CASE HISTORIES OF TEMPORARY AND PERMANENT RETAINING STRUCTURES IN THE LOWER MAINLAND

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ABSTRACT

The rapid expansion of residential development in the mountainous terrain of the Lower Mainland and Fraser Valley has increased demand for temporary and permanent retaining structures. The need to create roads, and relatively level building sites on steep terrain has resulted in considerable opportunity for the introduction of innovative retaining wall concepts. This paper outlines five case histories of wall design and construction. These case histories represent a few of the many retaining options available, and illustrate some of the problems which may arise.

INTRODUCTION

The rapid expansion of residential development in the mountainous terrain of the Lower Mainland and Fraser Valley has increased demand for temporary and permanent retaining structures. The need to create roads, and relatively level building sites on steep terrain has resulted in considerable opportunity for the introduction of innovative retaining wall concepts, particularly for walls over 4.5 m high (considered to be the practical height limit for traditional gravity wall structures). The most significant advancements have occurred in the use of Mechanically Stabilized Earth (MSE) and segmental block structures, wherein retaining walls are created by reinforcing earth fill with steel strips, geosynthetics, wire mesh, precast counterforts, etc. A large number of wall facings are available.

Factors which generally influence the selection and feasibility of a wall system for a specific site include the soil conditions, accessibility, boundary conditions, topography, aesthetics and cost of construction. Design of all retaining walls, and MSE walls in particular, requires consideration of soil-structure interaction. This applies to both the internal stability of a wall system and to the effects of wall installation on the ground retained, i.e. exteranl stability, as wall installation changes the stress and drainage conditions of the surrounding soil both during construction and over the long term. Many of the wall systems are proprietary and the division of responsibilities between the wall designer and the geotechnical engineer of record for the project can be problematic, especially for high walls on sloping ground. As a wall cannot be considered in isolation of its surroundings, wall designers should be responsible for local and global stability of the wall.

This paper outlines a number of cases where the authors' company was responsible for all aspects of wall design and construction. These case histories represent a few of the

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many retaining options available, and illustrate some of the problems which may arise. The cases to be discussed are summarized in Table 1, which briefly describe the purpose of the wall, design issues which had to be addressed and construction issues that were considered or arose during construction. The case histories presented include four MSE walls using extensible reinforcement with a variety of wall fascia and one contiguous pipe pile wall.

DESIGN APPROACH

Design, based on limit equilibrium, methods, followed conventional practice, with minor modifications to take account of special conditions. The design static factor of safety for internal and global stability was 1.5 except for Case History No. 3 where a factor of safety of 1.4 was used. The external static factors of safety were 1.5 and 2.0 for sliding and overturning, respectively. For the permanent walls, seismic surcharges were evaluated using pseudo static analyses in conjunction with consideration of the relationship between displacement and the critical horizontal acceleration coefficient. The permanent walls were designed to a minimum pseudo static factor of safety of 1.1. In both permanent wall case histories, static analyses dictated the final design.

All of the MSE walls were designed with polymer (extensible) reinforcement. The design against grid pull-out and tensile failure was based on the use of interaction coefficients and strength reduction factors. The reduction (performance) factors accounted for durability, construction damage and creep effects. Biological and chemical degradation factors were not considered applicable. Interaction coefficients and performance factors were generally obtained from manufacturers' literature and were dependent on the type of backfill and expected life span of the structure. Suggested creep reduction factors were reduced for the temporary walls and the values used were based on unconfined creep performance tests supplied by the manufacturers' local representatives. The Long Term Allowable Design Strength (LTADS) of the polymer reinforcement was determined using the following expression:

$$T_a = T_{ult}*(F_D*F_{CD}*F_{CR})$$

where
T_a = Allowable Design Strength
T_{ult} = Ultimate Wide Width Tensile Strength (ASTM D-4595)
F_D = Reduction Factor for Durability
F_{CD} = Reduction Factor for Construction Damage
F_{CR} = Reduction Factor for time dependent strength loss (creep)

In addition to the performance factors mentioned above, stress-strain compatibility between the matrix soil and the earth reinforcement was also considered for the MSE walls. As extensible reinforcement requires large strains to mobilize working strengths 4% to 6% based on unconfined stress-strain testing), large strain or ultimate shear strengths of the soil matrix were employed in these designs to maintain strain compatibility.

CASE HISTORIES

B.C. RAIL TEMPORARY DIVERSION, WEST VANCOUVER, B.C.

In order to gain vehicle access to a shoreline residential development, a permanent large diameter culvert underpass was to be constructed underneath the B.C. Rail line requiring a temporary railway diversion to be constructed. The proposed diversion site was located near the toe of a steeply sloping hillside covered with mature trees and undergrowth. The diversion required embankment fills up to 3.5 m thick, except at the culvert location where excavation requirements and spatial restrictions required construction of an 8 m high wall, with the new track located no more than 1.7 metres from the outside wall face. Specific design requirements are shown in Table 1.

The soil conditions at the site typically consisted of a 1.0 to 1.5 metre overburden mantle of loose to compact sand and gravel with occasional cobbles and boulders, overlying very dense sand and silt. The sand and silt was underlain by a water bearing stratum of dense sand and gravel stratum at a depth of about 6 metres below the site surface. Excavation for the culvert was to extend about 2.0 m into the lower sand and gravel, with seepage and potential loss of ground being of concern. Specific design requirements are noted in Table 1.

Construction of the wall was completed over a period of approximately 9 days. The completed wall is presented in Figures 1 and 2. Minor ground water seepage encountered near the base of the excavation was managed by a drainage blanket and sump system. Construction of the wall was monitored on a regular basis to confirm proper placement of the reinforcement and quality of the fill material placed between the geogrid layers. Compaction of the fills was carried out with a ride on vibratory roller, or a large plate tamper where access was restricted. Two inclinometers were installed along the back of the Lock Block wall located near the centre (SI#2) and east end (SI#1) of the wall. The inclinometers were segmentally installed as the wall was constructed. Ground monitoring stakes were also placed and initial readings taken with a tape measure prior to proof loading of the wall.

Prior to the wall being placed into service, it was proof loaded using two excavators. The inclinometer readings did not capture the initial deformations as the zero readings were taken after the initial loading, but the very small movements recorded over about two weeks of service showed that the wall performed well. After completion of the culvert, the wall was dismantled and the components were used for numerous small walls in the subdivision.

TEMPORARY EXCAVATION SUPPORT, WHITE ROCK, B.C.

A proposed multifamily residential development required a relatively deep excavation along the north and east perimeters of a sloping site to achieve the design subgrade elevation for an underground parking structure. Due to the proximity of the structure relative to the north property boundary, unsupported excavation slopes could not be completed within the property boundary without what was considered a high risk of failure or large displacements.

The soil conditions at the site comprised 4.6 m to 6 m of stiff to firm desiccated silty clay/clayey silt overlying soft to firm, normally to lightly overconsolidated clayey silt which extended to depths varying from approximately 18 m to 21 m below the site surface. Ground water levels measured at the time of the drilling were near the interface between the desiccated zone and the underlying soft to firm deposits.

Stabilizing the excavation slope with conventional shotcrete and soil anchors was considered but later abandoned as an encroachment agreement could not be negotiated with the property owner on the north (uphill) side of the site. After a review of few available options, it was concluded that the most feasible method of excavation support was a contiguous pile wall as shown in Figure 3. The proposed system would address concerns such as lateral support to the cut slope, base and slope instability, as well as taking advantage of foundation pipe piles which had been delivered to the site for support of the proposed structure. The unsupported excavation height of the wall varied from about 4.4 m at the east end of the wall to about 3 m at the west end. The length of the wall was about 19 m and was offset approximately 2.1 m from the north property boundary. The back slope behind the top of the piles was 1.5H:1V (Figure 4).

Construction of the wall was carried out over a period of approximately 2 weeks during the summer of 1990. The piles were installed with a track mounted pile rig to an installation tolerance of 25 mm maximum spacing between the piles. The piles were driven open ended to reduce disturbance to the underlying soft deposits and were filled with sand upon completion of driving. Excavation proceeded from the front of the site during pile driving, with a 4.5 m berm being maintained against the wall until completion of driving. Two inclinometers were installed directly behind the wall prior to removal of the bench and slope in front of the wall. The inclinometers were located near the east end and central portion of the wall and extended to a depth of about 15 m below the top of the piles. The top of the wall moved about 13 mm relative to the base of the piles during removal of the berm and only a further 5 mm displacement was observed during subsequent pile driving and construction. These movements were considerably less than the 50 mm estimated. Ground surface monitoring stakes on the slope behind the wall confirmed that movements of the slope and the neighbouring property were negligible.

The final cost of the contiguous pile wall was reported to be in the order of $50,000.00.

Salvage of the piles was considered as a means to further reduce the cost of the excavation but it was judged that this posed a risk of movement of the nearby residential structure and they were left in place.

TEMPORARY PRELOAD, RICHMOND, B.C.

To reduce the post construction settlements of a proposed twin residential tower, 7.6 m high preloads were designed. In order to achieve the required loading over the full building envelope of each tower, it was necessary that the side slopes of the preloads be about 1H:3.5V immediately adjacent to the property boundaries. Consequently, some form of temporary slope retention was necessary. Site conditions were typical of the Richmond area, being essentially flat and underlain by recent alluvial deposits comprising surficial compressible silts over loose to compact fine to medium sands. The sands gave way to silts at about 40 m below grade, which extended to a great depth.

A river sand fill reinforced with uniaxial geogrids was designed. The original design called for the slope face to comprise a non-woven geotextile wrap placed in direct contact with the geogrid. The vertical spacing between the geogrids (and height of wrap) was constant at 0.6 m. Guidelines for construction were provided, but the details of forming the face were left to the contractor. The successful bidder, who had no experience of MSE wall construction, commenced construction using L-shaped forms fabricated of heavy aluminum sheeting (3 m long and 600 mm high) to create the battered face of the slope. The soil was packed tight against the face of the form. In the early stages of construction, failure of the wrap was reported in the form of numerous bulges along the face of the wall and occasional complete pull-out of the grid which had resulted in over steepening of the fill. These problems were found to be associated with the removal of the forms. In addition, the geotextile wrap had been slashed in numerous places by vandals leading to the escape of backfill.

The wall was re-designed as shown in figures 5 and 6. The lower 2.5 m of the slope was to be vertical, consisting of 3 courses of Lock Blocks to reduce the opportunities for vandalism, and permit a flatter slope over the remaining height of the preload. The forms were modified to reduce the surface area in contact with the geogrid by 90% to reduce pull-out problems and only nominal compaction was specified within 600 mm of the form face. The original preload was removed and the new design was implemented. Review of the wall after completion of the preload to full height indicated apparent outward rotation at the top of the vertical section. This outward rotation apparently occurred during placement of the Lock Block wall. Subsequent monitoring of the wall indicated that the wall was actually moving into the preload as a result of preload settlement as shown in Figure 6.

Construction of an identical preload in Phase 2 also taught some lessons. Construction of the initial 4.5 m of preload proceeded without field review by the consultant. The geogrid from Phase 1 was reused. Evidence of the use of damaged geogrid led to instructions for the preload to be dismantled to the top of the Lock Block wall. The upper portion of the preload was redesigned on the assumption that the grid behind the Lock block wall had a 50% reduction in efficiency. Additional lengths of geogrid were added in the upper portion. The Phase 2 preload has performed satisfactorily to date.

REMEDIATION OF SLOPE FAILURE, WEST VANCOUVER, B.C.

In March 1994, a large land slump occurred in the backyard of a private residence in West Vancouver during an extreme rainfall event. A significant portion of the property moved and the slump threatened to retreat further towards an existing residential structure. Temporary remediation of the slope was achieved by grading the slope to 1H:1V which resulted in a 5.8 m high slope with the crest being only 1.8 m to 4.3 m from the rear of the structure. The soil conditions on the cut slope consisted of compact granular fills with occasional boulders, construction debris, logs and other organics. The fill extended to the base of the cut slope and was underlain by very dense glacial till soils. Minor ground water seepage existed near the base of the cut slope. The owners wished to maximize the lateral extent of the backyard.

Options for reconstruction of the slope were severely restricted due to limited access for equipment, a West Vancouver By-Law which prohibits any wall from extending above a line projected at 1H:1V from a point 1.2 m above ground at the property line, and requirement to tie into an existing mortar and granite wall which had been constructed in three benches ranging from 1.5 to 1.7 m high. The selected wall type was a 3 to 4 tier MSE wall constructed of uniaxial geogrids and faced with a locally available landscape block. The wall configuration is shown in Figures 7 and 8. The wall components could be moved by hand and bobcat. Encroachment agreements were required to allow storage of materials within a reasonable distance of the site and to allow sufficient room for placement of the geogrids.

Excavation slopes in the random fills were cut back at 1H:2V slopes to provide sufficient room for the placement of the geogrids. Surface ravelling was prevented by driving rebar dowels into the slope with a pneumatic hammer to secure welded wire mesh and polyethylene sheeting to the slope. An articulated conveyor system was used to transport the imported backfill material which was placed in 300 mm lifts and was compacted with a 4 cycle plate compactor. Density testing and construction monitoring were carried out on a regular basis. The wall was completed over a period of 5 weeks for a cost of about \$36.00/ft^2.

PERMANENT VEGETATED FACE MSE WALLS, WEST VANCOUVER, B.C.

A subdivision near the west end of West Vancouver District required numerous 2 to 4 metre high retaining structures to support cuts and fills required to achieve roadway and driveway access into and around the development. The style of wall was to be consistent throughout the subdivision and concrete, metal or modular block facing were not considered desirable by the developer. The site was relatively steep with occasional bedrock outcrops. The soil conditions typically consisted of a 1 m to 4 m thick layer of loose to compact silty sand, gravel, cobbles and occasional boulders over dense sand and gravel with occasional boulders. Occasional ground water seepage was evident.

A vegetated face MSE wall was selected as shown in Figures 9 & 10. Native sands and gravels were used as backfill and uniaxial geogrid was used as reinforcement. The front face of the walls was contained by welded wire mesh forms which were lined on the inside with biaxial grid and geotextile. A 200 mm wide zone of topsoil contained within the geotextile was to be planted with Ivy or Dogwood shoots.

During construction, delays to the overall development required a change in the planned vegetation to a grass seed to provide temporary growth along the wall face. Grass seed was mixed with the topsoil resulting in very rapid growth and cover of the facing elements as seen in Figure 10 Figure 11 shows the wall 1 year later an illustrates the difficulty of maintaining steep grass covered slopes.

CONCLUSION

The case histories have illustrated some options available for temporary and permanent retaining walls and have illustrated some of the pitfalls to be avoided. In particular, it is clear that working through the construction procedures with the contractor in advance can greatly reduce the potential for delays during construction. For temporary construction and phased developments, the issue of re-use of geogrids has been raised. Although satisfactory outcomes have occurred in the two case histories cited, the effects of previous use will be greatly dependent on the specific conditions of that use and so it is suggested that re-use of geosynthetics should be considered with caution and only in comparatively low stress environments.

TABLE 1
Case History Summary

Case No.	Project	Purpose	Design Issues	Wall Details
1	B.C. Rail Diversion, West Vancouver	Temporary railway diversion to allow construction of a permanent railway underpass.	Owners requirements - Materials used in diversion wall were to be re-cycled for smaller walls in the subdivision. Design Requirements - Wall was to support loads generated by two 2500 kN locomotives followed by a 115 kN/m line load. The wall alignment was controlled by the local geometry and maximum curvature criteria provided by B.C. Rail.	Wall Type: MSE - Geogrids & Lock Blocks Wall Maximum Height: 8 m Wall Length @ Crest: 24 m Wall Length @ Base: 7.5 m
2	Excavation Support, White Rock	Temporary support to an excavation slope.	Owner's Requirements - Provide maximum access for equipment in front of wall. Design Requirements - Minimize ground displacements of uplsope property.	Wall Type:Contiguous Pile Wall Wall Height: 10 ft to 14.5 ft. Wall Length: 63 ft.
3	Preload Construction, Richmond	Precompression of a site proposed to support a highrise.	Design Requirements - Steep slopes required to stay within property boundaries, soft foundation conditions.	Wall Type: MSE - Geogrids & combination Lock Blocks and Fabric wrapped facing. Wall Height: 7.7 m Wall Length: 35 m
4	Slope Failure Remediation, West Vancouver	Provide permanent support to an unstable fill slope.	Owner's Requirements - Maximize the lateral extent of the backyard and provide pleasing aesthetics. Design Requirements - Meet West Vancouver's By-Law for retaining wall configuration.	Wall Type: MSE - Geogrids & Landscape Blocks Wall Height: 8 ft. to 22 ft. Wall Length: 105 ft.
5	Subdivision Access Roads, West Vancouver	Provide permanent support to subdivision road fills.	Owners Requirements - Wall aesthetics to blend in with natural environment.	Wall Type: MSE - Geogrids & fabric wrapped facing Wall Maximum Height: 4 m Wall Lengths: Variable

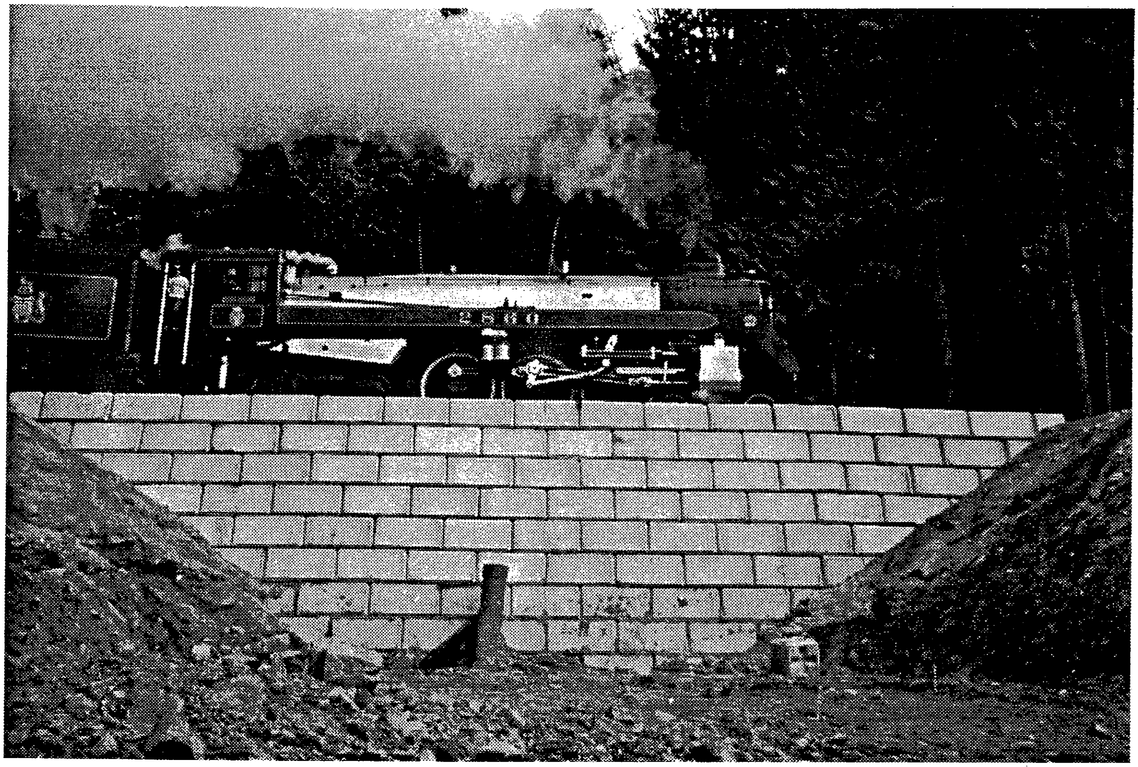

Figure 1 - Completed Wall

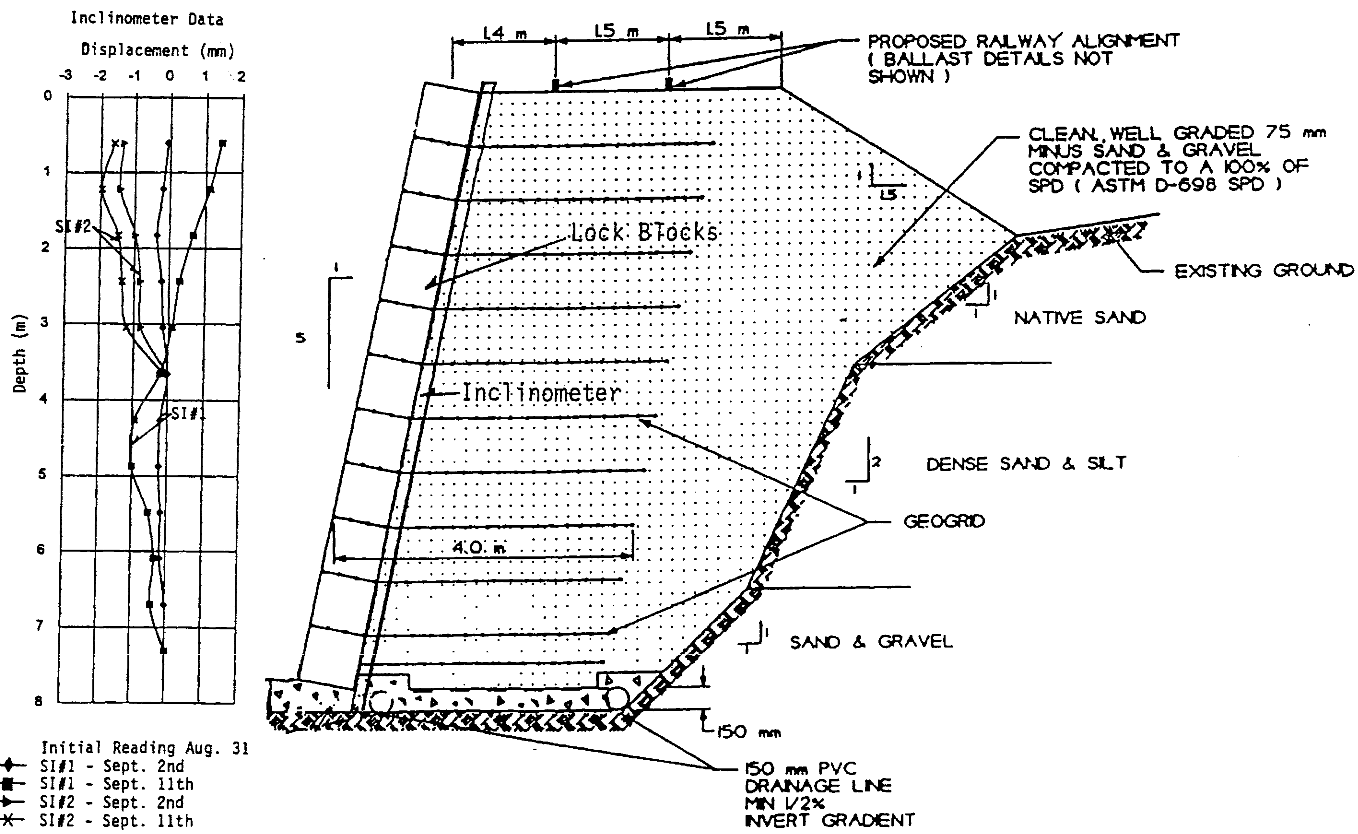

Figure 2 - Section Details & Displacement Map

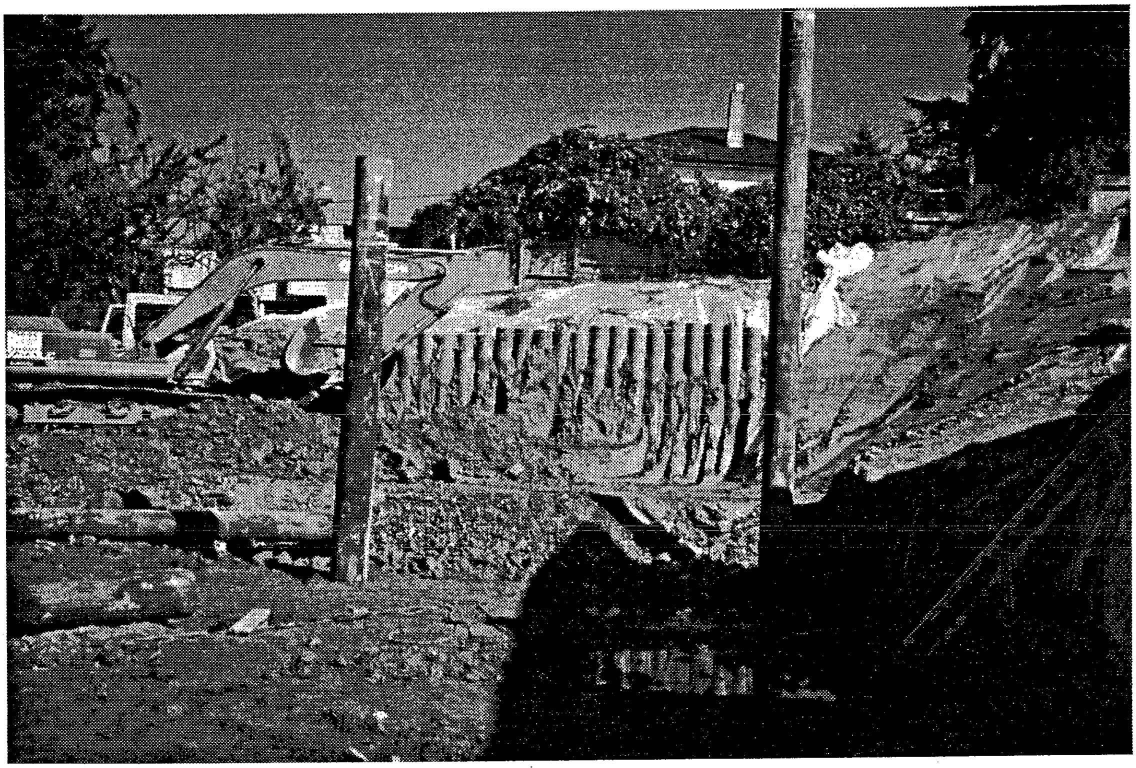

Figure 3 - Partial excavation in front of wall

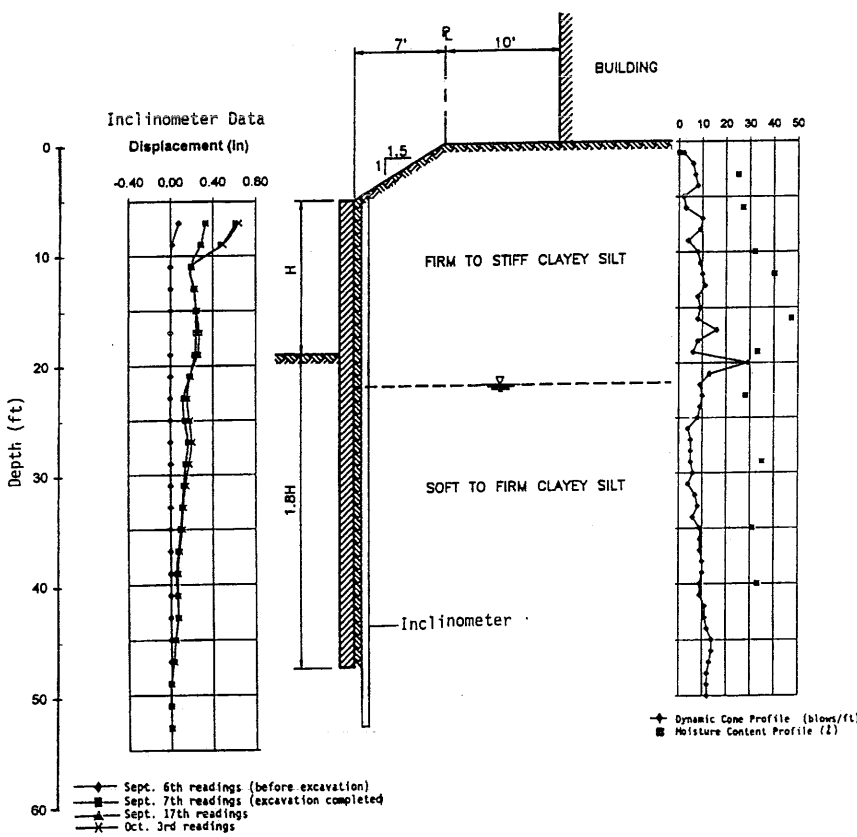

Figure 4 - Section detail and displacement data

Figure 5 - Preload configuration

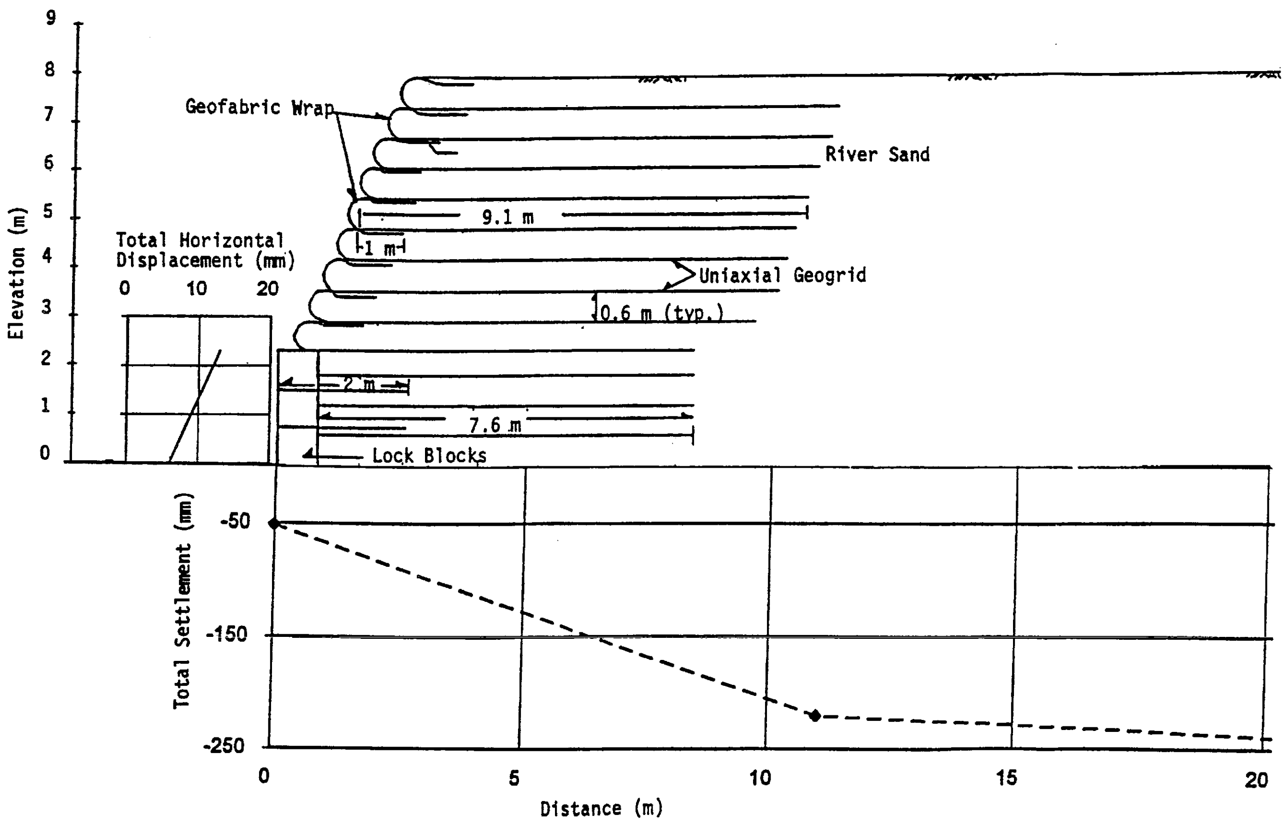

Figure 6 - Section, displacement and settlement details

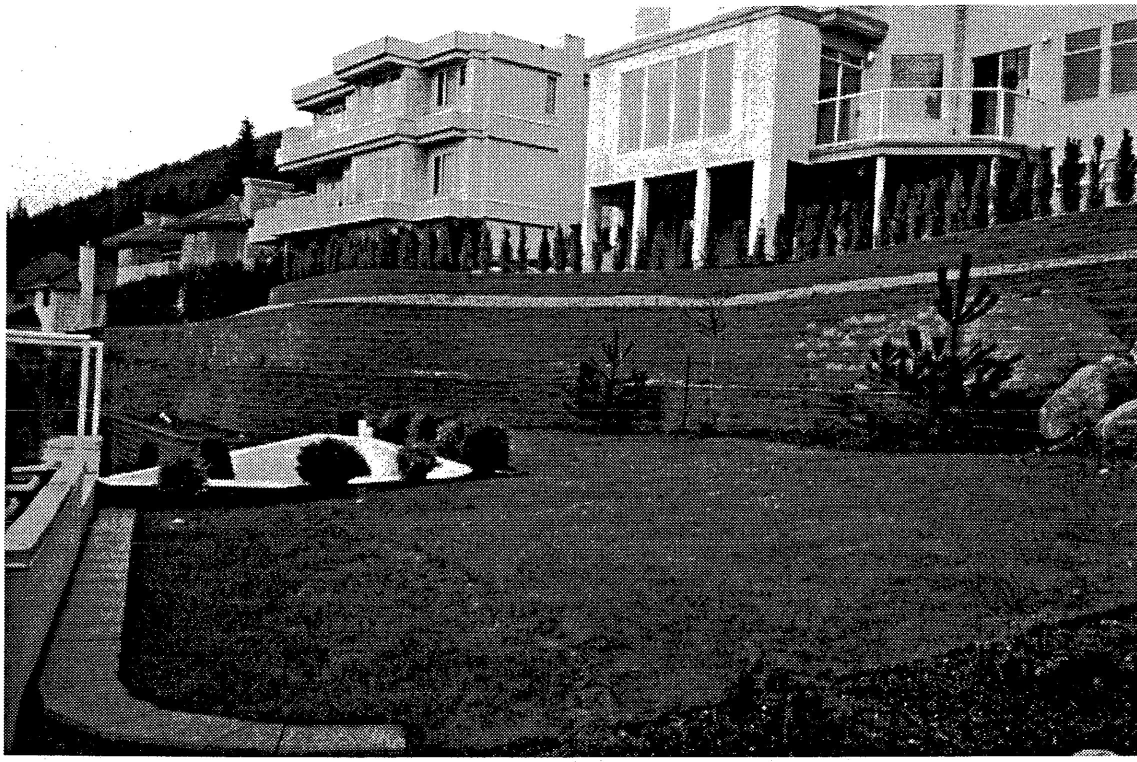

Figure 7 - Finished Wall

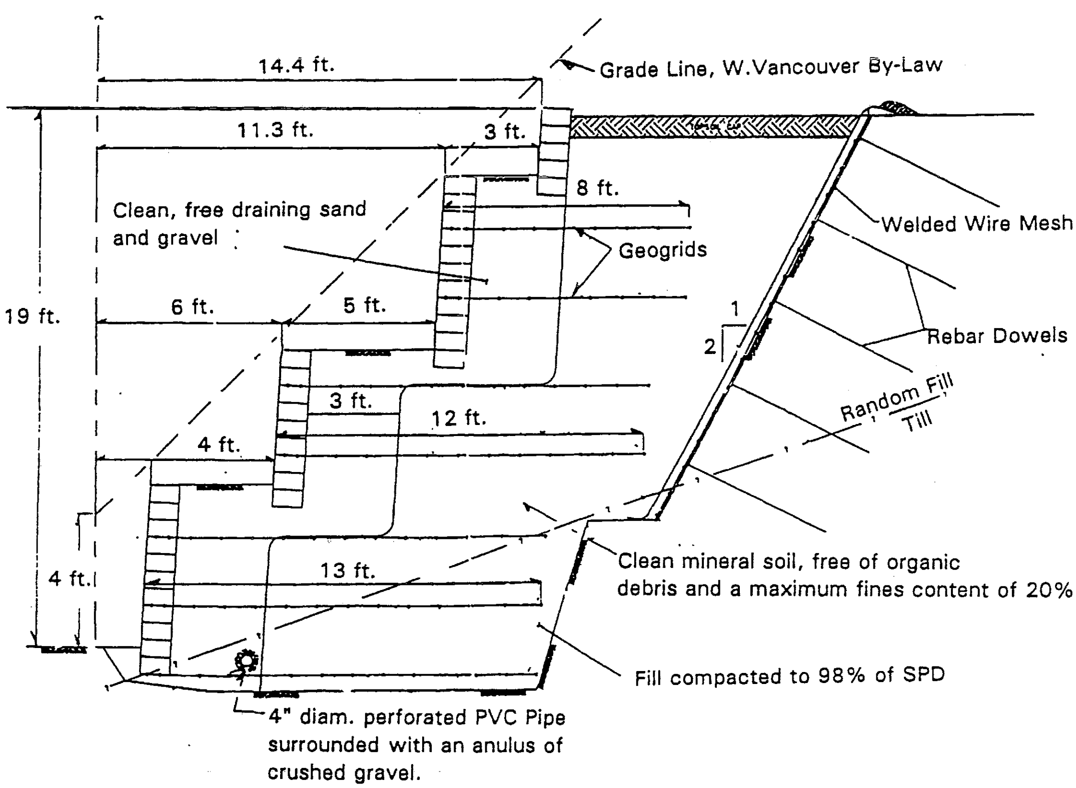

Figure 8 - Section detail

Figure 9 - 4 m high wall 1 month after completion

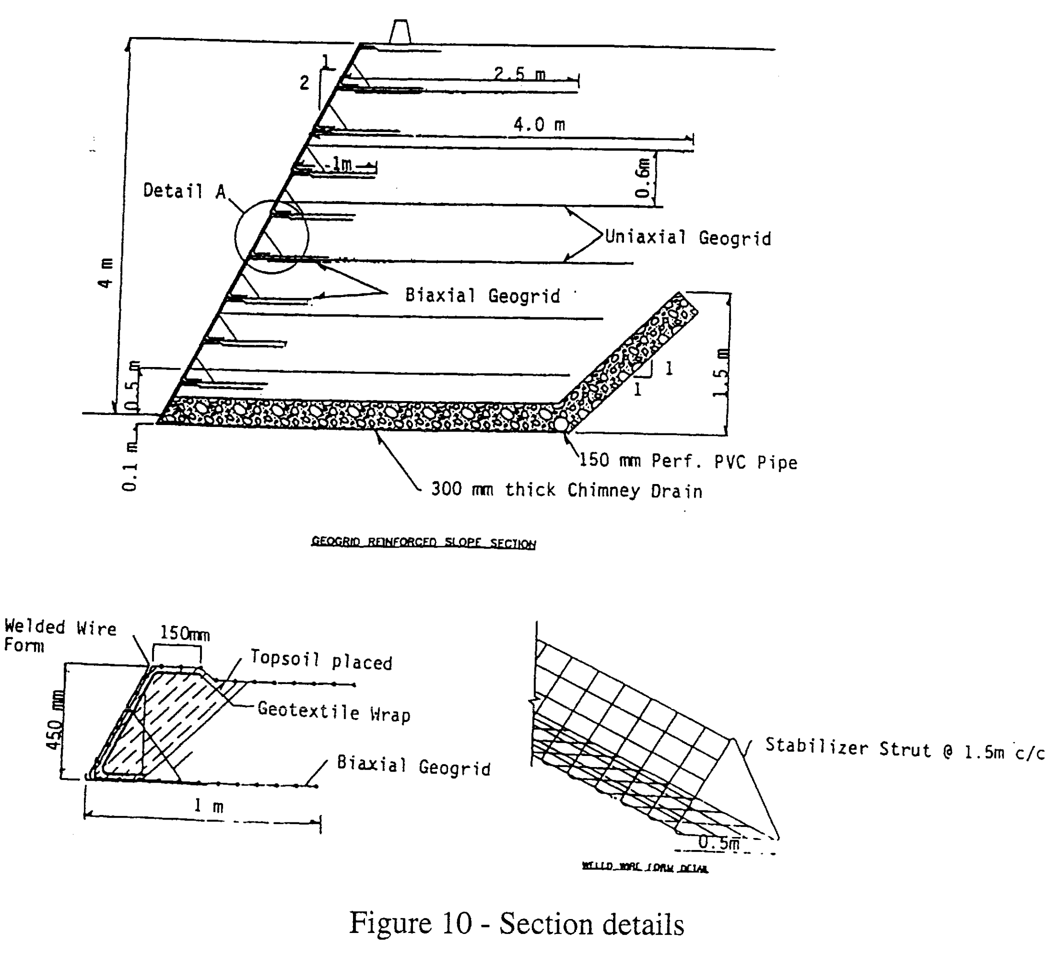

Figure 10 - Section details

Figure 11 - 1 year after construction