Use of tiebacks in Southern Ontario

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ABSTRACT
The application of tiebacks at six sites in Southern Ontario is presented in this paper. The testing methods for tiebacks used and tieback performance at these sites are discussed in this paper. The tieback load transfer (bond strength) and bond capacity (skin friction) obtained from tieback performance tests are related to the soil conditions. Typical values of tieback load transfer and bond capacity in various soils are recommended. Correlations between load transfer/bond capacity and SPT N-value are proposed.

RÉSUMÉ
L'application des tiebacks à six loctations au sud de l'Ontario est présentée dans cet article. Les méthodes d'essai pour des tiebacks utilisés et la performance des tieback dans ces locations sont discutées dans cet article. Le transfert de charge du tieback (force en esclavage) et la capacité de lien (frottement de peau) obtenue des essais de performance de tieback sont liés aux conditions dans le sol. Les valeurs typiques de la transfèrt de charge du tieback et la capacité de lien dans divers sols sont recommandées. On propose des corrélations entre le transfert de charge/capacité en esclavage et la valeur-N de SPT.

1 INTRODUCTION

Tiebacks are cement grouted, prestressed tendons installed in soil or bedrock. Vertical or near vertical tiebacks are called tiedowns. Tiebacks are also referred to as ground anchors. The basic components of a tieback include anchorage, free stressing (unbonded) length and bond (fixed) length. The anchorage is made up of an anchorage head or nut and a bearing plate that can transmit the prestressing force from the pressing steel (bar or strand) to the ground surface or the supported structure. The free length is the portion of the prestressing steel that is free to elongate elastically and transfer the resisting force from the bond length to the ground surface or the supported structure. A bondbreaker is a smooth plastic sleeve that is placed over the tendon in the free length to prevent the prestressing steel from bonding to the surrounding grout. The bond length is the length of the prestressing steel that is bonded to the grout and is capable of transmitting the applied tensile load to ground.

The first application of tiebacks in civil engineering was used for the strengthening of Cheurfas Dam in 1933. Hanna (1982) gave a detailed historical review of the development and the application of tiebacks. Tiebacks have been in use for more than 30 years in North America. Today the use of tiebacks represents a common technique for earth retention and slope stabilization. In certain design and construction conditions, tieback support systems offer several advantages over more conventional systems that have resulted in economic and technical benefits such as unobstructed workspace for deep excavation, ability to resist large earth pressures, reduced construction time, etc.

The detailed recommendation for design and construction of tiebacks are stipulated in Canadian Geotechnical Society (2006), FHWA (1999) and British Standards Institution (1989).

This paper presents the application of tiebacks in six sites in Southern Ontario. The tieback load transfer (bond strength) and bond capacity (skin friction) obtained from tieback performance tests are related to soil conditions. Typical values of tieback load transfer and bond capacity in various soils are recommended. Correlations between load transfer/bond capacity and SPT N-value are proposed.

2 APPLICATION IN SOUTHERN ONTARIO

In southern Ontario, tiebacks are mainly used for temporary support in deep excavation. Occasionally, tiebacks are also used for the permanent support of structure walls. Tiebacks are usually designed to support caisson walls or soldier pile walls. In most cases, the tieback is made up of a few numbers of 15mm strand tendons installed within 150mm diameter cased boreholes and post-grouted the day after installation.

Performance tests are usually conducted on selected tiebacks to verify tieback capacity, establish load-deformation behaviour, identify causes of tieback movement and to estimate the apparent free length prior to the installation of production tiebacks. The apparent free length is calculated as follows

$$\text{Apparent free length} = A_t E_s \delta_e / P \quad [1]$$

where A_t is the cross section of the tendon, E_s is the elastic modulus of the tendon, P is the applied net test load (total test load minus the alignment load) and δ_e is the elastic movement of tieback which is the difference between the total movement at the test load and the permanent movement at the alignment load.

The load procedure for performance tests is generally in accordance with PTI (1996), but also follows that of the Canadian Geotechnical Society (2006) for some projects. During the test, the tieback is stressed by a hydraulic jack and the movement is measured by a dial gauge on the pulling head of the jack. The dial gauge is supported on an independent tripod. For temporary tiebacks, test load ranges from 133% to 200% of the design load which is maintained for 10 to 60 minutes to assess creep. For permanent tiebacks, test loads range from 200% to 300% of the design load which is maintained for 1 to 10 hours to assess creep.

Tieback performance tests are evaluated in accordance with the acceptance criteria spelled out in PTI (1996). The criteria are also addressed in FHWA (1999) and Canadian Geotechnical Society (2006). The acceptance criteria can be simplified as follows

(1) The minimum apparent free length at the test load shall not be less than 80% of the free length plus the jack length;
(2) The maximum apparent free length at the test load shall be less than 100% of the free length plus 50% of the bond length plus the jack length;
(3) The creep amount shall not exceed 1mm at test load during the period of 1 to 10 minutes or not exceed 2mm within the period of 6 to 60 minutes.

The maximum apparent free length criterion can be relaxed if the creep test is acceptable.

Proof testing is conducted on each production tieback. The test procedure follows the guideline of PTI (1996). Test load is normally 133% of the design load which is maintained for 10 minutes if the creep movement between 1 and 10 minutes did not exceed 1mm. In the case that the creep movement during the period of 1 to 10 minutes exceeded 1mm, the test load is maintained for an additional 50 minutes. The tieback is normally adjusted to the lock-off load at 110% of the design load after 10 minute hold without the measurement of permanent movement. The permanent movement is estimated from the performance test. In the case that the performance test cannot be compared with the proof test, the permanent movement is measured in the proof test. The acceptance criteria for the proof test are the same for the performance test.

Application of tiebacks at six sites in Southern Ontario is discussed as follows.

2.1 Waterloo Wastewater Treatment Plant Upgrade

The site is located at approximately 200m north of University Avenue East and approximately 100m west of Conestoga Parkway in the City of Waterloo, Ontario. Subsurface investigation revealed that the site stratigraphy is made up of approximately 10m of firm to hard silty clay overlying very dense sand and silt till. The groundwater table was about 4.5m below grade.

A contiguous caisson wall consisting of 1m diameter caissons at spacing of 1.5m and 0.88m diameter filler caissons was constructed. Three rows of tiebacks were used to support the maximum 10.8m deep excavation and the permanent structural wall. The king piles were

Photo 1. Caisson wall supported by tiebacks at Waterloo WWTP

reinforced with W510x113 H-sections installed a maximum of 16.8m below existing grade. Three rows of permanent tiebacks were installed at 2.2m, 5.8m and 8.8m below grade. Each tieback was made up of 2 to 5 15mm strand tendons installed within 150mm dia. cased boreholes.

The overview of tieback installation at Waterloos wastewater treatment plant (WWTP) is shown in Photo 1. All tiebacks were installed at 45^{o} to the horizontal. The upper and middle tiebacks were bonded in hard silty clay to very dense sand and silt till. The lower tiebacks were bonded in very dense sand and silt till. For the bond zone, encapsulated tendon or Class I protection was used. Table 1 summarizes the tieback characteristics.

Table 1. Tieback characteristics at Waterloo WWTP

Tieback level	Design load (kN)	Free length (m)	Bond length (m)
Upper	290 - 405	4.5 - 8.0	5.5 - 8.2
Middle	520 - 695	4.5 - 5.4	10.4 – 13.9
Lower	695	4.5	13.9

The load procedure for performance tests conformed to Canadian Geotechnical Society (2006). The test load was 200% or 300% of the design load which was maintained for 10 hours to assess creep for the permanent tiebacks. Figure 1 shows a performance test conducted on an upper tieback. The tieback performance test results were evaluated in accordance with the acceptance criteria spelled out in PTI (1996). All performance tests carried out in the project met the acceptance criteria.

Proof tests were conducted on each production tieback. The test load was 133% of the design load and the test load was maintained for 15 minutes in most cases. A few of tiebacks were held for 1 hour at the test load as the creep movement was greater than 1mm

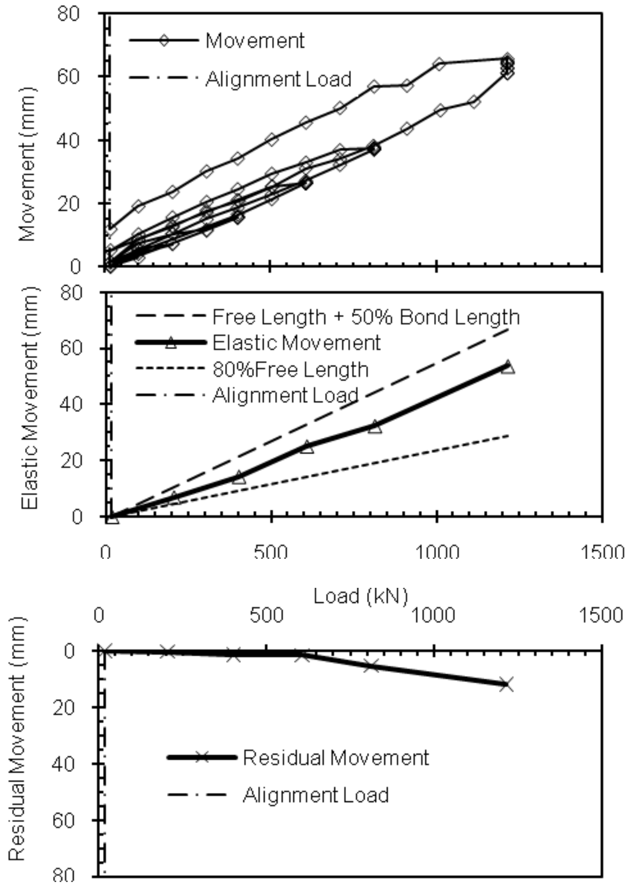

Figure 1. A typical performance test at Waterloo WWTP

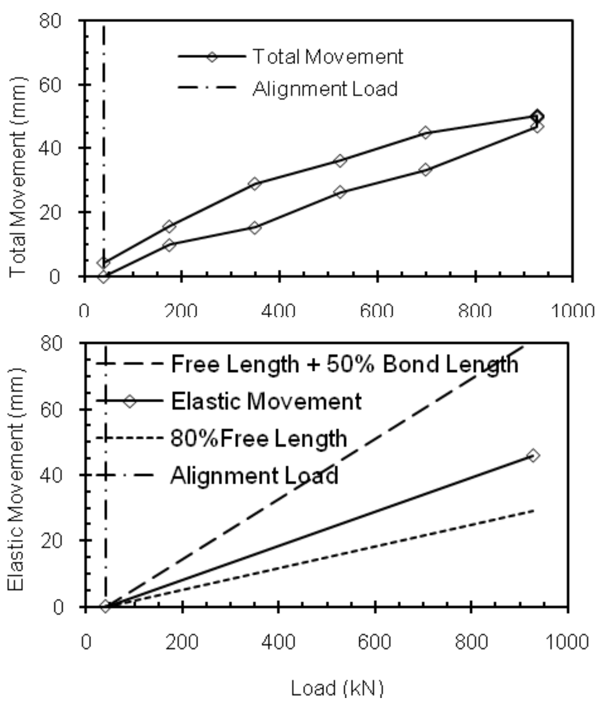

Figure 2. A typical proof test at Waterloo WWTP

during the period from 1 to 10 minutes. Most tiebacks were adjusted to the lock-off load at 110% of the design load after 15 minute hold with an exception of a few of tiebacks which were returned to alignment load to permit the determination of permanent and elastic movements at the test load. Figure 2 shows a proof test conducted on a middle tieback. All tiebacks met the PTI criteria.

2.2 R.C. Harris Water Filtration Plant (WFP)

The site is located at Queen Street East, Toronto, Ontario. Subsurface investigation using drilled boreholes indicates that the site stratigraphy is representative of Eastern Toronto or Scarborough bluff soil strata made up of approximately 1.5m of firm silty clay to clayey silt fill material overlying an approximately 10m thick layer of very loose to dense silty sand to sandy silt which in turn rests on approximately 9m of stiff to very stiff silty clay to clayey silt. The silty clay to clayey silt is underlain by very dense sandy silt to gravely sand over silty clay till. Ordovician Georgian Bay shale and limestone/siltstone bedrock was encountered at 29m below existing ground surface. The groundwater table was about 5 m below grade.

Contiguous continuous flight auger (CFA) pile walls were employed to support a 10.5m deep excavation. Three rows of tiebacks were installed at 1.2m, 2.5m and 7m below existing ground surface to support the caisson wall. The upper tiebacks were installed within 150mm diameter cased boreholes and were bonded in very loose to dense silty sand to sandy silt. The middle and lower tiebacks were installed within 460mm diameter CFA bores in compact to dense silty sand to sandy silt. Each tieback was made up of 6 to 7 15mm strand tendons. The tiebacks were generally post-grouted the day after they were installed. Table 2 summarizes the tieback characteristics.

Table 2. Tieback characteristics at R.C. Harris WFP

Tieback level	Inclination	Design load (kN)	Free length (m)	Bond length (m)
Upper	15º	550	6.5	16.0
	20º	250	5.5	3.9
	5º	300	5.0	5.0
Middle	20º	750	5.5	11.5
	20º	750	6.5	16.0
Lower	30º	750	5.0	12.0

The tieback design load was verified by performance tests in which the test load was 133% or 150% of the design load and the test load was generally maintained for

10 minutes. The sequence of loading followed the recommendations of PTI (1996). The majority of the performance tests met the acceptance criteria. A few of performance tests for the tiebacks installed using both CFA and cased hole methods could not meet the criteria due to significant creep movement or shorter free length.

Proof tests were carried out for all tiebacks. The test load was 180% of the design load for some of the upper tiebacks and 133% for other tiebacks and the test load was generally maintained for 10 minutes. Similar to the performance tests, the majority of tiebacks met the acceptance criteria but a few of tiebacks could not meet the criteria due to significant creep movement or shorter free length. A lower design load was used for the tiebacks which could not meet the acceptance criteria.

The lesson from this project is that the bond capacity for silty sand to sandy silt was overestimated in the tieback design probably due to the disturbance of saturated sandy soils during the tieback installation.

2.3 Waterloo Region Consolidated Courthouse

The site is located within the block bounded by Weber Street, Scott Street, Duke Street and Frederick Street in the City of Kitchener, Ontario. Subsurface investigation using drilled boreholes indicates that the site stratigraphy is made up of approximately 1.5 to 3.1m of loose sandy silt and soft to very stiff clayey silt fill material overlying dense to very dense silt to sand which locally contains very stiff to hard clayey silt till. The groundwater table was 4.5 to 10.3m below grade.

Soldier piles at 3m spacing with wood lagging were employed to support a maximum 7m deep excavation. The soldier piles were installed a maximum of 13m below existing grade. One row of tiebacks was installed at approximately 2m below existing ground surface to support the solider pile wall. The tiebacks were installed within 150mm dia. cased boreholes and were bonded in dense to very dense sandy silt to sand. Each tieback was made up of 2 to 5 15mm strand tendons. The tiebacks were generally post-grouted the day after they were installed. Table 3 summarizes the tieback characteristics.

Table 3. Tieback characteristics at Courthouse

Inclination	Design load (kN)	Free length (m)	Bond length (m)
20º	300	3	3
20º	400	3	4
20º	500	3	5
20º	700	3	7

Performance testing to 200% of the design load was carried out to verify the tieback design. However the test load could not be maintained. When the tieback was retested to 133% of the design load, the test met the PTI criteria.

Proof tests were carried out for all tiebacks. The test load was 133% of the design load. 10% of the proof tests failed as the load could not reach the test load or the test load could not maintained for 10 minutes. Lower working load was used for the failed tiebacks.

The bond capacity for the sandy silt to sand used in the tieback design was overestimated in the project.

2.4 Maple Pumping Station

The site is located in the City of Vaughan, Ontario. Soldier pile walls with two rows of tiebacks were employed to support an approximately 10m deep excavation. The tiebacks were bonded into very stiff to hard clayey silt till to dense to very dense sandy silt till. Table 4 summarizes the tieback characteristics.

Table 4. Tieback characteristics at Maple pumping station

Tieback level	Inclination	Design load (kN)	Free length (m)	Bond length (m)
Upper	30º	600	6.0	8.0
	30º	700	6.5	9.5
	35º	700	4.9	9.5
	30º	750	6.5	10.0
	30º	800	6.5	11.0
	30º	900	7.0	12.0
Lower	30º	700	4.5	9.5
	30º	850	4.5	11.5
	30º	900	4.5	12.0
	30º	1100	4.5	15.0

Four performance tests to 200% of the design load were carried out to verify the tieback design. The test load was maintained for half an hour. All performance tests met the PTI criteria.

Proof tests were carried out for all tiebacks. The test load was 130% to 140% of the design load. Two tiebacks could not reach the test load due to the significant movement of the seats. One tieback could not reach the test load as post grouting was not conducted.

2.5 Lorne Park Water Treatment Plant

The site is located in the City of Mississauga, Ontario. The tiebacks and soldier pile walls were employed to support an approximately 10m deep excavation. The tiebacks were installed within 150mm diameter cased boreholes and were bonded into dense to very dense sand and silt and the design working load was 700 kN with 7m long bond length. The tiebacks were generally post-grouted the day after they were installed.

Two performance tests were carried out to verify the tieback design. One test could not be maintained at 200% of the design load and another test could not reach 200% of design load.

Since the performance tests failed, the tiebacks were redesigned with additional 1m bond length. Proof tests were carried out for all tiebacks and these tests met the PTI criteria.

2.6 FLY Condo Building

The site is located at 352 Front Street West in Toronto, Ontario. Field investigation with drilled boreholes revealed that the site stratigraphy is made up of 4.7 to 6.4m of loose to compact sandy silt overlying stiff to very stiff clayey silt till. Georgian Bay shale and limestone/siltstone bedrock was encountered at 8.8 to 9.5m below existing ground surface. The groundwater table is about 5 m below grade.

Soldier piles at 3m spacing with wood lagging were employed to support an approximately 13.6m deep excavation in which 10.6m of excavation was within overburden soils and 3.3m of excavation inside bedrock. The soldier piles were installed approximately 16.6m below existing grade. Two rows of tiebacks were installed at approximately 3.3m and 8.6 below existing ground surface to support the solider pile walls. The tiebacks were installed within 150mm dia. cased boreholes and were bonded in bedrock. Each tieback was made up of 6 to 7 15mm strand tendons. Table 5 summarizes the tieback characteristics.

Table 5. Tieback characteristics at FLY condo building

Tieback level	Inclination	Design load (kN)	Free length (m)	Bond length (m)
Upper	45º	940	9.2 – 10.8	3 - 4.6
	45º	1000	9.2 – 9.6	5.0
	45º	1100	9.2 – 9.6	5.7
Lower	25º	530	3.9	3.0
	25º	600	3.9 – 4.3	3.0
	25º	630	4.3	3.3

A performance test for the upper row of tiebacks was tested to 138% of the design load. The test load was maintained for 1 hour and the test met the PTI criteria. A performance test for the lower row of tiebacks was tested to 200% of the design load. The test load was maintained for half an hour and the test also met the PTI criteria.

Proof tests were carried out for all tiebacks. The test load was 133% of the design load and was maintained for 10 minutes. All tiebacks except for seven tiebacks met the PTI criteria. Two tiebacks did not meet the maximum free length of the PTI criteria. As the creep at the test load was less than 1mm, these two tiebacks were considered as acceptable. Two tiebacks could not reach the test load in the initial testing due to the insufficient support of seats/wales. After improving the seats, the retests met the PTI criteria. Three tiebacks could not reach the test load due to the broken wires. A lower design load was used for these three tiebacks.

Two inclinometers were installed behind the soldier pile wall prior to excavation. The inclinometers were monitored during and after the excavation and tieback installation. Figure 3 shows the monitoring results of one inclinometer. The maximum cumulative lateral deflection of the wall was 13.6mm occurring 0.5m below the existing ground surface. At the excavation level, the lateral deflection of the wall was 2.3mm. It is also noted that there was 0.2mm lateral movement in the bedrock at the bottom of the soldier pile wall. The lateral movement toward the excavation measured in the second

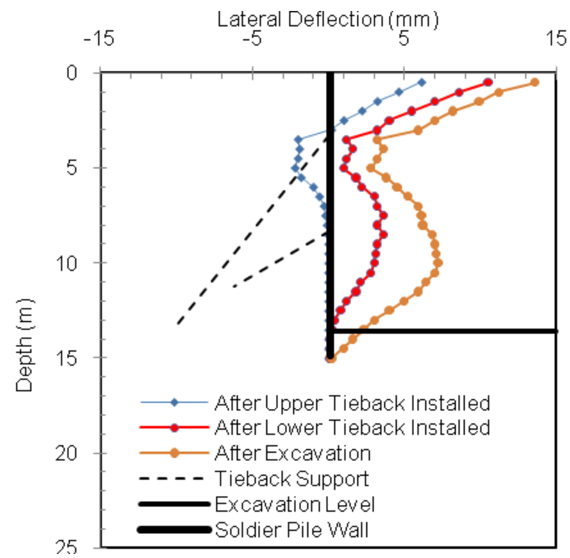

Figure 3. Lateral deflection of soldier pile wall

inclinometer was limited to 2mm. The monitoring of lateral movement confirmed the success of the tiebacks.

3 DISCUSSIONS

Based on the tieback performance and proof tests conducted at six sites, the main reasons for the poor tieback performance can be summarized as follows:

(1) Overestimated bond capacity (skin friction) for tieback installed in silty sand to sandy silt;
(2) Insufficient support of seats/wales;
(3) Broken wires;
(4) No post-grouting or poor grouting;
(5) Over-grouting in the portion of free length or poor tendon debonding design detailing of free zone.

The ultimate load transfer (bond strength) and bond capacity (skin friction) obtained from limited performance tests are presented in Figure 4. If a performance test failed at the test load, a resistance factor of 0.6 was used to obtain the ultimate value. Thus the values shown in Figure 4 can be considered as factored values at ultimate limit states (ULS). The ultimate load transfer ranges from 52 to 131 kN/m for sandy silt to silty sand, from 99 to 148 kN/m for glacial till of silty clay to silty sand mixture and from 400 to 417 kN/m for shale bedrock. The ultimate bond capacity (skin friction) ranges from 91 to 259 kN/m^2 for sandy silt to silty sand, from 211 to 314 kN/m^2 for silty clay till to silty sand till and from 849 to 884 kN/m^2 for shale bedrock. There is not clear relationship between ultimate load transfer/bond capacity and tieback bond (fixed) length.

An attempt was made to relate ultimate load transfer (bond strength) and bond capacity (skin friction) with the Standard Penetration Test (SPT) N-value as shown in Figure 5. The ultimate load transfer and bond capacity (skin friction) are found to increase with the SPT N-value

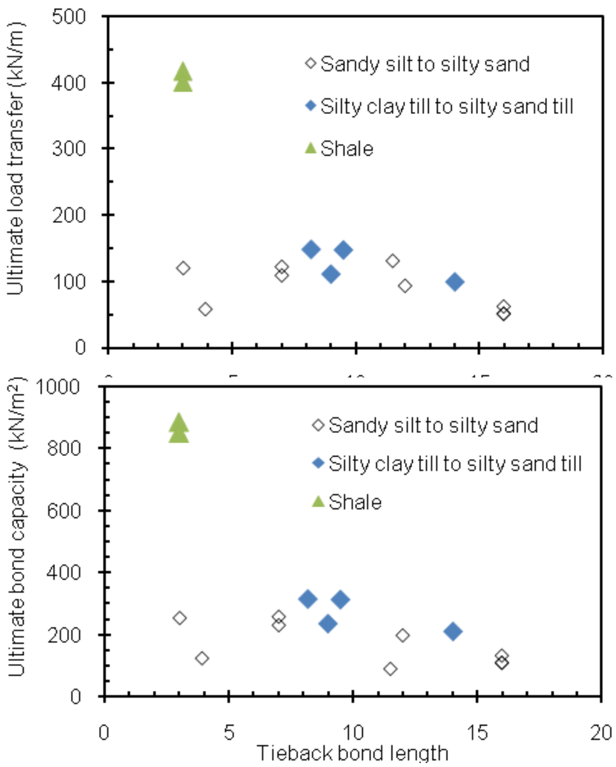

Figure 4. Ultimate load transfer/bond capacity with bond length

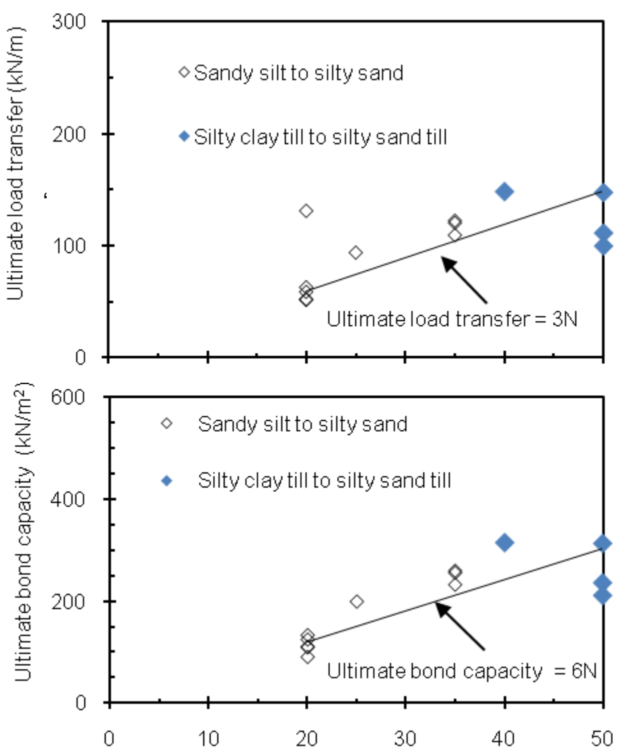

Figure 5. Ultimate load transfer/bond capacity with SPT N-value

as one would expect.

For the preliminary design of post-grouted tiebacks, the following relationships may be used to estimate the factored bond capacity at ULS for soils with SPT N-values not greater than 50 blows per 300mm penetration:

Ultimate load transfer (bond strength) (kN/m) = 3N [2]

Ultimate bond capacity (skin friction) (kN/m^2) = 6N [3]

The proposed correlations should be verified by performance tests.

4 CONCLUSIONS

Tieback performance and proof tests carried out at six sites in Southern Ontario indicated the bond capacity used in current tieback design for glacial till and shale bedrock is reasonable, but generally overestimated bond capacity in granular soils probably due to the soil disturbance during the tieback installation. The tieback load transfer (bond strength) and bond capacity (skin friction) in various soils summarized in this paper can be used as a starting point of reference for tieback design. Statistical correlations between load transfer/capacity and SPT N-value have been derived from limited data. Use of the correlations should be verified by tieback performance tests.

REFERENCES

British Standards Institution 1989. *Ground Anchorages*, BS 8081, BSI, London, England.

Canadian Geotechnical Society 2006. *Canadian Foundation Engineering Manual*, 4th ed., BiTech Publisher Ltd, Richmond, BC, Canada.

FHWA 1999. *Ground Anchors and Anchored Systems*, FHWA-IF-99-015, Federal Highway Administration, Washington, D.C., USA.

Hanna, T.H. 1982. *Foundations in Tension Ground Anchors*, 5th ed., McGraw Hill, New York, NY, USA.

PTI 1996. *Recommendations for Prestressed Rock and Soil Anchors*, 3rd *ed.*, Post-Tensioning Institute, Phoenix, Arizona, USA.