ROYAL ROADS MILITARY COLLEGE

GEOGRID WALL DESIGN

by

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ABSTRACT

This paper describes the selection, design and construction history of a reinforced-soil-and-block-wall system near Victoria, British Columbia. The design basis is explained, including design assumptions, ground conditions, reinforcement and loading parameters, wall geometries, seismic design considerations, and the results of internal and external stability analyses. Construction experiences, as these relate to design details, are also presented.

INTRODUCTION

The Department of National Defence (DND) Canada recently expanded its facilities at the Royal Roads Military College (RRMC), Colwood, British Columbia, with the construction of a 90-unit cadet residence. The four-to-six storey residence, 77 m x 12 m in plan, was positioned perpendicular to an existing building and the resulting alignment placed the axis of the new residence skew to the natural topography, a slope dipping at approximately 3H:1V. To construct the building the slope had to be partially excavated and regraded.

A prime objective of the DND was to preserve the aesthetics of the college grounds. These include formal gardens and a castle, while the slope to be developed bore trees over 200 years old. The project architect and a landscape architect worked together to design the area around the building with minimal disturbance to existing vegetation. The architects' scheme, shown in Figure 1, consisted of terraced walls and graded slopes with overall angles up to 52 degrees and a maximum slope height of 13 m.

DESIGN CONSIDERATIONS

Geotechnical engineering solutions were required to allow construction of building foundations, and to permanently restrain the slopes and walls proposed by the architects. Several solutions were considered: concrete cantilever walls; sheet pile walls; H-piles/lagging; slurry trench walls; and geogrid-reinforced soil slopes.

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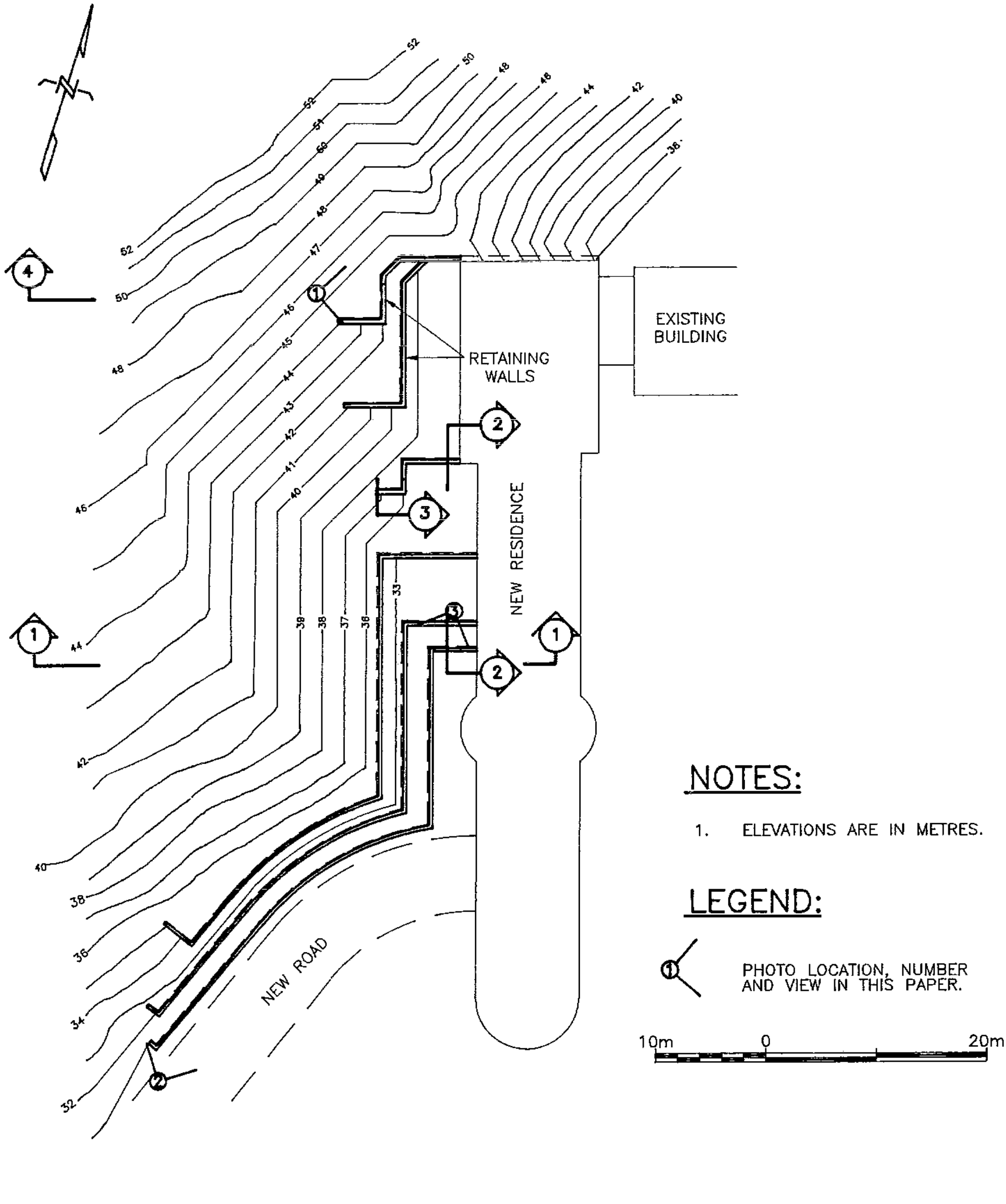


Figure 1 - Site Plan

The following factors were considered in selecting a preferred solution:

1. Site soils: dense granular materials with cobbles;
2. Site geometry: steep original and proposed slopes;
3. Site access: very restricted;
4. Site location: high seismic acceleration zone;
5. Local weather: possibility of intense precipitation;
6. Construction materials and equipment available;
7. Contractor's experience;
8. Construction schedule;
9. Wall configuration and alignment: battered and curved walls;
10. Structure performance: tolerance to settlement and seismic behaviour;
11. Durability;
12. Site disturbance: to be minimized;
13. Costs; and
14. Aesthetics.

A two-stage solution was chosen. A soil-nail-and-shotcrete wall, 85 m in length and to a maximum height of 13 m was selected to restrain soils temporarily during foundation construction; and a geogrid-reinforced slope with precast interlocking blocks was chosen to provide permanent support. Only the latter is discussed herein.

ANALYSIS

INTRODUCTION

The wall system, shown in Figure 1, was divided into four representative sections and analyzed in two-dimensions for static and psuedo-static conditions.

The geotechnical design of geogrid-reinforced slopes for static conditions has evolved over the past decade through research, modelling and instrumented case studies, to become accepted practice. In comparison, the design of reinforced slopes subject to seismic loading is in its infancy (Patterson et al., 1990). Seismic and static design approaches are broadly similar in that reinforcement layer strength, length, number and spacing are determined using slope stability methods. However, design details, such as applied loads, soil and reinforcement strengths, and factors of safety, differ in static and pseudo-static cases.

WALL AND SLOPE GEOMETRY

Figure 2 shows one of the four sections analyzed. Internal stability was analyzed using circular and wedge-shaped failure surfaces through and between reinforcement layers. External stability was analyzed for base sliding and overall failure.

The soil-nail-and-shotcrete wall used for temporary slope support was designed independent of the geogrid-reinforced slope. However, the position of the shotcrete wall was controlled strictly by the fixed location of the block walls and the minimum length of geogrid required to achieve adequate stability of the permanent reconstructed slope. The unprotected soil nails were assumed to corrode completely in the long term and thus impart no additional strength to the slope outside of the geogrid-reinforced mass. The shotcrete wall/soil interface, however, was assigned a conservative fictional angle of 20° and evaluated as a potential slip surface.

Slopes were analyzed in two-dimensional sections rather than three-dimensional space, and a degree of conservatism accepted for walls concave in plan, because of the complexity of optimizing geogrid length and spacing in three dimensions. The interior corners of walls were reinforced perpendicular to both faces.

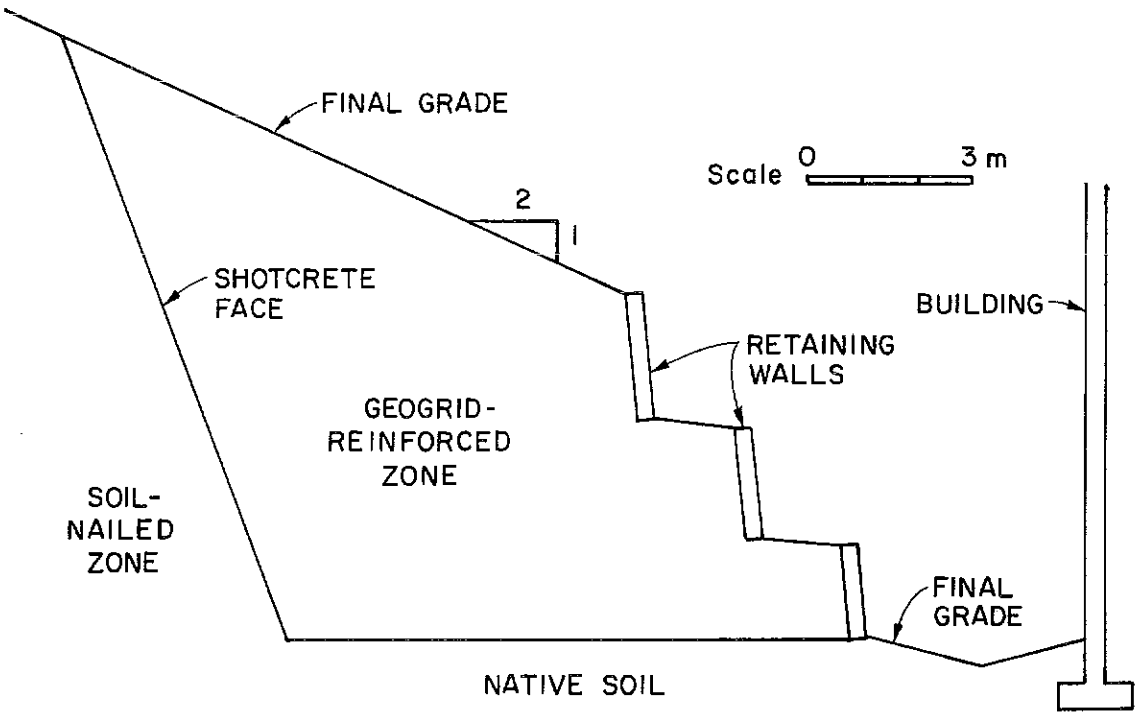


Figure 2 - Section 1

ASSUMPTIONS

The following assumptions were made to simplify the analyses:

1. Soil reinforcement is placed horizontally.

2. Block walls are vertical and their mass provides no increase in stability to the reinforced-soil-wall system.

3. Wall foundations are competent.

4. Geogrid to block connection is adequate to resist pull-out during and after backfill placement and compaction.

5. Material properties for each design component (i.e. native soil, backfill, geogrid type) are isotropic.

6. There are no hydrostatic forces and pore pressures do not arise during a seismic event.

7. The loads caused by seismic events can be modelled satisfactorily pseudo-statically.

DESIGN PARAMETERS

SOIL

Field investigations were carried out prior to selecting a preferred slope-retention solution, to determine site soils and groundwater conditions. Drillholes and a test pit exposed weakly cemented dense to very dense glacio-fluvial sands and gravels with 10 to 35 percent cobbles. Trace amounts of fine particles were also present.

In stability analyses, cohesion was assumed to be zero for native material in its undisturbed state and when used as backfill. Static and pseudo-static friction angles of 38° and 42° were used for backfill and undisturbed native material, respectively. Effective unit weights were estimated using engineering experience.

Groundwater levels were monitored and found to be well below the excavation/reconstruction zone. Slopes were designed accordingly for the conditions of no hydrostatic and no seismically-induced pore water pressures.

REINFORCEMENT

Four strengths of geogrid, produced by a single manufacturer, were used in design. The allowable long-term static strength of each geogrid type was based on values published by the manufacturer. The values implicitly included the effects of creep behaviour, damage during installation, and biological and chemical degradation.

Whereas static geogrid strengths discussed above are, primarily, based on creep rate/permissible strain criteria, dynamic geogrid strengths are controlled by loading rate. The reason for this is that a polymer with a high degree of molecular orientation, such as HDPE, can achieve a substantially greater percentage of its ultimate strength when loaded rapidly versus slowly, for a given strain. This characteristic of increasing geogrid strength with increasing strain rate is beneficial during seismic events. However, to establish a site-specific dynamic design strength requires knowledge of seismic loads and loading rates, and the execution of rate-specific geogrid strain tests. The process is sophisticated and yields, at best, a range of possible dynamic geogrid strengths. In the RRMC analyses a simpler approach was taken in which the manufacturer's recommended allowable dynamic strength of 90% of the ultimate strength was factored to 80% because of the uncertainty of seismic loading rates.

SEISMIC LOADING

During a seismic event reinforced and natural slopes were assumed to behave as rigid monoliths. The assumption is based on the dense, weakly cemented nature of undisturbed site soils and the high degree of compaction specified for backfill materials.

In a slope which does not behave monolithically, seismic loads will vary throughout the slope based on deformation characteristics viz. soil shear moduli and damping variations with shear strain and the characteristics of bedrock motions. These characteristics are exceedingly difficult to quantify accurately. Further, analytical methods using vertical slices then must account for variable force distributions with height in addition to the presence of reinforcement.

The National Building Code of Canada states that the peak horizontal ground acceleration ('a') for Victoria is 0.3 g. For design purposes, a pseudo-static coefficient ('k') is used. 'a' and 'k' values are broadly correlated and engineering judgement is required to select an appropriate relationship. Seed and Whitman (1970) recommend $k = 0.85a$ for retaining structures. Correspondingly, RRMC slopes were designed for a horizontal acceleration of 0.26 g.

SOIL-REINFORCEMENT INTERACTION

During loading, soil and reinforcement interact both frictionally and by interlocking, and these interactions generate frictional and passive resistances to pull out, respectively. The degree of interaction determines the effectiveness of reinforcement, that is, the resistance it offers to sliding of the soil mass above a reinforcement layer, and the length of reinforcement embedment required to prevent pull out.

The ratio of soil strength at the reinforcement interface to soil strength alone is called the soil-reinforcement interaction coefficient. Static interaction coefficients may be assigned either on the basis of manufacturer's data or, preferably, on the results of shear-box tests using site soils.

Dynamic interaction coefficients are more difficult to assign because large-scale shear-box tests which model seismic events are presently beyond the state of the art. One can make predictions about the dynamic behaviour of reinforcement and soil separately, but there are, at present, few precedents to guide the engineer in predicting the synergistic behaviour of soil <u>and</u> reinforcement during a seismic event. Caution during design is therefore required and contract documents should specify backfill materials and compaction criteria which minimize the risk of soil weakening during a seismic event.

On the RRMC project, static and dynamic interaction coefficients were assigned a common value of 0.9 because of good backfill materials and a specified high degree of compaction. Research is under way at the University of British Columbia which aims to provide a more informed basis for assigning dynamic interaction coefficient values.

FACTORS OF SAFETY

Because of the relatively low risk to life and property of slope failure at RRMC and the likelihood that failure would not be catastrophic, slopes were designed to equal or exceed static and pseudo-static factors of safety of 1.5 and 1.15, respectively.

DESIGN CALCULATIONS

Reinforced-soil slopes must be analyzed for internal and external stability. Stability external to the reinforced zone can be analyzed using well-established slope stability computer programs, design charts or other methods. Bishop's Simplified Method of Slices is commonly used and it yields satisfactory results in most cases.

Internal stability must be examined to determine the amount, length and distribution of reinforcement required to prevent failure through and between the reinforcement layers and reinforcement pull-out. In two-dimensional computer models, reinforcement may be analyzed as tensile line forces. The magnitude of line forces must decrease near the reinforcement's free end to model correctly decreasing pull-out resistance with decreasing embedment length.

Stability programs with these characteristics are not common but they are preferred to reinforced-soil design charts which are often product-specific and less versatile.

Taking as fixed the design parameters stated earlier, the geometry shown in Figure 2, and a wall module height of 0.75 m (chosen to conform with precast block heights), design variables simplified to reinforcement length and amount (i.e. strength times number).

Analyses using circular and wedge-shaped failure surfaces yielded the minimum reinforcement requirements shown in Figure 3. Critical failure surfaces for both static and dynamic cases terminated at the base of the bottom wall and intersected several of the lowermost geogrid layers. Interestingly, the shotcrete/reinforced-fill interface was not a critical failure plane even with an assumed friction angle of only 20°. In addition, the analyses showed that low strength reinforcement was required within the 2H:1V slope to prevent sloughing during a seismic event.

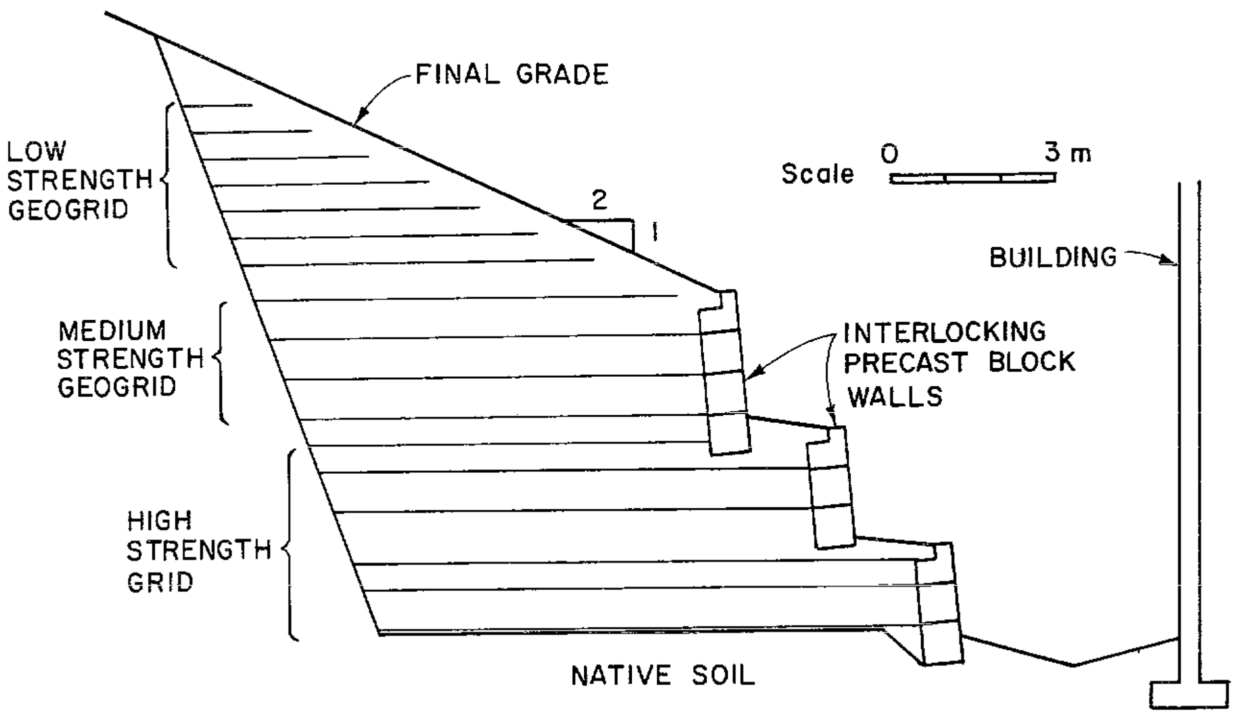


Figure 3 - Section 1 with Geogrid Reinforcement

The areal extent of reinforcement is shown in figure 4. Reinforcement was required behind all but one low wall and typically the reinforcement extended to the shotcrete wall at all elevations. Multiple reinforcement types were used to optimize the design.

DESIGN DEVELOPMENT

The initial design, based on stability analyses, was developed further to address practical considerations. The most northerly wall (see Figure 1 and Photograph 1) was redesigned as a permanent soil-anchored wall to eliminate substantial excavation, soil nailing and shotcreting, which would have been required to construct a geogrid-reinforced slope. Block walls were battered backwards for aesthetics. Light reinforcement within 2H:1V slopes above the walls was eliminated to simplify construction. Reinforcement layers at inside-corner walls on the same course of blocks were separated one-half block height, with backfill in between, to enable satisfactory soil-reinforcement interaction. Drainage was installed at the heel of all block walls and at the toe of the shotcrete wall to drain water which might infiltrate the slope.

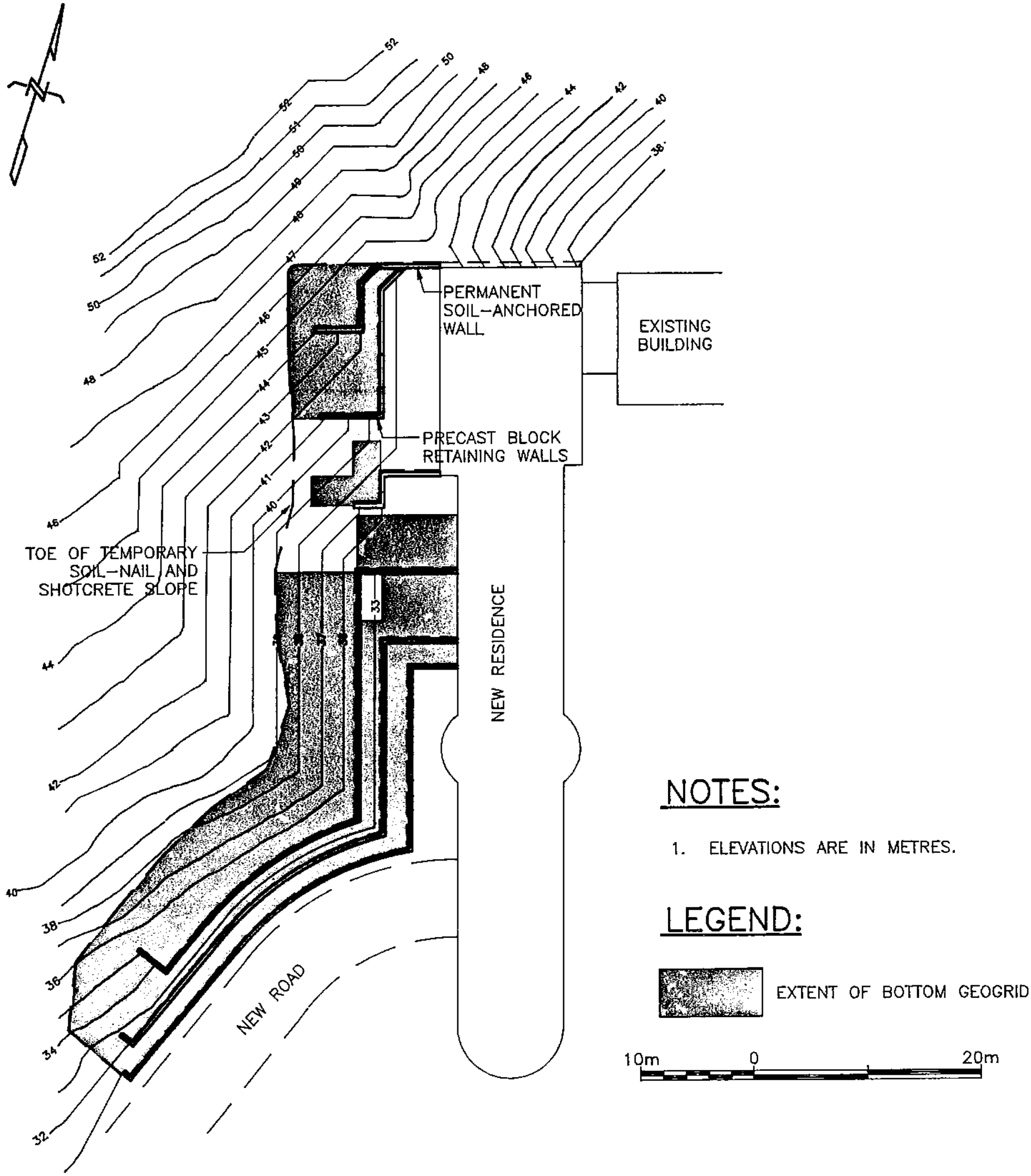
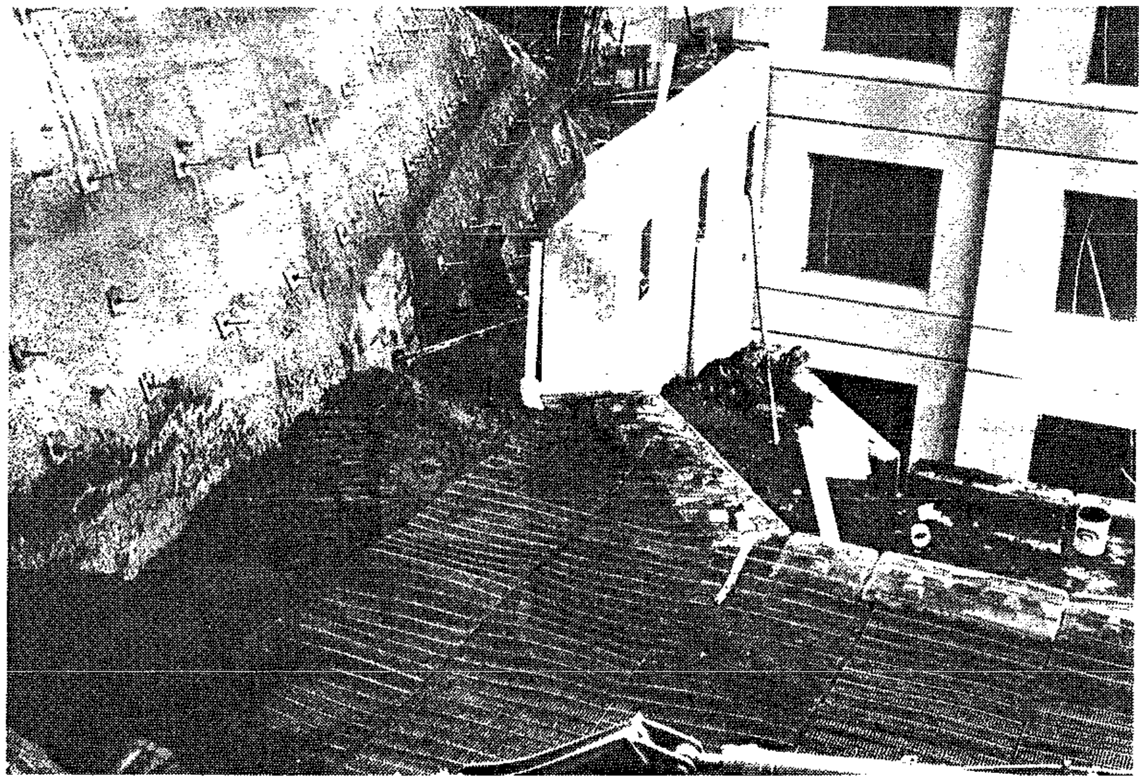


Figure 4 - Plan of Geogrid Reinforcement

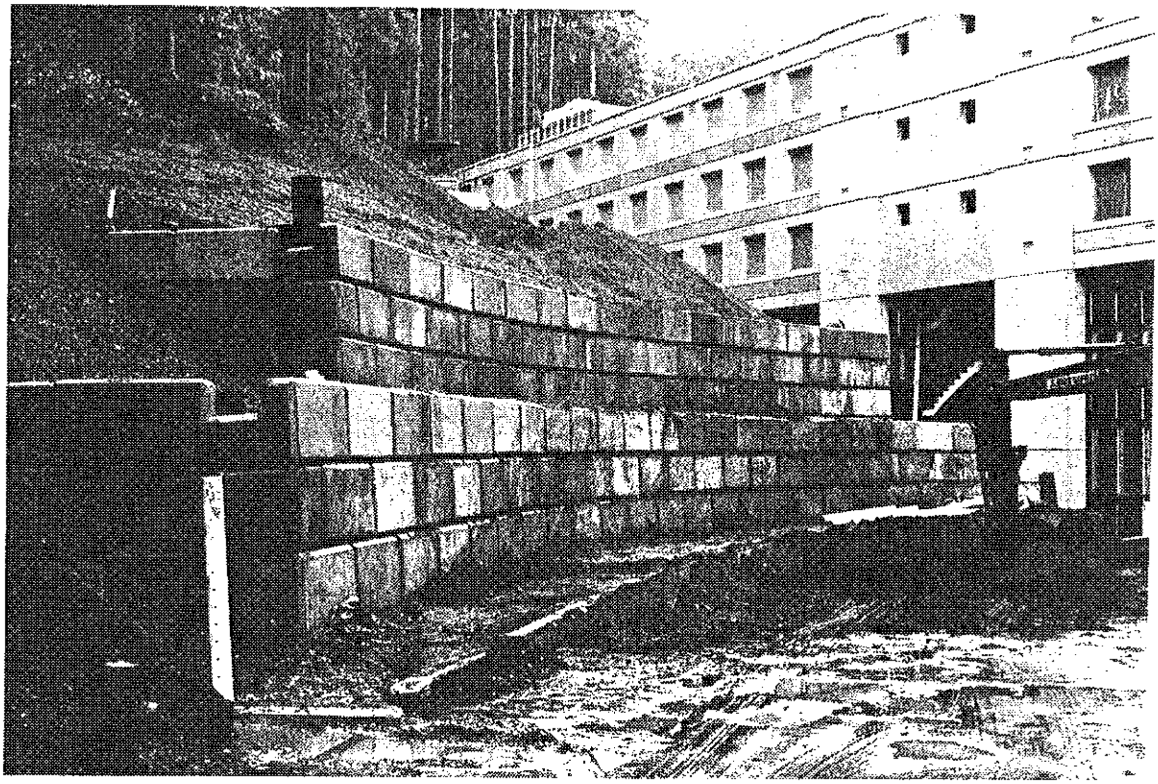


Photograph 1 - North End of Site With Soil-Nailed Wall, Soil-Anchored Wall and Geogrid Reinforcement

CONSTRUCTION EXPERIENCES

Contract drawings and specifications and construction monitoring were used to transmit design fundamentals into prototype construction. The RRMC contract documents specified both the characteristics of geotechnical materials and construction methods. The latter included preparation of block-wall foundations, reinforcement installation, and backfill placement and compaction procedures.

Lessons were learned during construction. For example, the specification that walls be battered backwards at 12V:1H was incompatible with the use of standard-size precast interlocking blocks for curved walls (Photograph 2). As wall length increased with wall height it became increasingly difficult to 'match' interlocking forms. Fortunately, dimensional tolerances of the blocks allowed adjustments to be made during placement.



Photograph 2 - Lower Precast Block Walls

The benefit of keeping things simple was also confirmed. Specification of compaction backfill requirements in a performance format eliminated compaction testing, which would have been difficult in granular soils with large particles (150 mm minus gradation). The requirement that reinforcement extend to the temporary shotcrete face simplified execution and monitoring of the work. And requesting that reinforcement types be colour-coded on site reduced the risk of similar-looking geogrid types, with significantly different strengths, being substituted inadvertently.

The contract specifications were augmented with guidance provided by the project engineer, acting as inspector, and the geogrid supplier, to enhance the quality of the reinforced-soil slope constructed. The contractor's personnel had no previous experience with reinforced-soil slope construction and yet the work was executed skilfully on a restricted site (Photographs 1 and 3).

Construction experiences confirmed the importance of an overall design appropriate for the problem, the site and the resources available. The reinforced-soil and block-wall solution chosen yielded a satisfactory engineering and aesthetic result with minimal disturbance to the original site. The solution's simple and modular character allowed it to be constructed by inexperienced personnel operating common construction equipment on a confined site with limited access. The project was completed ahead of schedule.

Photograph 3 - Site Conditions

CLOSURE

To construct foundations for a residence at the Royal Roads Military College and to achieve a particular aesthetic result around the building, cut and reconstructed soil slopes were retained with reinforcement. A system of soil nails and shotcrete provided temporary support. A geogrid-reinforced-soil-and-precast-block-wall system was used to provide permanent support.

Four typical sections were analyzed in two dimensions for static and pseudo-static conditions. Design parameters were assigned values based on test data and engineering experience. Designs based on analyses were developed further to accommodate practical considerations.

Construction experiences were instructive and showed both the benefits of simple designs and the importance of selecting engineering solutions appropriate to a problem and its context.

ACKNOWLEDGEMENTS

The Department of National Defence Canada kindly permitted publication of project information. John Kerr of Tensar Corporation and Terry Haigh of Nilex Geotechnical Products Inc. provided invaluable service during design and construction, respectively. Terry Haigh also took the photographs reproduced in this paper. Colleagues at Klohn Leonoff provided assistance throughout the RRMC project and during preparation of this paper. The contribution of all of these parties is gratefully acknowledged.

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