The Reinforced Soil Slope at Foley Creek, British Columbia

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Abstract
A section of road failed on the Foley Creek Forest Service Road, near Chilliwack, British Columbia. Evaluation of the options for repair of the section led to its rehabilitation, in May 1995, using techniques of reinforced soil. A 3.5 m high, geogrid reinforced soil slope was constructed using local soils. The geometry and arrangement of the reinforcement in the slope are described, and issues related to details of the facing then addressed. Following this review of the design, emphasis is then placed on the construction procedure followed in the field. A summary of costs is presented, together with comments on the subtotals and a discussion of the relevance to other similar sites. The performance of a similar instrumented sloped reinforced soil wall is then examined, with the intent of outlining "best practices" for such structures.

Introduction
Effective approaches to access road construction in landslide-prone terrain require economically and environmentally sound solutions. Common problems include maintenance of cut slope stability, improvement of slope stability, and control of fill slope stability (USFS, 1994). Solutions often involve a combination of careful route selection, slope drainage, reshaping of slopes, and at some locations the use of soil or rock retention systems.

Soil retention systems used in resource engineering include gabions, bin walls, crib structures, anchored and cantilevered H-piles, and geosynthetic reinforced soil. An informal review of over 30 retaining wall failures reported by the United States Forest Service (USFS, 1994) reveals a common series of contributing factors associated with low volume roads:

(1) Many walls are designed for steep, rugged terrain with no geotechnical investigation or involvement in construction. Most of these sites involve a combination of poor soils, wet conditions, steep slopes, and slope stability problems.

(2) Many designers are not familiar with requirements of walls in steep terrain. As a result, wall ends are not tied into soil natural ground, adequate bench width or embedment is not provided, and poor quality soils are used for backfill and walls are not properly drained.

(3) Adequate construction inspection is not provided. Many inspectors are not familiar with wall design and are unable to make design changes needed in the field. Often the contractor is left with little direct guidance from the agency. Important aspects of construction such as compaction, drainage, and a review of foundation conditions prior to construction of the wall are unheeded.

(4) Many contractors for low volume road agencies are inexperienced in wall construction.

(5) Proprietary earth retention systems are very conservatively designed and failures are usually related to foundation, backfill and drainage factors.

This paper describes the selection, design and construction of a soil retention system at Foley Creek, British Columbia. The geosynthetic reinforced soil structure was built to reinstate site access at the location of a fill slope failure on a B.C. Forest Service road (FSR). Aspects of the design, construction and project costs are examined. Experience gained on the project is evaluated, and used to make recommendations for future applications of geosynthetics in reinforced slopes and walls for soil stabilization.

Site description
This section of the Foley Creek FSR was built around 1975 to provide access along the south side of the valley, near Chilliwack, British Columbia (see Fig. 1). The geological history of the region is heavily influenced by the Fraser Glaciation, and by subsequent processes. The surficial materials are

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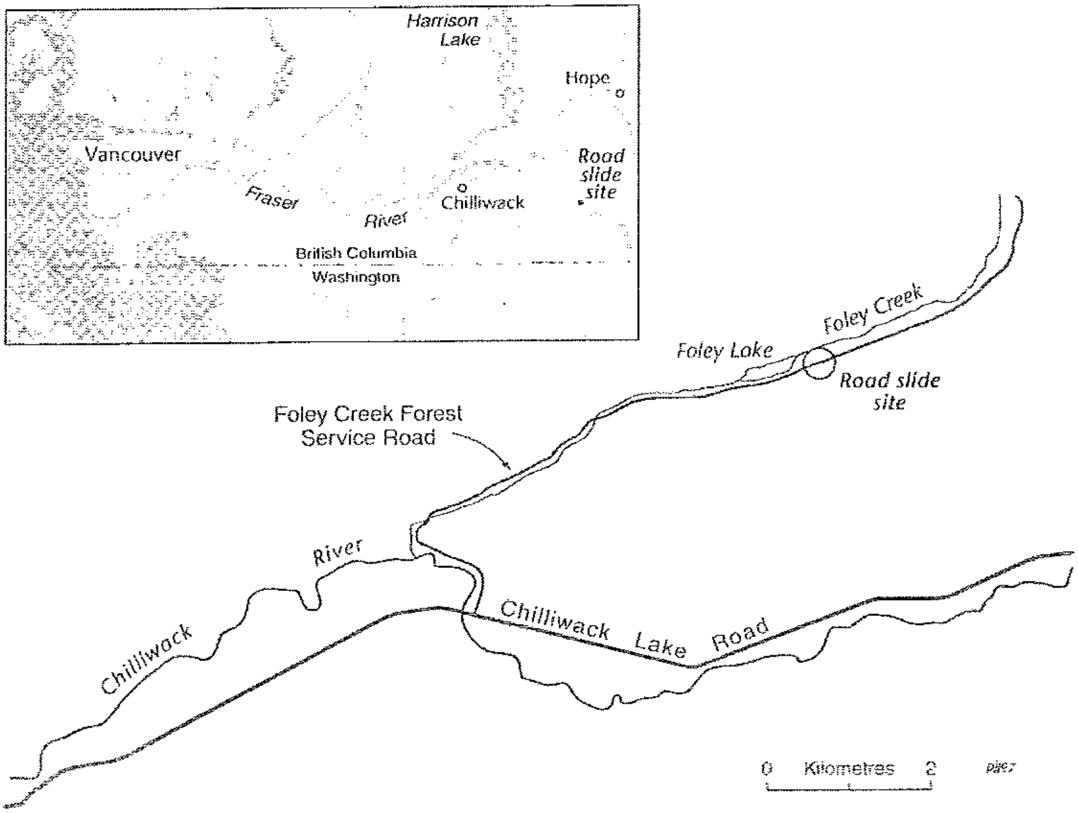


Figure 1. Site location.

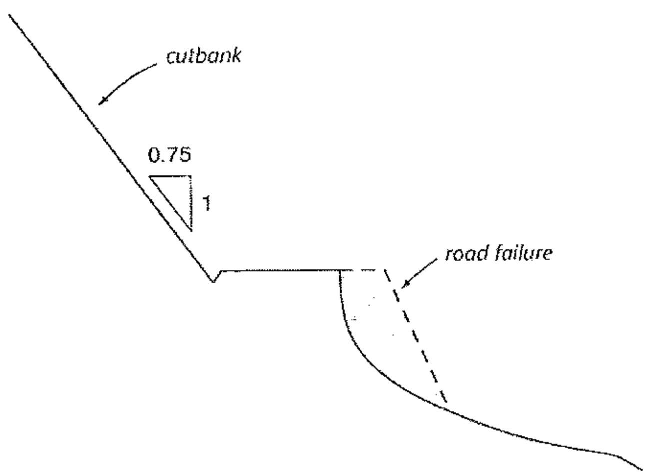
characterized by a relatively thick till blanket overlying the Chilliwack Group bedrock, which comprises a variety of volcanic and sedimentary rocks and metasediments of the Pennsylvanian and Permian period (Thomson, 1997).

The road was built by bulldozer and comprises a partial bench construction, with steep cut slopes (1V:0.75H), and moderate-to-steep fill slopes as a result of sidecasting excess soil and rock (see Fig. 2). The access supported harvesting activities in the watershed by a forest licencee, under the small business scheme, in 1994. The harvesting system used was a high lead tower, with yarding to landings adjacent to the road. Since then, the road has been used by the Small Business Program, and for silvicultural activities and recreation access by the public.

A failure occurred in February 1995, where the road traverses the lower part of a cutblock with hillslope gradients approximately 80 % (38.6°). The failure occurred in the sidecast fill and extended back 2 m into the main prism of the road. It was nearly 22 m long, and 6 m deep at the lowest point in the bedrock profile. It resulted in closure of the road for log hauling. The mass wasting event is characterized as a debris slide that progressed, with further entrainment of soil, rock and woody material, to a debris flow. It followed a path downslope toward the valley floor, a floodplain area used for camping and fishing. The path geometry causes it to be characterized as a Type 1 event according to Fannin and Rollerson (1993, 1996).

The failure was likely triggered by groundwater seepage along the bedrock surface. There is some evidence to suggest the failure occurred at a low point in the bedrock profile, where the shape of the bedrock surface would tend to promote a local convergence of groundwater flow. The influence of

Figure 2. Schematic diagram of the road failure.



hillslope topography on groundwater flow is well-documented (Pierson, 1980; Iverson and Major, 1986; Hetherington, 1987; Reddi and Wu, 1991; Jackson and Cundy, 1992). It is postulated that the failure was due partly to overloading of the slope by sidecast material, and partly to the inadequate support of that fill by buried stumps and logs. The failure plane is believed to be coincident with the underlying bedrock surface. The role, and any contributing influence, of logging activities on the road failure are difficult to discern at this site.

Construction options for soil stabilization

Given the need for continued site access to areas of active logging and for silivicultural treatment of harvested areas in the watershed, and the anticipated high recreational use of the valley, several options were considered for stabilization of the failed section of road. Realigning the road was considered, but quickly found to be undesirable given the existing high, steep cutbanks into the 80 % hillslope, the proximity of rock to the ground surface, and the lack of a suitable spoil area, all of which made this a prohibitively expensive solution. Consequently a soil retention system was selected for the oversteepened fill slope, to maintain the existing road alignment and grade. The approach was used in combination with new drainage provisions to address concerns for groundwater seepage.

Soil retention systems appropriate to the site include a bin wall, gabion structure and sloped reinforced soil wall. A consideration of site constraints, and desire to evaluate the potential for reinforced soil techniques for repair of sidecast failures, led to the final selection of the geosynthetic reinforced soil structure. Soil reinforcement is a simple concept that has been extensively used in transportation applications, and can be considered a "common practice". Reinforced soil structures require minimal foundation preparation and can accommodate significant post-construction differential settlements. The major advantages of this construction technique are ease of use, economy of materials, use of standard road construction equipment, and aesthetics of the finished structure.

Description of the structure

The structure, as designed, was a sloped (1V:0.5H) reinforced soil wall 3.5 m high. Such walls act as gravity retaining structures, and use the mass of the reinforced zone behind the facing to resist forces imposed by the retained backfill and external loadings. The arrangement and spacing of the geosynthetics are shown in Fig. 3. The longer primary reinforcement is a uniaxial geogrid, for which properties are reported in Table 1. The main function of the primary reinforcement is to provide tensile strength to the soil, and internal stability to the composite structure over the specified design life.

Table 1. *Properties of the geogrid reinforcement.*

Foley Creek Structure				
Product	Aperture MD	size (mm) CD	Wide width strength MD ult. (kN/m)	Type
Stratagrid 600	23.0	53.1	94.9	Polyester geogrid with PVC coating of fibers
Stratagrid 300	15.2	39.6	42.3	
Stratagrid 200	19.1	18.3	32.1	
Trevira 018/200	NA	NA	14.5	Polyester geotextile
Norway Structure (equivalent product data)				
Tensar UX1400	156.0	16.0	54.0	Punched and drawn polyethylene sheet
Tensar BX1100	28.0	38.0	12.5	

Notes:

MD = machine direction, CD = cross-machine direction

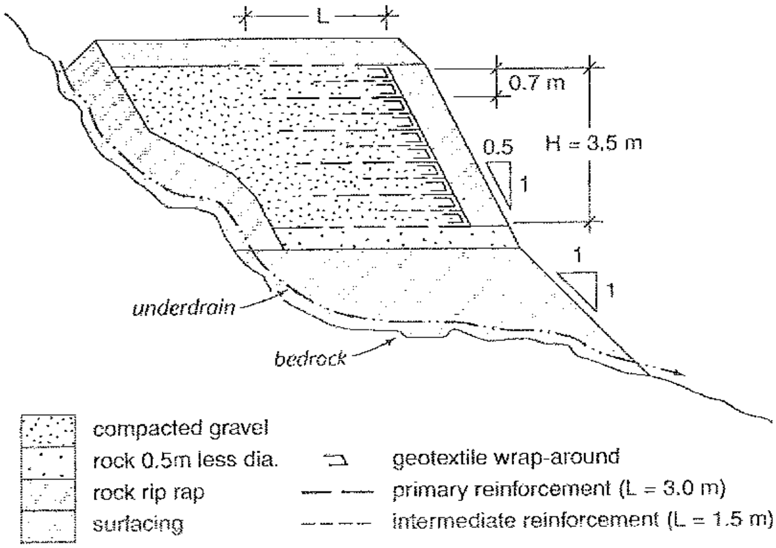


Figure 3. Foley Creek reinforced soil slope : cross-section view.

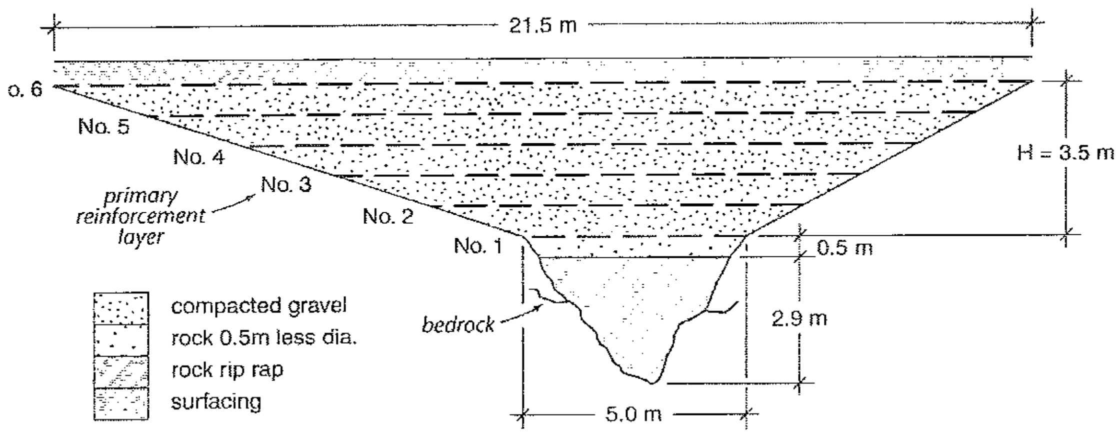
The shorter intermediate reinforcement is also a uniaxial geogrid (see Table 1), and is used solely to provide for localized stabilization adjacent to the facing.

A nonwoven geotextile, for which material properties are also reported in Table 1, was used in a wrap-around between the layers of geogrid to retain the compacted soil and behind the rip-rap facing. The unbound aggregate surfacing of the road, approximately 0.5m thick, was placed directly on the crest of the structure (see Fig. 4). The smooth bedrock profile was found to slope outwards, but was used to establish a suitable foundation for the structure.

Vehicle loading on the road, during log hauling, typically comprises an L-45 on-highway logging truck. An L-60 truck was used for purposes of design of the reinforced soil structure. Axle load configurations are reported for the L-60 truck in Fig. 5. The influence of points loads, such as wheel loads, may be evaluated in one of two ways in design. In a simplified approach the vehicle loading is treated as a uniform surcharge loading on the crest of the structure, and a uniform lateral pressure determined; alternatively, and using the principle of superposition and influence factors for point loads, the lateral thrust from the series of axle loadings can be determined and the resultant assumed to act at a given elevation above the base of the structure.

The design procedure for reinforced soil retaining walls involves evaluating the external stability of the structure for overturning stability, sliding stability and bearing stability as is routine for mass-gravity structures, together with an overall assess-

Figure 4. Foley Creek reinforced soil slope : front profile.



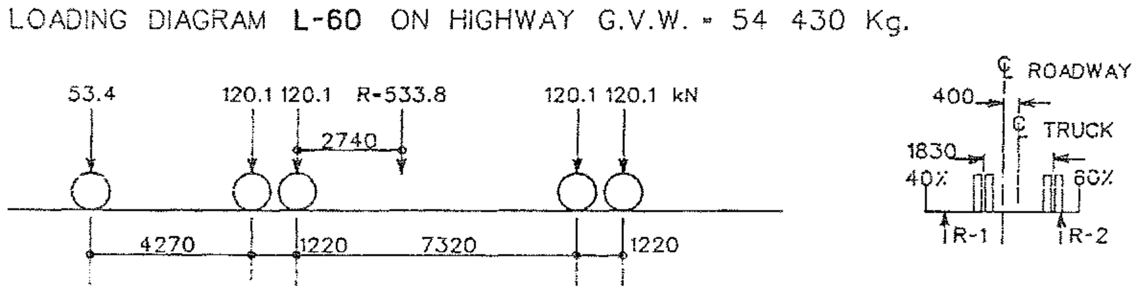


***Figure 5.** Vehicle loading diagram for L-60 logging truck.*

ment of the global stability. Additionally, an assessment of internal stability must be completed to ensure the reinforcement is adequately anchored, and to check that the force in each layer of reinforcement does not exceed the allowable long-term design strength, including a consideration of any connections at the facing. The type, length and vertical spacing of the reinforcement layers are established to satisfy these requirements, for target factors of safety. Limit equilibrium analyses are used in design, for which detailed procedures have been published (Christopher et al., 1990; USFS, 1994)) and software developed by the manufacturers of the geosynthetics.

The arrangement of the reinforcement, as specified by the geotechnical design engineer, comprised 6 layers of primary reinforcement, 3.0 m long, at a vertical spacing of 0.7 m.

The intermediate geogrid reinforcement, 1.5 m long, was also placed at a vertical spacing of 0.7 m, and the nonwoven geotextile wrap-around was embedded a nominal distance of 1.0 m.

The outsloping bedrock profile exposed during construction, and convergence of groundwater flow at the site, led to the structure being founded on a prepared rock mat underlain by a substantial drainage blanket that was taken up the back of the retained fill to provide a sloped drain.

Construction procedure

Construction of the reinforced sloped wall took place between 19 May and 22 May, 1995. Excavation to bedrock below the road failure on 19 May, 1995 revealed the longitudinal profile narrowed too much to allow for proper seating of the wall and placement of the base layer (No.1, see Fig. 4) of geogrid reinforcement. Consideration was given to blasting the bedrock to create a foundation pad, however the presence of members of the public camping below led the supervisory engineer to the preferred option of placing the underdrain and rock mat (see Figs. 3 and 4). The underdrain comprised angular rock that was keyed into the bedrock profile. This represents a variation from the design to account for site conditions.

The base layer of reinforcement placed is a Stratagrid 600 uniaxial geogrid which, although not necessary for purposes of design, was substituted from excess material at another project following on-site consultation. The backfill soil was placed and spread with a rubber-tyred backhoe/front-end loader (J. Deere, model 290D), working in tandem with a tracked excavator (Hitachi 270). The soil was placed in loose lift thicknesses between 0.2 and 0.35 m, at the natural moisture content, and compacted with a small vibrating plate. Compaction was carried out to the satisfaction of the supervising engineer.

The primary geogrid reinforcement for all layers except the base is a Stratagrid 300. Intermediate reinforcement, a Stratagrid 200, was placed at the same vertical spacing of the primary reinforcement, but on alternating lifts of backfill soil. The geogrid is supplied in rolls 2m wide by 100 m long. It was placed with the machine direction (strong direction) perpendicular to the slope face, with adjacent panels overlapped slightly to ensure continuity of coverage.

The original concept of the geotechnical design engineer was to have the reinforcement taken to the sloped face of the structure, and use biotechnical methods of stabilization in combination with the nonwoven geotextile wrap-around to provide for surface stability. The geotextile used is a Trevira ProEarth 018/200, manufactured in rolls 4.6 m wide and 91.4 m long. It is a continuous filament needle-punched nonwoven fabric, manufactured with 10 % post-consumer recycled PET resin. At the time of construction, and taking into consideration the on-site availability of rock and concerns for adequate compaction of the backfill soil at the face of the structure, the supervisory engineer elected to key large boulders into the backfill soil and create a rip-rap facing to the structure. The rock was available within 200 m of the job, without any requirement for drilling or blasting. Although this use of on-site materials had little significant impact on cost for this project, it may at other sites: alternative facing systems exist and are discussed below for future applications.

Costs

Construction was completed in 3 working days, including site preparation and excavation, as well as placement and compaction of the backfill soil and drainage layers. A breakdown of direct project costs for materials, equipment, labor and supervisory engineering services is given in Table 2. Equipment costs account for 56% of the total direct costs, and the geosynthetic materials comprise 27%. Labor accounts for 7% and the engineering supervision for 10%. The component of these equipment, labor and engineering supervision costs attributed to placement of the rip-rap facing is estimated to be in the range 15 to 20 % of the total direct costs.

Indirect costs that are not explicitly addressed in this summary include the provision of engineering design services by the manufacturer of the geogrid, through the local distributor of the product. All backfill soil and rock used in construction were available on-site.

Long-term behavior of reinforced soil structures

Considerable attention has been given to the long-term performance of geosynthetic reinforced soil structures by regulatory agencies, to properly address the approval of proprietary systems, geosynthetic products and any limits to construction applications. Currently the British Columbia Ministry of Transportation and Highways (BCMoTH) is reviewing its height restriction of 5 m on permanent vertical or near-vertical wall structures. The Washington State Department of Transportation (WSDOT) has recently relaxed its 5 m height restriction on such structures, given pre-approval of materials on a qualified products list and submission of materials test data on tensile strength and creep behavior. Many structures exist

***Table 2.** Project costs at Foley Creek.*

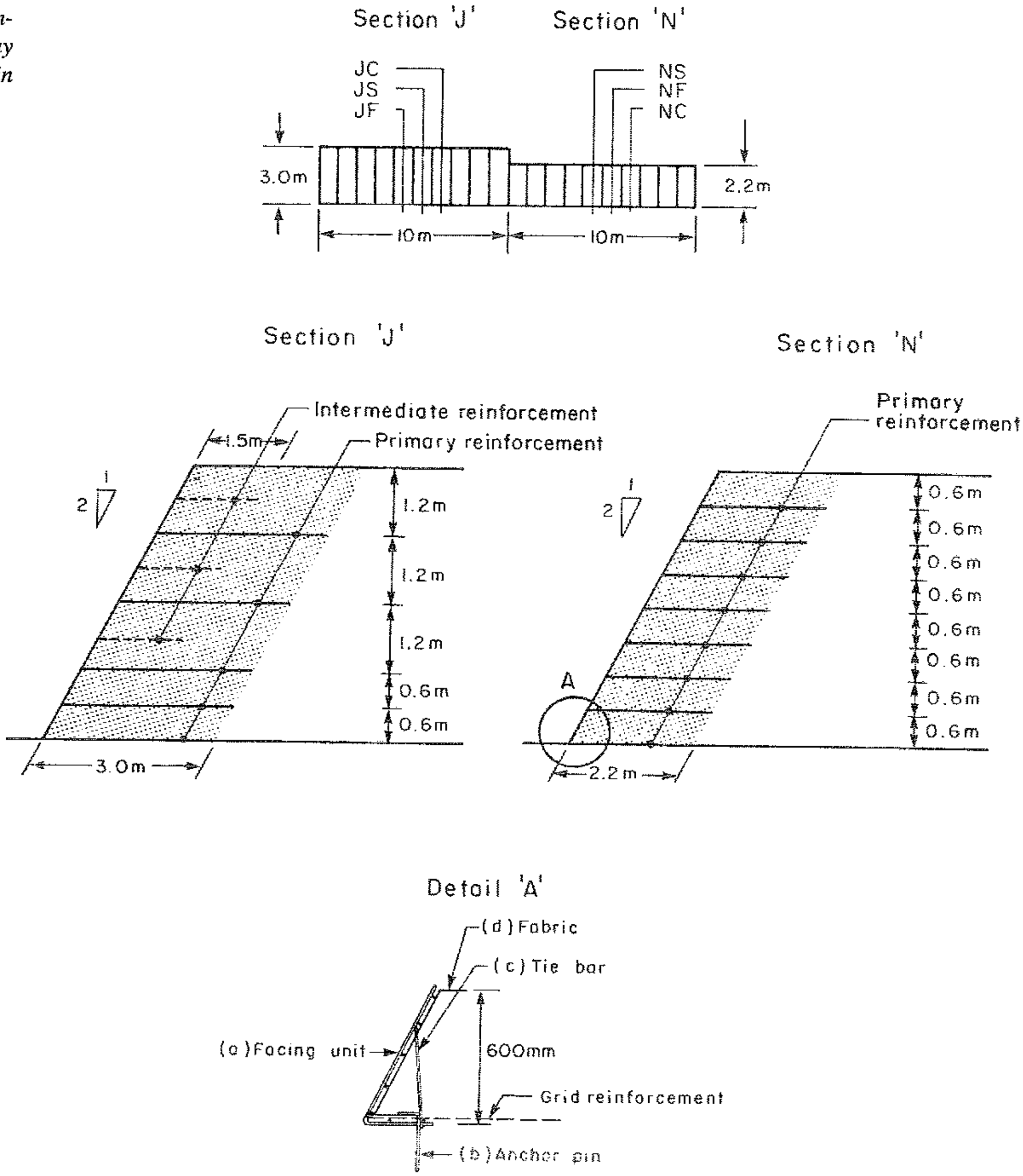
TABLE 2 Project costs at Foley Creek

Item	Unit Cost	Quantity	Subtotal ($)	Total
Equipment				
Hitachi 270,tracked excavator	$140/hr	36 hrs	5040	
J. Deere 290D, backhoe/front-end loader	$ 60/hr	20.5 hrs	1230	
Truck	$ 70 hr	36 hrs	2520	
Compressor	$130/day	4 days	520	
Generator	$50/day	4 days	200	
Vibrating plate compactor	$125/day	4 days	500	$ 10,010
Labor				
Foreman	$30/hr	18.5	555	
General	$20/hr	35	700	$ 1,255
Engineering Supervision				
Engineer	$450/day	4 days	1,800	$ 1,800
Geosynthetic Materials				
Geogrid Stratagrid 600	$ 34.80/m	8 m	278	
Stratagrid 300	$ 16.34/m	150 m	2451	
Stratagrid 200	$ 14.88	80 m	1191	
Geotextile Trevira Pro Earth 018/200	$ 6.40	160 m	1027	$ 4,947
			Direct costs	$ 18,012

in British Columbia and other regions of Canada and the United States which are greater than 5m height, and indeed structures less than 5m high would be considered to be relatively small.

To give insight to the likely behavior of the structure at Foley Creek, a summary follows of the performance of a similar sloped (1V:0.5H) reinforced soil wall built in Norway (Fannin and Hermann, 1990). The structure is 4.8m high, and comprises two sections, each with a different arrangement and spacing of reinforcement (see Fig. 6). It is founded on a competent foundation. Section 'J' has 5 layers of primary geogrid reinforcement, a uniaxial Tensar geogrid (UX1400 equivalent), and 3 layers of intermediate reinforcement, a biaxial Tensar geogrid (BX1100 equivalent). Section 'N' has 8 layers of primary reinforcement. The backfill soil is a uniformly graded medium-to-fine sand, for which $\gamma = 17$ kN/m³, $d_{50} = 0.25$ mm and $C_U = 3$. Three months after construction, and following a period of variable surcharge loading with water-filled tanks, a 3.0 m high earth berm was placed on the crest of the structure, increasing the height to

***Figure 6.** Arrangement and spacing of the reinforcement : Norway structure (after Fannin and Hermannm, 1990)*



7.8 m and imposing a permanent surcharge loading.

Instrumentation was placed to measure, independently, the force and strain in the layers of primary reinforcement using vibrating-wire force cells and Bison inductance coils respectively. Strains in the primary reinforcement immediately post-construction, during the period of surcharge loading, were found to be small and not exceeding 0.5 % (see Fig. 7). The application of permanent surcharge loading led to the development of larger strains, which is to be expected as the layers of reinforcement took up the increment of lateral thrust. Typically the strains increased to values in the range 0.5 to 1.0 %. Polymeric geogrid reinforcement, like many materials, exhibit creep and long-term observations of the structure have shown continued very small strains at nearly constant load which confirm this behavior. Comparisons of field performance with laboratory data used to determine appropriate long-term design strengths have shown excellent agreement (Fannin, 1994). The measured force in each layer of reinforcement, see Fig. 8, suggests a nonuniform spacing of primary geogrid (closer spacing near the base of the structure) promotes a more uniform distribution of force between the layers. It is interesting to note that all layers of reinforcement except the base layer reflect the placement of the 3.0 m high earth berm in October 1987, the time of permanent surcharge loading. Analysis has shown the sum of the force in the primary reinforcement agrees well with a coefficient of lateral earth pressure derived for the soil type and geometry of the sloped wall structure.

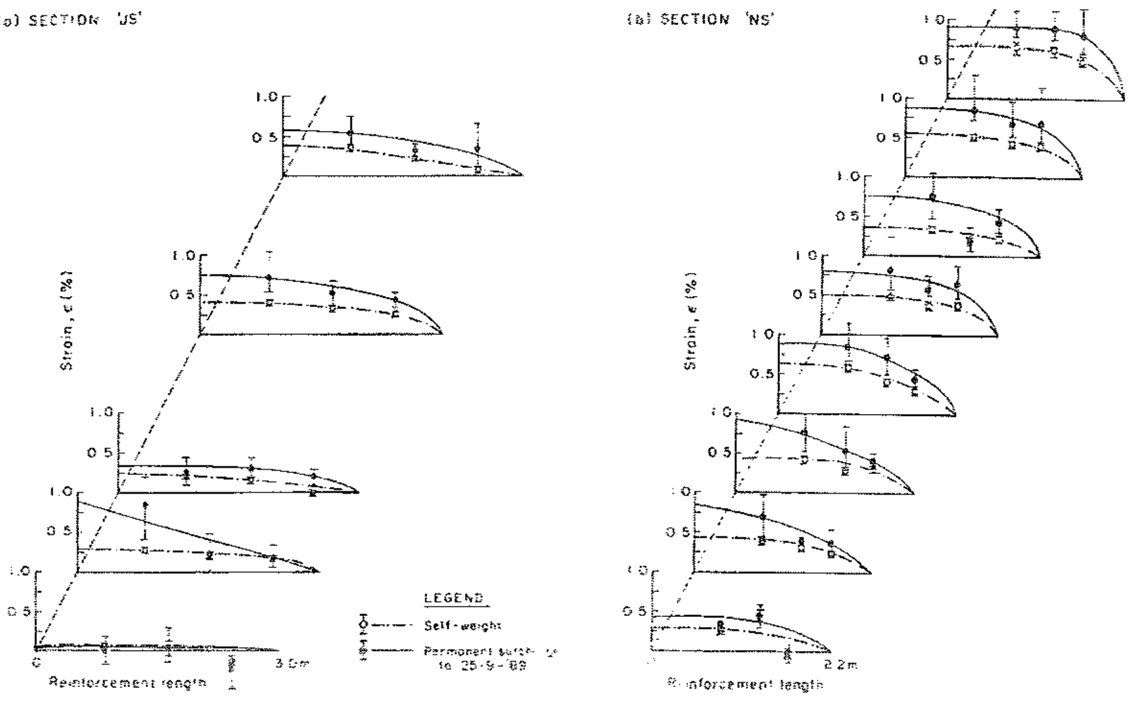
Concluding remarks

Factors influencing the selection of a reinforced soil retention system for repair of a sidecast failure in a section of the Foley Creek FSR have been reported. The geogrid reinforced soil structure was built to maintain the existing alignment and grade of the failed road, at a location where the hillslope configuration and steep cutbanks constrained other options for reinstatement of access. The general arrangement of the reinforcement at Foley Creek is similar to other structures, and direct comparison with an instrumented structure in Norway is used to illustrate the likely behavior of the polymeric geogrid in the long-term. The following general observations are made:

1. It is recommended, for such relatively low reinforced soil structures, that a uniform vertical spacing of the primary reinforcement be specified: there is little efficiency to be gained from optimizing the spacing, and it simplifies the construction routine. It is further recommended, in design, that the same type and grade of reinforcement be specified for all layers of primary reinforcement: this is to avoid any potential for incorrect placement of the layers during construction.

2. Intermediate reinforcement is typically used

***Figure 7.** Force in the reinforcement : Norway structure (after Fannin and Hermann, 1990)*



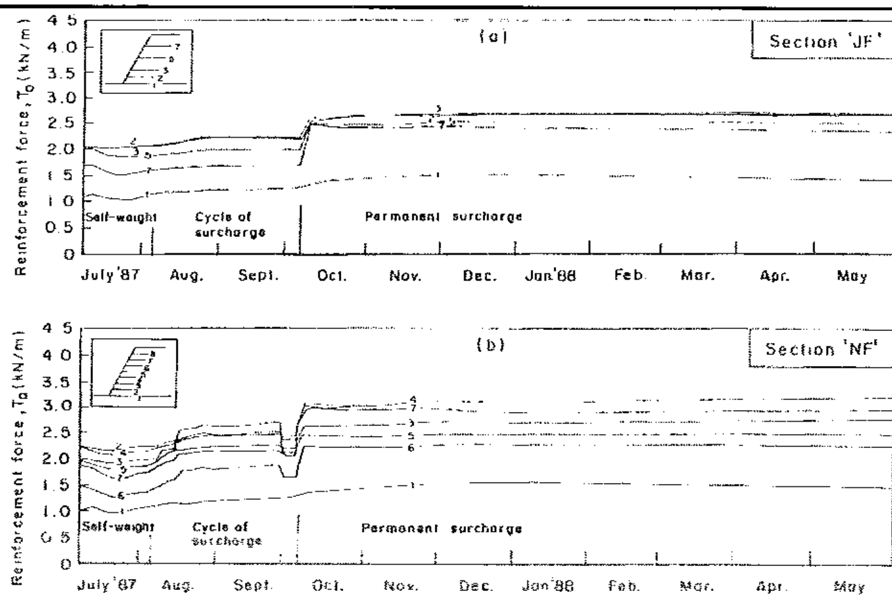


***Figure 8.** Force in the reinforcement : Norway structure (after Fannin and Hermann, 1990)*

when the spacing of the primary reinforcement exceeds 0.5 m. At Foley Creek it was used, together with a geotextile wrap-around, to prevent sloughing and erosion of soil at the face of the structure. Rip-rap was brought up the face to provide further protection, and assist with compaction to the edge of the fill slope. Where such rock is not available, preferred alternatives may include the use of gabion baskets (Simac et al., 1997), erosion control matting with surface vegetation, bar mesh facing units (Fannin and Hermann, 1990), or in some situations an asphalt emulsion or similar coating.

Experience gained on the project at Foley Creek further suggests that:

3. Many road failures on steep terrain involve a localized concentration of groundwater flow. Design of remedial structures must account for both surface water drainage (ditch and road surface run-off), and sub-surface drainage (preferential flow routes, often controlled by bedrock). Good provision for surface and sub-surface drainage, proper compaction of the backfill soil, and adequate "keying" of the structure into the underlying foundation and into the road approaches on either side are essential to the long-term performance of these reinforced soil structures on steep terrain.

4. The use of geosynthetics is unfamiliar to many contractors in the forest engineering sector. The project supervisor for construction should be familiar with the use of these materials, and a pre-construction meeting should occur to review design details, installation procedures and site-specific requirements.

5. The use of geosynthetic reinforcement with local soils, where they are appropriate, appears to be a cost-effective solution when a sufficient source of large angular rock (shot-rock or talus) is not available.

Acknowledgements

The geotechnical design for internal stability of the structure was provided by Armtec Construction Products, with assistance from Solmer International of Mississauga, Ontario. Site supervision during construction was provided by RB Engineering of Port Coquitlam. The Ministry of Forests does not, through the reporting of this project, seek to endorse any specific geosynthetic products for slope stabilization projects.

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