Economical retaining wall design using soil and rock anchors

Conception de mur de soutènement économique en utilisant des ancres de sol et de roche

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SYNOPSIS: Several projects are described where soil and rock tieback anchors were used in lieu of conventional retaining walls to support both temporary and permanent retaining walls in confined areas thereby allowing adjacent mainline railways and controlled access highways to operate during construction. This resulted in savings of about 10% of the total cost. A summary of full-scale tests on soil and rock anchors, strength tests on rock (shale) cores, proof tests on production anchors, and associated costs is included.

1 INTRODUCTION

Toronto, Ontario, is a large cosmopolitan city with a population of about 3.4 million. Adjacent urban centres are linked to the city through an extensive and complex transportation system that is continually being modified and upgraded. On the east side of the city, a major controlled-access highway (Hwy 401) closely parallels the Canadian National Railway (C.N.R.) mainline, which handles both commuter and freight traffic. These transport systems are frequently crossed by other roads with bridges and access ramps.

In order to handle rapidly increasing numbers of commuters, construction of a rapid transit system (GO Transit) commenced in the early 1980's. This project was planned and supervised by GO Transit and the Ministry of Transportation of Ontario. Geotechnical investigations and design were carried out and construction was awarded on a contract basis to different portions of the project which covers a length of about 15 km. This paper deals with the tieback retaining walls for five separate contracts which were designated GGE 310, 311, 312, 313 and 315. The GO Transit system consists of twin tracks within a right-of-way which is designated as the "guideway". In general, this guideway lies between the existing C.N. Rail and Hwy. 401.

In the past, conventional reinforced-concrete cantilevered retaining walls were usually used for such projects. Experience, however, demonstrated that these walls often required extensive excavation to provide adequate working space and to ensure that the associated temporary cut slopes remained stable during construction. Furthermore, the time and costs associated with controlled backfilling operations were substantial. In confined areas, particularly adjacent to facilities such as mainline railways and controlled access highways, additional expensive protective measures and detours were often required and traffic had to be temporarily disrupted.

For the GO Transit project, it was essential to minimize or completely eliminate interruption of the existing transportation systems. Accordingly, the use of a tieback support system employing soil anchors was introduced. The success of this system led to the use of tieback systems using soil and rock anchors on subsequent contracts. These tieback systems were used for both temporary and permanent structures.

This paper summarizes the main features of each contract as well as the results of the extensive geotechnical investigations, strength tests on cores of shale bedrock, full-scale loading tests on soil and rock anchors, proof-tests on production anchors, and associated costs.

2 MAIN FEATURES OF FIVE SEPARATE CONTRACTS

The main features of five separate contracts associated with the GO Transit project are summarized in Table I (Canadian dollars quoted throughout).

Table I - Main features of five GO Transit contracts

GGE 310 — Zones of soft to firm clay overlying bedrock
- Bridge + three permanent walls on piles
- Four temporary walls: H-piles with timber lagging tied back with rock anchors; length 200 m, average height 6.5 m
- Four full-scale rock anchor tests
- Compression tests on six cores of shale bedrock
- Detour of rail spur line eliminated
- Total cost $4.3 million; savings $0.65 million

GGE 311 — Zones of soft to firm clay overlying till and bedrock
- One new bridge; modifications to 2 existing bridges
- Permanent tie-back wall with rock anchors; length 61 m, average height 9.5 m
- Full-scale anchor tests; 2 in rock and 1 in soil
- Total cost $5.8 million; savings $0.24 million

GGE 312 — 16 m of very dense silt overlying bedrock
- Two new subway structures
- Four conventional permanent walls; average height 4.5 m
- One temporary tie-back wall with soil anchors; length 74 m, average height 9 m.
- Total cost $9.3 million; savings $0.36 million

GGE 313 — Hard and very dense glacial till
- Two new bridges
- Permanent tie-back wall with soil anchors, H-piles and precast concrete elements: length 460 m, average height 9.4 m
- Nine full-scale anchor tests in soil
- Total cost $4.8 million; savings $1.1 million

GGE 315 — Realignment of existing road and new bridge
- 17 m of very dense sandy silt overlying bedrock
- Two temporary tieback walls with rock anchors; length 135 m, average height 9 m
- Three full-scale anchor tests in soil
- Total cost $7.3 million; savings $0.8 million

3 FOUNDATION INVESTIGATION

Boreholes were put down at horizontal spacings of about 30 m at the proposed locations of the major structures. In general, samples of the overburden were taken with a standard penetration sampler and the N values (blows/0.3 m) were determined. Two representative stratigraphic sections are shown in Fig. 1.

As shown in Fig. 1a, a zone of very soft to firm clay was encountered in the overburden. These conditions were not favourable for soil anchors, and therefore, rock anchors were used (GGE 310 and 311).

Subsoil conditions were much more favourable at the other sites (Fig. 1b), and therefore, conventional walls with spread footings or tieback walls with either soil or rock anchors could have been used.

At the GGE 310 site (Fig. 1a), cores were taken of the shale bedrock (Whitby Formation), and subjected to uniaxial compression tests; strengths ranged from 12 to 30 MPa.

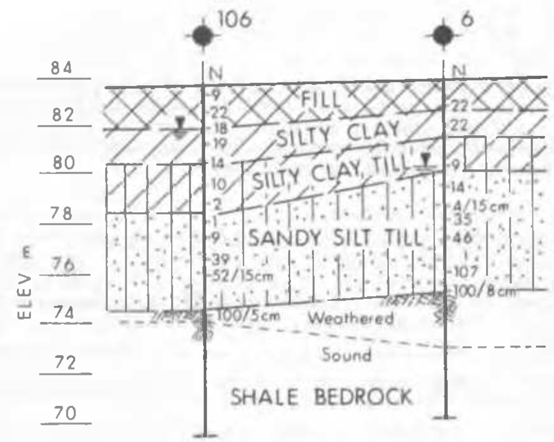

Figure 1a. Stratigraphic section at GGE-310

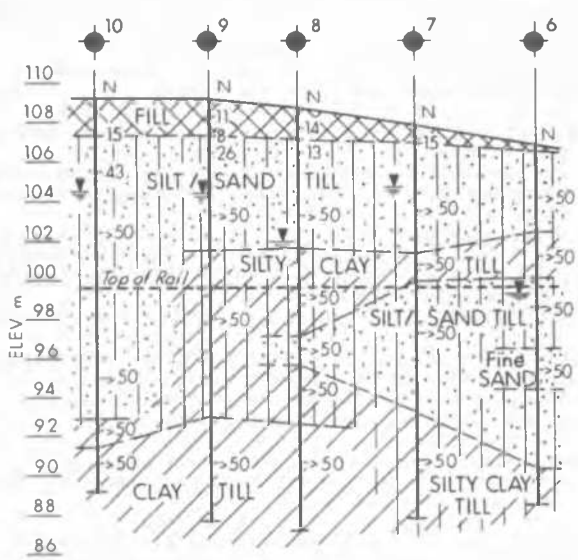

Figure 1b. Stratigraphic section at GGE-313

4 PREDICTED BOND STRESS FOR ROCK AND SOIL ANCHORS

The prediction of anchor capacity in rock is usually based on the allowable grout-to-rock bond stress acting over the fixed anchorage length. The Canadian Foundation Engineering Manual (1985) states that the allowable bond stress should be smaller than 1/30 times the unconfined compressive strength of the rock and 1/30 times the unconfined compressive strength of the grout. It should not exceed 1300 kPa.

Based on the core strengths of the shale, the allowable bond stress for rock anchors ranges from about 410 to 1020 kPa. A 7-day grout strength of 20 kPa results in an allowable bond stress of about 670 kPa. Initially, a bond stress of 500 kPa was selected; this value was modified as required following performance tests on site-specific full-scale anchors.

For the predominantly granular glacial till encountered at several sites (Fig. 1b), the capacity of a grouted soil anchor was based on an allowable bond stress of about 100 kPa. This estimate was based on recommendations given in the literature e.g., Hanna (1982), Lee et al (1983) and CFEM (1985).

5 PERFORMANCE TESTS ON ANCHORS

The design, supply, installation and testing of soil and rock anchors were in accordance with a document issued by the Ministry. This document was written specifically for the use of either Dywidag Threadbar or BBR Cona Multi-Strand tendons with double corrosion protection encased for their full length in corrugated PVC sheathing and the annular space grouted prior to installation (corrosion protection for permanent installations only). Smooth PVC tubing was to be used in the free-stressing length.

For the soil anchors, inclined holes were typically advanced with a 300-mm diameter hollow-stem auger. The tendon was inserted in the auger prior to advancing the hole with the tip of the tendon acting as a plug. The grout was injected and pumped at the lowest point of the bond length and pumping was continued to the top of the bond length as the auger was withdrawn. A soil plug about 0.3 m in length, was formed at the interface of the bonded length and the free-stressing length by reversing the rotation of the augers. This plug was intended to support and centre the tendon. A bentonite slurry was pumped into the free-stressing length.

When the grout in the bond length achieved its specified strength, performance tests were made on each rock or soil anchor. The anchors were loaded in conformance with prescribed cyclic load increments.

An anchor test configuration for testing anchors is shown in Fig. 2. Precautions were taken to ensure that the dial gauges used to measure displacements were mounted on reference beams which were independently supported.

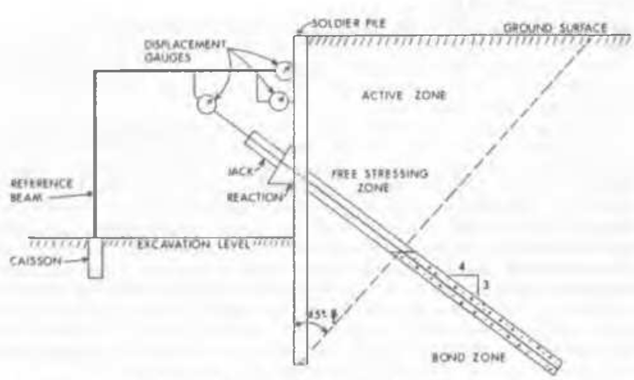

Figure 2. Anchor test configuration

In order to ensure that the free-stressing length was effective, it was specified that the total elastic movement must be at least 80% of the theoretical elastic elongation of the tendon but not greater than the elongation of the free-stressing length plus half the bond length.

The load and deformation data for the performance tests on rock and soil anchors are summarized in Table II. The elastic and plastic movements are given for each load cycle. The total deformation can be obtained by adding the elastic and plastic deformation.

Table II - Summary of performance tests on anchors

No.	Tendon	Bond Length m	Cycle No.	Max. Load kN	Min. Load kN	Plastic Def. mm	Elastic Def. mm
Rock anchors GGE-310							
1	Dywidag	2	1	505	125	5	30
			2	755	125	6	46
			3	1000	0	10	62
2	BBR	2	1	610	125	30	33
			2	750	130	35	58
			3	1000	0	53	75
3	Dywidag	3	1	500	140	19	24
			2	760	140	22	56
			3	950	0	30	61
4	BBR	3	1	500	130	19	32
			2	750	130	23	56
			3	1000	0	36	87
Rock anchors GGE-311							
1	BBR	3	1	760	0	92	142
			2	1000	0	134	202
2	BBR	3	1	510	65	28	94
			2	760	0	32 - 96	166
			3	690	0	225	115
			4	860	0	312	202
			5	1000	0	386	234
Soil anchor GGE 311							
1	BBR	6	1	510	135	26	67
			2	760	135	32	122
			3	995*			
*violent failure occurred when total movement reached 212 mm							
Soil anchors GGE-313							
1	Dywidag	5.6	1	240	50	2	5
			2	365	50	4	8
			3	485	50	8	13
			4	615	35	12	18
			5	750	50	23	23
			6	825	35	36	25
			7	900	60	39	22
			8	1000	100	43	29
2	Dywidag	5.6	1	245	45	2	7
			2	380	45	5	4
			3	490	45	12	14
			4	620	45	20	18
			5	755	45	29	21
			6	830	45	31	20
			7	880	45	35	47
			8	995	45	48	75
3	Dywidag	5.6	1	1000	490	22	10
			2	700	490	23	5
			3	700	490	23	5
			4	1000	0	11	23
4	Dywidag	5.6	1	490	130	9	7
			2	700	130	18	16
			3	1000	130	32 - 54	29
5	Dywidag	5.6	1	570	60	8 - 52	14
6	Dywidag	5.6	1	490	120	5	7
			2	760	120	7	15
			3	1000	40	10 - 18	26
7	Dywidag	5.6	1	500	0	92	18
8	Dywidag	5.6	1	690	40	73	11
9	Dywidag	5.6	1	500	130	5	7
			2	750	130	9	15
Soil anchors GGE 315							
1	Dywidag	2.5	1	245	65	15	5
			2	325	0	26	13
2	Dywidag	2.5	1	250	65	2	5
			2	375	130	5	7
			3	490	15	9	13
3	Dywidag	2.0	1	245	65	9	3
			2	245	15	20	4

Note: Dywidag tendons were 25 - 36 mm ϕ
BBR tendons were 7 strand, each strand 16 mm ϕ

The data in Table II were used to plot load vs. elastic, plastic or total deformation. In each case, the plots indicated that the elastic elongation was within the specified limits and that the method of construction was effective in providing a free-stressing length of tendon.

Plots (Fig. 3) of load vs. total deformation (elastic + plastic) were used in conjunction with time-deformation measurements to determine the allowable load on each anchor. In many tests, for example Figs. 3a and 3d, acceptable performance was obtained at loads up to 1000 kN and "failure" was not achieved. With respect to Fig. 3c, sudden failure occurred at the 1000 kN load. With respect to several other tests, for example Test 2 in Fig. 3b, relatively large deformations occurred at constant load

Typically, each load increment was held for a period of at least 15 minutes and deformation measurements were taken at about 5 minute intervals. If the rate of deformation did not exceed about 0.25 mm/h, the next increment was applied. If the rate of deformation exceeded 0.25 mm/h, an attempt was made to sustain the applied load and deformation measurements were continued for a longer period of time. "Failure" was assumed when the deformation exceeded approximately 1.5 mm during the last log time cycle. A log time cycle was defined as the time interval between the final time and one-tenth of the final time, e.g., 3 to 30 minutes.

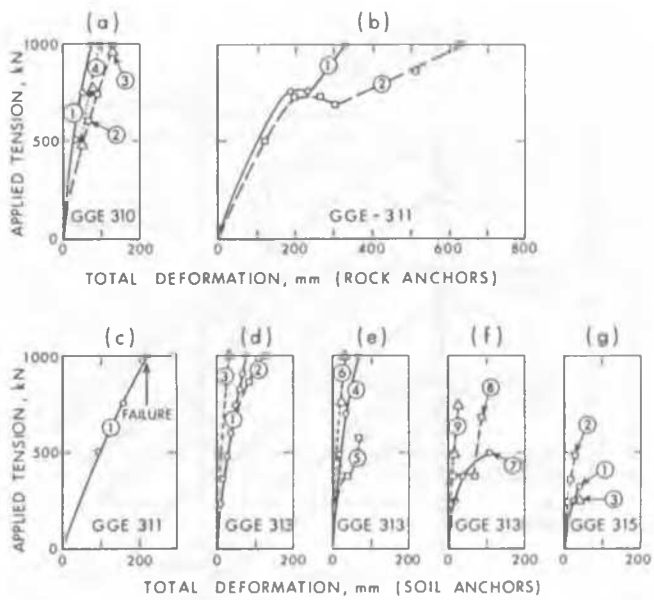

Figure 3. Results of performance tests

6 PRODUCTION ANCHORS

Representative installations for both rock and soil anchors are illustrated in Fig. 4. Rock anchors were used on Contract GGE-310 and were inclined 1:1 (H:V), as shown in Fig. 4a. Soldier piles were reinforced with driving shoes and were driven to refusal in sound shale bedrock. Soil anchors were used on Contract GGE-313 and were inclined 4:3 as shown in Fig. 4b. A minimum depth of embedment of 2.5 m was specified for the soldier piles.

Each production anchor was accepted or rejected on the basis of the specified criteria with respect to the free-stressing length and the rate of deformation (creep). As an example, observations made with respect to 264 production soil anchors at the GGE 313 site are discussed below.

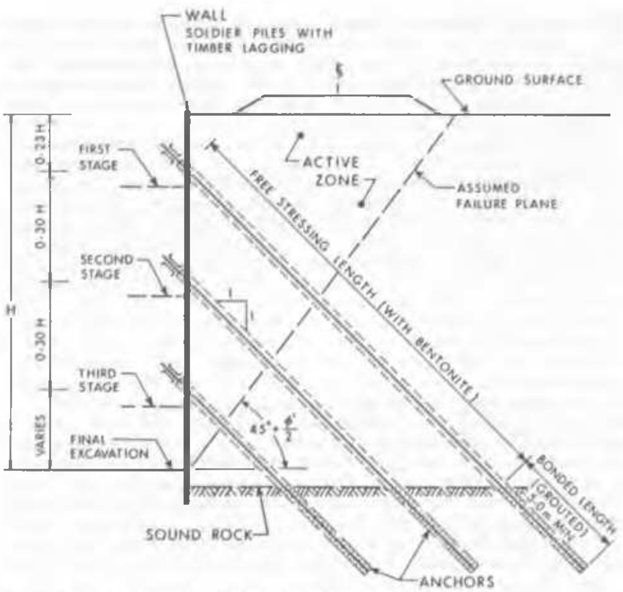

Figure 4a. Rock anchor tieback system

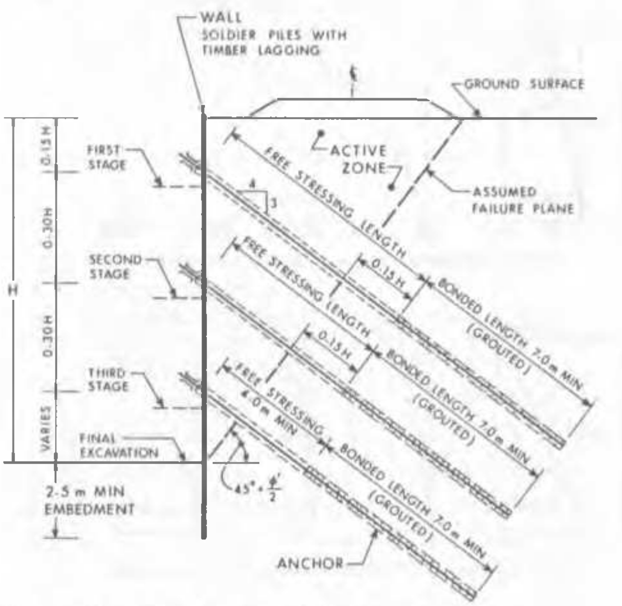

Figure 4b. Soil anchor tieback system

Eight anchors failed the elastic elongation acceptability criterion. Of these, five did not reach the 80% of the elastic elongation in the free-stressing length. None of the eight anchors failed the creep criterion. All of these anchors were successfully replaced.

Table III indicates the sensitivity of the acceptance level with respect to creep values. These values were determined at 150% of the working load and between the period of 3 to 30 minutes after the working load was applied. As shown, 95% of the anchors passed the 1.5 mm criterion.

A review of the stratigraphic sections indicated that most of the unacceptable anchors were probably installed, at least in part, in clay or silty clay and these soils would result in lower capacities. It wasn't possible to define the extent of these clayey deposits in advance due to the heterogeneity of the soil deposit (Fig. 1b).

Table III - Record of production anchors at GGE 313

Creep, mm	% Passing	# Passing	# Failing
1.0	82	206	46
1.1	87	219	33
1.2	90	226	26
1.3	92	232	20
1.4	93	235	17
1.5	95	239	13

7 COSTS

As indicated in Table I, savings up to 20% were achieved on some contracts. These savings were attained primarily through the use of tieback support systems rather than conventional reinforced concrete retaining walls. Costs are given in Table IV.

Table IV - Summary of some approximate unit costs

Item	Approximate cost ($Cdn)
Full-scale performance tests	$10,000-12,000/test
Rock tieback anchors	$130-180/m total length
Soil tieback anchors	$100-110/m total length
Tieback wall	$350/m^2
Conventional wall on spread	$360/m^2 for 4 m height
footings (not including	$440/m^2 for 6 m height
backfilling)	$550/m^2 for 8 m height

8 CONCLUSIONS

A tieback support system with anchors in rock or soil was used successfully for temporary and permanent retaining walls in the construction of a rapid transit rail line in a built-up urban centre. Several site-specific performance tests were required to evaluate the design holding capacity of the anchors due to the heterogeneous soil conditions along the project length which covered about 15 km.

With few exceptions, tiebacks were used in lieu of conventional retaining walls and this resulted in savings of about 10% of the total cost. The tiebacks also permitted uninterrupted use of existing rail and highway corridors.

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