Design and Construction of Three Instrumented Test Piles to Examine Time Dependent Pile Capacity Gain

ABSTRACT: Three heavily instrumented test piles were designed, constructed, and installed at a bridge reconstruction site in Newbury, Massachusetts as part of a research initiative into time dependent pile capacity gain. Pile instrumentation consisted primarily of an alternating pattern of piezometers and strain gages, allowing for correlation between pile capacity gain and excess pore pressure dissipation along discrete pile segments. Additional instrumentation within the piles included accelerometers, telltales, and radial pressure cells, allowing monitoring of total pressure at the pile wall. Standard dynamic gages (strain gages and accelerometers) were also attached to the piles during dynamic testing. A total of 86 vibrating wire, 17 electrical resistance, 17 telltales, and 4 piezo-resistive gages were installed within the test piles to record strain, displacement, pressure, and acceleration.

This paper describes (i) the initial installation location of the test piles, (ii) the design and layout of the individual test piles, (iii) the selection and installation design of the individual instrumentation within the test piles, and (iv) assembly of the test piles. Sample measurements of the various instruments and a summary of the instrumentation performance are also presented.

KEYWORDS: pile, pore water pressure, dissipation, capacity gain, piezometers, total pressure cells, strain gages

Introduction

As part of a long-term research initiative into time dependent pile capacity gain conducted by the Geotechnical Engineering Research Laboratory at the University of Massachusetts—Lowell and supported by the Massachusetts Highway Department, three full-scale instrumented test piles were designed, constructed, and installed at a bridge reconstruction site in Newbury, Massachusetts (hereafter referred to as the Newbury site). Although configured for initial installation at the Newbury site, all three test piles were designed to be reusable for future research. The three instrumented test piles consisted of one 32.4-cm (12.75-in.) diameter by 31.4-m (103-ft) long closed ended steel pipe end-bearing pile (designated Test Pile #1 or TP#1), one 32.4-cm (12.75-in.) diameter by 24.4-m (80-ft) long closed ended steel pipe friction pile (TP#2), and one 35.6-cm (14-in.) square by 24.4-m (80-ft) prestressed concrete friction pile (TP#3). Steel pipe piles were selected for TP#1 and TP#2, since the majority of the adjacent bridge pilings would be of the same size and type. This selection would not inconvenience the pile-driving contractor, and it would allow for the instrumented test pile data to be correlated to the production test pile data without shape and size influences. A pre-stressed concrete pile was selected for the remaining test pile, since (i) the majority of piles used in the Boston area are of this pile type and (ii) it would provide an additional pile size, type, and surface roughness for the research study.

Newbury Test Site

The testing location is a bridge reconstruction site along US Route 1 on the border between Newbury and Newburyport, Massachusetts. The general soil profile at the site (from ground surface downward) consists of the soil strata listed in Table 1 and presented in a graphical form in Fig. 1. The soil stratigraphy is typical of the general subsurface formations found in the Boston area. The site was considered suitable for the project since,

1. An approximately 13-m (42.5-ft) thick deposit of Boston Blue Clay was located close to the surface, extensive enough to allow assessment of time dependent pile capacity gain over a finite pile length sufficient to assume radial consolidation,
2. Previous testing conducted at the site by the University of Massachusetts—Lowell showed that the site was easily accessible and a short driving distance from the University (Paikowsky and Peach 1995).

These factors also made the site a useful location for Multiple Deployment Model Pile (MDMP) testing (see Paikowsky and Hart 2000). An extensive field and laboratory study of the site was conducted to detail the physical characteristics and engineering parameters of the subsurface (Paikowsky and Chen 1998), aiding in the analysis of the MDMP and instrumented test pile design and interpretation.

Test Pile Design—Overview

The design of the test piles followed the routine of engineering design effort by defining an objective and proceeding through a series of logical steps to prepare plans and specifications (Dunnicliff...
TABLE 1—General soil profile at test pile cluster location (Paikowsky and Chen 1998).

<table>
<thead>
<tr>
<th>Depth, m</th>
<th>Soil Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0–2.4</td>
<td>Granular Fill/Concrete Outwash (Cased)</td>
</tr>
<tr>
<td>2.4–2.7</td>
<td>Organic Silt and Peat</td>
</tr>
<tr>
<td>2.7–5.4</td>
<td>O.C. Clay</td>
</tr>
<tr>
<td>5.4–11.5</td>
<td>Soft N.C. Clay</td>
</tr>
<tr>
<td>11.5–16.4</td>
<td>N.C. Clay</td>
</tr>
<tr>
<td>16.4–19.3</td>
<td>Interbedded Silt, Sand, and Clay</td>
</tr>
<tr>
<td>19.3–21.6</td>
<td>Silty Sand</td>
</tr>
<tr>
<td>21.6–23.9</td>
<td>Interbedded Silt, Sand, and Clay</td>
</tr>
<tr>
<td>23.9–26.3</td>
<td>Fine to Medium Sand</td>
</tr>
<tr>
<td>26.3–30.5</td>
<td>Till</td>
</tr>
<tr>
<td>30.5+</td>
<td>Bedrock</td>
</tr>
</tbody>
</table>

FIG. 1—General soil profile at the Newbury site and test pile instrumentation layouts.

TABLE 2—Summary of measured parameters, instruments, gage types, and gage abbreviations.

<table>
<thead>
<tr>
<th>Measured Parameter</th>
<th>Measurement Instrument</th>
<th>Gage Type</th>
<th>Gage Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excess Pore Pressure Buildup and Dissipation</td>
<td>Piezometers</td>
<td>Electrical Resistance (ER)</td>
<td>ERPG</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vibrating Wire (VW)</td>
<td>VWPG</td>
</tr>
<tr>
<td>Total Radial Pressure along Pile</td>
<td>Total Soil Pressure Gages</td>
<td>Pressure measurement w/VW piezometer</td>
<td>TSPG</td>
</tr>
<tr>
<td>Dynamic Pile Testing – Stress Wave Measurements</td>
<td>Strain Gages and Accelerometers</td>
<td>Electrical Resistance</td>
<td>ERS</td>
</tr>
<tr>
<td>Load Distribution along pile during static testing</td>
<td>Strain Gages</td>
<td>Piezo-resistive (interior)</td>
<td>PCB</td>
</tr>
<tr>
<td>Movement along the Pile</td>
<td>Telltales</td>
<td>Electrical Resistance</td>
<td>ERS</td>
</tr>
<tr>
<td>Pile Temperature</td>
<td>Thermistor</td>
<td>Slender Rod</td>
<td>TT</td>
</tr>
</tbody>
</table>

Review of prior research and test projects (Mynampaty 1993; Paikowsky et al. 1995), experience gained via model pile design and testing experience (Paikowsky and Hart 2000), and a methodology for a practical evaluation of the phenomena of time dependent pile capacity gain resulted in identifying a total of six parameters to be measured using the instrumented test piles. The primary measurement was pore water pressure buildup and dissipation, the controlling factor of time dependent pile capacity gain for piles driven in cohesive soils. Dynamic testing during driving and restrikes, standard load distribution (i.e., strain gages and telltales), and typical static load test instruments (e.g., pile top load and displacement) were selected to determine pile capacity. Total radial stress measurements coupled with pore pressure measurements to produce effective stresses, providing valuable insight into the capacity gain phenomenon. Table 2 provides a summary of the selected parameters to be measured and the means to achieve it.

Nine types of instruments were selected for installation within the test piles based on (i) the requirement to have backup and/or redundant instrumentation to measure various parameters, (ii) previous design experience with certain instrumentation types, and (iii) available instrumentation. Vibrating wire gages were selected for the majority of the strain gages and piezometers based on (i) long-term durability and stability, (ii) the ability to take measurements in unfavorable environmental conditions (e.g., such as excessive moisture which could short circuit electrical gages), and (iii) the ability to have long cable lengths which would not adversely affect the gage measurements. Electrical resistance strain gages and piezometers were chosen for installation in the two steel pipe piles to supplement the vibrating wire gages due to the ability to sample the data provided by them at high frequencies and as such to monitor some of the parameters during dynamic measurements (e.g., stresswaves in the pile and pore pressures along the pile wall during driving). Hajduk and Paikowsky (2000) illustrate and discuss the limitations of vibrating wire gage measurements to follow pore pressure variations during driving. Additionally, the electrical transducers provided redundancy, thereby allowing for continuous measurements in case of failure of one of the data acquisition systems. Piezo-resistive accelerometers were added to the two steel
pipe piles in order to complement the electrical resistance strain gages during dynamic measurements, allowing for examination of dynamic changes along these piles during driving and hammer restrikes. Piezo-resistive accelerometers were selected based on the general practice of using them on steel piles and their performance in low acceleration environments, such as those expected at the test pile tip and mid-length. Telltales were installed in all the piles to provide backup measurements for the strain gages and measure movements along the pile during static load testing. Total soil pressure gages were installed in the two steel pipe piles to provide measurements of radial stresses at discrete locations. Accompanied with pore pressure measurements, the influence of effective stress changes on time dependent pile capacity gain could be examined. A pressure measurement system, which incorporated a pressure cell built into the pile wall and a vibrating wire pressure transducer, were selected for the two steel pipe piles. Thermistors of selected gages in all three test piles would be monitored to record pile temperature throughout the testing period.

A summary of the selected instrumentation types, the associated measured parameter, and the abbreviations for the selected instrumentation gages installed within the test piles is presented in Table 2. Instrumentation that was installed in the ground around the piles or attached to the piles during dynamic and static testing is not discussed in this paper.

### Estimated Magnitudes of Measurements

The following measurements are of key importance: the maximum pore pressure buildup along the pile wall ($U_{max}$), the total radial pressure along the pile wall ($\sigma_{r, \text{max}}$), the maximum acceleration during pile driving and hammer restrikes ($A$), the maximum compressive and tensile stresses during pile driving and hammer restrikes ($\sigma_c$ and $\sigma_t$), the ultimate skin and total static pile capacities ($R_s$ and $R_u$), and pile top and tip movement during static load testing ($\delta_{\text{tip}}$ and $\delta_{\text{tip}}$). Variations in the temperature magnitudes were expected to be within the seasonal averages for the area (i.e., $0^\circ$C to $32^\circ$C) before reaching a constant temperature of approximately $10^\circ$C after the piles’ installation within the ground.

Prediction of the magnitudes and change for the desired parameters were made based on (i) normalized relationships between distance from the pile wall and excess pore pressure buildup, (ii) estimates of static pile capacity, and (iii) dynamic wave equation analysis of piles (i.e., WEAP). Estimations of pore pressure buildup and dissipation were based on the research presented by Paikowsky et al. (1995). Total pile radial pressure estimates were based on these estimates and coefficients of lateral earth pressure around driven piles of $1.20–1.30$ for pipe piles and $1.45–1.65$ for precast square concrete piles suggested by Mansur and Hunter (1970). Both the expected total pore and radial stress estimates were calculated over one standard deviation range of the mean of the predicted initial excess pore pressure buildup (see equation in Table 3). Ultimate pile capacity predictions were calculated based on the average of several commonly used design methods, dependent on the soil type (cohesive or noncohesive). Paikowsky and Hajduk (1999) present detailed calculations for skin and point resistance estimations for each method for the various soil layers. Pile movement estimates were determined using the computer program STAPRO, a static load test simulation program developed by Samuel Paikowsky and Les Chernauskas at the University of Massachusetts—Lowell (Chernauskas 1992). The computer software program GRLWEAP™ was used to estimate the pile accelerations and stresses from driving and hammer restrikes. Values for soil quake and damping were based on analysis of the bridge production piles (GTR 1996). Available site data from Paikowsky and Chen (2000), such as groundwater levels and soil engineering properties, were incorporated where appropriate. A summary of the relevant equations and references for the magnitude of change estimations and the calculated values of the key parameters are provided in Table 3.

### Instrumentation Selection

Instrumentation selection was based on (i) the ability of the gage to measure the desired parameter in terms of magnitude and frequency, (ii) past performance under similar conditions, (iii) designer experience/familiarity, and (iv) budget considerations. A summary of the various instruments selected for the test piles is presented in Table 4. The details of the instrumentation design within the test piles were based on previous experience and knowledge of the parameters to be measured. In addition, the individual instrumentation details incorporated the lessons learned during the design of the Multiple Deployment Model Pile (MDMP) (Paikowsky and Hart 2000). Previous research examined during the MDMP design included the Piezo-Lateral Stress Cell (PLS) (Wissa et al. 1975), Grosch and Reese (G&R) model pile (Grosch and Reese 1980), and Norwegian Geotechnical Institute (NGI) model pile (Karlstrud and
were installed at each location. For the two steel pipe piles, two strain gages for constant pore pressure measurement in case of a data acquisition system failure. For the two steel pipe piles, two strain gages was used along the length of the two steel pipe piles to provide alternating pattern of electrical resistance and vibrating wire piezometers between sets of two strain gages was used on each test pile. An additional requirement, a reoccurring pattern of piezometers located halfway (iv) the soil profile at the Newbury Site. To satisfy the primary requirement for repetitive and/or backup measurements, and (iv) the soil profile at the Newbury Site. To satisfy the primary requirement, a reoccurring pattern of piezometers located halfway between sets of two strain gages was used on each test pile. An alternating pattern of electrical resistance and vibrating wire piezometers was used along the length of the two steel pipe piles to provide for constant pore pressure measurement in case of a data acquisition system failure. For the two steel pipe piles, two strain gages were installed at each location 180° apart to allow for backup and eccentric loading measurements. Due to the confined space within the pile, only one strain gage was used at each location for TP#3.

The electrical resistance strain gages and piezo-resistive accelerometers were located at the middle and tips of the two steel pipe piles in order to record accelerations (and calculate velocities) along the test pile lengths. The remaining instrumentation was placed according to the soil profile. Telltales were located at the top and bottom of the clay layer as well as at the pile tip for each test pile. The total soil pressure gages were located within the middle of the clay and interbedded silt, sand, and clay layers and near the same elevations of the ground piezometers installed around the test pile locations. Figure 1 shows the final instrumentation layouts for the test piles relative to the soil profile. Table 4 summarizes the instrumentation installed within each of the test piles; see Table 2 for notations.

The two steel test piles were cut into smaller segments to allow for easier installation of the instrumentation. Typical pile segments for each steel test pile measured 1.83-m (6.0-ft) long. The various gages were typically installed within each segment at a distance of 0.46 m (1.5 ft) from both open ends. Refer to Paikowsky and Hajduk (1999) for additional details concerning the segment instrumentation.

**Instrument Identification System**

An instrument identification system was developed to catalog the test pile, relative location, orientation, and instrument type for each gage within the cluster. The identification system also assisted in the development and installation of the data acquisition array, described in detail by Paikowsky and Hajduk (1999). The instrument identification system used the following format:

\[ X-Y-Z-Q \]

Where:

- \( X \) = Pile Number (1, 2, or 3).
- \( Y \) = Segment Number (for TP#1 & 2) or Gage Number (TP#3).
- \( Z \) = Gage Type (see Table 2).
- \( Q \) = Gage Orientation within the specific test pile (A or B: See Figs. 2–4)

Examples of gage identifications are the following:

- Example #1: 1-6-VWSG-A
- Example #2: 2-10-TSPG-B

The first identification is of vibrating wire strain gage (VWSG) A of Segment 6 in TP#1. The second identification is that of the total soil pressure gage (TSPG) B of TP#2 in Segment 10. A TH in front of the gage identification would specify the thermistor of that gage.

**Instrumentation Details**

**Interior Accelerometers**

The piezo-resistive accelerometers (Model EGE-72B-51B-1000 manufactured by Enthan Devices of Fairfield, New Jersey and calibrated by Pile Dynamics of Cleveland, Ohio) were bolted to mounting blocks welded onto interior pile walls. A typical interior accelerometer assembly cross section for the two steel pipe piles is shown in Fig. 5.

**Test Pile Piezometers**

The basic piezometer configuration for each pile consisted of porous stone(s), located flush with the pile surface, connected by small diameter steel tubes to pressure transducers located below the stones. Each piezometer was located below the porous stone to provide a gravity flow of water into the system and allow for any air trapped within the system to escape. An aluminum oxide porous stone, 25 mm (1 in.) in diameter, separated the soil from the piezometers while allowing for free water flow. Each assembly had an additional access tube installed slightly above each piezometer to allow for resaturation and deairing of the system before pile installation (if necessary) and for future reuse. A similar system used on the MDM provided excellent results (Paikowsky and Hart 2000).

Figure 6 shows a typical piezometer assembly for TP#1 and TP#2. Due to the limited space inside the steel pipe piles, each piezometer would be connected to only one porous stone. Figure 7 shows a typical piezometer assembly for Test Pile #3, consisting of two porous stones pressed into stainless steel porous stone holders.
located 180° apart from each other and connected to Geokon Model 4500H vibrating wire piezometer.

Each piezometer assembly was deaired and saturated prior to pile assembly (TP#1 & TP#2) or instrument installation (TP#3). Satur- 
aton was maintained in TP#1 and TP#2 through use of a piezometer cap system (see Fig. 6). For TP#3, each porous stone holder was 
sealed using rubber prior to the installation of the instrumentation into the pile. The access tubes were used to re-saturate/deair any 
assembly that had experienced a loss of saturation prior to installa-
tion.

**Strain Gages**

The electrical resistance (Model LWK-06-W250B-350 by the Measurements Group of Raleigh, North Carolina) and the vibrating wire strain gages (Model VSM-4500 by Geokon of Lebanon, New Hampshire) were attached to the interior of the steel pipe piles in accordance with the manufacturer guidelines. A typical vibrating wire strain gage installation cross section for the steel pipe piles is shown in Fig. 8. The vibrating wire strain gages for TP#3 were cast in the center of the pile in accordance with the manufacturer guidelines. A cross section of a typical VWSG installation for TP#3 is shown in Fig. 9.

**Total Soil Pressure Cells**

A unique pressure gage was designed to measure total lateral soil pressure on the two steel test piles (TP#1 and #2). Each gage consisted of a thin stainless steel membrane that covered a 180° cutout
A section of the pile filled with a thin layer of fluid connected to a vibrating wire pressure transducer. Two TSPG gages were installed into special 32.4-cm (12.75-in.) diameter by 2.14-cm (0.843-in.) thick by 0.61-m (2-ft) long pile segments with accompanying vibrating wire piezometers located below the gages. A typical TSPG cell is presented in Fig. 10.

A total stress measurement system, using confined fluid, records pressure measurement due to applied soil pressure or due to volume changes of the confined fluid as a result of temperature variations. Each total pressure cell was, therefore, calibrated for pressure and temperature, the summary of which is presented in Table 5.

The pressure calibrations were performed using two specially built calibration chambers. Each chamber followed the principles of the MDMP calibration device, consisting of a sleeve placed over the section of the pile containing the soil pressure cell (see Paikowsky and Hart 2000). The laboratory calibration chamber is shown in Fig. 11. This system was used to calibrate the total soil pressure gages of TP#1 and 2-6-TSPG-A & B for TP#2.

During construction of the total pressure cells, it was suggested that the ratio between the before and after installation pressure calibrations of the vibrating wire transducer were unique for all the gages, thereby eliminating the need for individual calibration of all the gages. In spite of this approach, it was decided that each gage should be individually calibrated and a field calibration device was built for 2-10-TSPG-A and B. The field calibration device consisted of a thin strip of mild steel, molded to a diameter slightly greater than the outside diameter of the pile, which was then welded to the pile, forming an air-tight seal. The
chamber was removed prior to pile installation without any apparent damage to the gages. Comparison of the field calibrations of Gages 2-10-TSPG-A and B to the suggested pressure calibration based on the pressure transducer-completed gage ratio of the other gages showed that this ratio was not a unique constant for all the gages.

Temperature calibrations for each total soil pressure gage were conducted by varying the temperature while keeping the pressure constant. Typical temperature calibration tests using the laboratory calibration chamber were conducted by submerging the TSPG section in water and subjecting to overnight temperature variations. The temperature range of these tests, from approximately 7 to 15°C, was the temperature range expected during installation (underground) and, therefore, considered adequate.

The results of the individual calibrations showed that the gages were significantly more sensitive to temperature than to pressure. For example, in a comparison of the pressure calibration factor of Gage 1-6-TSPG-A (0.1941 kPa/digit) to the thermal calibration factor (−27.3 kPa/°C), the higher sensitivity of the gage to temperature relative to a change in the pressure reading is clearly shown. In spite of the high sensitivity, the temperature in the ground below a relative shallow depth of approximately 3 m (10 ft) remains constant, such that most variations relate to pressure changes in the soil. Temperature measurements taken during the installation of TP#1 and TP#2 showed that at the TSPG gage locations, temperatures stabilized to near 10.5°C within 15 h from the start of driving.

Telltales

Steel telltale rods, with diameters of 0.64 cm (0.25 in.) and 0.96 cm (0.375 in.), were installed in each of the test piles. Telltale elevations within the piles are provided in Fig. 1. Various types of casing were installed within the individual test piles to protect and guide the telltale rods during installation and testing. Within TP#1 and TP#2, 1/2-in. Schedule 40 steel pipe was used for the telltale protective tubing. The locations of the telltale tubing for TP#1 and TP#2 are shown in Figs. 2 and 3, respectively. The steel telltale tubing was connected to support strips installed within the test pile sections to maintain stability. Telltale casing alignment during pile assembly was accomplished through use of steel pipe couplings attached to the upper ends of the telltale tubing sections. This connection also acted as a guide during segment assembly. Figure 12
shows a typical connection of the telltale tubing between pile segments for TP#1 and TP#2. For TP#3, 3/4-in. diameter Schedule 40 PVC pipe was chosen as the protective telltale casing. The locations of the telltale tubing in TP#3 are shown in Fig. 4. Due to the limited space inside TP#3, only one telltale tube was installed at each location. The PVC telltale tubing was attached to the steel tension cables.

For TP#1 and TP#2, the telltale rod displacement measurements were made relative to the pile top and not a reference beam by using LVDT holders that attached directly to the telltale protective tubing. This method of measurement was chosen, due to the difficulty of extending the telltale rods out to be measured from a reference beam. A cross section of a typical interior LVDT setup during static and Osterberg load cell tests for TP#1 and TP#2 is presented in Fig. 12. A bayonet system was used to secure the telltale rods at the desired locations. A cross section of the bayonet connection system is also shown in Fig. 12.

**Osterberg Load Cell (O-Cell™)**

In order to measure capacity as soon as possible after driving and periodically thereafter without using the reaction frame, an Osterberg Load Cell (O-Cell™) was installed in TP#1. Selection of the proper Osterberg Load Cell for TP#1 was dependent on the pile size and predicted pile capacity. For TP#1, an ultimate load of 2.67 MN (300 tons) was predicted, with a distribution

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3 Loadtest, Inc., Gainesville, FL 32606.
of 1.54 MN (173 tons) end-bearing and 1.13 MN (127 tons) side resistance. Although standard Osterberg Load Cell sizes existed that could attach directly to the end of TP#1, it was decided to use a modified O-Cell™ design in order to:

1. Maintain a uniform outside pile diameter, since the standard O-Cells™ available did not exactly match the selected pile size,
2. Prevent water penetration into the test pile through the O-Cell™,
3. Ensure retrieval of the O-Cell™ for future pile testing,
4. Select an appropriate capacity of the O-Cell™ based on the pile capacity predictions.

The modified design consisted of placing a smaller, standard O-Cell™ into a 32.4-cm (12.75-in.) diameter by 1.3-cm (0.5-in.) thick by 0.91-m (3-ft) long pile segment. Based on the interior dimensions of the pile segment selected for the O-Cell™ and the predicted pile capacity, a 2.67 MN (300 ton) capacity O-Cell™ was chosen. This O-Cell™, typically used for 22.9-cm (9-in.) diameter pipe piles, was chosen for TP#1. Figures 13 and 14 show a plan view and cross section, respectively, of the modified Osterberg Load Cell design in TP#1.

**SMARTPILE™ System**

In addition to the instrumentation detailed above, the SMARTPILE™ system, comprised of impulse force sensors embedded approximately 15.2 cm (6 in.) from the pile head and toe, was installed in TP#3. Further details regarding the SMARTPILE™ system are provided by Ooi and Frederick (2003).

**Test Pile Fabrication**

**Instrument Wire Exits**

As the instrumentation wiring was embedded within the test piles, a wiring exit was required near the pile head in order to attach these wires to the data acquisition systems. The instrument wires could not exit through the pile tops, due to the necessity of placing a helmet cap over the piles during installation and hammer restrikes. The instrument wire bundles from each pile were wrapped in protective tape, placed in rubber sleeves, and then attached to the piles using wire mesh strain relievers to minimize strains and damage during pile installation and hammer restrikes. Additional
details of the instrument wire exits are provided by Paikowsky and Hajduk (1999).

**Test Piles #1 and #2 Fabrication**

Fabrication of the two steel test piles consisted of a four-step process:

1. Cutting of the steel piles into various segment lengths,
2. Installation of the planned instrumentation,
3. Shop assembly of the pile segments into two sections of approximately equal length,
4. Field assembly of the two pile sections at the test site.

The arrangement between shop and field assembly would allow a controlled environment for pile reassembly and simplify the transport of the piles to the test site. The steel used for the test piles was ASTM A252 Grade 50.
Pile shop assembly consisted of placing two pile segments horizontally on adjustable roller supports. The lower pile section was attached to a lathe, which rotated the pile segments when required. Alignment of the two pile segments was achieved through the use of pile markings and the telltale connections. Levels were used to insure alignment of the segments along the pile axis. Instrumentation cables were secured at every other support strip via nylon wire ties placed close to the pile center to protect the cables from the heat of the welding process. A full penetration, v-groove butt weld was used to join the steel test pile segments during shop assembly. A continuous steel backing plate with intermittent guides was used to protect the instrumentation wires during welding and maintain the required gap between pile segments. A cooling system was constructed to provide additional protection to the instrumentation from heat during the welding process. The cooling system setup during shop pile assembly is shown in Fig. 15. Instrumentation readings taken before and after the welding of each pile segment showed that the heat from the welding process did not significantly affect each instrument. Figure 15 shows a typical shop assembly setup.

A standard steel conical pile tip was welded to the end of TP#2 during shop assembly of the pile. For TP#1, the final shop assembled sections were 14.0-m (46-ft) and 17.4-m (57-ft) long for the top and bottom, respectively. For TP#2, the final shop assembled top pile section was 11.0-m (36-ft) long and the bottom was 13.4-m (44-ft) long.

In order to study the relationship between surface roughness and pile capacity, each test pile would have a different pile surface within the granular soil layers. Part of TP#2 was coated with an epoxy coating providing a third surface, in addition to steel (TP#1) and concrete (TP#3). M45/M46, a two-component epoxy mastic coating was selected for its exceptional adhesion, coverage, flexibility, impact resistance characteristics, and immersion use in fresh or salt water. The epoxy coating was applied over the lower 24 ft of the pile, measured from the conical pile tip.

Field assembly of TP#1 and TP#2 was accomplished using welding and cooling techniques nearly identical to those used during the shop assembly. The field welds for both piles were not ground to match the pile surface. The assembled TP#2, 24.4-m long (80-ft), prepared for driving is shown in Fig. 16.

Test Pile #3 Casting

TP#3 was designed to have an effective prestress of 4.83 MPa (700 psi) and a nominal compressive strength concrete of 41.4 MPa (6000 psi). Details of the reinforcement placed with the pile are described by Paikowsky and Hajduk (1999). TP#3 was cast in

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4 Manufactured by Benjamin Moore & Company of Montvale, New Jersey.
Typical Pore and Radial Pressure Measurements

Data from the various test-pile instrumentation were collected from the time of installation to the end of the testing program. Details, results, and analysis of the test pile program data are thoroughly presented by Paikowsky and Hajduk (2004). Typical results from the vibrating wire piezometers, electrical resistance piezometers, and total soil pressure gages within the Boston Blue Clay (BBC) deposit are presented with depth and time in Figs. 17 and 18, respectively. The electrical and vibrating wire gage (ERPG and VWPG) measurements with time show (Fig. 17) that each gage type was capable of measuring pore pressure buildup and dissipation. The shift in pore pressure buildup between 1 and 100 h from the start of driving is associated with the locations of the gages on the pile, pile installation, and porous stone saturation and is not attributed to a difference between the gages. Examination of Fig. 17 shows a difference in measurements between the two gages at approximately 3 h from the start of driving. At this time, it appears that the ERPG records a higher pressure before each gage experiences a sudden drop in pressure. By plotting the same gage readings with depth, a better understanding of the measurement difference is observed.

Figure 18 shows that during penetration into the overconsolidated clay layer, each gage experiences a loss in pressure. Before this loss of pressure occurs, a slight pressure buildup was registered by each gage. The ERPG readings, which are being recorded at a higher frequency than the VWPG, provide more data points of the buildup. The lower recording frequency of the VWPG misses the majority of this buildup. Examination of similar comparisons between the two gage types for the installation of each of the steel pipe piles reveals comparable patterns of measurement.

Figures 17 and 18 also show that the total soil pressure gage (TSPG) measurements are similar in magnitude to the pore pressure measurements. This indicates that the magnitude of the effective radial stresses on the pile wall in the BBC during and after pile installation is about zero. These results match closely with the MDMP measurements (Paikowsky and Hart 2000), which were made with a dog-bone radial stress cell, indicating that the TSPG functioned properly.

Instrumentation Performance

In order to assess the overall durability of the test pile instrumentation, the percentage of working instruments at the end of the testing, labeled as % Durable, was computed to allow comparisons between the major instrument types. The results of this analysis
are presented in Table 6. Hajduk and Paikowsky (2000) provide details concerning the performance of the test pile instrumentation throughout the project. The key findings of Hajduk and Paikowsky (2000) are the following:

- The majority of vibrating wire instrumentation could withstand the harsh conditions throughout the installation and testing period.
- Failure of the various vibrating wire instruments did not align with specific testing events.
- Of the electrical resistance gages, only the electrical resistance piezometers continued to function throughout the project duration. Since failure of all the electrical resistance strain gages and piezo-resistive accelerometers did not occur during specific tests, their failures were attributed to the intrusion of water into the two steel pipe piles.
FIG. 16—Assembled Test Pile #2 before installation.
Summary and Conclusions

Three full-scale instrumented driven test piles were designed, built, and calibrated for monitoring time dependent capacity gain in cohesive soils. Each pile was designed to record pore pressure buildup and dissipation and skin friction at discrete segments along its lengths by means of strain gages and piezometers. Additional instrumentation included total lateral soil pressure cells, telltales, and accelerometers. Vibrating wire and electrical gage based instrumentation was used alternately to provide the ability for data collection during dynamic events and long-term monitoring, as well as redundancy in measuring systems. In total, 86 vibrating wire and 17 electrical resistance transducers were used, along with 17 telltales, and 4 piezo-resistive accelerometers. The cluster also contained an on-site data acquisition array to monitor the various types of instrumentation.
TABLE 6—Test pile instrumentation durability summary (after Hajduk and Paikowsky 2000).

<table>
<thead>
<tr>
<th>Gage Type</th>
<th>Test Pile</th>
<th>Stage</th>
<th>APCB</th>
<th>ERGP</th>
<th>ERSG</th>
<th>TSPG</th>
<th>VWPG</th>
<th>VWSG</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Start of Testing</td>
<td>2</td>
<td>7</td>
<td>4</td>
<td>12</td>
<td>28</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of Testing</td>
<td>0</td>
<td>7</td>
<td>0</td>
<td>4</td>
<td>11</td>
<td>28</td>
</tr>
<tr>
<td>% Durable</td>
<td></td>
<td></td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>92</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Start of Testing</td>
<td>2</td>
<td>6</td>
<td>4</td>
<td>8</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of Testing</td>
<td>0</td>
<td>5</td>
<td>0</td>
<td>3</td>
<td>7</td>
<td>20</td>
</tr>
<tr>
<td>% Durable</td>
<td></td>
<td></td>
<td>83</td>
<td>0</td>
<td>75</td>
<td>88</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Start of Testing</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>End of Testing</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>% Durable</td>
<td></td>
<td></td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

Evaluation of the design and construction of the test pile cluster lead to the following conclusions:

1. Careful selection of gages requires: (i) a determination of the magnitude of the parameters to be measured, (ii) an evaluation of the measurement frequency needed to capture accurately the variations in the measured parameter, and (iii) the ability of the instrument to record accurately the variation of the measured value with time during rapid changes (e.g., driving), and long-term monitoring.

2. Due to the robustness of the vibrating wire gages and the fast response of the electrical gages, both transducer types were utilized, resulting in two data acquisition systems and providing redundancy in the monitoring.

3. Special thin film liquid based cells were built into the pile’s wall to monitor radial total soil pressures. Such devices are temperature sensitive, and careful pressure and temperature calibration is required in order to enable accurate interpretation of their response. The calibration of all eight halves (four complete circumferential) Total Soil Pressure Gages (TSPG) showed that (i) the gages were significantly more sensitive to temperature than pressure and (ii) each gage is unique and, therefore, requires individual temperature and pressure calibrations.

4. The installation of multiple instrumentation into a long element and a confined space demanded careful construction planning and highly qualified execution. The piles were cut to sections and prepared for assembly, then shipped for gages installation. The sections were reconstructed to become a whole pile (TP#2) or two segments that were connected to become one pile on site (TP#1). Special cooling systems and numerous other technical details were required for the gage installation and pile assembly to take place without damaging the instruments and cables.

5. The various instruments required a large number of special features to ensure the instrument’s performance and to accommodate its installation. Such features included mounting blocks, deairing vents, caps to maintain saturation of piezometers, supporting strips for tell-tales and cables, tell-tales assemblies, and numerous similar technical details to guarantee desired performance.

6. Although the test piles were primarily designed for the use at the Newbury Site, the alternating layout of strain gages and piezometers along the length of the three piles allows for reuse of the instrumented test piles in future testing programs.

Acknowledgments

The instrumented test piles are a part of a long-term research project supported by the Massachusetts Highway Department (MHD) and assisted by the Federal Highway Administration (FHWA). The authors would like to acknowledge specifically the assistance of Mr. Nabil Hourani and Mr. John Pettis of the MHD and Mr. Al DiMilio and Mr. Carl Ealy of the FHWA. The instrumentation design team for TP#1 and TP#2 consisted of Shannon and Wilson, Inc., of Seattle, Washington; Geokon Incorporated of Lebanon, New Hampshire; and the Geotechnical Engineering Research Laboratory of the University of Massachusetts—Lowell. The TSPG was designed by Alex Feldman of Shannon and Wilson, Inc., and John McRae of Geokon, Inc. Dr. Samuel Paikowsky and Edward Hajduk of the University of Massachusetts—Lowell, performed selection and design of instrumentation and installation arrangements for TP#3. TP#3 was cast at Northeast Concrete Products Incorporated, located in Plainville, Massachusetts.

Also acknowledged are the support and cooperation of all the individuals and companies associated with this research project, in particular:

Pile Driving Contractor: P.A. Frisco, Inc., Ms. Paula Frisco, President
Geotechnical Design Firm: Shannon and Wilson, Inc., Mr. Gerard Buechel, Senior Associate; Mr. Alex Feldman, Senior Engineer
Instrumentation Firm: Geokon, Inc., Mr. John McRae; Mr. Ken Alger, Machinist
Steel Pile Fabricator: H&H Engineering, Mr. Mike Helfrich, President
Concrete Pile Donator: AM&FC, Mr. Charlie Guild.
UML Geotechnical Engineering Research Lab: Gary Howe, Director of Laboratories, Mr. Leo Hart, Mr. Chris Palmer, Mr. John Chen, Mr. Steve Tien, and Mrs. Mary Canniff

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