CHAPTER 3

FIELD LOAD TEST FACILITY

3.1 INTRODUCTION

During the period from July 1997 through June 1998, a field test facility was developed at Virginia Tech for performing lateral load tests on deep foundations. The test site is located at Virginia Tech's Kentland Farms, approximately 10 miles west of Blacksburg, Virginia. The facility is designed to investigate the factors that control the lateral load resistance of pile caps and integral bridge abutments. This chapter describes details of the in-ground facilities, the equipment that was used to apply horizontal loads to the foundations, the instrumentation that was used to measure deflections and loads, and the data acquisition system.

3.2 IN-GROUND FACILITY

The test foundations consist of three groups of four piles each, one with a cap 18 inches deep and two with 36 inch deep caps, two individual test piles, and an embedded bulkhead with no piles, as shown in Figure 3.1. The features of the facility are described in the following paragraphs. Figure 3.2 contains photographs taken during construction of the facility.

3.2.1 Piles

The piles, all HP10x42, were installed on August 28, 1997, by Coalfield Services, Inc. of Wytheville, Virginia. The piles were driven using an International Construction Equipment (ICE) model 30S diesel pile hammer, with a rated energy of 22,500 foot-pounds at a stroke of 7.5 feet. Table B.1, in Appendix B, contains detailed information about the pile hammer and pile driving system components.

The piles in all of the groups were installed at a center-to-center spacing of 4D (40 inches). The piles beneath the southeast cap were driven 10 feet and the others were driven to a maximum depth of 19.3 feet, or to refusal, whichever occurred first. Refusal was defined as 15 blows per inch. The two northern piles in the northeast cap met with refusal at depths between 17 and 18 feet, and the two individual piles reached refusal at 19 feet. Detailed pile driving data is summarized in Table B.2.

3.2.2 Concrete for Pile Caps and Bulkhead

The three pile caps and the bulkhead were constructed of reinforced concrete, with their tops at or below ground. The caps are 5 ft by 5 ft in plan dimensions and are located at three corners of a quadrant, such that each cap can be loaded against its neighbor, as shown in Figure 3.1. The fourth corner of the quadrant contained the bulkhead, which had no piles, and consisted of a monolithic block of reinforced concrete approximately 6.3 feet in length, 3 feet in width, and 3.5 feet in depth.

The caps and the bulkhead were positioned so that the loading axis passed through their centroids. Excavations were trimmed to neat lines and grades using a small backhoe and hand shovels. Concrete was poured against undisturbed ground wherever possible, so that the first series of load tests could measure the resistance of the pile caps in contact with the natural ground at the site.

It was necessary to use wood forms at some locations to provide access for the threaded anchor rods used for attaching loading equipment. The anchor rods were cast in the concrete and extend horizontally approximately 4 inches outside the caps. Each of the pile caps and the bulkhead had 4 anchor rods extending outwards in each direction of loading, as shown in Figure 3.3. The anchor rods provided a means for attaching the loading equipment directly to the caps and the bulkhead.

Three separate concrete pours were involved in constructing the pile caps and the bulkhead:

- the NE and NW pile caps were poured on October 24, 1997,
- the SE pile cap was poured on November 19, 1997, and
- the bulkhead was poured on February 17, 1998.

Concrete was delivered to the site in ready-mix trucks and was placed in one continuos pour at each pile cap or integral abutment; no construction joints were necessary. Concrete slump tests were performed on each truckload in conformance with ASTM C-143. The concrete slump ranged from 4 in to 4.5 in. High early strength concrete with a design strength of 4,000 psi was used, and was vibrated in place using hand-operated concrete vibrators. To minimize curing cracks, the concrete surfaces were kept moist using wet burlap covered with plastic for 28 days. No loads were applied to the concrete structures until after the 28 day concrete curing period was complete. The strength of the concrete was checked by performing compression tests, in general accordance with the ASTM C-39 test procedure. The 28 day compressive strengths ranged from 4,200 psi to 5,200 psi, with an average value of 4,770 psi.

3.2.3 Reinforcing Steel for Pile Caps and Bulkhead

The pile caps and bulkhead were reinforced as follows:

- The 36 in deep caps were reinforced in the transverse and longitudinal directions with No. 8 bars at 6 in spacing in the bottom face and No.6 bars at 5 in spacing in the top face.
- The 18 in deep cap was reinforced in the transverse and longitudinal directions with No. 6 bars at 7 in spacing in the bottom face and No.4 bars at 14 in spacing in the top face.

- The bulkhead was reinforced with No. 4 bars spaced at 5 inches on both faces.

Pile cap design standards vary widely in regards to reinforcement and pile embedment requirements. AASHTO specifications for Highway Bridges (AASHTO 1994) recommends at least 12 in of pile embedment; however, for special cases, it may be reduced to 6 in. According to Kim (1984), standards for minimum pile embedment vary between state agencies over a range of 6 to 24 inches. Some states require reinforcement only near the top of the cap, while others call for reinforcement at both the bottom and top of the cap. The Virginia Department of Transportation typically requires that steel piles be embedded a minimum of 12 in, and that steel reinforcement be placed at the top of the piles (personal communication with Ashton Lawler, 1998).

Based on the results of full-scale lateral load tests, Kim (1984) determined the optimum location for steel reinforcement was near the base of the cap. Kim's results indicate that lateral deflections and rotations of pile caps with reinforcement in the top are approximately twice as large as for pile caps with reinforcement located at the base. These differences are attributed to the fixity of the pile to the pile cap. Paduana (1971) performed lateral load tests on full-scale embankment piles with fixed-head and free-head conditions, and found that fixed-head piles resist approximately twice as high lateral load as free-head piles at the same lateral deflection. Thus, to minimize lateral deflections, a fixed condition (no rotation or zero slope) at the pile to pile cap connection is desirable, although 100% restraint between the pile and cap cannot be achieved in the field.

To achieve as much pile restraint as possible, and because of ambiguities regarding the optimum placement of the reinforcing steel, the pile caps in this study were heavily reinforced in both the top and bottom faces. In accordance with ACI 7.7, a minimum of 3 inches of cover was used on reinforcing bars, and at least 5 inches of cover was used on the piles.

The piles extended to within 0.3 ft of the top surface of the caps. Reinforcing bars were welded between the flanges of the piles and the bottom grid of reinforcement to securely connect the rebar cage to the piles. As noted by Kim (1984), the research and development division of the Cement and Concrete Association of England recommends welding reinforcing bars to embedded piles to achieve optimum anchorage. Because of the unknown effects that welding may have on the properties of standard carbon grade reinforcing steel, the welded bars were not considered to be part of the bending or shear reinforcement. Attaching the rebar cage to the piles minimized movement of the reinforcing bars and the threaded anchor bars during concrete placement.

Provisions in the ACI code for steel reinforcement were exceeded for bending (ACI 10.5), shear (ACI 11.5), shrinkage and temperature cracking (ACI 7.12.2). As recommended by Clarke (1973), for added shear reinforcement, ACI standard stirrup hooks were used to form an enclosed cubical rebar cage.

High strength (60 ksi yield strength) one-inch-diameter steel anchor rods were embedded in the caps and abutment at the locations shown in Figure 3.3 The anchor rods were threaded at both ends and were spaced to match the holes in the clevis base plates. A steel frame made of 2 in x 2 in x 1/8-thick welded angles was fabricated and inserted inside of the bulkhead rebar cage. The steel frame was used to support the anchor rods and ½-inch-thick anchor plate. Additional No. 4 and No. 5 bars were used as needed to support the threaded 1-inch-diameter anchor rods and to prevent them from moving during concrete placement.

Except for the east-west aligned rods in the NE cap, the embedded ends of the anchor rods were secured inside of the cap by bolting a ½-inch-thick steel plate to the rods. This helps distribute the applied load evenly between the rods and prevents the rods from pulling out or pushing through the cap when loaded. The east-west aligned rods in the NE cap pass completely through the cap and are threaded at both ends. At the end of the 28-day concrete curing period, the rods were secured within the caps by bolting ½-

inch-thick steel plates onto both ends. The anchor bars used in the bulkhead were threaded throughout their length.

3.2.4 Roads, Drainage and Weatherproofing

The entrance road leading to the site was covered with a layer of geotextile reinforcing fabric and gravel to facilitate wet-weather access. Additional site grading was performed for diversion of surface runoff, and 160 feet of 4-inch-diameter slotted ADS drain pipe was installed to help keep the site dry.

A large tent shelter, manufactured by Cover-It, Inc. of West Haven Connecticut, was erected to protect the test area during inclement weather, and to provide a degree of environmental control during load testing. The tent was 18 ft by 28 ft in plan dimensions and had approximately 8 ft of vertical clearance. The frame was made from 15 gauge galvanized steel tubing and the cover consisted of a reinforced waterproof nylon fabric. The tent was large enough to shelter a test setup, including reference beams and monitoring equipment, as shown in Figure 3.4. After testing at one location, the tent ground anchors were moved and positioned at the next test location.

3.3 LOADING EQUIPMENT

Lateral loads were applied to the pile caps and the bulkhead using an Enerpac 200-ton double-acting hydraulic ram (model RR-20013) which was purchased for the project. The struts and connections in the load path were designed to apply maximum compressive loads of 150 kips and maximum tensile loads of 100 kips. The configuration of the loading apparatus is shown in Figure 3.5. Loads were applied through the pipe struts and clevis connections by pressurizing the ram cylinder using an Enerpac hydraulic pump (model PEM 3405BR). As can be seen in Figure 3.5, the ram and struts were located below the ground surface in a 2-foot-wide trench that was excavated between the caps.

The pipe struts were fabricated by welding one-inch-thick steel plates to the ends of 6-inch-diameter schedule 80 steel pipe (6.625 in outside diameter and 0.432 in wall thickness). The clevis pin connector consisted of two pieces fabricated from 1-inch-thick ASTM A-36 steel plate stock. The eye bracket or tongue piece was attached to the anchor rods that extend from the sides of the cap and bulkhead. The other piece of the connection, called a female clevis bracket, was attached to the end of the pipe strut. The two clevis pieces were joined together using a 1.5-inch-diameter solid steel pin, which allowed the pipe strut to rotate upwards or downwards around an axis aligned perpendicular to the direction of loading.

The ram plunger attached to a load cell connected to the shorter (15.5-inch-long) pipe strut. The base of the ram was bolted to the ram base adapter, which was bolted to the longer (27.8-inch-long) pipe strut. When the individual piles were tested, the long pipe strut was removed from the lineup, and the ram base adapter was connected directly to the female clevis bracket. The clevis connectors, pipe struts, and ram base adapter were fabricated in the Structural and Materials Laboratory at Virginia Tech.

The 9-inch-diameter hydraulic ram was held in place by a steel cradle fabricated from ½-inch steel plates. Only the bottom half of the steel cradle was in position at the time of the photograph shown in Figure 3.5. The cradle was anchored to a 12 cubic foot block of reinforced concrete embedded in the bottom of the loading trench. The ram could move freely in either direction parallel to the axis of the trench, but was restrained from buckling by the steel cradle and the weight of the concrete block.

The hydraulic pump was powered by a 5000 watt Northstar generator purchased for the project. The ram force was controlled by a remote pendant switch, which operated a 24-volt solenoid valve that advanced or retracted the ram.

After testing was completed at a setup, the loading system was disassembled and moved to the next test location. The system was used to test the individual piles by removing the large pipe strut and connecting the ram base adapter directly to the clevis connection.

3.4 INSTRUMENTATION

The magnitude of the applied force was monitored using load cells and a data acquisition system that were built for the project. Two different load cells, identified as the 200 kip and the 150 kip load cells, were used during the tests.

The 200 kip load cell was capable of measuring either compressive or tensile forces. It was fabricated from a 16-inch-long, 2.5-inch-diameter, high strength bar of cold-rolled steel with a full bridge of strain gauges attached to the outside, and waterproofed, as shown in Figure B.1. Prior to attaching the strain gauges, one end of the bar was welded to a 1.5-inch-thick steel plate, and the other end was cut with threads to match those of the ram plunger. Eight ¼-inch strain gauges were installed on the exterior of the round bar using M-Bond 200 polyurethane strain gauge cement. The strain gauges used were type CEA-06-125UN-120, gage factor 2.065, made by Micro-Measurements Division of Measurements Group, Inc. Each gauge level consisted of a pair of gauges on either side of the round bar. The 4 pairs allowed completion of a full Wheatstone bridge circuit, as shown in Figure B.2, which canceled out the bending effects.

Prior to testing the SE cap and bulkhead (location B in Figure 3.1), the 200 kip load cell was damaged as a result of flooding in load trench B. Because of difficulties in repairing the load cell in the narrow loading trench, a second load cell was calibrated and substituted for the 200 kip load cell. This load cell had a capacity of 150 kips, and was used for the remainder of the tests.

The 150 kip load cell was constructed of high strength cold-rolled steel and had an hourglass shape, as can be seen in the photographs in Figure B.3. The ends of the load cell were 5 inches in diameter and about one inch thick. The ends had a slightly convex surface for centering the load. The central portion of the load cell was 6 inches long and 3inches in diameter, and contained the strain gauges, which were mounted and wired using the same techniques that were used to construct the 200 kip load cell. The load cell was temporarily held in place during testing by sandwiching it between two steel plates held together with four ¾-inch-diameter threaded rods, as shown in Figure B.3(a). The 150 kip load cell could be removed between tests and stored inside to protect it from environmental damage.

The load cells were calibrated in the lab under controlled conditions using a Satek universal testing machine. The data acquisition equipment used during calibration, including excitation voltage, power and signal lead lengths, shielding, data acquisition hardware, data channel assignment, and cable connectors, was identical to those used during the field tests. Prior to calibration, the load cells were subjected to approximately 50 load/unload cycles. As indicated by the calibration curves shown in Figure B.4, the load cells exhibited linear responses with no measurable hysteresis. The resolution of the load cells were determined during calibration to be approximately 1 kip.

Displacements and rotations of the caps were measured using 12 linear deflection transducers (6 per cap). These provided sufficient data to evaluate displacements of the caps along three mutually perpendicular directions (parallel to the direction of loading, perpendicular to the direction of loading, and vertical). The instruments were mounted on temperature-stable wood references beams, which were mounted on posts located outside the zone of influence of the foundations. The instruments were positioned at the locations shown in Figure 3.6, which made it possible to determine rotations as well as deflections of the pile caps.

Two types of electronic displacement transducers were used during the tests: 1) Longfellow linear transducers manufactured by Waters Manufacturing of Wayland, Massachusetts and 2) Celesco cable-extension position transducers, manufactured by Celesco Transducer Products, Inc. of Canoga Park, California. Photographs of the transducers are shown in Figure 3.7. Standard specifications are provided in Table B.3.

Six Celesco cable-extension position transducers were used to measure deflections and slopes of the individual piles. The Celesco transducers were clamped onto the pile flanges and onto the vertical telltale, at the locations shown in Figure 3.8. The instrument cables were attached to wood reference beams, which were mounted on posts driven into the ground, outside of the zone of influence caused by loading.

Data from the instruments were electronically recorded with the Keithley 500 data acquisition system using the Lab Tech Notebook Pro software package running on a personal computer, as described in the following sections. Dial gauges were used for mechanical verification of the electronic data collection system. The dial gauges used were model A1-921 manufactured by Teclock Corportation.

The transducers were calibrated in the laboratory under controlled conditions using a Mitutoyo height gauge (model number 192-116). The data acquisition equipment used for calibration, including: excitation voltage, excitation and signal lead lengths, shielding, data acquisition hardware, data channel assignment, and cable connectors were identical to those used during the field tests. Scale factors or calibration curves were developed for the instruments, and in all cases, the voltage versus displacement curves exhibited linear response with no measurable hysteresis. An example calibration curve for one of the displacement transducer is shown in Figure B.5. The calibration factors for all of the displacement transducers are provided in Table B.4. The zero intercept of the calibration curves was not needed because zero readings were obtained at the beginning of each test. The resolution of the instruments were determined during calibration to be approximately 0.002 in.

3.5 DATA ACQUISITION HARDWARE

The electronic deflection measuring devices and the load cell strain gauges produce a voltage signal that was monitored through an analog-to-digital data acquisition system. The hardware for the analog-to-digital system included a Keithley model 500A data acquisition system and a personal computer.

The Keithley 500A is a general purpose data acquisition and control unit that served as an interface between the computer and electronic measurement devices. All incoming commands from the computer were decoded and directed through the Keithley internal motherboard circuitry. The motherboard controlled the 0.8 amp, +15 volt, excitation power supply and served as a link between the computer and the input modules. The Keithley motherboard contained slots for up to 10 input modules. A variety of input modules are available, depending on the system requirements. This study incorporated one AMM2 module and three AIM3A modules, as described below.

The AMM2 module was connected to the first slot of the Keithley motherboard. The AMM2 is a global conditioning master measurement module that consists of a high-speed software-controlled gain amplifier that controls the signal conditioning and switching circuitry for all the other modules. During load testing, the local amplifier gain was set at x1 for the electronic transducers and x10 for the load cell strain gauges. A 2 kHz low-pass filter was used to minimize analog/digital noise. After analog conditioning, signals were routed to the 16-bit analog/digital (A/D) converter section of the module for analog-to-digital conversion using a 16-bit successive approximation converter, with controllable sampling rates of up to 50 kHz. The AMM2 converter range was set at 10 V for the transducers and the load cell strain gauges.

The load cell strain gauges and the deflection transducers were connected to individual terminal channels of AIM3A modules, which were installed in slots 2 through 4 of the Keithley system. The AIM3A is an analog input module that accepts input signals of 100 mV full scale through 10 V full scale, and outputs a signal of 10 V full scale to the AMM2 A/D converter module. The AIM3A module produces full 16-bit precision results at a rated linearity of 0.005 percent. Input signals from the measuring devices were connected to individual channels of the module using screw terminal strips, which were accessed through the back of the Keithley control box. A maximum of 32 single-ended or 16 differential (floating) inputs can be connected to a single AIM3A module. Shielded cables were used for all the instruments, which were connected as differential inputs to the AIM3A modules. This type of wiring scheme is often used when there are multiple inputs with different ground points. The voltage input is limited to 10 V per channel when differential inputs are used, but the possibility of ground loop errors and common mode noise is reduced.

Cables from the electronic instruments were routed to plastic terminal strips mounted on a plywood board as shown in Figure 3.9(a). The board functioned as an electronic junction between the instruments and the Keithley control system. The instrument cables contained leads for positive and negative signals, positive voltage, ground, and shield lines. Electromotive force was routed from the Keithley system to a positive voltage regulator on the terminal board. The regulator reduced the voltage from +15 V to +10 V. The voltage was jumped to each terminal on the board using 20 gauge lead wires.

A Gateway 2000 4DX-33V personal computer with 640 kilobytes of system memory and a floppy disk drive was used for data acquisition. The computer controlled the operation of the system through an interface card that contained data and address bus buffers, address decoding circuits, and programmable interval timers. The computer and Keithley 500A were powered by a 120 volt AC dedicated power source produced by a 1600-watt Honda generator. The generator was used solely for powering the data acquisition hardware during load testing. A separate generator was used for powering the hydraulic pump.

The complete data collection system including the Keithley 500, personal computer, and terminal board were attached to a steel frame mounted inside a minivan, as shown in Figure 3.9(b). This protected the sensitive electronic instruments from the elements and vandalism, and provided a means for transporting the system to and around the site.

3.6 DATA ACQUISITION SOFTWARE

The computer program Labtech Notebook Pro (LNP), by Laboratory Technologies Corporation, was used to control the data acquisition hardware. LNP was written using BASIC Version 3.0 programming language and was configured to interface directly with the Keithley data acquisition system. The data acquisition and process control functions were established in LNP using block functions. The block functions established characteristics such as type of input signal, block name, sampling rate, channel number, interface device, scale factor, and offset constant.

Individual block functions were established for each type of electronic instrument. Various sampling rates, ranging from 0.2 Hz to 1 Hz were used during the initial load tests. For static testing, a sampling rate of 0.2 Hz was found optimal. A summary of the instrument channel numbers and interface devices is provided in Table B.4. The block scale factor corresponds to the calibration slope shown in the table.

The block offset constant is a value that is added to the input value, and was set equal to zero for all of the transducers. Separate blocks were established for elapsed time and for the load cell.

During data acquisition, LNP places raw data from the analog-to-digital converter in a buffer, and subsequently copies it to a file on the computer's hard disk. When LNP writes the data from a block's buffer to a file, it is converted from voltage units to engineering units in ASCII real number format using the block offset constant and scale factor. The buffer was transferred to a file on the computer's hard disk continually during the tests. (LNP offers a number of formats for downloading data; the format described above was used in this study) After a test, the data file was copied to a diskette and stored for later processing. Items in the output file such as field width, number of significant digits, column headings, time, date, and comments can be controlled during the initial software configuration. The data file created in this manner can be readily copied into a spreadsheet for manipulation and plotting.

The software was configured to display 5 different screens on the computer monitor during testing. The screens showed real time plots of deflection versus time and load versus time, and digital displays of deflection and load. The display was monitored during testing, and deflection readings of selected instruments were manually recorded at various load intervals to serve as a backup to the electronic data.

3.7 CONSTRUCTION SCHEDULE AND COST

Design of the field test facility began in June 1997. The field investigation was initiated about the same time and continued intermittently throughout the fall and winter months. Construction activities started in September 1997 and were essentially complete in April 1998. Work on the instrumentation was conducted in May 1998. Load testing began in June 1998 and ended in October 1998.

Approximate costs of materials and supplies for building the field test facility are listed in Table B.5. The total was about $34,000. The majority of design and construction work was performed by Virginia Tech graduate research assistants. Salary costs are not reflected in the table, but are estimated to be about $20,000. In addition, because the test site was developed at Virginia Tech, where there are well-equipped geotechnical and structural laboratory facilities, it was possible to utilize other equipment and supplies at no cost to the project. The total estimated purchase or rental value of this other equipment is about $16,000. Thus, in total, it has been possible to develop a test site with an estimated value of $70,000 under a research project with a total budget of $36,000.

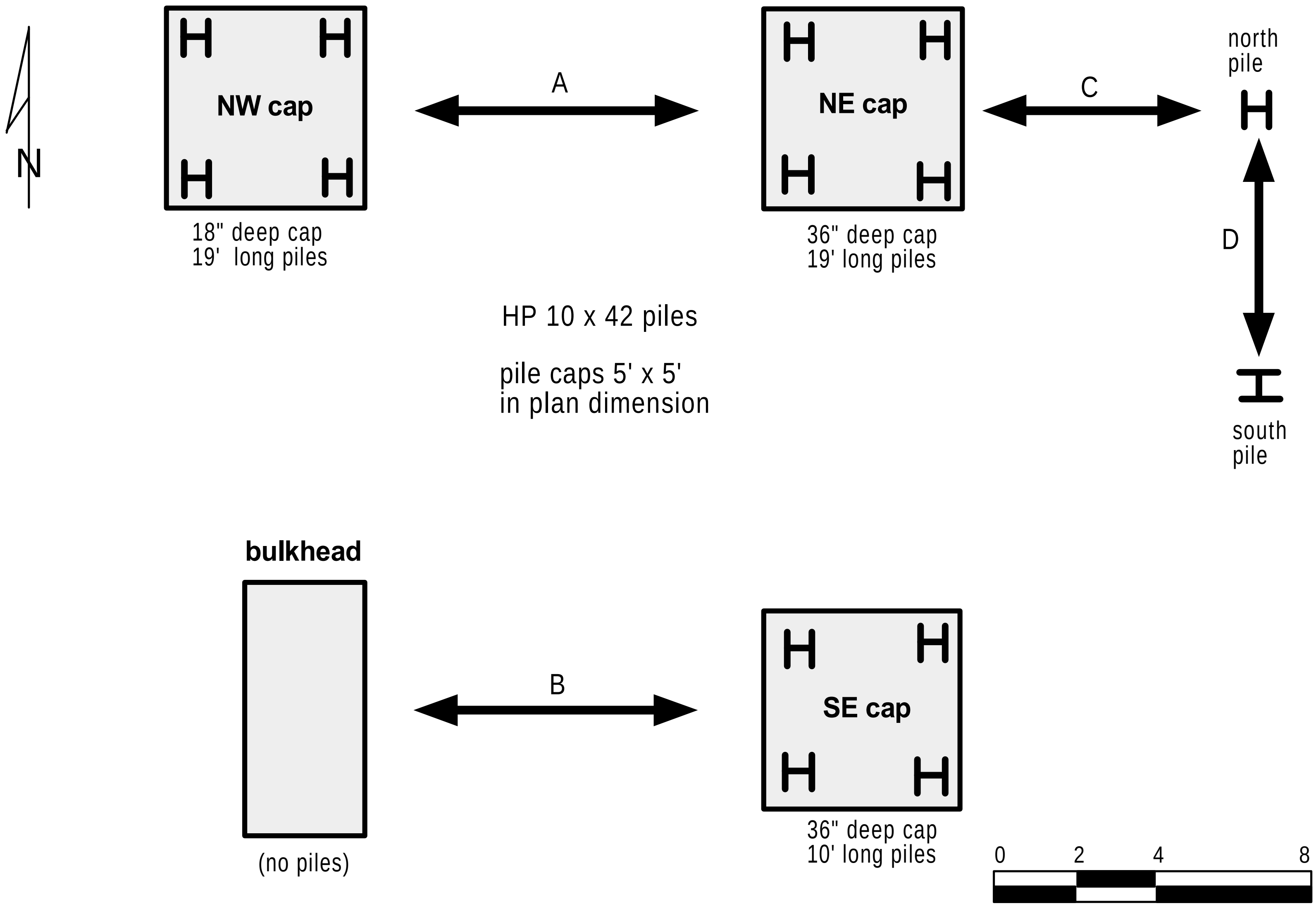


Figure 3.1. Plan view of field test site.

(a) Rebar in place for 18” deep cap.

(b) Finishing concrete surface of 36” deep cap.

(c) Trench at setup A, ready for loading equipment.

(d) Installing hydraulic ram in loading trench.

Figure 3.2. Foundation construction photos.

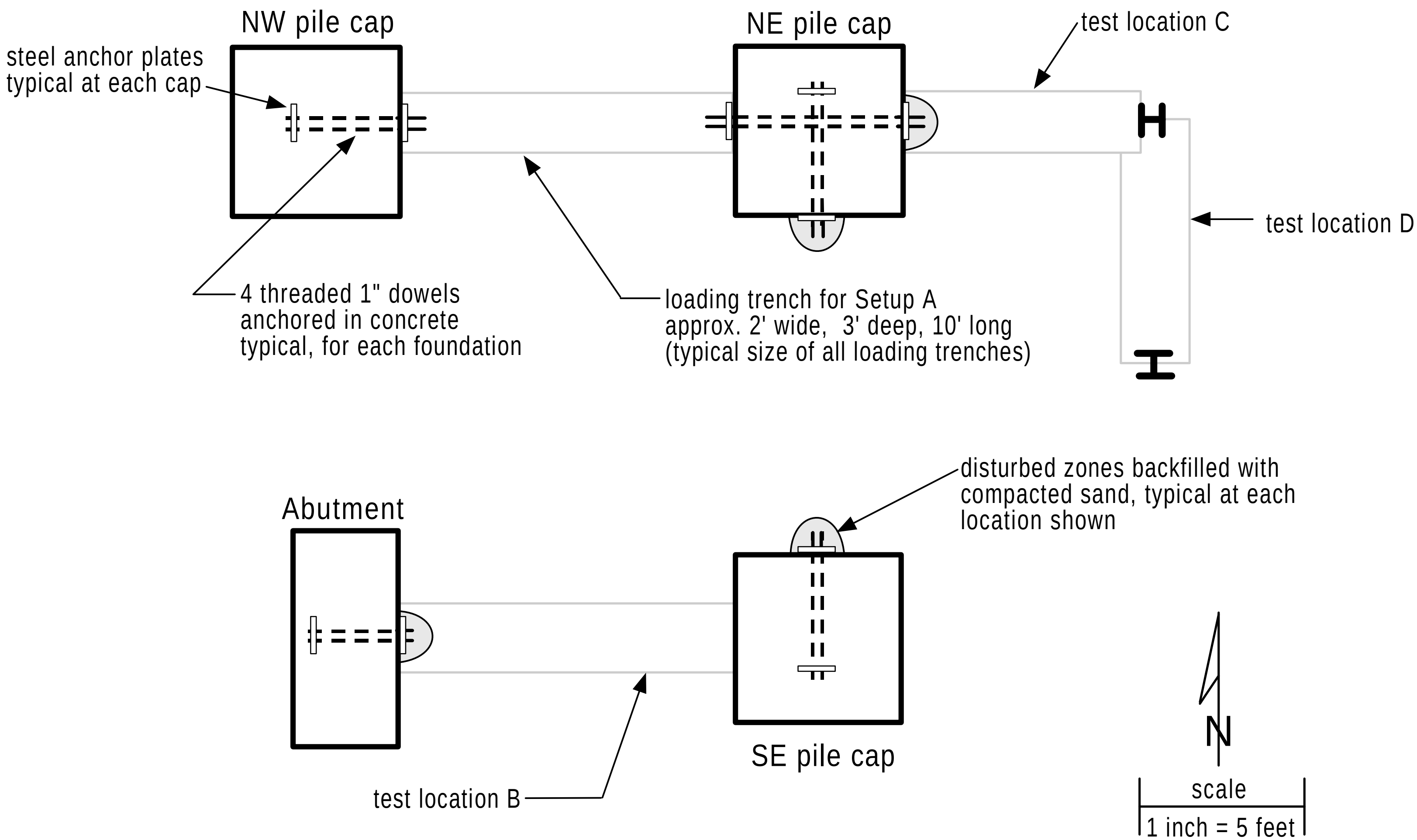


Figure 3.3. Plan view of anchor rod layout.

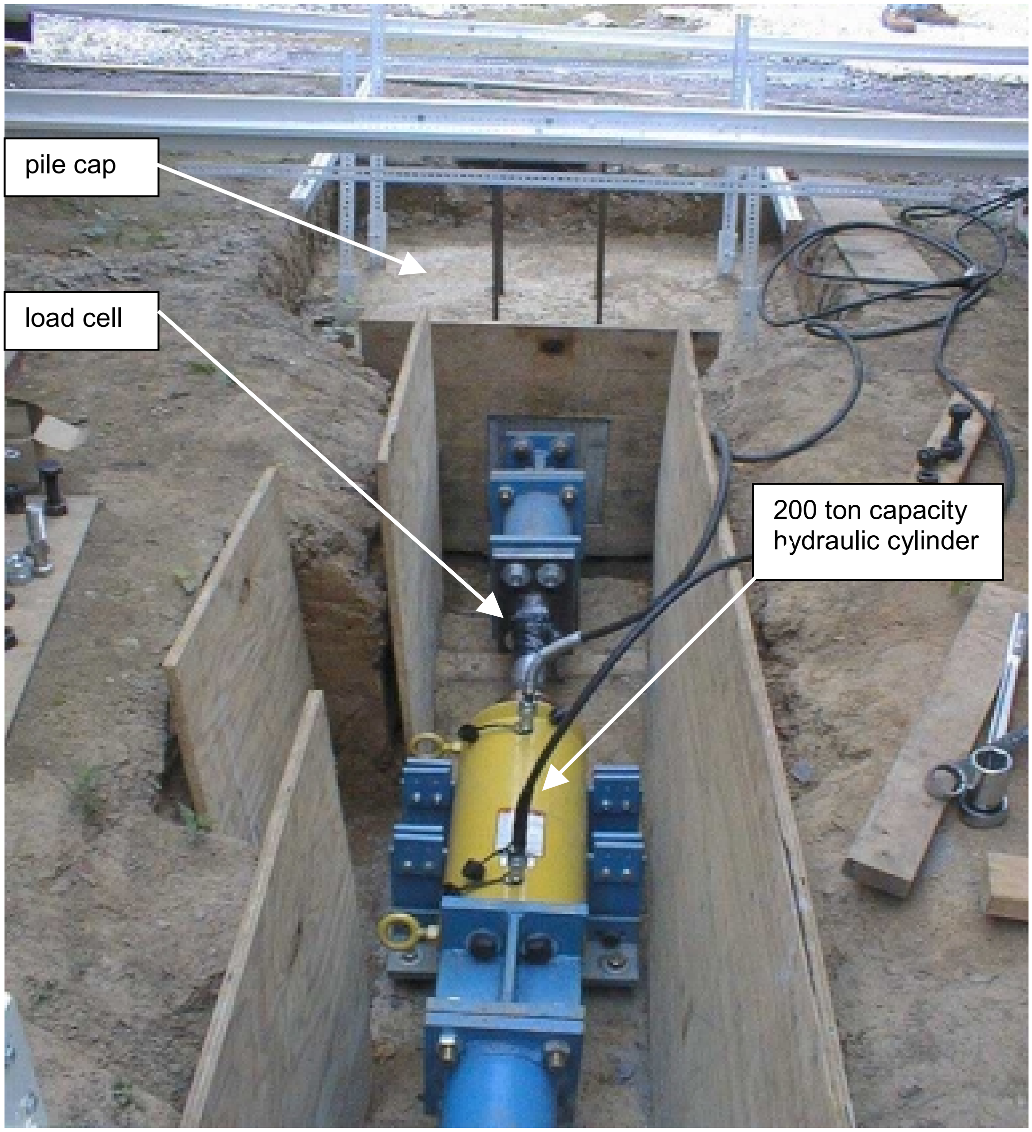
(a) Erecting tent frame.

(b) Tent frame under construction.

(c) View of tent from the east.

(d) View of tent from the west.

Figure 3.4. Tent shelter photos.



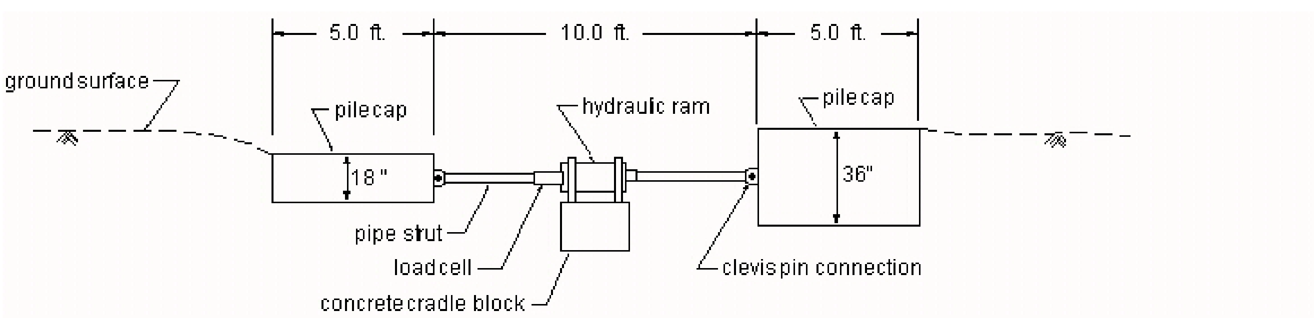
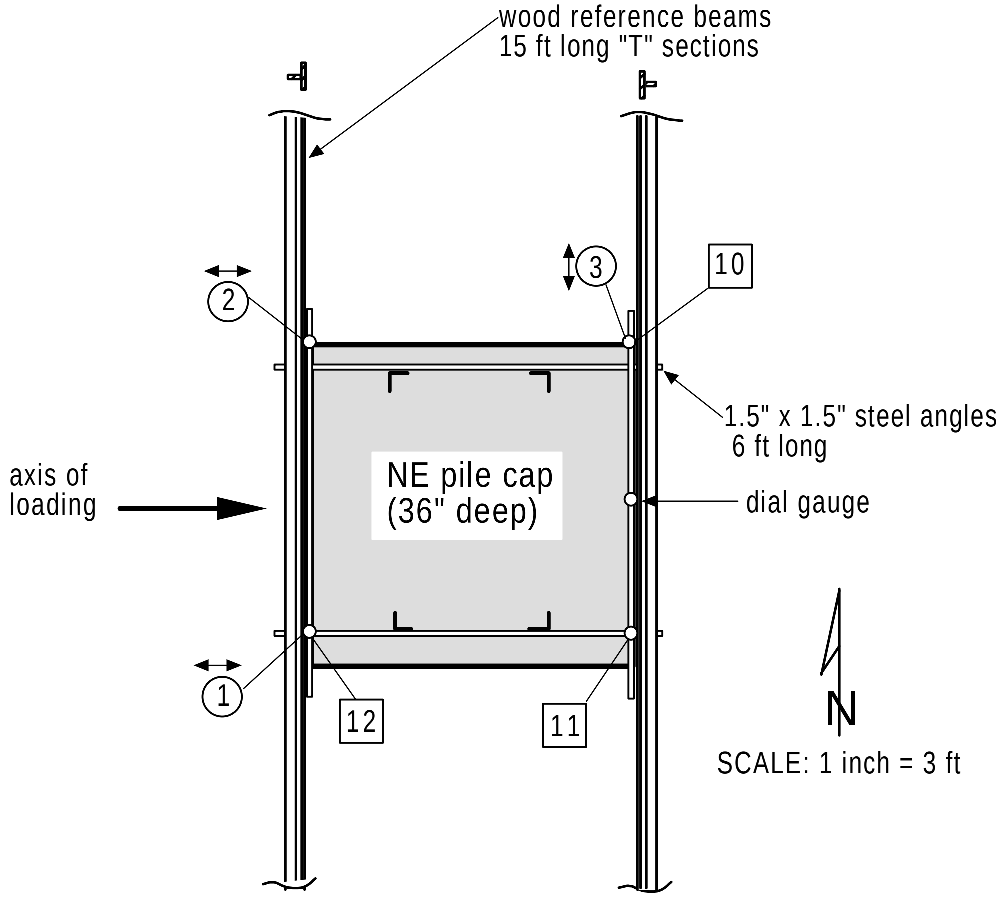


Figure 3.5. Hydraulic ram and steel struts positioned in loading trench.



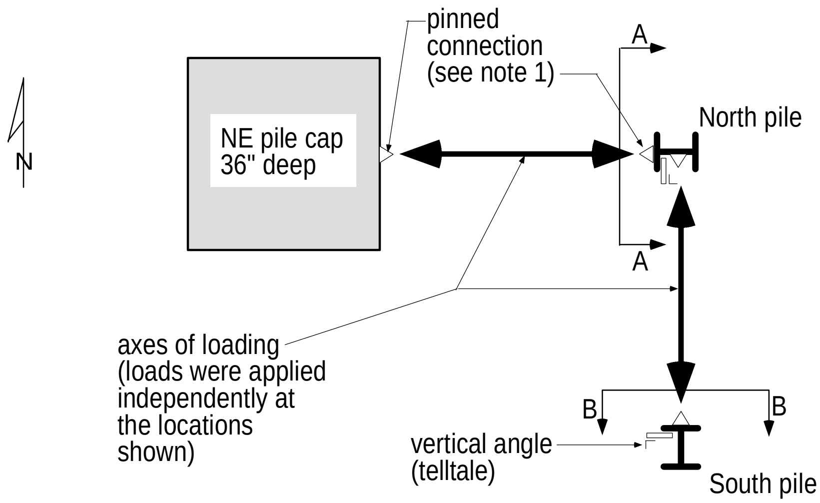
LEGEND

(X) Linear potentiometer positioned in the horizontal plane, arrows designate direction of measurement, "x" indicates intrument number.

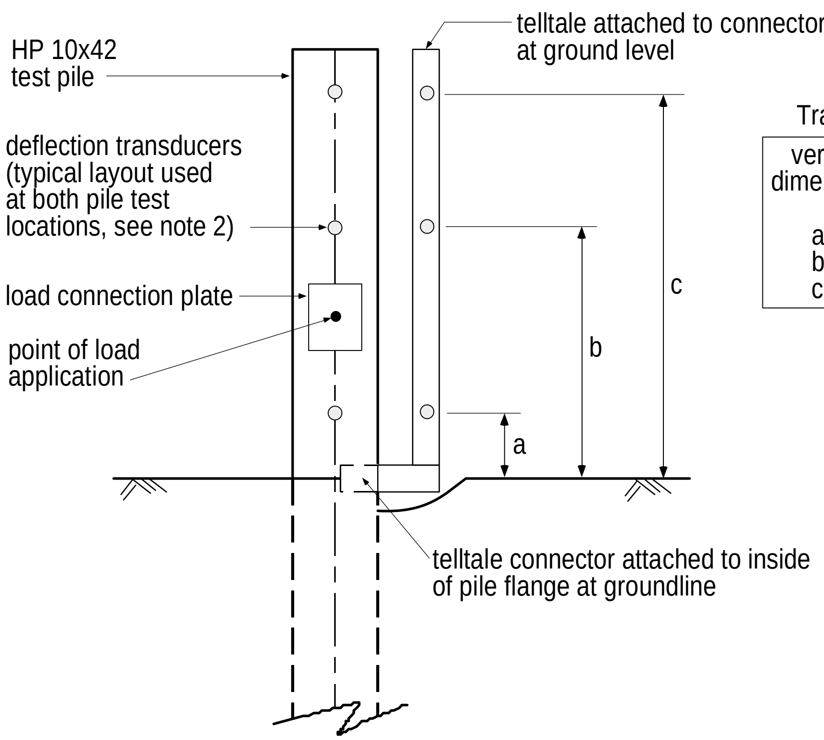
$\boxed{X}$ Linear potentiometer positioned for vertical deflection measurement, "x" indicates instrument number.

Figure 3.6. NE pile cap instrumentation plan.

Figure 3.7. Instrumentation in place for measuring deflections during lateral load test.



(a.) Plan view - scale: 1 in = 5 ft.

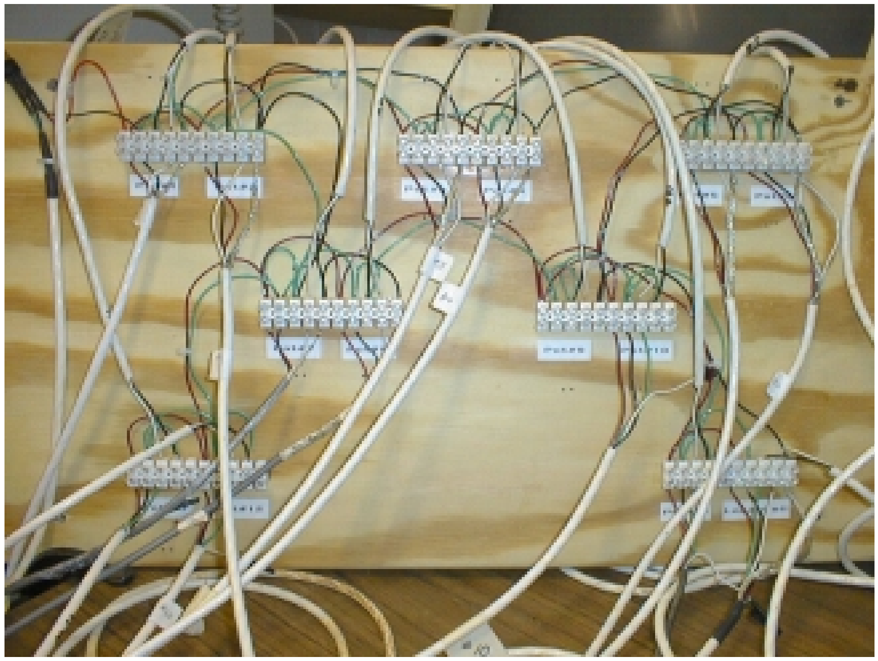


Transducer Locations (ft)

vertical dimension	north pile	south pile
a	0.7	0.35
b	2.6	2.35
c	4.0	3.55

(b.) Sections A-A (north pile) and B-B (south pile) - scale: 1 in = 2 ft.

Figure 3.8. Instrumentation layout for individual piles.


(a) Instrument terminal board.


(b) Data collection system mounted inside of minivan.

Figure 3.9. Photos of data collection system.