus, the upper soil nails could be shorter than the lower ranks of nail, as these need only be designed to provide reinforcement to the upper soil terraces.

After a feasible installation method was determined for a soil nail system for the site, design of slope reinforcement was then able to proceed according to conventional design methods (*1–6*). Local stability of the three low-terraced retaining walls was carried out using a two-part wedge analytical method. The global stability of the terrace system was undertaken using conventional circular arc failure surfaces and limit equilibrium methods, with the lower three ranks of soil nails providing sufficient restraining forces to stabilize the hillside, as is illustrated in Figure 6.

Earth pressures acting on the soil face between the ranks of soil nails were transferred to the nails by a series of rectangular polymeric blocks ("Geoblocks"), which in turn were connected to the soil nails by steel angles that spanned adjacent nails horizontally. To provide an acceptable visual appearance, the stone facing was reconstructed in front of the support system; the facia was connected to the spanning angles, and thereby the nails, with geogrid reinforcing (multi-strand polyester geogrid, long-term allowable design load 65 kN/m) (*7*). The stone facia and the soil face were separated by a prism of clear stone material encased in filtration geotextile (filtration opening size < 90 μm, fabric weight > 240 g/m^2); this element of the system also provided for drainage. The detail of the facia is shown in Figure 6.

Construction proceeded in accordance with the projected method, in the following work units:

1. A 75-mm-diameter steel tube was installed to the desired length using an air percussive hammer suspended from a specially designed and constructed gantry and driving against a restraint mounted on a timber platform constructed above and across the swimming pool (Figure 7).
2. At appropriate increments of penetration, the advancement of the steel casing into the ground was rested and the inside of the casing was cleaned out by air flush.
3. The Duckbill 88 anchor that was attached to the stainless steel tendon was inserted into the casing and driven beyond the tip of the casing by about 0.5 m.
4. The stainless steel cable was then tensioned against the mouth of the steel casing and the annulus between the casing, and the cable was grouted to form the soil nail.

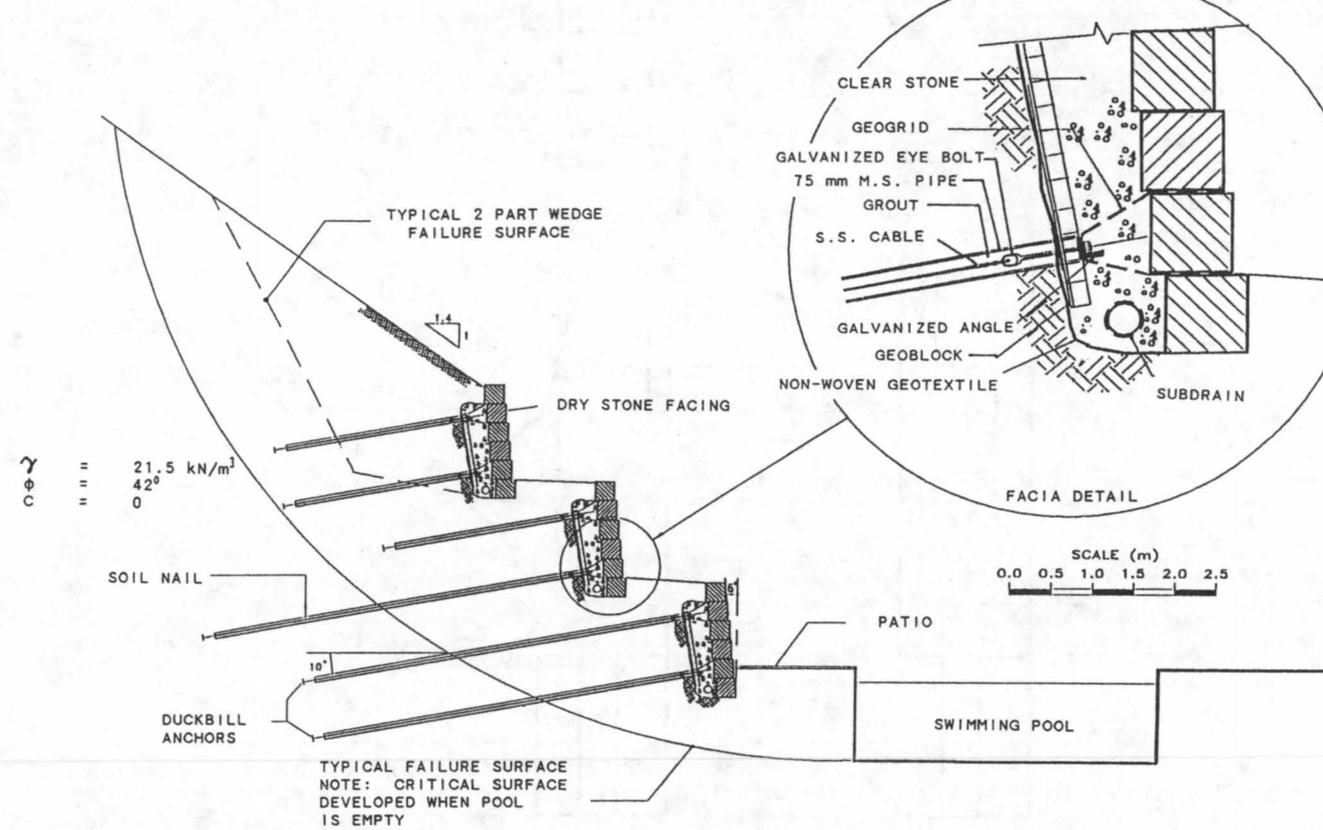

FIGURE 6 Section through terraced soil nail–reinforced retaining walls, Old Mill Drive site.

5. The facia system was constructed and attached to the soil nails with geogrid.
6. Reconstruction of the terraces commenced on the lowest level, and earth support on each terrace was completed before a start was made on the next higher wall.

DUFFERIN STREET SITE

At the Dufferin Street site, the subsurface conditions consist of a hard silty clay till of low plasticity. A representative borehole log is shown in Figure 8, and a typical gradation of this material is given in Figure 9. Standard penetration test *N*-values of this material typically range from 30 to 50 blows per 300 mm in the upper 3.5 m of the soil profile and exceed 100 below this depth. The water content of the silty clay is about 10 percent, which is below the plastic limit of the soil. This soil is heavily overconsolidated and extensively fissured; its long-term behavior is governed by an effective angle of internal friction of about 30 degrees.

FIGURE 7 Reconstruction of lower terrace of retaining walls, Old Mill Drive site.

At this site, the profile shown in Figure 2 had been cut prior to wall design, on the assumption that a retaining wall could be designed to fit this geometry. After the cut profile had been allowed to stand for several weeks (no constructable design had been produced in that time), the geotechnical engineer responsible for certifying the slope declined to extend his certification. The design requirements of the retaining wall at this site were, therefore, that a retaining system be installed that did not require any modification of the profiled slope and that, furthermore, could be installed safely from the ground at the base of the slope. At this stage, the authors were contacted and asked to design and effect the construction of a retaining wall that met the site requirements. It is also pertinent to the design that the owner of the development was not able to accept any changes in the footprint of land occupation at the base of the slope, which would have enabled the slope profile to have been left as it was and other retaining wall systems considered. The design solution that was developed to provide the horizontal restraint to the wall, and to accommodate the site requirements, involved the installation

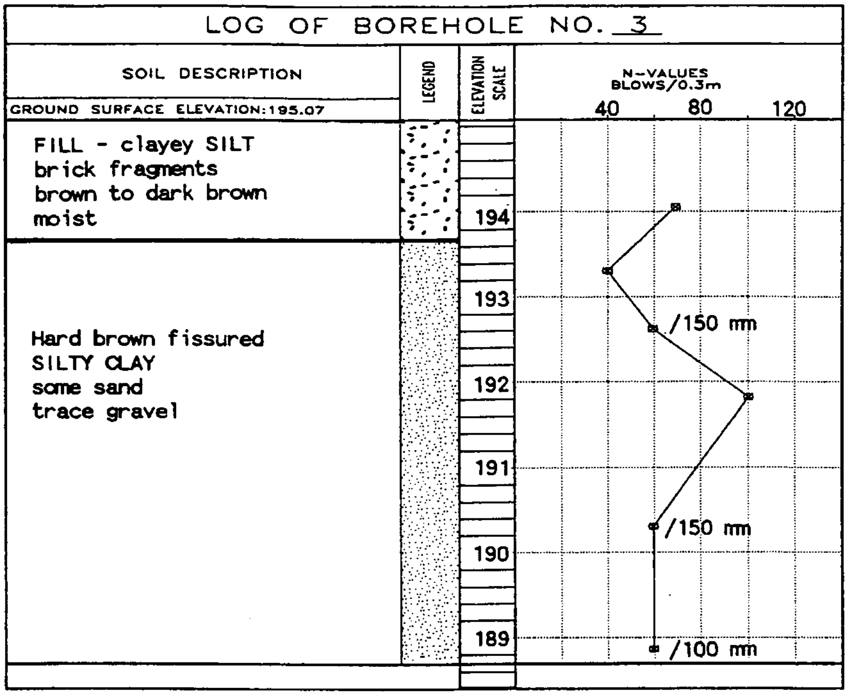

FIGURE 8 Typical soil profile, Dufferin Street site.

of a series of helical plate screw anchors into the vertical soil face. This installation could be effected by using a torque head mounted on a backhoe to drive the anchor into the ground; the anchor was aligned by supporting the tip of the anchor from a remote boom. By using this method, it was possible to keep all personnel somewhat remote from the face of the soil bank to ensure their safety.

The earth anchoring system was designed using the Kranz method of analysis (*8*) and additionally positioning the anchors to satisfy the empirical method given in the *Canadian Foundation Engineering Manual* (*9*).

The modular masonry face was attached to the tensile tendon units using a system that was similar to that previously developed (*7*) and that was used at the Old Mill Drive site—

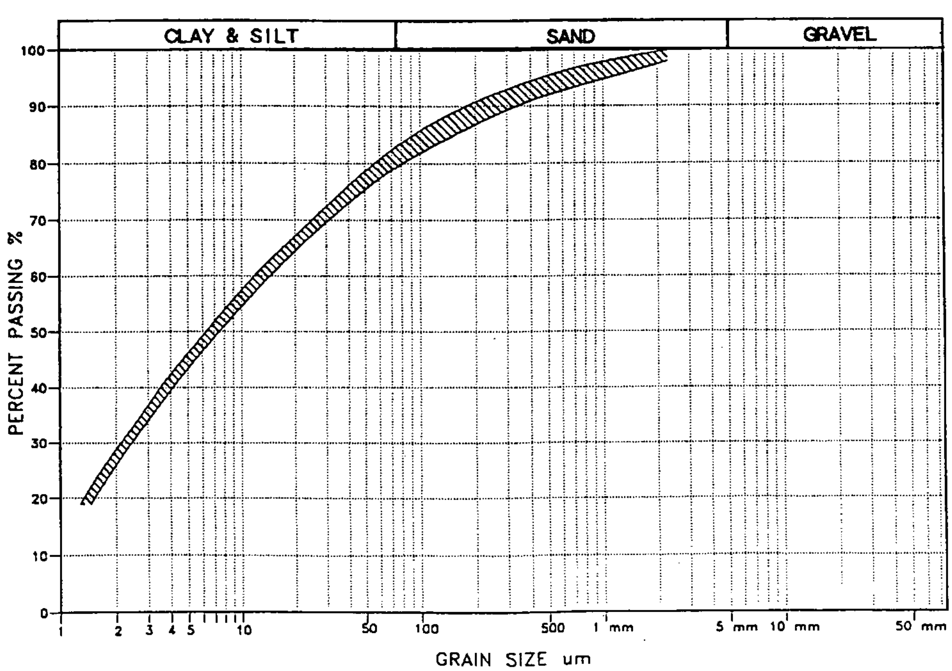

FIGURE 9 Envelope of grain size distribution, native silty clay, Dufferin Street site.

namely, a system of geogrids connecting the facia to the anchors through steel angles spanning between the earth anchors and a prism between the soil bank and the facia filled with self-compacting clear stone material (Figure 2). Figure 10 also illustrates this project.

TESTING AND MONITORING

To prove the design capacity of the horizontal restraint tendons, cyclic testing was carried out at both sites on working anchors. The results of these tests are summarized in Figures 11 and 12 for the Old Mill Drive and Dufferin Street sites, respectively. The ultimate loads of the various anchors were analyzed using the method developed by Chin for estimating the ultimate load-carrying capacity of piles not taken to failure (*10,11*).

Thus, ultimate tensile loads of 19 and 84 kN were estimated for the 2.5- and 5-m-long soil nails, respectively, installed at the Old Mill Drive site. These results indicate an ultimate value of average adhesion between the steel casing and the enclosing very dense sandy silt to be 41 and 80 kPa, respectively. The average overburden loads on the short and long nails are about 33 and 66 kPa, respectively. Considering the reinforcing elements (soil nails) to be similar to horizontal piles for analytical purposes, these values of adhesion may be compared with values calculated by the method of Broms (*12*). The comparison is poor if a steel-to-soil contact is assumed for analysis, but the measured adhesion is very close to that which would be estimated if a grout-soil contact face were assumed.

The average ultimate load capacity of the 150-mm-diameter helical plate anchors installed at the Dufferin Street site, estimated by the Chin method, was found to be about 90 kN. This value of holding capacity may be compared to a value

FIGURE 10 Installation of screw plate anchors into soil bank, Dufferin Street site.

of about 140 kN that would be estimated using the holding capacity–versus–installation torque relationship developed by the A. B. Chance Company (*13*). The fissured character of the soil probably accounts for this decrease in measured pull-out capacity compared with the manufacturer's estimate.

The tensile load tests showed minimum factors of safety of 3.0 and 2.1 for the Old Mill Drive and Dufferin Street sites, respectively.

Where a wall system is finished with a modular facia that is erected at a certain angle of inclination, deformation may be monitored by marking representative sections and measuring movement by conventional survey techniques. At these sites, the wall facias were monitored for inclination at various locations and, to date, movement has been found to be negligible (less than 1 degree of rotation since the completion of construction).

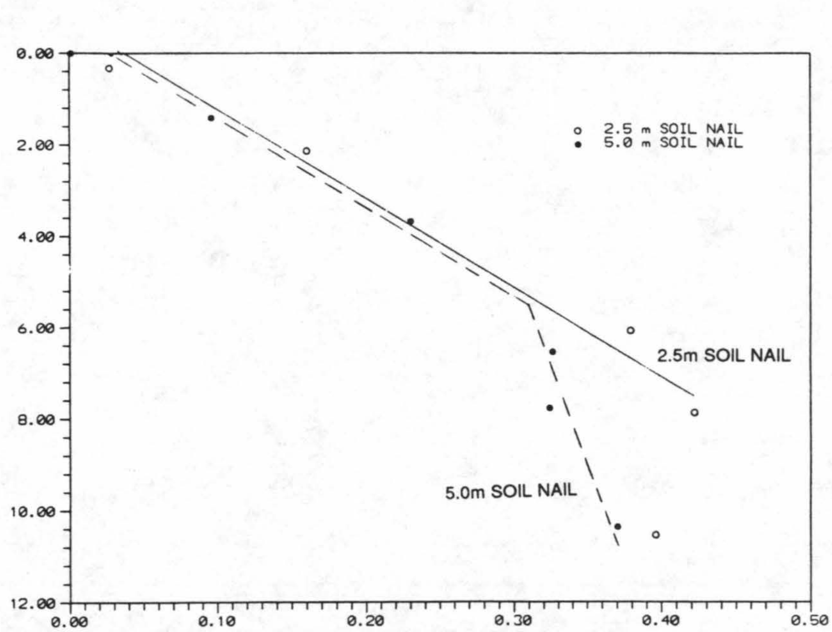

FIGURE 11 Summary of tensile tests on soil nails, Old Mill Drive site.

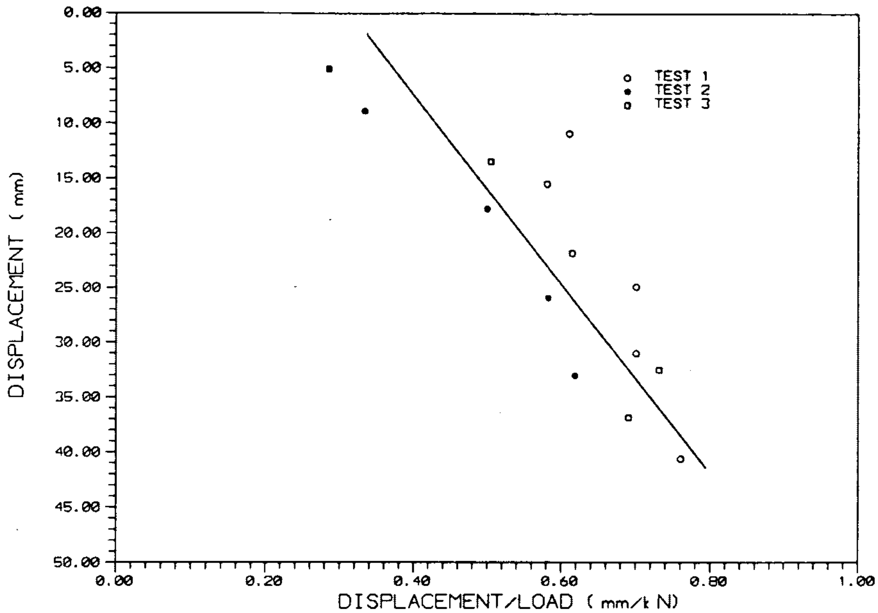

FIGURE 12 Summary of tensile tests on screw plate anchors, Dufferin Street site.

CONCLUSIONS

The use of the design techniques and construction methods described in this paper has enabled the two potentially unstable subject slopes to be supported satisfactorily, economically, and safely. The application of these techniques has illustrated that the installation of near-horizontal ground reinforcement systems can be appropriate to areas that are not accessible by conventional soil nail installation equipment.

More widely, these case histories illustrate how many of the problems associated with the construction of conventional earth retaining structures very near property lines can be eliminated by use of soil nail–reinforced structures. Especially at sites with difficult access and small site storage areas, the disposal of excavated materials is becoming increasingly difficult and more expensive. Application of these simple nailing techniques can greatly reduce such problems.

These projects serve as a reminder that it is both necessary and desirable to consider, in detail, the practicality of construction at the design stage, to scheduling of the construction, and to providing maximum support to the soil bank in a minimum period of time. To effect satisfactory completion of the described projects, it was necessary for the designer and constructor to agree on matters such as the design of soil nails in their entirety, positioning and support of installation equipment (Old Mill Drive site), positioning of construction equipment (Dufferin Street site), and means of constructing the wall facias. In the particular set of circumstances that applied to each of the projects, the benefits of preselecting the contractor so that designer and constructor are able to work in cooperation were significant.

ACKNOWLEDGMENTS

The design for the project at Old Mill Drive was carried out for R. Stephenson of Stephenson and Stephenson, Barristers and Solicitors, who represented the owner. The owner of the Dufferin Street project was Graywood Developments, and its manager was J. Hershkovich. Gratitude is expressed to both men for accepting new design methods and approving the contractual arrangements put in place for each of these sites, as well as for their permission to use site data in this paper. T. Richardson and J. Walls were the structural engineers for the Old Mill Drive and Dufferin Street sites, respectively. Both contributed to the adopted solutions with advice, friendly criticism, and analysis of structural components of the design. I. P. Lieszkowszky of Geo-Canada Ltd. provided encouragement during design preparation, and his contribution is also acknowledged.

REFERENCES

1. Gassler, G., and G. Gudehus. Soil Nailing—Some Aspects of a New Technique. *Proc., 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Sweden, Vol. 3, 1981, pp. 665–670.
2. Jones, C. J. F. P. *Earth Reinforcement and Soil Structures*. Butterworths, London, England, 1985.
3. Mitchell, J. K., and W. C. B. Villet. *NCHRP Report 290: Reinforcement of Earth Slopes and Embankments*. TRB, National Research Council, Washington, D.C., 1987.
4. Schlosser, F., and P. Unterreiner. Soil Nailing in France: Research and Practice. In *Transportation Research Record 1330*, TRB, National Research Council, Washington, D.C., 1991.
5. Stocker, M. F., G. W. Kerber, G. Gassler, and G. Gudehus. Soil Nailing. *Proc., International Conference on Soil Reinforcement*, Vol. 2, Paris, France, March 1979, pp. 469–474.
6. Stocker, M. F., and G. Riedinger. Nailed Retained Structures Behaviour. *Proc., ASCE Special Conference on Design and Performance of Earth Retaining Structures*, Ithaca, N.Y., 1990, pp. 612–628.
7. Alston, C. Construction of a Geogrid- and Geocomposite-Faced Soil-Nailed Slope Reinforcement Project in Eastern Canada. In *Transportation Research Record 1330*, TRB, National Research Council, Washington, D.C., 1991.
8. Hanna, T. H. *Foundations in Tension—Ground Anchors*. McGraw-Hill Book Company, New York, 1982.

9. *Canadian Foundation Engineering Manual*, 2nd ed. Canadian Geotechnical Society, Rexdale, Ontario, 1986.
10. Chin, F. K. Diagnosis of Pile Condition. *Geotechnical Engineering*, Vol. 9, 1978, pp. 85–104.
11. Hanna, T. H. Ground Anchorages: Ultimate Load Estimation by the Chin Method. *Proc., Institution of Civil Engineers*, Part 1, 1987, pp. 601–605.
12. Broms, B. Methods of Calculating the Ultimate Bearing Capacity of Piles, A Summary. *Sols-Soils*, Vol. 5, Nos. 18–19, 1966, pp. 21–31.
13. *Encyclopedia of Anchoring*. A. B. Chance Company, 1990.

Publication of this paper sponsored by Committee on Mechanics of Earth Masses and Layered Systems.