Full-Scale Tiedown Tests for the Central Artery/ Tunnel Project

A test program was developed to obtain data that would provide the basis for more accurately predicting tiedown capacity for the preliminary design of this large project.

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During the geotechnical investigation phase of the Central Artery/Tunnel (CA/T) Project in Boston, Massachusetts, full-scale tiedown tests were performed at two test sites in the Central Area of the project (see Figure 1). A total of 14 instrumented tiedowns — seven at each of two ventilation buildings — were installed and tested. The tiedowns were installed with anchorages in specific soil or rock strata, including glaciomarine deposits, glacial till and rock strata that ranged from completely weathered to slightly weathered argillite. Three types of tiedown tests were performed. Initially, a load test to failure was performed on one tiedown anchored in each stratum at each site. Based on the results, reference design loads were selected and load tests (that included a 72-hour constant load) were performed on the remaining tiedowns. Instrumentation of the tiedowns included vibrating wire strain gages mounted on the tiedown strands at various locations within the anchor zone to provide data to evaluate load transfer. The data that were collected have been evaluated according to current methodologies used to predict tiedown capacity and behavior.

Site preparation began in January 1992, tiedown installation proceeded through the middle of February 1992 and most of the testing was completed by April 1992. Long-term monitoring of four tiedowns (two at each site) was also performed for approximately two years.

The purpose of the tiedown element test program was to obtain site-specific data regarding the capacity and performance of tiedowns in the Central Area. The objectives of the tiedown element test program were:

- To obtain data on the grout-to-ground bond capacity of the tiedowns with anchorages in the glaciomarine, glacial till or rock (argillite) strata;
- To study the load transfer and the long-term creep behavior of the tiedown anchorages; and,
- To obtain data that would provide the basis for the design of permanent tiedowns that

FIGURE 1. Location of the tiedown test sites.

would resist hydrostatic uplift pressures on the proposed vent buildings.

Methods for Predicting Capacity

The methods that can be used to predict tiedown capacity can range from the use of empirical data on bond stress from previous tests in similar subsurface conditions to estimates of capacity based on in-situ or laboratory test parameters.

The estimated design capacities of similar anchors in similar ground conditions are determined by multiplying the surface area of the sides of the anchor bond zone by the "bond stress" appropriate for the ground conditions. This simplified approach does not account for the inherent variability of subsurface conditions and construction methods, or the effects of foundation geometry and stiffness. However, it does provide a useful starting point before considering more refined approaches.

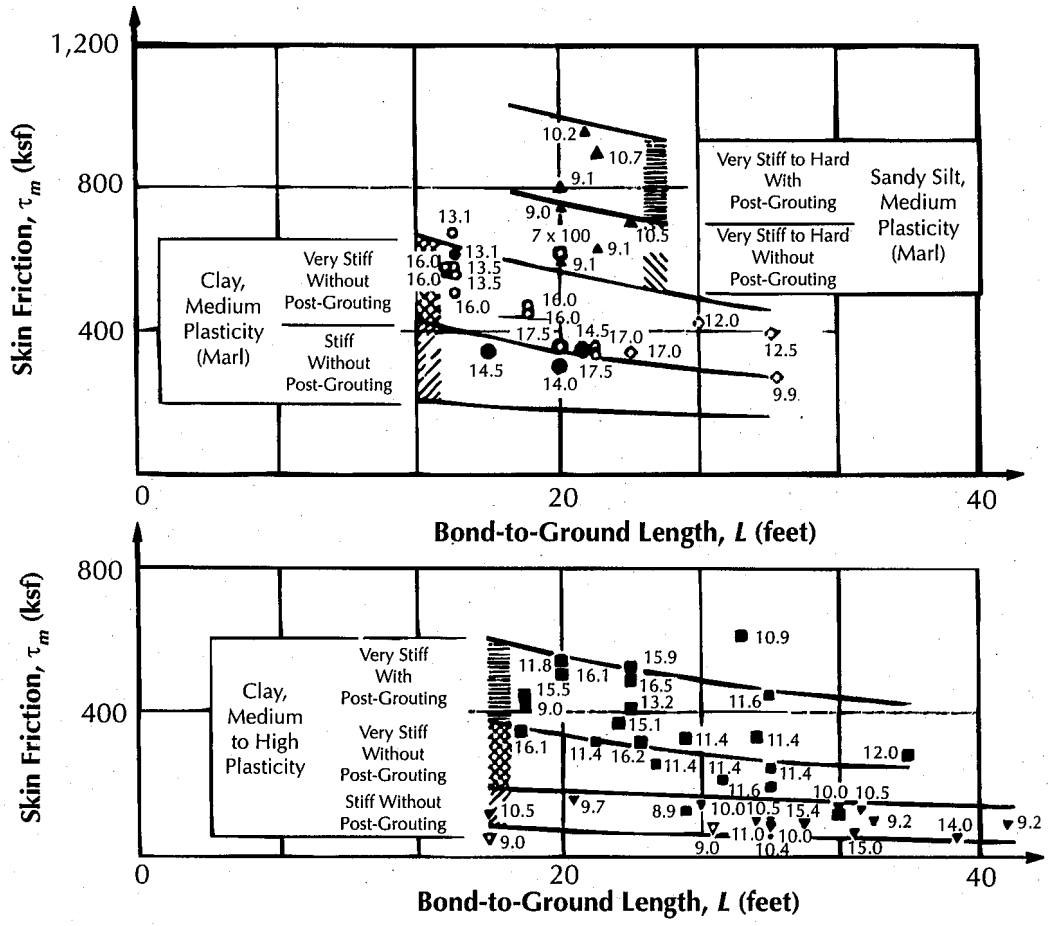
Use of empirical data from the literature may be appropriate for smaller projects, but can be uneconomical for many projects due to the large factor of safety required to account for the uncertainty inherent in the method. Empirical values of bond stress contained in the literature and used for calculating tiedown capacity in various soil and rock materials are provided in Table 1 and in Figures 2 and 3.

Other approaches that are available for predicting anchor capacity are based on correlations with in-situ and laboratory tests. These methods are generally based on direct comparisons of the measured soil or rock parameters and the measured tiedown capacity at several sites and are extrapolated to estimate tiedown capacities at other sites where similar field or laboratory tests have been performed.

Among the test parameters used to estimate anchor capacity are standard penetration test (SPT) N-values, pressuremeter limit pressures, the unconfined compressive strength of rock and the compressive strength of concrete. Some common correlations of SPT N-values to observed side shear resistance and anchor capacity are shown in Figures 4 and 5. Similarly, correlations of side shear resistance versus

TABLE 1.
Selected Empirical Values of Bond Stress

Bond Stress Ultimate or [Working] (ksf)	Pile or Anchor & Soil/Rock Characteristics	Source (see References)
0.5-11	Clay till (Boston, Mass.)	Ref. 1
8.8	L = 4.5 meters, d = 11.4 centimeters	Ref. 2 (Figure 4-19b)
4.2	L= 5 meters, clay, medium plasticity (marl)	Ref. 2 (Figure 4-24)
2	L= 5 meters, clay, medium to high plasticity	Ref. 2 (Figure 4-24)
5	L= 5 meters, clay, medium to high plasticity, post-grouted	Ref. 2 (Figure 4-24)
9	L= 5 meters, sandy silt, medium plasticity (marl), post-grouted	Ref. 2 (Figure 4-24)
8	L= 5 meters, sandy silt, medium plasticity (marl)	Ref. 2 (Figure 4-24)
[0.5]	Friction pile in inorganic clay	Ref. 3 (Section 1214.3.2)
0.15-0.6	Caissons in silt & soft clay	Ref. 4 (Table 21.1)
1.0-4.0	Caissons in very stiff clay	Ref. 4 (Table 21.1)
2.09-6.27	Stiff to very stiff clay	Ref. 5
6.27-10.44	Sandy silt	Ref. 5
4.2-7.6	Sand & silt for 11.4-centimeter diameter	Ref. 6
1.7-3.4	Silt/clay mixture for 11.4-centimeter diameter	Ref. 6
8.56	L = 4.8 meters, d = 11.4 centimeters	Ref. 2 (Figure 4-19b)
2.0	Weathered sandstone & shale; failure not reached	Ref. 4 (Table 20.5)
[25]	For f_c' = 3,500 psi	Ref. 4 (Table 20.5)
[28.8]	Sound, hard rock (allowable)	Ref. 4 (Table 20.5)
17.3-28.8	Slates & hard shales	Ref. 2 (Table 4-4)
2.9-17.3	Soft shales	Ref. 2 (Table 4-4)
4.39-17.3	Soft shale	Ref. 2 (Table 4-5)
20	Weathered sandstone & shale	Ref. 4 (Table 20.5)
8.5	Soft shale, 11.4-centimeter diameter	Ref. 4 (Table 20.5)
7.3	Weak shale	Ref. 5
2.1-8.4	Soft sandstone & shale	Ref. 5
7.7	Soft sandstone & shale	Ref. 5
< 87.7	Competent rock (unconfined compressive strength > 20 N/mm^2)	Ref. 5
7.3-14.6	Weak rock	Ref. 5
8.2	Very weathered shale (argillaceous)	Ref. 7
[12.95]	Shale	Ref. 7
[6.3]	Shale (safety factor = 2.1)	Ref. 7
[2.7-5.0]	Shale	Ref. 7
[12.95]	Grey siltstone	Ref. 7
[17.1]	Argillite	Ref. 7
[13.2]	Mudstone	Ref. 7
[5.2-10.4]	Bedded sandstone & shale	Ref. 7
[1.5]	Shale & sandstone (safety factor = 1.5)	Ref. 7
2.1	Shale & sandstone	Ref. 7



Failure Load Was Reached	Failure Load Was Not Reached	Post-Grouting	Type of Soil	W_L%	I_P%	I_C%
▲	△	Without	Silt, Very Sandy (Marl) Medium Plasticity	~45	~22	~1.25
▲	△	With				
●	○	Without	Clay (Marl) Medium Plasticity	32-45	14-25	1.03-1.14
●	○	With				
●	○	Without		36-45	14-17	1.3-1.5
●	○	With				
	◇	Without	Silt, Medium Plasticity	23-28	5-11	0.7-0.85
■		Without	Clay Medium to High Plasticity	48-58	23-35	1.1-1.2
■		With				
▼	▽	Without		45-59	16-32	0.8-1.0

Bond Resistance in Cohesive Soil for Various Fixed-Anchor Lengths With or Without Post-Grouting (From Ref. 8)

FIGURE 2. Empirical side shear resistance in cohesive soils.

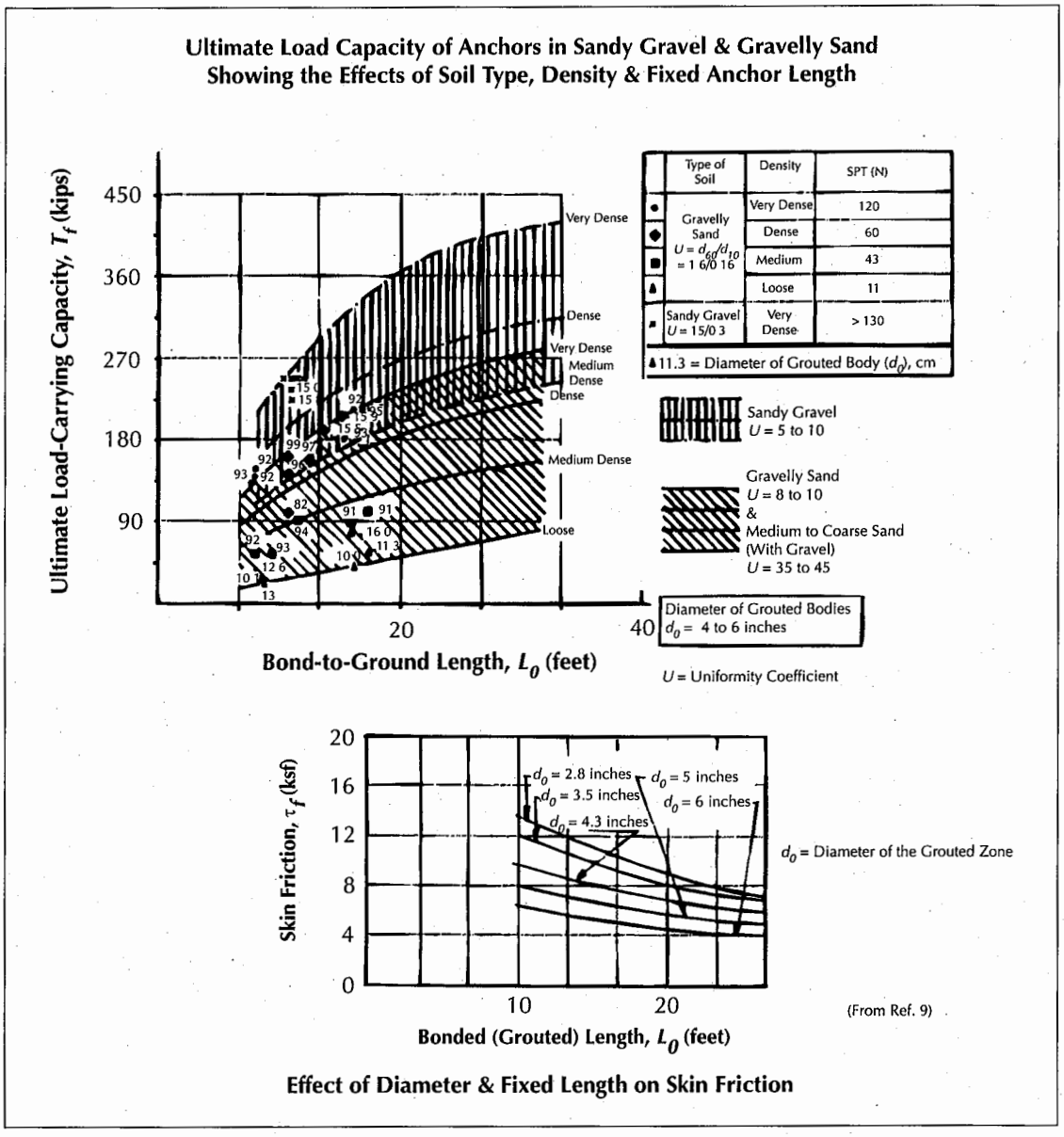


FIGURE 3. Empirical side shear resistance in sandy gravel.

pressuremeter limit pressure and side shear resistance versus unconfined compressive strength of rock are provided in Figures 6 and 7.

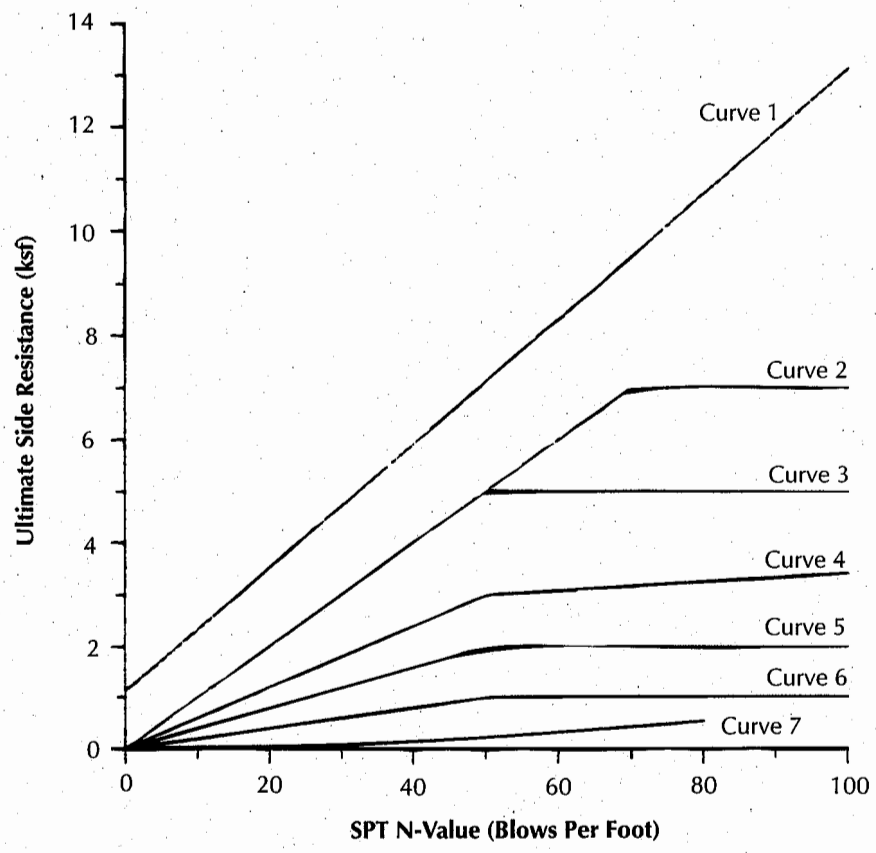
Finally, there are theoretical approaches that may be used to estimate anchor capacity in soils based on empirical parameters commonly used to calculated side resistance for piles. Three such methods are:

- The alpha (α) method;
- The beta (β) method; and,
- The lambda (λ) method.

The α-method is a total stress analysis and assumes undrained conditions.[18] The ultimate capacity, Q_u, of an anchor in soil can be calculated using the following equation:

$$Q_u = \pi D \int_0^L \alpha(z) S_u(z) dz$$

Where:



Curve 1 — Analytical Model & Field Tests on Piles in Sand (Ref. 10)
Curve 2 — Rough Drilled Shafts, Silty Clays or Silty Sands (Ref. 11)
Curve 3 — Smooth Drilled Shafts (Ref. 11)
Curve 4 — Cohesionless Soils (Ref. 12)
Curve 5 — Full Displacement Piles in Sand (Ref. 13)
Curve 6 — Non-Displacement Piles in Sand (Ref. 13)
Curve 7 — Lab & Field Testing of Two Driven Piles in Sand (Ref. 14)

FIGURE 4. Correlations between side shear resistance and SPT N-value.

D = Diameter of anchor bond zone
L = Length of the bond zone
z = Depth along the bond zone
S_u = Undrained shear strength of the soil
α = An empirical reduction factor to account for the effects of construction on undrained shear strength

The value for α is obtained empirically. Several sources have suggested values for α.[18-22]

The λ-method also assumes that the side shear resistance is a function of undrained shear strength. However, it also considers the effect of the mean effective overburden stress using Rankine earth pressure theory. The λ-method is described by the equation:

$$f_s = \lambda(\sigma'_m + 2S_u)$$

Where:

f_s = Unit side shear capacity
σ'_m = Mean effective overburden stress
λ = Empirical reduction factor

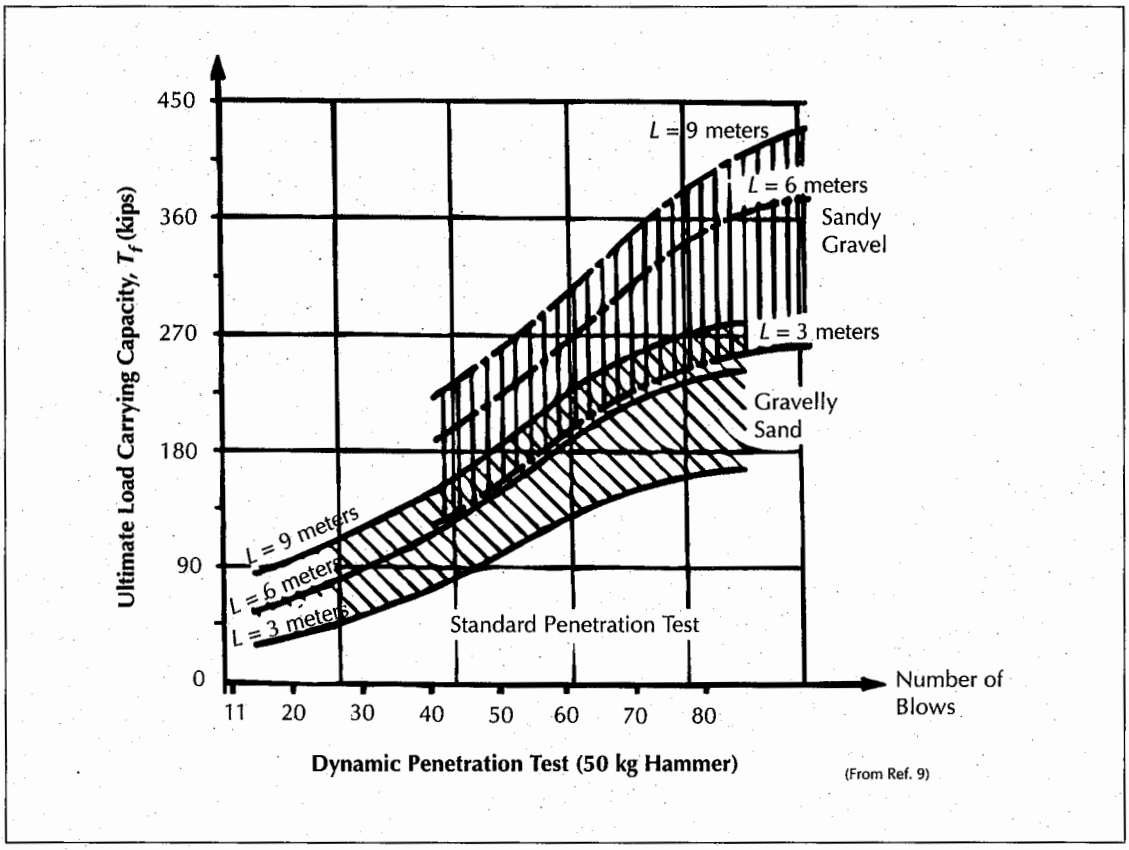


FIGURE 5. Empirical anchor capacity versus SPT N-value.

Several sources give values for λ.[16,23]

The β-method is an effective stress analysis that can be used for either drained or undrained conditions. The equation used to determine anchor capacity in the β-method is:

$$Q_u = \pi D \int_0^L \sigma'(z)\beta(z)dz$$

Where:

β = $K\tan\delta$
δ = Friction angle of the anchor/ground interface
K = Coefficient of horizontal stress
σ' = Effective vertical stress

Values for the various parameters can be found in several sources.[2,16,24-26]

Site & Subsurface Conditions

Seven tiedowns were installed and tested at each test site. Both test sites have been used as parking areas and the ground surface at each site is relatively level. The ground surface of Test Site No. 1 is paved with bituminous concrete. The ground surface of Test Site No. 2 is unpaved. Three tiedowns were installed and tested in the soil stratum of interest at each test site — the glacial till and the glaciomarine deposit at Test Site Nos. 1 and 2, respectively. In addition, two tiedowns were installed in the upper, less competent rock, and two tiedowns were installed in the lower, more competent rock at each site. The arrangement of the tiedown elements and the subsurface conditions at Test Site Nos. 1 and 2 are illustrated in Figures 8 and 9, respectively.

The tiedowns at Test Site No. 1 were installed approximately 13 feet apart along a straight line. Three of the tiedowns were installed with anchor zones in the glacial till. The anchor zones in the glacial till were about 15 feet long and centered about 55 feet below the ground surface. Post-grouting was performed on one of

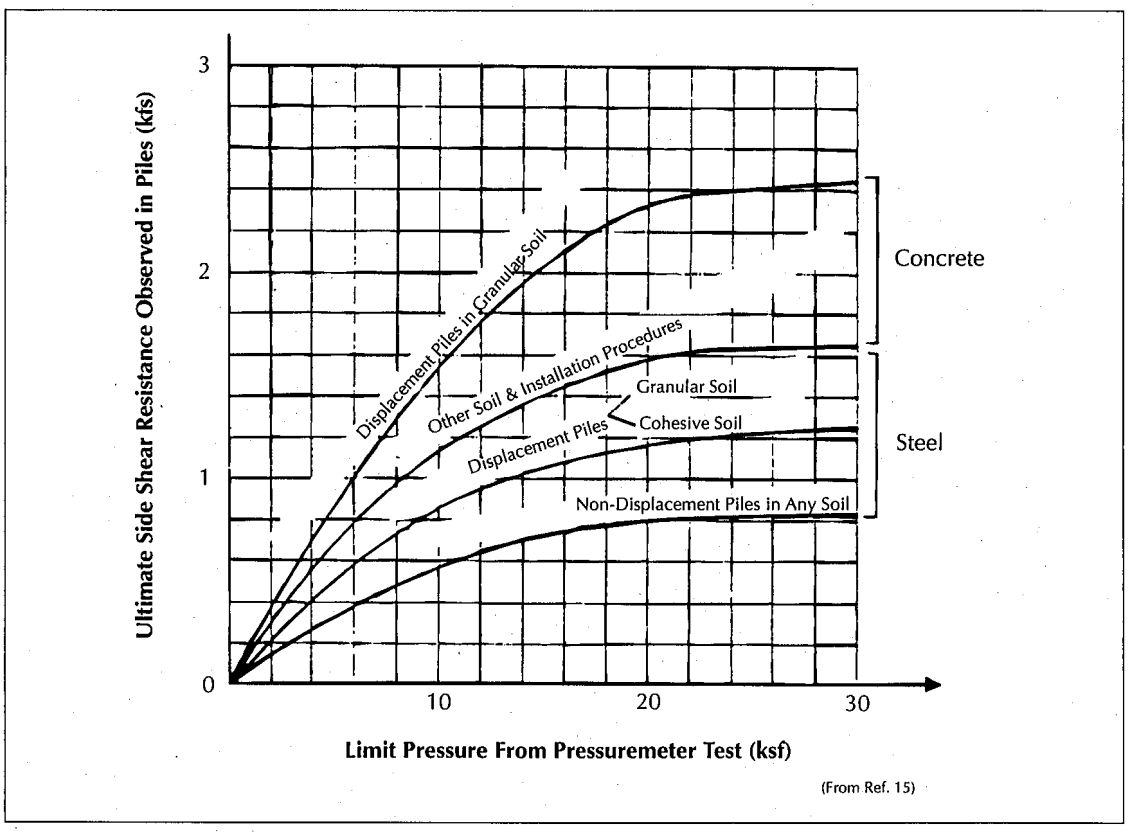


FIGURE 6. Correlations between side shear resistance and pressuremeter limit pressure.

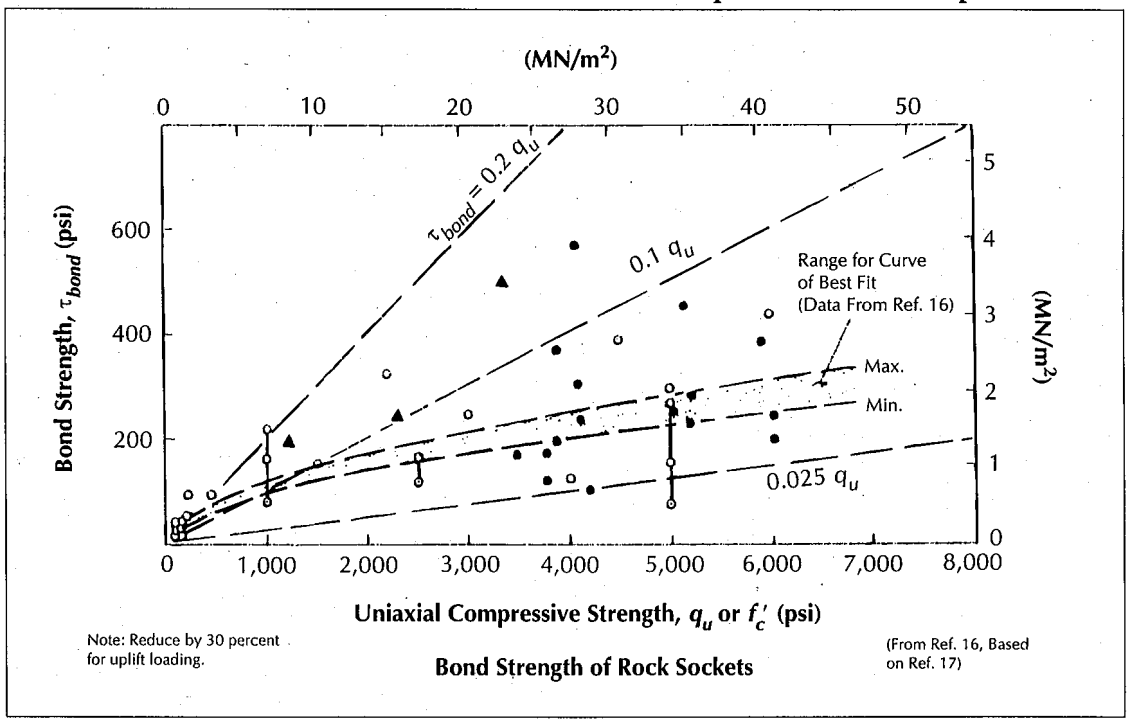


FIGURE 7. Correlation between side shear resistance and unconfined compressive strength.

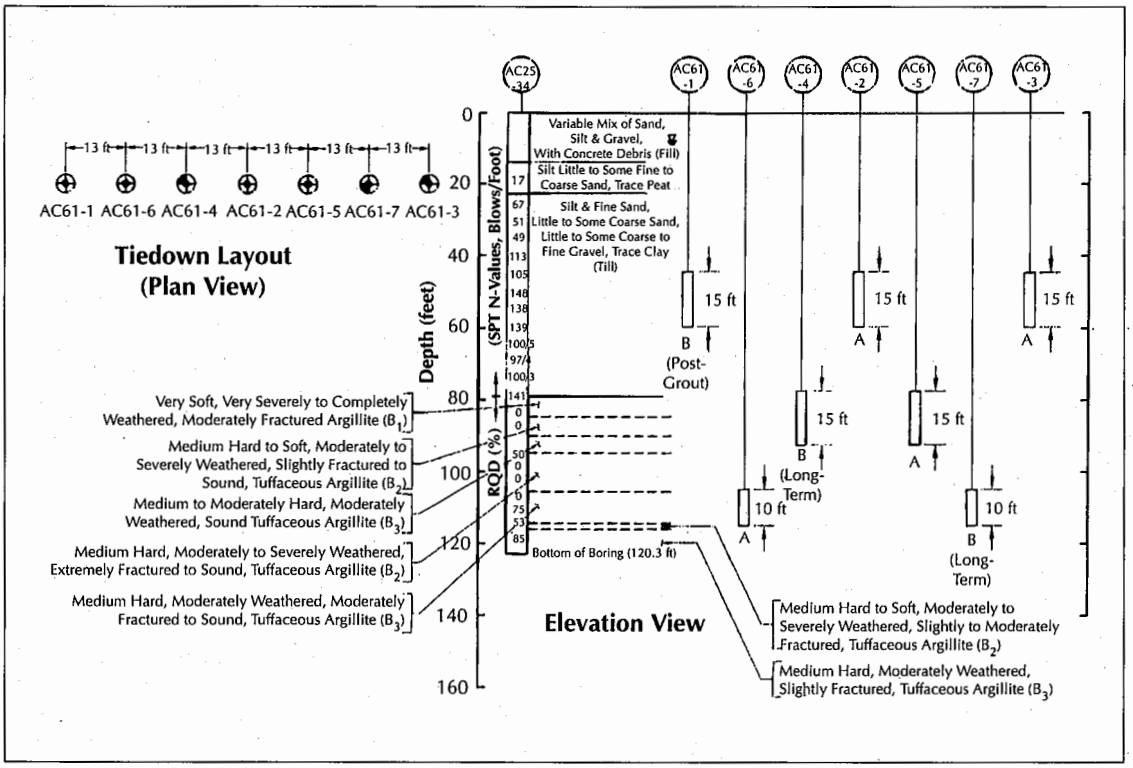


FIGURE 8. Tiedown layout and subsurface data for Test Site No. 1.

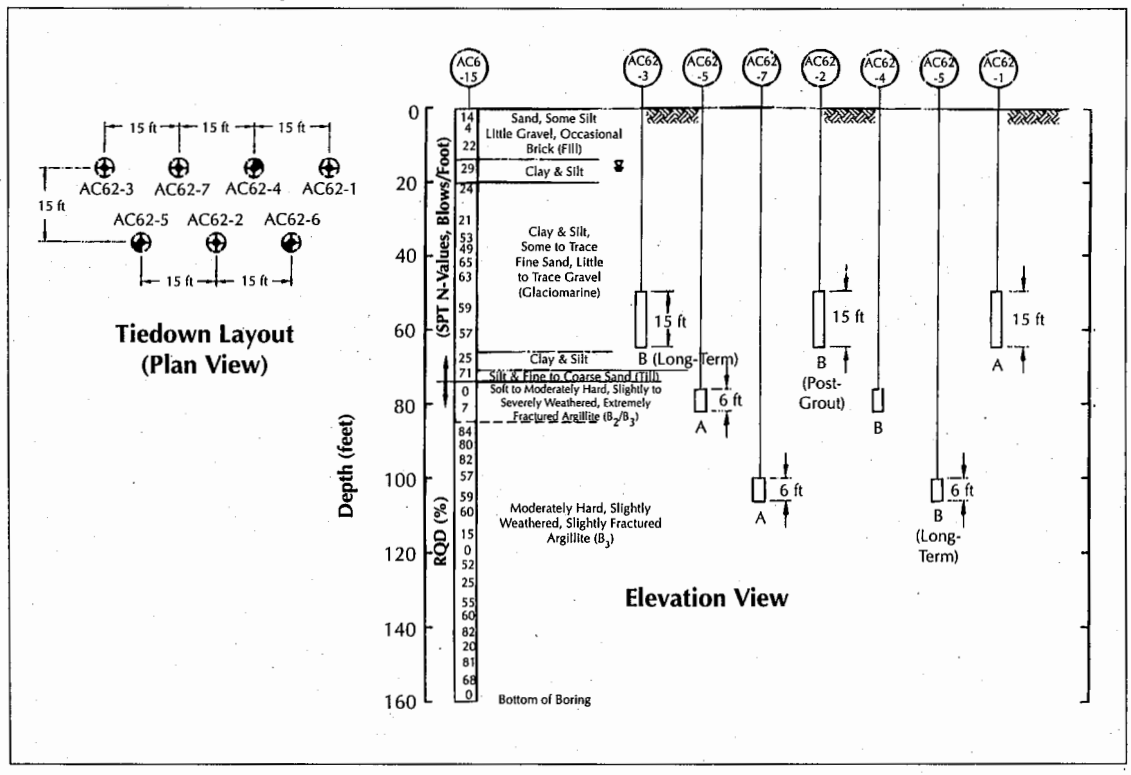


FIGURE 9. Tiedown layout and subsurface data for Test Site No. 2.

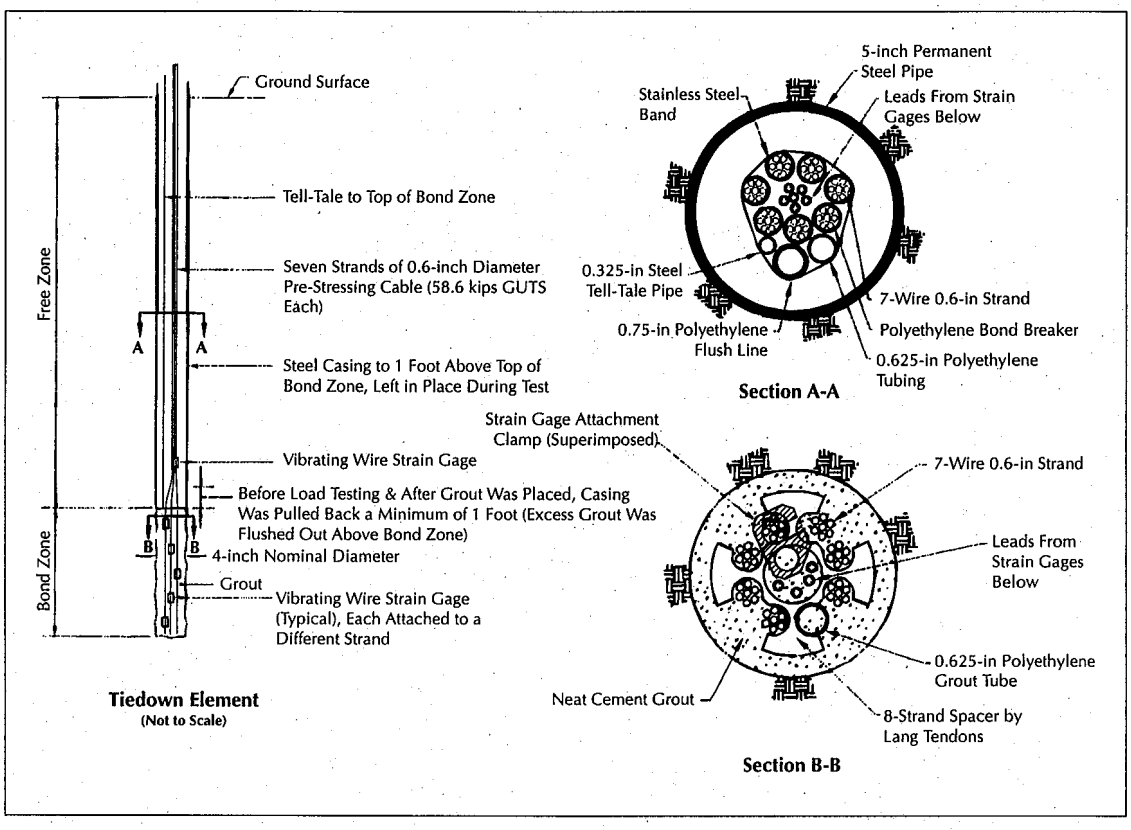


FIGURE 10. Tiedown element details.

the tiedowns (AC61-1) in the glacial till. Two tiedowns were installed with anchor zones in an area of severely to completely weathered argillite just below the top of rock. The anchor zones were approximately 15 feet long and were centered about 85 feet below the ground surface. The remaining two tiedowns were installed with 10-foot-long anchor zones in an area of severely weathered argillite. The anchor zones for this last pair of tiedowns were centered about 110 feet below the ground surface.

The tiedowns at Test Site No. 2 were installed approximately 15 feet apart in a staggered pattern. Three tiedowns were installed with anchor zones in the glaciomarine deposit. The anchor zones in the glaciomarine were about 15 feet long and were centered about 57 feet below the ground surface. Post-grouting was performed on one of the tiedowns (AC62-2) in the glaciomarine deposit. Two tiedowns were installed with anchor zones in an area of slightly to severely weathered argillite just below the top of rock. The anchor zones were approximately 6 feet long and centered approximately 85 feet below the ground surface. The short anchor length was chosen so that grout/grout bond failure could be achieved and an ultimate bond value determined. The remaining two tiedowns were installed with 6-foot-long anchor zones in an area of slightly weathered argillite. The anchor zones for this last pair of tiedowns were centered at about 110 feet below site grade.

Additional information regarding the test sites, soil and rock conditions, and test procedures, details and results can be found in several reports to the CA/T Project.[27-32]

Tiedown Elements

The tendons for the tiedown elements consisted of seven strands of nominal 0.6-inch-diameter, seven-wire prestressing steel (see Figure 10). Each strand had a nominal cross-sectional area of 0.217 square inch, a guaranteed ultimate tensile strength (GUTS) of 58.6 kips (which corresponds to an ultimate stress of 270 ksi) and an

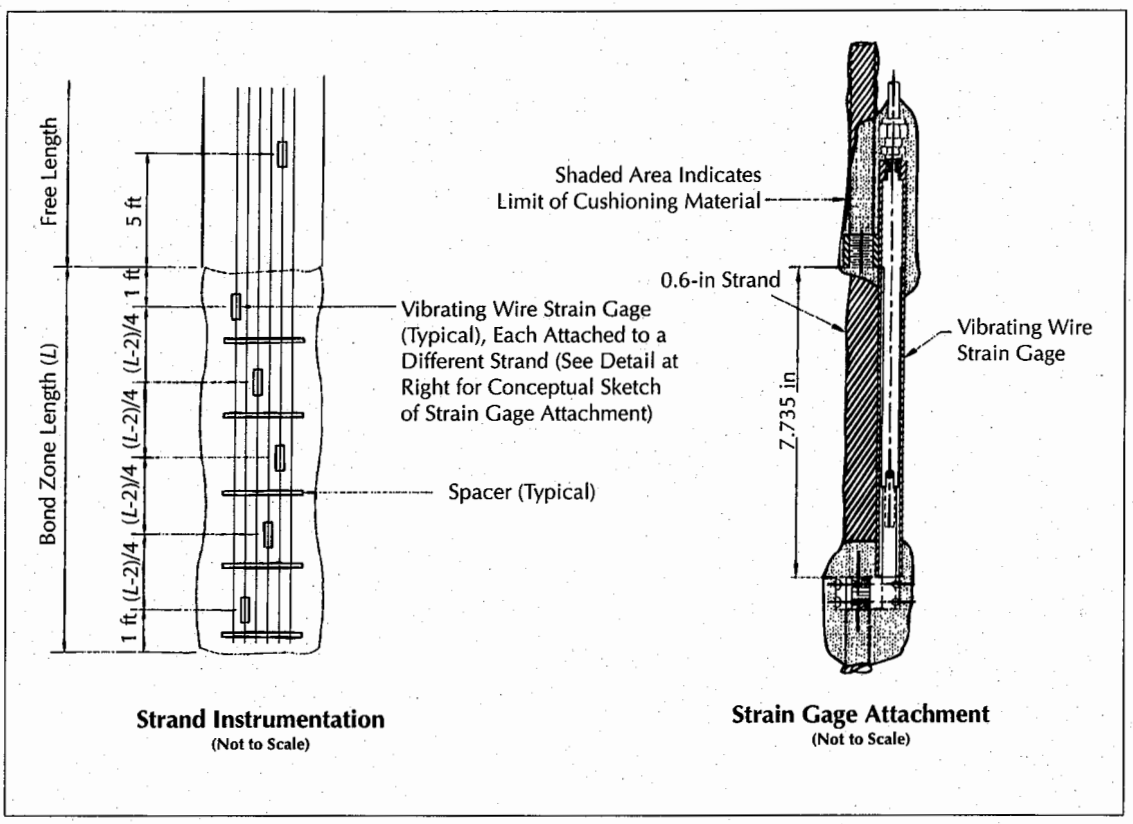


FIGURE 11. Strand instrumentation and strain gage attachment.

approximate modulus of elasticity of 28,600 ksi. The bond zones were grouted with Portland Type I/Type II neat cement grout. Cube samples of the grout used in each tiedown were made, and an independent laboratory tested the cube samples. The results of tests on the cube samples indicate 28-day grout compressive strengths of at least 5,000 psi.

Tiedown Installation

The contractor developed and implemented procedures for installing the tiedowns, including instrumentation and instrumentation testing. Prior to installing the tiedowns, the contractor performed laboratory strain gage attachment tests to demonstrate that the strain gage installation method that was selected would work. The components of a typical tiedown are shown in Figure 10. The unbonded length section of each strand was greased and sheathed with polyethylene to provide a bond breaker in the event that grout remained above the proposed top of bond zone. Five or six of the seven strands in each tendon were equipped with vibrating wire strain gages, depending on the length of the bond zone. Six vibrating wire strain gages were installed on tiedowns with 10- and 15-foot-long bond zones, and five gages were installed on tiedowns with 6-foot-long bond zones. One gage was mounted on the free length of each tendon. The remainder of the gages were mounted in the anchorage bond zone.

During fabrication, each strand was uncoiled and laid flat. The strain gages were attached to the strands with steel clamps (see Figure 11). Plastic spacers designed for eight-strand anchors were used to hold the seven strands of the tiedown, the grout tube and the instrumentation cables in place in the bond zone. The wires from the strain gages ran through the opening in the center of the plastic spacers and up along the strands to the top of the tendons. A 0.625-inch-diameter grout tube was placed in the eighth strand location of the plastic spacer. For tendons AC61-1 and AC62-2,

a tube-a-manchette was placed in the eighth tendon slot for post-grouting, and the grout tube was placed with the strain gage wires in the center of the tendon assembly. The center of the top strain gage in the bond zone was located approximately 1 foot from the top of the bond zone. The center of the bottom strain gage was located roughly 1 foot above the bottom end of the tendon. The center of the free length gage was located approximately 5 feet above the top of bond zone. Strain gages located between the top and bottom of the bond zone were evenly spaced with no two gages attached to the same strand.

During shipping to the site, the tendon free length was rolled and tied so that the free length was in a 6-foot-diameter coil. The zone where strain gages were present was not coiled and was kept straight at all times in order to avoid causing strain gage displacements beyond the range of the instruments. Operation of the strain gages was checked when the tendon arrived at the site and before installation.

At each tiedown location, a nominal 9-inch diameter casing was advanced to below the fill layer. Then a 6.75-inch diameter, tricone roller rock-cutting bit was used to drill to the top of the bond zone. A 5-inch inside diameter casing was lowered into the open hole after drilling was completed. In cases where the 5-inch casing could not be inserted to the proper depth, the drill rig was used to spin the casing in place. Once the 5-inch casing was in place, the 9-inch casing was removed. All tiedown holes were drilled to the tops of the bond zones, and 5-inch casings were placed prior to the arrival of the tendons at a site. When the tendon arrived at the site, a 4.5-inch, tricone roller rock-cutting bit was used to drill the bond zone. The bond zone was overdrilled by 1 to 2 feet to allow for the settlement of suspended drill cuttings upon completion of drilling. Water was the only fluid used during the drilling of the bond zones. Fresh water was obtained from a hydrant at each site. Recirculated water was collected during drilling and pumped into a tank, suspended drill cuttings allowed to settle out and then the water was reused.

After the bond zone of a hole was completed, the tendon was unloaded from the boom truck, unrolled and laid flat at the site. The boom truck then picked up the tendon at a point approximately 50 feet from the bottom of the tendon and set it over the hole. The tell-tale and the flush tube were attached during the installation process. During installation, the lowering of the tendon into the hole was temporarily stopped as sections of tell-tale were added. When the tendon was at the proper depth, the drill rig supported it until grouting was completed.

Primary grouting of all tiedown elements was performed using low-pressure tremie grouting methods. Portland Type I/Type II cement was used at a ratio of 5 gallons of fresh water to 1 bag (1 cubic foot) of cement for a water-cement ratio (by weight) of approximately 0.44. After the tendon was placed and the strain gages were found to be operating satisfactorily, the grout was mixed and placed. The amount of water and number of bags of cement used for each batch of grout mixed were recorded. After the grout was mixed, it was pumped into the hole via the grout tube attached to the tendon. A pressure gage was mounted between the grout tube and the hose from the pump and was monitored during pumping. Grout was pumped into the tube until grout of the same consistency being pumped in began exiting the top of the casing. Compression tests (ASTM Standard C942-86) were performed on the grout cubes. Following the completion of low-pressure grouting, any grout in the free length portion of the tiedown was removed by pumping fresh water through the flush tubes until clear water was observed exiting the top of the casing. After flushing was completed, the 5-inch diameter casing was pulled up 1 foot, and the hole was flushed again until clear water flowed from the top of the casing. The strain gages mounted on the tiedown were checked before and after grouting.

For the post-grouting of selected tiedowns (AC61-1 and AC62-2), up to four bags (4 cubic feet) of cement were mixed with water in a ratio of one bag of cement to 5 gallons of water. The grout mix was subsequently injected via the pre-installed tube-a-manchette into the bond zone until no further grout take occurred or until the rate of grout take indicated that leakage into the open casing was occurring. Post-grouting was done at least 24 hours — but not more than 72 hours — after installation of the

primary grout. At AC61-1, grout pressures up to 200 psi were recorded during post grouting, with a grout take of 4 cubic feet 1 day after the primary grouting. At AC62-2, post-grouting was performed on two consecutive days. The first day had a grout take of 2.5 cubic feet with grout pressures up to 400 psi, and the second day registered grout pressures as much as 620 psi with 2.5 cubic feet of grout injected. After post-grouting, the free length of the tiedown was thoroughly flushed with water to remove any fresh grout inside the casing above the anchor zone.

Test Setup & Instrumentation

Center-hole, double-acting hydraulic rams controlled by an electrically operated pump were used to apply the test load. Two 200-ton rams, each with a maximum travel of 12 inches, were used during the tests. For tests on the tiedowns anchored in the deeper rock (AC61-6 and AC62-7), the two rams were mounted in series — one on top of the other — to provide sufficient travel to accommodate the potential movement of the relatively long tiedown during load testing to failure. For the remainder of the tiedowns, one jack was sufficient for each anchor test. With two jacks available, there were also cases when two tiedown tests were performed simultaneously. Reaction for a tiedown test was provided by reaction beams supported by two wood reaction pads. The reaction beams consisted of double HP14x102 steel beams. The timber reaction pads were approximately 1 foot thick, 4 feet wide and 11 feet long each.

The instrumentation was installed to permit monitoring of the loads, displacements, pressures and strains associated with the various elements of the tiedown test program. The instrumentation included load cells, pressure transducers, dial gages, strain gages and optical survey equipment. The pressure in the hydraulic line to the loading ram was measured using a vibrating wire pressure transducer and was used to calculate the load in the ram. The load on the tiedown during testing was also monitored using an electrical resistance load cell. For the long-term setup, a vibrating wire load cell was used to monitor the force on the tiedown after it was locked off at the reference design load (RDL).

Dial gages were set to measure the movements of the tiedown relative to a fixed reference beam (which was supported independent of the reaction system during testing). Three dial gages were positioned roughly 120 degrees apart on top of the stressing head, about 5 inches radially from the axis of the tiedown, to monitor the vertical displacement. The main dial gage was a combination dial gage and digital counter with an 18-inch travel and 0.001-inch graduations. The remaining two gages were standard dial gages, each with a 2-inch travel and 0.001-inch graduations. Occasionally, the two dial gages with the lesser travel were reset when the full travel range was exhausted. A dial gage with a 2-inch travel and 0.001-inch graduations was also used to monitor the movement of the tell-tale rod that was attached to the top of the bond zone during the test. An optical surveying system was utilized both as a secondary check for the movement of the stressing head and to detect movement of the reference beams and bearing pads relative to a fixed temporary bench mark.

The vibrating wire strain gages were attached to the tiedown strands to provide information on the load in the strand at various locations along the length of the bond zone and in the free length. Calibration of the strain gages was performed by the manufacturer, and the operation of all the gages was tested on delivery, during tendon fabrication as well as before, during and after tiedown installation. The strain gages and all other vibrating wire instruments (pressure transducer and load cell) were monitored using a readout box and data acquisition system.

Before the fabricated strands were rolled and bundled for shipping, readings were taken and recorded for each strain gage. When the tendons arrived at the site, the readings were taken again before unbundling the tendon. After the tendons were unloaded and unrolled, another set of readings was taken. Readings were taken once the tendon was placed in the hole and again shortly after grouting and flushing were completed. During installation of the tiedowns at Test Site No. 2, a combination of extremely cold weather, slippage associated with the use of non-hardened set-screws to attach the strain gages to the tendons and other factors associ-

ated with the placement of the instrumented tiedown in the pre-drilled hole caused gage deformations greater than the strain gages' range and resulted in about 30 percent of the strain gages failing. Later, during the installation of tiedowns at Test Site No. 1, hardened set-screws coated with Loktite were used, the weather was warmer and minor changes in instrumentation placement and installation procedures were made that improved the survival rate of the strain gages to greater than 90 percent.

Testing Procedures

Four types of load tests were performed. First, a loading test to failure (referred to as a Type A test) was performed on one tiedown anchored in each stratum at each site. Based on the results of the Type A testing, an RDL was determined for each tiedown using procedures described by Cheney.[33] The RDL was used to select the loading level for the three-day constant-load test (Type B test) on the other tiedown(s) in each stratum. In addition, the relaxation behavior of tiedowns with anchor zones in the completely to severely weathered rock (AC61-4), in the severely weathered rock (AC61-7), in the glaciomarine (AC62-3) and in the slightly weathered rock (AC62-6) were monitored long term. The long-term relaxation monitoring continued for approximately two years after the lock-off of the last tiedown. The Type B test anchors not used for long-term relaxation tests were subjected to quick-load tests to failure following completion of the Type B tests.

The Type A test was designed to provide data on the overall load capacity of the tiedown. The Type B test was performed to provide data on the creep and relaxation behavior of the tiedown at the RDL. The RDL was generally determined based on results of the Type A test, but was modified in some cases, depending on the performance of the tiedown observed during the Type B test. During a Type A test, the load was applied in increments of 10 percent of GUTS for all seven strands (or approximately 40 kips per increment). Each increment was held for 1 hour, with readings at intervals of 1, 2, 5, 10, 20, 30 and 60 minutes after the load was applied. This sequence was followed until the tiedown could no longer carry the load without significant displacement or until the total load had reached 80 percent of GUTS (approximately 328 kips for all seven strands). After reaching failure or the maximum load, the load was reduced in four equal steps to zero. Each step was held for about 15 minutes, during which time readings were taken at intervals of 1, 2, 5, 10 and 15 minutes.

The RDL was the maximum load attained during the Type A test divided by a factor of safety of 1.5 or 90 percent of the critical creep tension, whichever was less. The critical creep tension was estimated from the Type A test results using the method described by Cheney.[33] This method, also known as the French method,[2] uses a plot of the anchor head displacement (in inches) versus time (in minutes, logarithmic scale). The creep rate, or the slope of the displacement versus log time curve, for each load increment is determined, and then the arc-tangent of the creep rate in degrees — denoted as the creep index (α) — is plotted versus load. The load corresponding to break in the creep index-load curve is denoted as the critical creep tension.

In a Type B test, a tiedown was loaded to 125 percent of the RDL (in increments of 25 percent of the RDL), with each load increment being held for 1 hour. Readings were taken at the same time intervals as during a Type A test. The load was then decreased to the RDL, and readings were taken at 1, 2, 5, 10, 20, 30 and 60 minutes. The RDL was maintained for 72 hours, with readings recorded every hour. At the end of the 72-hour hold, the load on selected tiedowns (two at each site) was locked off for long-term monitoring. The tiedowns not used for the long-term tests were unloaded to zero load in four equal steps with 1-, 2-, 5-, 10- and 15-minute readings at each step.

A quick-load retest was performed after the Type B test on the four tiedowns not used for long-term testing (AC61-1, AC61-3, AC62-2 and AC62-4) to provide additional data on tiedown capacity. During a quick-load retest, the tiedown was loaded to failure or to 80 percent of GUTS (whichever was less) in increments of 10 percent of GUTS. Each increment was held for 5 minutes, with 1-, 2- and 5-minute readings taken at each load. After maximum load was reached, the tiedown was unloaded in four equal decrements, again holding each load for 5 minutes.

TABLE 2.
Summary of Tiedown Tests at Test Site No. 1

Tiedown Number	AC61-2	ACI61-3	AC61-1 (Post-Grouted)	AC61-5	AC61-4	AC61-6	AC61-7
Soil/Rock Type	Till	Till	Till	Severely to Completely Weathered Argillite	Severely to Completely Weathered Argillite	Severely Weathered Argillite	Severely Weathered Argillite
Test Type	A	B	B	A	B††	A	B††
Length of Bond Zone (feet)	15	15	15†††	15	15	10	10
Initial Free Length (feet)*	53	53	53	85.5	85.5	115	113
Maximum Load Attained (kips)	292	198	325	160	110	212	150
Critical Creep Tension (kips)**	207	NA	NA	120	NA	160	NA
Failure Load (kips)***	200	150	250	120	NA	160	NA
Displacement at Failure Load (inches)§	3.4	2.5	3.5	3.4	3.8	5.6	NA
RDL (kips)	200	125	200	90	90	120	120
Average Load Transfer Based on Failure Load (kips/ft)§§	13.3	10.0	16.7	8.0	NA	16.0	NA
Average Side Shear Based on Failure Load (ksf)§§§	11.3 (78.6 psi)	8.5 (59.0 psi)	14.1 (98.2 psi)	6.8 (47.2 psi)	NA	13.6 (94.3 psi)	NA
Average Side Shear Based on RDL (ksf)†	11.3 (78.6 psi)	7.1 (49.1 psi)	11.3 (78.6 psi)	5.1 (35.4 psi)	5.1 (35.4 psi)	10.2 (70.7 psi)	10.2 (70.7 psi)

Notes: * The initial free length is the distance from the top of the anchor head to the top of the bond zone at the beginning of the test.
** The critical creep tension was determined using the method described by Cheney (Ref. 33).
*** The failure load was computed using the slope-tangent method, which is similar to Davisson's method for piles (Ref. 34).
§ Movement of the anchor head due to seating has been subtracted to obtain the displacement values shown.
§§ Calculated using nominal bond zone length and failure load.
§§§ Calculated using nominal side area and failure load.
† Calculated using nominal side area and the RDL.
†† This tiedown was used for the long-term creep test.
††† Post-grouting was performed on tiedown AC61-1.

For selected tiedown elements (AC61-4, AC61-7, AC62-3 and AC62-6), the tops of the tiedowns were prepared prior to the Type B test so that the RDL could be locked off for the long-term relaxation test. A vibrating wire load cell was mounted between steel bearing plates at the top of the tiedown to measure the anchor load, and a dial gage was attached at the top of the tell-tale rod so that movement of the top of the bond zone could be monitored. In addition, a steel engineer's scale was mounted on a bench mark at each site, and a slot for inserting a leveling rod on the concrete bearing pad was made for subsequent monitoring of bearing pad movement using optical survey methods.

Load-Displacement Results

The results from the load-displacement tests are summarized in Tables 2 and 3. Example plots of the results are shown in Figures 12 and 13. The failure load for each tiedown was estimated using the slope-tangent method,[35] which is similar to Davisson's method for determining the capacity of piles.[34] In the slope-

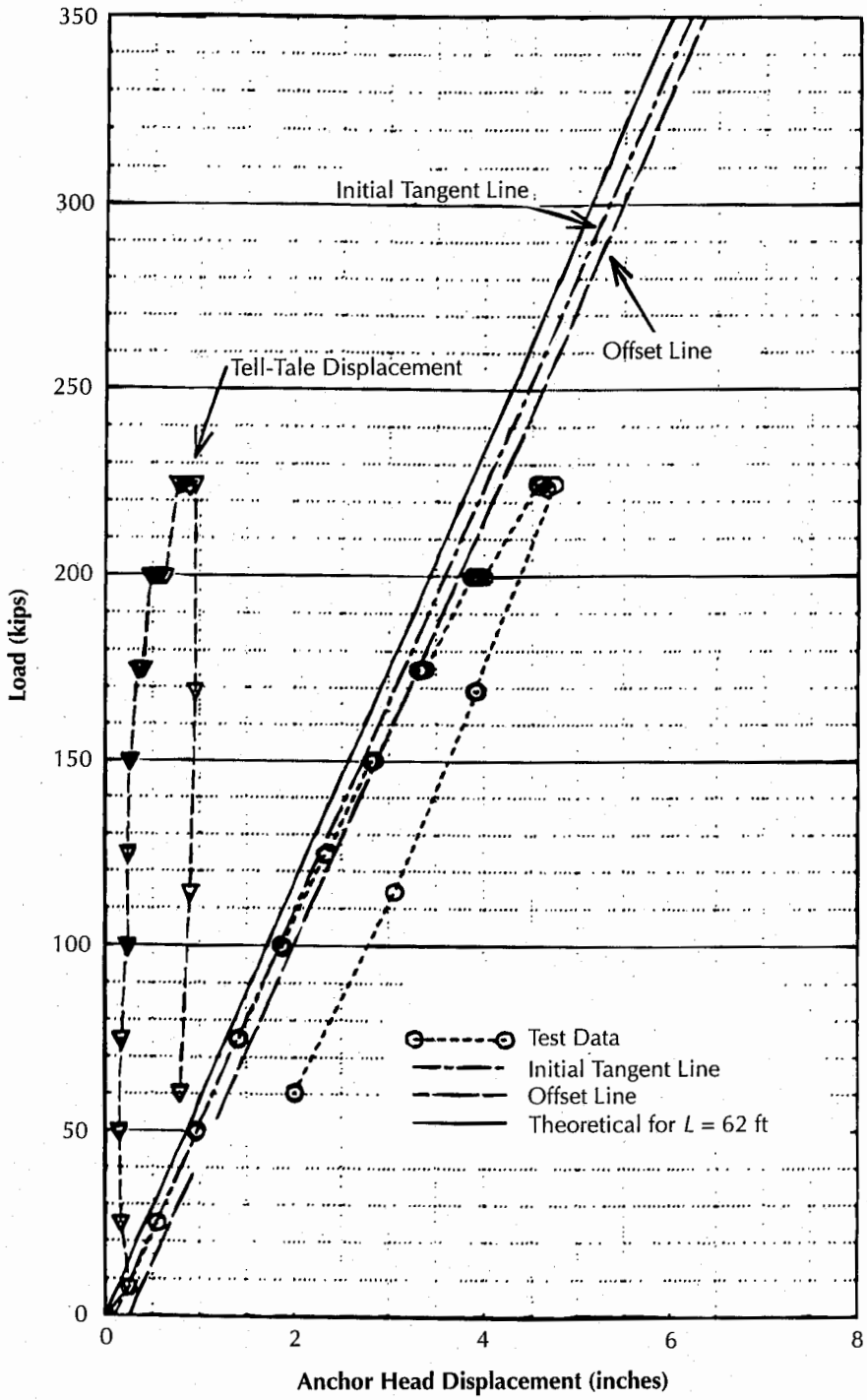
TABLE 3.
Summary of Tiedown Tests at Test Site No. 2

Tiedown Number	AC62-1	AC62-2 (Post-Grouted)	AC62-3	AC62-5	AC62-4	AC62-7	AC62-6
Soil/Rock Type	Glacio-marine	Glacio-marine	Glacio-marine	Slightly to Severely Weathered Argillite	Slightly to Severely Weathered Argillite	Slightly Weathered Argillite	Slightly Weathered Argillite
Test Type	A	B	B††	A	B	A	B††
Length of Bond Zone (feet)	15	15†††	15	6	6	6	6
Initial Free Length (feet)*	63.5	63.5	63.5	87	87	111	109
Maximum Load Attained (kips)	225	300	135	120	210	270	210
Critical Creep Tension (kips)**	120	NA	NA	90	NA	210	NA
Failure Load (kips)***	175	200	NA	120	180	225	NA
Displacement at Failure Load (inches)§	3.20	3.25	NA	3.05	4.25	6.45	NA
RDL (kips)	108	108	108	80	80	168	168
Average Load Transfer Based on Failure Load (kips/ft)§§	11.7	13.3	NA	20.0	30.0	37.5	NA
Average Side Shear Based on Failure Load (ksf)§§§	9.9 (68.6 psi)	11.3 (78.6 psi)	NA	17.0 (118 psi)	25.5 (177 psi)	31.8 (221 psi)	NA
Average Side Shear Based on RDL (ksf)†	6.1 (42.2 psi)	6.1 (42.2 psi)	6.1 (42.2 psi)	11.3 (78.5 psi)	11.3 (78.5 psi)	23.8 (165 psi)	23.8 (165 psi)

Notes: * The initial free length is the distance from the top of the anchor head to the top of the bond zone at the beginning of the test.
** The critical creep tension was determined using the method described by Cheney (Ref. 33).
*** The failure load was computed using the slope-tangent method, which is similar to Davisson's method for piles (Ref. 34).
§ Movement of the anchor head due to seating has been subtracted to obtain the displacement values shown.
§§ Calculated using nominal bond zone length and failure load.
§§§ Calculated using nominal side area and failure load.
† Calculated using nominal side area and the RDL.
†† This tiedown was used for the long-term creep test.
††† Post-grouting was performed on tiedown AC62-2.

tangent method, a tangent to the initial portion of the load versus displacement curve is drawn. A line parallel to this tangent is then drawn at a horizontal distance of 0.15 inch to the right of the initial tangent line. The intersection of this second line and the load versus displacement curve is considered the failure load. (The x-intercept of the initial tangent line provides a reasonable estimate of the movement of the anchor head due to seating.) The displacement values listed in Tables 2 and 3 correspond to the net displacements of the tiedown anchor head at the failure load — *i.e.*, the abscissa of the intersection of the second line and the load versus displacement curve minus the estimated displacement due to seating.

Test Site No. 1. Based on the Type A test on AC61-2, an RDL of 200 kips was determined for tiedowns in the glacial till. However, based on the tiedown response observed during the Type B test on AC61-3 (also anchored in till), the RDL was revised lower twice. First, when imminent failure of AC61-3 was observed at a load of approximately 200 kips, the RDL was reduced



Notes: Bond zone is 15 feet long in glaciomarine. Failure load = 175 kips.

FIGURE 12. Load versus displacement for tiedown AC62-1 (Type A test).

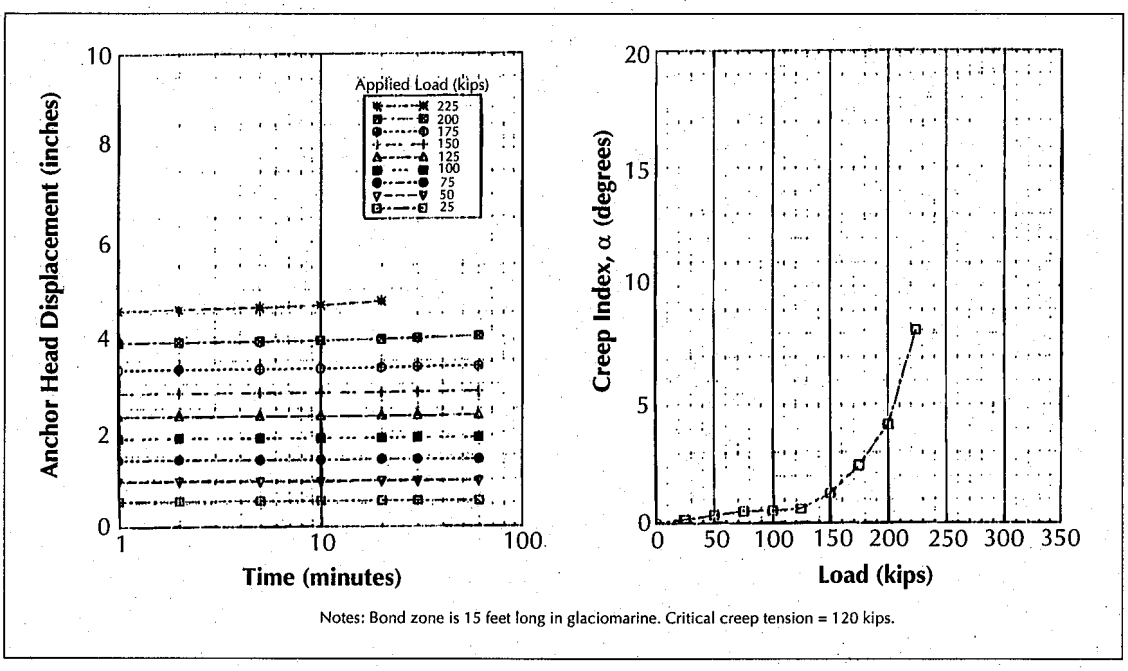


FIGURE 13. Creep and critical tension for tiedown AC62-1.

to 160 kips. Later, when significant creep was observed during the load hold at 160 kips, the RDL was further reduced to 125 kips for the 72-hour test hold portion of the test. The quick-load test on the AC61-3 tiedown indicated a failure load of approximately 150 kips. During the Type B test, the post-grouted tiedown in till (AC61-1) was first tested to a load of 250 kips (125 percent of the RDL determined from AC61-2) and then a load of 200 kips (100 percent of the RDL) was maintained during the 72-hour hold portion of the test. Later during the quick-load test on the post-grouted tiedown in till, a maximum load of 325 kips (about 80 percent of GUTS) was reached and a failure of 250 kips was determined based on the slope-tangent method.

During the Type A test on AC61-5 with its anchor zone in the severely to completely weathered argillite, the tiedown failure load measured was approximately 120 kips, which is 60 to 80 percent of the capacity measured for the tiedowns with anchor zones in the glacial till. Tiedown AC61-4, also with its anchor zone in the severely to completely weathered argillite, behaved satisfactorily during the Type B test, with no signs of significant creep during the 72-hour hold at the RDL of 90 kips (from AC61-5). The tiedown load for long-term monitoring in the severely to completely weathered argillite was initially locked off at 75 kips, due to slippage in the strand gripping system. It was later re-tensioned and locked off at about 90 kips and monitored long-term. The load in AC61-4 appears to be staying relatively constant, with less than 2 percent drop off in load over two years.

Tiedown AC61-6, anchored in the severely weathered argillite rock, was subjected to a Type A test and yielded a failure load and critical creep tension of 160 kips for a 10-foot-long bond zone. The average unit side shear stress at failure in this case (14 ksf or 94 psi) is about 1.2 to 1.6 times the average unit side shear stress at failure determined for the tiedowns in the glacial till. During the 72-hour hold portion of the Type B test on AC61-7, the load was maintained at the RDL of 120 kips, and there was little change in deformations or strains. After locking off the wedges on the anchor head for the long-term test, the load cell readout registered about 140 kips. Tiedown AC61-7 maintained the load during the long-term test with less than 2 percent drop off in load over two years.

Test Site No. 2. During the Type A test on tiedown AC62-1, anchored in the glaciomarine deposit, the maximum load recorded was 225

kips. The failure load for the tiedown, determined using the slope-tangent method, was 175 kips, which corresponds to an average side shear stress at failure of about 10 ksf (70 psi). The RDL selected for the remaining tests in the glaciomarine deposit was 108 kips (based on a critical creep tension of 120 kips). During the Type B tests, the remaining two tiedowns in the glaciomarine (AC62-2 and AC62-3) were loaded to 125 percent of the RDL and then held at 100 percent of the RDL for 72 hours with minimal creep observed. A quick-load test on the post-grouted tiedown (AC62-2) resulted in a failure load of approximately 200 kips, which is about 14 percent greater than the failure load reached in non-post-grouted tiedown AC62-1 in the glaciomarine deposit. The data from AC62-3 for the 72-hour hold at the RDL indicate no significant creep. The anchor load for AC62-3 for long-term monitoring in the glaciomarine was locked off at 92 kips. Tiedown AC62-3 was to be re-tensioned to increase the lock-off load to the RDL of 108 kips, but the strands protruding at the surface were inadvertently cut off during preparations for the remaining tiedown tests and re-tensioning could not be performed. The data indicate a 4 to 5 percent reduction in the anchor load in AC62-3 over two years.

The Type A test on AC62-5, which was anchored in the slightly to severely weathered argillite near the soil-rock interface, resulted in a failure load of 120 kips and a critical creep tension of 90 kips. The average unit side shear stress at failure in the slightly to severely weathered argillite was approximately 17 ksf (118 psi), which is about 50 to 70 percent higher than the average unit side shear stress at failure that was determined for the tiedowns anchored in the glaciomarine deposit. In the subsequent Type B test on AC62-4, a tensile force of 80 kips (RDL) was applied and held for 72 hours with no significant creep recorded. Upon completion of the Type B test, a quick-load retest was performed on AC62-4 in which a failure load of 180 kips (1.5 times the failure load as determined in the Type A test) was reached. The difference in the failure load between AC62-5 and AC62-4 was likely due to the variability of rock, especially the weathering and fracturing.

The capacity of tiedown AC62-7 in the slightly weathered argillite was 20 to 75 percent greater than in the slightly to severely weathered argillite, with a failure load (from the slope-tangent method) of around 225 kips and a critical creep tension of 210 kips. At the failure load, the average side shear stress was about 32 ksf (221 psi) for the 6-foot bond zone of AC62-7. Tiedown AC62-6, also in the slightly weathered argillite, behaved satisfactorily and did not exhibit any sign of significant creep during loading to 125 percent of the RDL for 1 hour and during the 72-hour hold at the RDL (168 kips). In preparation for the long-term test, the tension in the tiedown was increased to about 185 kips, with the expectation of around a 10 percent loss during the locking-off process based on the experience from the previous long-term tests. After locking off the wedges on the anchor head, the read-out registered about 177 kips on the vibrating wire load cell. The data indicate a 9+ percent reduction in the anchor load (which appears to correspond with settlement of the reaction block supporting the stressing head and load cell and not creep or relaxation associated with the anchor zone).

Load Transfer Results

The rate at which the applied load is transferred – or simply "load transfer" – from the tiedown anchor to the surrounding soil or rock (in kips per foot length of anchorage) was evaluated using the strain gage measurements during each test. If the axial strain, ϵ, on the tendon at a particular gage location is known, then the corresponding tensile force, T_s, in the steel strands can be calculated based on the cross-sectional area, A, and the elastic modulus of the strands, E:

$$T_s = AE\epsilon$$

The average load transfer to the ground between two adjacent gage locations was calculated by taking the difference between the tensile forces on the strands on those two locations and dividing the difference by the distance between the two gages. The inherent assumption is that the tiedown grout cannot resist tension, and the portion of the load not carried by the steel strands is transferred to the surrounding soil or rock through shear at the grout-ground interface:

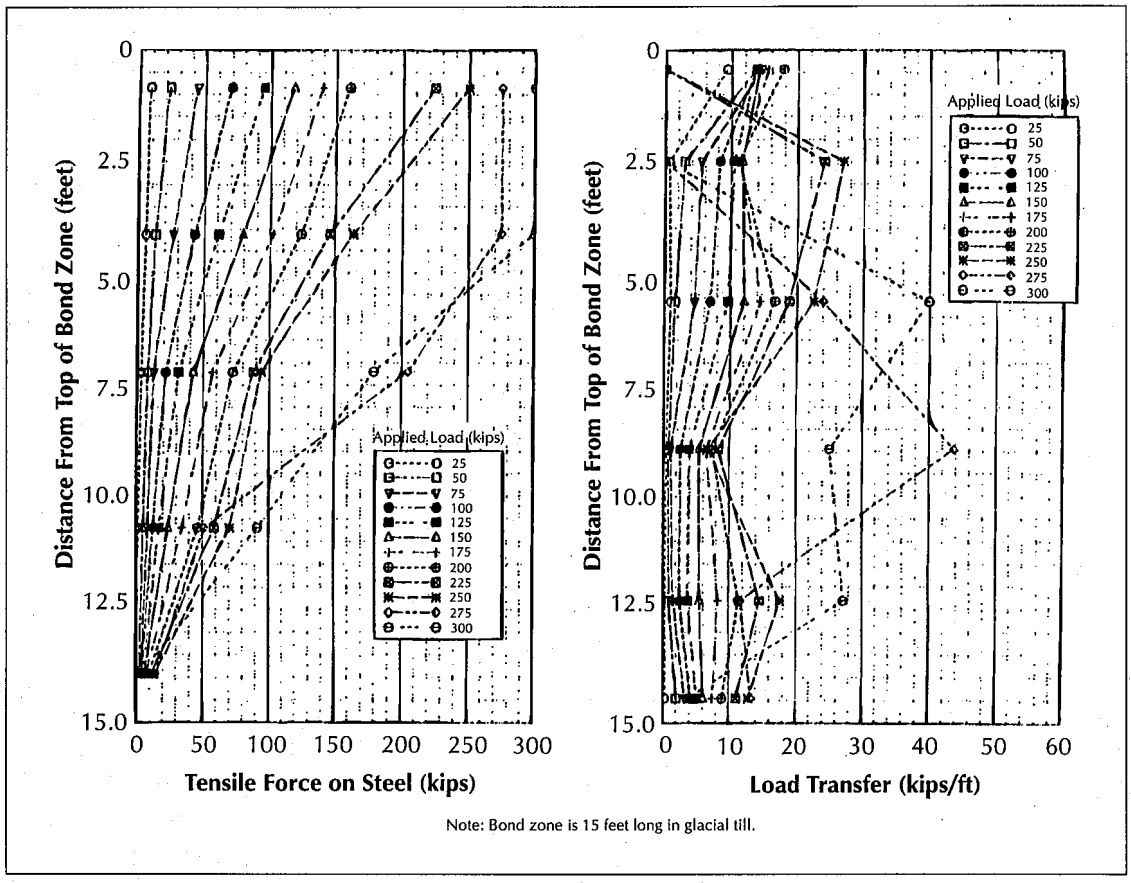


FIGURE 14. Load transfer for tiedown AC61-2 (Type A test in glacial till).

$$\tau_{Ti} = \frac{T_{si} - T_{si+1}}{L_i}$$

Where:

τ_{Ti} = The average load transfer along segment i with length L_i, with i increasing from the top to the bottom of the bond zone

T_{si} = The tensile force in the steel strands at the top of the segment of steel strand

T_{si+1} = The tensile force in the steel strands at the bottom of the segment of steel strand

In these analyses it has been assumed that the tensile force on the steel strands at the bottom end of the bond zone is zero.

Test Site No. 1. The progressive transfer of load during the Type A test on tiedown AC61-2, which is in glacial till, is illustrated in Figure 14. Although the maximum load attained was 300 kips, the strain gage data showed signs of imminent failure at around 275 kips. When the applied load was 275 kips, the strain gage at the middle of the bond zone indicated a tensile force on the steel strands of 205 kips (or 75 percent of the applied load), compared to 25 percent of the applied load at 25 kips and to 49 percent of the applied load at 125 kips. The plot of load transfer versus distance from the top of the bond zone (see the right side of Figure 14) also showed the progressive mobilization of shear along the grout-soil interface with increasing load. The load transfer along the bond zone length, which was initially greater near the top and tapering down below, appeared to resemble a uniform distribution in the upper part of the bond zone as the applied load became greater. At 275 kips the load transfer near the bottom of the bond zone approached 13 kips/foot (or about 77 psi in equivalent side shear stress). When the load was at least 275

kips, the computed load transfer reached as high as 44 kips/foot (259 psi equivalent shear) in the middle of the bond zone, and the erratic strain gage data could be attributed to the onset of failure along the grout-ground interface of the tiedown.

The strain gage data from the Type B test on AC61-1 (the post-grouted tiedown in glacial till) also reflect a decrease in the transfer of load along the length of the bond zone. However, at 250 kips (or 125 percent of the RDL), the tensile force on the steel in the middle of the bond zone was 66 kips (or 26 percent of the applied load), which is considerably less than the 205 kips (or 75 percent of the applied load) in the Type A test on AC61-2 that was not post-grouted. The calculated load transfer values near the bottom of the bond zone were also much less in the post-grouted tiedown and appeared greater at the top of the bond zone. During the quick-load test on AC61-1 (following the Type B test), the tiedown anchorage load-deformation behavior did not indicate a failure condition at a load of 325 kips, but the strain gage data manifested symptoms of imminent grout to ground bond failure. At 300 kips, the load transfer near the second strain gage from the top reached about 45 kips/foot (265 psi side shear) then dropped to 29 kips/foot (171 psi side shear) when the load was increased to 325 kips, with evident redistribution of load within the bond zone downward.

In AC61-3, which was also in the till, the strain gage data during Type B test was similar to the data from AC61-2 and reflected a similar load transfer distribution along the bond zone length. As the load was increased in steps, the tensile force on the steel in the bottom portion of the bond zone was much less than the applied load, but the load transfer distribution was approximately a uniform distribution even at small loads. This behavior suggests that the shear resistance along the soil-grout interface was mobilized over the entire bond zone length at a relatively small load. The load transfer mobilized was on the order of 10 kips/foot, which is equivalent to a side shear stress of about 60 psi.

The load transfer data from the Type A test on AC61-5, which was anchored in the severely to completely weathered argillite, are shown in Figure 15. (The strain gage in the middle of the bond zone ceased to operate shortly after the installation of this tiedown.) At a load of 160 kips, the force on the steel at the bottom of the bond zone was about 16 kips (or 10 percent of the applied load), but the load transfer at the bottom was around 16 kips/foot (equivalent to 94 psi side shear) and the tell-tale dial gage indicated a 2-inch upward movement of the top of the bond zone. The load transfer distribution indicated that the bottom half of the bond zone carried most of the load as the force on the steel at the top half of the bond zone approached about 90 percent of the applied load, implying that the upper portion of the rock was softer and weaker than the rock in the middle and bottom portions of the tiedown bond zone. These results are consistent with the pattern of argillite weathering that was inferred from the rock core samples retrieved from nearby borings in which the argillite shallower than about 90 feet is described as completely weathered and the deeper argillite is described as severely weathered.

The load transfer in the subsequent Type B test on AC61-4 in the severely to completely weathered argillite exhibited a relatively high load transfer (25 kips/foot) near the top of the bond zone, possibly due to a thin, more competent rock layer in the upper part of the bond zone. However, the distribution of load along the bond zone length gave no sign of imminent failure of the tiedown. Data from the long-term monitoring phase for AC61-4 indicate no significant change in the load transfer trend.

The variation of the tensile force in the steel along the bond zone for the tiedowns anchored in the severely weathered argillite (AC61-6 and AC61-7) is evident from the strain gage data. During the Type A test on AC61-6, a maximum load of 220 kips was applied, with 64 percent carried by the steel strands at the middle of the 10-foot bond zone and about 15 percent near the bottom. The load transferred to the adjacent rock through shear averaged about 16 kips/foot (94 psi equivalent side shear) but reached a maximum of nearly 40 kips/foot (236 psi equivalent side shear) about 7.5 feet below the top of the bond zone. In the corresponding Type B test on AC61-7, the calculated load transfer did not exceed 20 kips/foot (118 psi equivalent

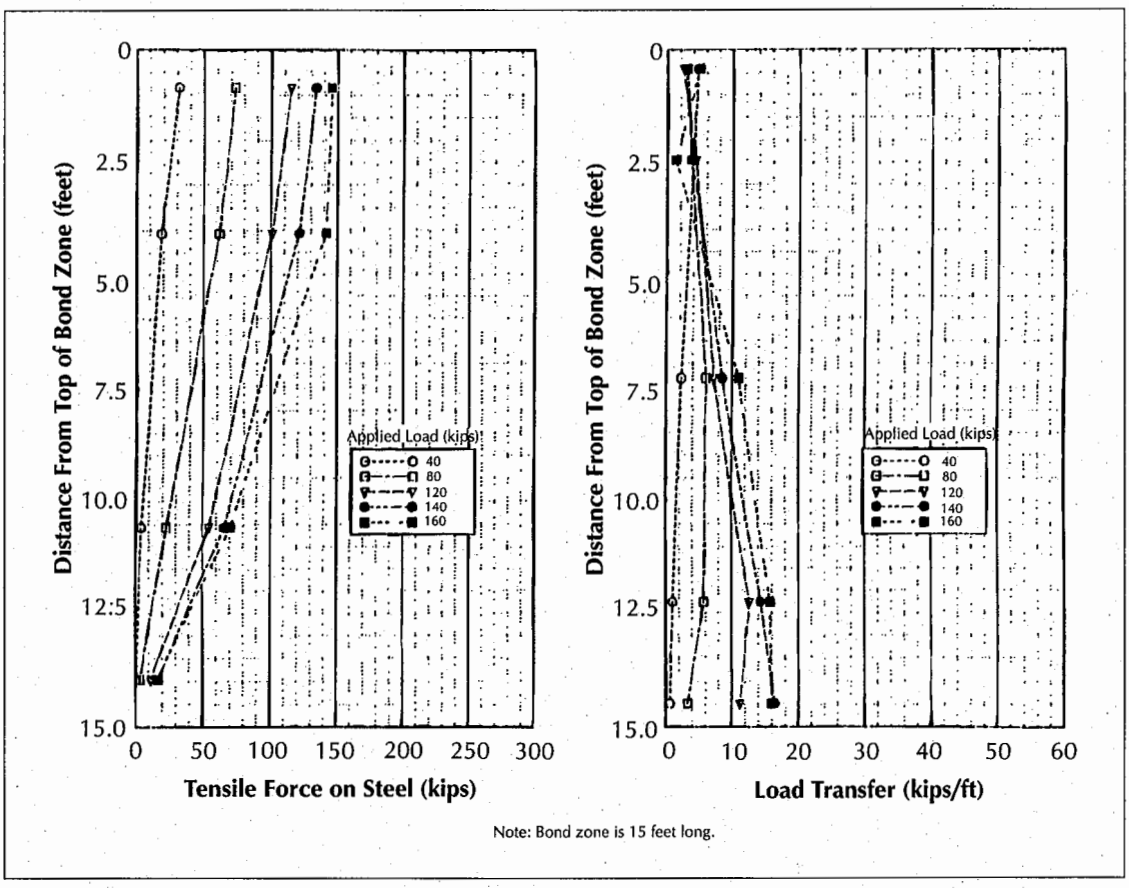


FIGURE 15. Load transfer for tiedown AC61-5 (Type A test in severely weathered to completely weathered argillite).

side shear) at a load of 150 kips (125 percent RDL). Near the bottom of the bond zone, the load transfer was roughly 10 kips/foot (59 psi equivalent side shear) at a load 150 kips. At 150 kips, the force on the steel was about 43 percent of the applied load at the middle of the bond zone and was about 8 percent of the applied load near the bottom of the bond zone. Both the force in the steel and the load transfer tended to decrease with depth along the bond zone. The deformation and strain readings have remained stable through the long-term monitoring portion of the test.

Test Site No. 2. The progressive transfer of load during the Type A test on AC62-1, which was anchored in the glaciomarine deposit, was observed even though two of the five strain gages in the bond zone ceased to operate after tiedown installation. When the applied load was 225 kips, the strain gage 10 feet down from the top of the bond zone indicated a tensile force on the steel strands of 96 kips (or 43 percent of the applied load) compared to 3 percent of the applied load at 25 kips and to 16 percent of the applied load at 125 kips (see Figure 16). The progressive mobilization of shear resistance along the grout-soil interface with increasing load is evident in Figure 16. The load transfer along the bond zone length, which was initially greater near the top and tapered down toward the bottom of the bond zone, appeared to resemble a uniform distribution as the applied load increased. At 175 kips, the load transfer along the bond zone approached a nearly uniform 12 kips/foot, or about 71 psi equivalent side shear stress. When the force applied to the tiedown was increased to 225 kips, the computed load transfer was approximately 23 kips/foot (136 psi equivalent side shear) about 12 feet below the top of the bond zone. The load

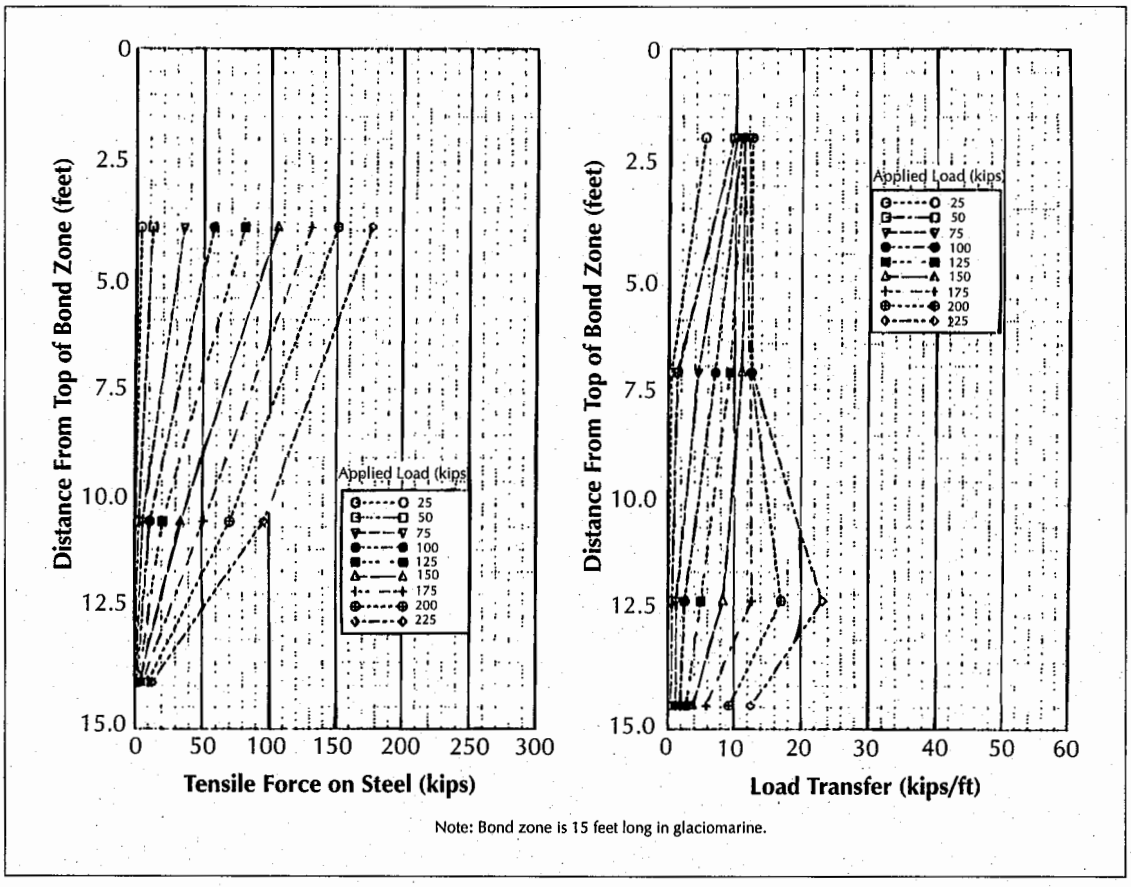


FIGURE 16. Transfer for tiedown AC62-1 (Type A test in glaciomarine soils).

transfer in the top half of the bond zone did not exceed 12 kips/foot, and when the anchor load transfer near the bottom of the bond zone approached 12 kips/foot at an applied load of 225 kips, the anchor deformation exceeded the slope-tangent method failure criterion.

The strain gage data from the Type B test on AC62-2 (the post-grouted tiedown anchored in the glaciomarine deposit) also indicated transfer of load down the length of the bond zone similar to that observed in AC62-1. At 135 kips (125 percent of the RDL), the tensile force on the steel about 4 feet below the top of the bond zone was 36 kips (or 27 percent of the applied load). This force is considerably less than the load in the steel at 125 kips applied load during the Type A test on AC62-1, which was not post-grouted. The load transfer was greater at the top of the bond zone in AC62-2, with values amounting to 25 kips/foot (147 psi equivalent side shear), surpassing the apparent threshold of 12 kips/foot in the Type A test on AC62-1. During the quick-load retest after the Type B test, a maximum load of 300 kips was applied. At 300 kips, the force on the steel near the bottom of the bond zone reached 97 kips (or 32 percent of the applied load). With increasing tiedown load near failure, a redistribution of the load transfer within the bond zone occurs wherein the shear stress at the top appears to approach a limiting value then drops off with further loading. As a consequence, the lower portion of the bond zone must produce resistance to the applied load until the tiedown fails. The load transfer near the top of the bond zone dropped off when the applied load was about 160 kips, and the load transfer for most of the bottom portion was an almost uniform 40 kips/foot (157 psi equivalent side shear). At the maximum load of 300 kips, the maximum calculated load transfer was 83 kips/foot (1,500 psi) over a short tributary length of about 0.6 foot.

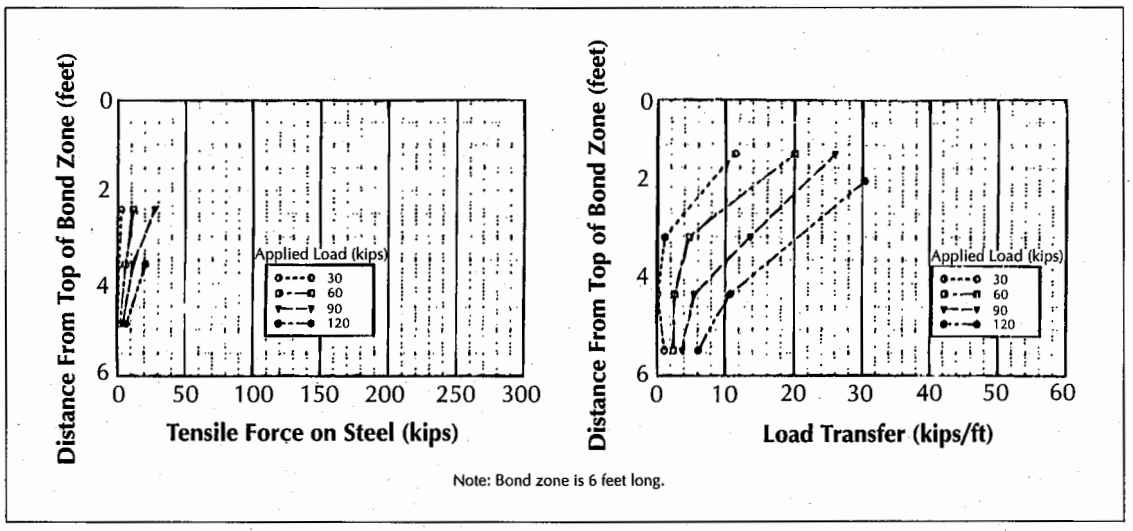


FIGURE 17. Load transfer for tiedown AC61-5 (Type A test in slightly weathered to severely weathered argillite).

The transfer of load along the bond zone length during the Type A test on AC62-5, anchored in the slightly to severely weathered argillite is shown in Figure 17. At a load of 120 kips, the force on the steel at the bottom of the bond zone was only about 7 kips, or 6 percent of the applied load and a maximum load transfer of about 30 kips/foot (177 psi equivalent side shear) occurred in the upper 2 feet of the bond zone.

The load transfer during the subsequent Type B test on AC62-4, also located in the slightly to severely weathered argillite, show most of the load was resisted over the upper half of the bond zone as the tiedown was loaded to 100 kips (125 percent of the RDL) for 1 hour and during the 72-hour load hold at the RDL of 80 kips. At the 100-kip load, the steel strands near the top of the bond zone carried 68 percent of the applied load, and the force on the steel near the bottom was only 2.6 percent of the applied load. The load transfer near the top of the bond zone was about 46 kips/ foot (271 psi equivalent side shear), while the load transfer near the bottom of the bond zone was only 2 kips/foot (12 psi equivalent side shear) for the 100-kip applied load.

During the quick-load retest on tiedown AC62-4, the load was redistributed over more of the bond zone as the anchor was loaded to the maximum load of 210 kips. At the maximum load of 210 kips, 166 kips (or 79 percent of the applied load) were carried by the steel strands near the top of the bond zone, and 42 kips (20 percent of the load) were transferred to the steel near the bottom. A maximum load transfer of 57kips/foot (336 psi equivalent side shear) was calculated near the middle of the bond zone.

At the maximum load of 270 kips during the Type A test on AC62-7, which was anchored in the slightly weathered argillite, the steel strands in the upper half of the 6-foot bond zone carried the entire load with little or no load transfer. During the first few load increments, the load transfer appeared to increase more rapidly in the upper portion of the bond zone. The load transfer reached a maximum of about 111 kips/foot (654 psi equivalent side shear) at 1.5 feet below the top of the bond zone when the applied load was 180 kips. As the tiedown loading continued, the load transfer appeared to shift downward until there was essentially zero load transfer in the upper 1.5 feet of the bond zone and up to 66 kips/foot (389 psi equivalent side shear) at 4 feet below the top of the bond zone at a load of 270 kips.

In the corresponding Type B test on AC62-6, the calculated load transfer reached a maximum of 46 kips/foot (271 psi eqivalent side shear) at 3.5 feet below the top of the bond zone and 34 kips/foot (200 psi equivalent side shear)

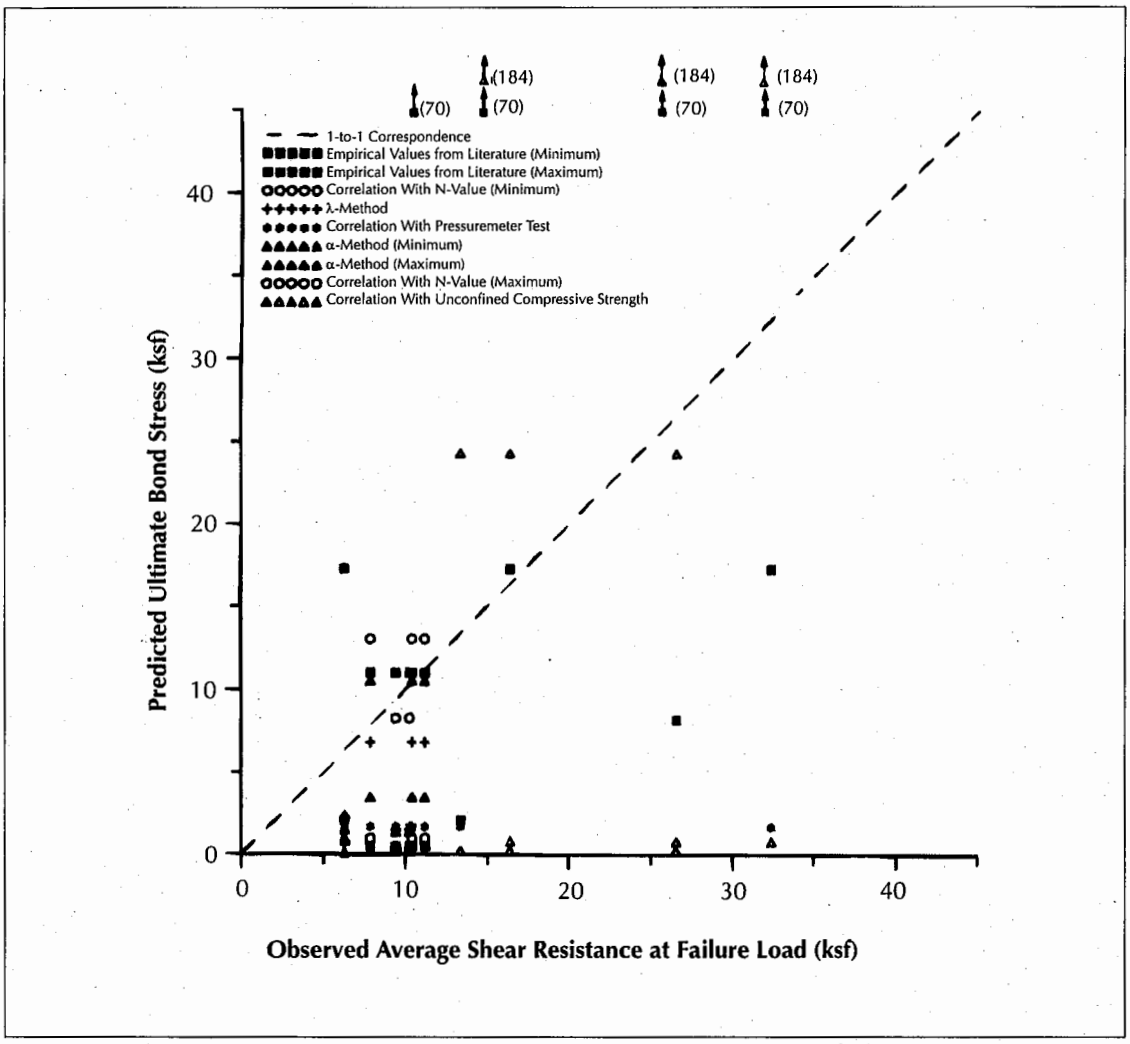


FIGURE 18. Comparison of observed and predicted side shear resistance.

at the bottom of the bond zone during the application of 210 kips (125 percent of the RDL) for 60 minutes. At an applied load of 210 kips, the force in the steel was about 87 percent (183 kips) of the applied load near the top of the bond zone and was about 16 percent (39 kips) of the applied load of the bond zone near the bottom. At an applied load of 168 kips, the tension in the steel strands was approximately 143 kips (or 85 percent of the applied load) near the top of the bond zone and about 14 kips (or 8.6 percent of the applied load) near the bottom. At the 168-kip load, the calculated load transfer was greatest 3.5 feet below the top of the bond zone with a value of 39 kips/foot (230 psi equivalent side shear), while the load transfer near the bottom of the bond zone amounted to only 12 kips/foot. During the 72-hour monitoring at 168 kips (the RDL) and the long-term hold at 177 kips, the load transfer appeared to remain relatively stable.

Comparison of Measured & Predicted Tiedown Capacity

The capacities of the anchors measured in the field tests were compared to the capacities predicted using the methods previously described. The predicted versus measured capacities are summarized in Figure 18. In general, the design methods used in the comparison tend to underpredict the capacity of the soil anchors. Some methods contained significant scatter, particu-

larly those based on empirical values for a particular ground type. The methods that appeared most successful in predicting the capacities of the anchors in soil were the high end of the λ- and α-methods. The good correlation with the N-value method is considered to be coincidental because the materials in the anchor zone were stiff clay and silt whereas the method is based on analysis of piles in sand. The underprediction of the soil anchors may be related to the depths (which were greater than 50 feet) to the tops of the anchor zones and the relatively short lengths of the anchor zones. The results for the rock anchorages indicate that the predictions tended to vary over a wide range and the measured values tended toward the lower end of the predicted range.

Summary & Conclusions

Fourteen instrumented tiedowns — seven at each of two sites — were installed and tested as part of the geotechnical investigation phase of the work in the Central Area of the CA/T Project in Boston, Massachusetts. Three of the tiedowns were anchored in glaciomarine soils and another three were anchored in glacial till. One of each trio of anchors in the two soil conditions was post-grouted. Four pairs of tiedowns were installed and tested in rock conditions ranging from completely weathered argillite to slightly weathered argillite. Anchor zone lengths ranged from 6 to 15 feet so that side shear failure could be achieved during the tests. Four of the tiedowns were locked off at the RDL and long-term (two years) relaxation behaviors were monitored for anchorages in the glaciomarine soil and completely weathered, severely weathered and slightly weathered argillite.

For the glaciomarine tiedowns, the average side shear stress at failure (based on the slope-tangent method) was about 10 ksf and an average side shear stress of 6 ksf corresponded to the RDL. The side shear stress at failure for the post-grouted tiedown in the glaciomarine soil was about 11 ksf. The average side shear stress at failure for the glacial till tiedowns ranged from about 8.5 to 11 ksf and the side shear stresses at the RDL ranged from about 7 to 11 ksf. The post-grouted tiedown in glacial till achieved an average side shear stress of about 14 ksf at the maximum test load that was limited by the capacity of the tiedown tendons to 325 kips. The average side shear stress at failure for tiedowns anchored in the rock ranged from about 7 to 32 ksf depending on the degree of weathering of the argillite. The average side shear stress corresponding the RDL ranged from about 5 to 24 ksf for rock tiedowns.

Load transfer observed in the test anchors generally conformed to expected patterns. At low loads, the load transfer at the top of the anchor zone was typically the greatest and decreased with depth along the anchor zone. At higher loads, load transfer increased lower down the bond zone, and at failure the generally short lengths of the anchors resulted in relatively uniform load transfer along the bond zone. In the rock anchors, the presence of stiffer layers (in the form of less weathered and perhaps less fractured rock) was evident in local, higher load transfer either near the top or the bottom of the bond zone while other parts of the bond zone had apparently reached a failure condition. In the post-grouted anchors, more of the load appears to have been transferred to the ground near the top of the bond zone. Post-grouting increased the capacity of the anchors by increasing the shear capacity of the soil so that higher load transfer can develop before slip occurs.

The long-term behaviors of the anchors at the loads tested appeared to be satisfactory. After two years, relaxation was less than 2 percent of the applied load for the rock tiedowns and about 5 percent of the applied load for glaciomarine tiedown and little change in load transfer was apparent. Loads for the long-term tests were determined based on the RDL method suggested by Cheney.[33]

In general, the tiedown capacity prediction methods examined tended to underpredict the capacities for the tiedowns with anchorages in the soil. The methods that appeared most successful in predicting the load capacity of the soil anchors were the upper ends of the λ- and α-methods. The predictions of the rock tiedown capacities tended to vary widely and the measured capacities were near the lower end of the predicted range.

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