Augered Cast-in-Place Pile Foundation Design and Construction for the MLK Bridge, New Stadium Project, Atlanta

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ABSTRACT

Augered Cast-in-Place (ACIP) piles were installed for an elevated roadway in the City of Atlanta, as part of the infrastructure improvements for a new stadium project. The design-build project team opted to support the continuous span bridge on groups of 610 mm diameter ACIP piles, principally to avoid noise and vibration issues in the downtown environment. The ground conditions comprised a sequence of fill, Piedmont residual soil and partially weathered rock (PWR), overlying Precambrian metamorphic rocks. Results of two instrumented static load tests on non-production piles demonstrate the axial loads are transferred principally in shaft resistance. Mobilized unit side resistance values of 73 kN/m^2 in the Piedmont residual soil and 199 kN/m^2 in the PWR were derived from strain gauge arrays. For production piling, confirmation of the embedment lengths and grout volumes were key quality control parameters, verified during construction with Automated Monitoring Equipment (AME).

INTRODUCTION

This paper presents details of the use of augered cast-in-place (ACIP) piles for a new elevated roadway in the City of Atlanta. The new bridge supports MLK Jr. Drive over Mangum Street on the south side of the new stadium (Figure 1). The infrastructure was required to improve the east-west commute routes.

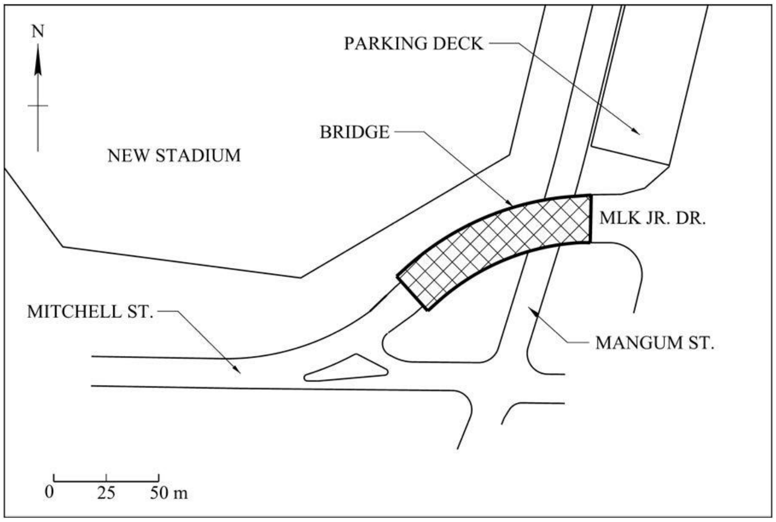

Figure 1. Location plan.

Although ACIP piles are a common deep foundation type for buildings in the south eastern USA (e.g. Hebner 2003; NeSmith 2003), their use as foundations for bridges is rare, despite FHWA guidance published a decade ago (Brown *et al*. 2007), seminal work on this pile type published over twenty years ago (Fleming 1995, DFI 1990) and the original patent and subsequent licenses for this pile type being granted in the United States in the late 1950's. Local practice on highway bridges is to use driven H piles and drilled shafts. ACIP piles were selected here primarily to avoid noise and vibration issues and to achieve some economy, as the pile type was also being used for the adjacent parking deck. The results of two instrumented static load tests on non-production ACIP piles at the bridge site described in this paper provide insights into mobilized unit side resistance values for ACIP piles in Piedmont residual soil and partially weathered rock (PWR).

BRIDGE STRUCTURE

The curved bridge comprises seven steel girders in continuous span across three bents (Figure 2). The width of the bridge is 22.27 m (outside edge to outside edge). The total length of the bridge along the profile grade line is 94.72 m. The span from Bent 1 to Bent 2 is 46.86 m in length and the span from Bent 2 to Bent 3 is 44.38 m. From the center of the Bent 3 cap, the girders cantilever 3.48 m to join with the existing elevated roadway structure. The minimum vertical clearance over Mangum Street is 5.64 m. Bent 1 is supported on a row of seven ACIP piles joined in a bent cap on which the girders are seated, and the piles are encapsulated by a mechanically stabilized earth (MSE) wall. At Bent 2, two reinforced concrete piers are each founded on a group of nine ACIP piles in a 3 x 3 rectangular arrangement, spaced at 3 and 4 pile diameters. The two reinforced concrete piers at Bent 3 are each supported on a group of six ACIP piles, in a 3 x 2 arrangement spaced at 4 and 4.25 pile diameters. The cast in place concrete deck carries two lanes in each direction and two sidewalks. A new storm water cistern and pump house was constructed beneath the bridge. This required an excavation of about 6 m below the existing grade of Mangum Street, that was supported by a soldier pile and lagging wall with tieback anchors.

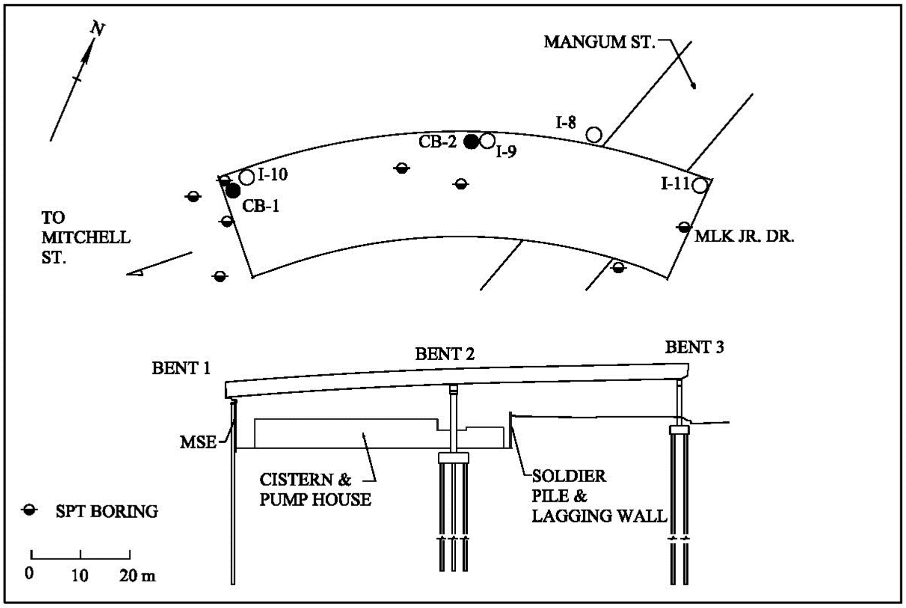

Figure 2. Bridge plan and profile, with locations of borings, test piles and indicator piles.

GROUND PROFILE

The ground profile is a sequence of fill, residual soil and PWR overlying Precambrian metamorphic rocks, typical of Atlanta. The residual soil consists of silty sands, sandy silts and clayey silts with mica. With increased depth the residual soils retain relict structures from the parent materials in an intermediate zone or 'saprolite', that grades into PWR. The contacts are gradational and highly variable, typical of the Piedmont Physiographic Complex (see Sowers and Richardson 1983). Eight standard penetration test (SPT) borings were drilled at the bridge site (Figure 2). A synthesis of the SPT N-values from all borings exhibited wide scatter, typical of Piedmont soils. Design of the foundations was therefore done using borings on a bent specific basis. Defining PWR as material with an N-value greater than 100 (see Sowers and Richardson 1983), the top surface of the PWR varied by about 13 m across the bridge from Bent 1 to Bent 3. The depth to sound competent rock was highly variable. It was encountered at Bent 1, but not at Bent 2 or Bent 3 within the depth of the borings drilled at these respective locations. Hard zones in the PWR were found, that caused auger refusal and a switch to coring, but the core recoveries in these zones were zero. The SPT N-values corresponding to the depths below the top of the two test piles are presented in Figure 3, for borings in the vicinity of Bent 1 (4 borings near test pile CB-1) and at Bent 2 (2 borings near test pile CB-2). Groundwater was encountered in the residual soil at 3.9 m below the top of test pile CB-1 and at 5.2 m below the top of test pile CB-2.

Figure 3a shows that at CB-1, the top of PWR was encountered at a depth of approximately 15 m below the top of test pile, with a distinctive break to higher N-values. The ground profile at CB-2 was far more variable. Although hard zones were encountered within the borings at around 13 m below the top of the pile, with SPT N-values greater than 100, the top of the PWR was not well defined and competent rock was not encountered despite the test pile being 33.5 m in length. N-values greater than 100 are plotted on Figure 3 at a value of 100 for simplicity.

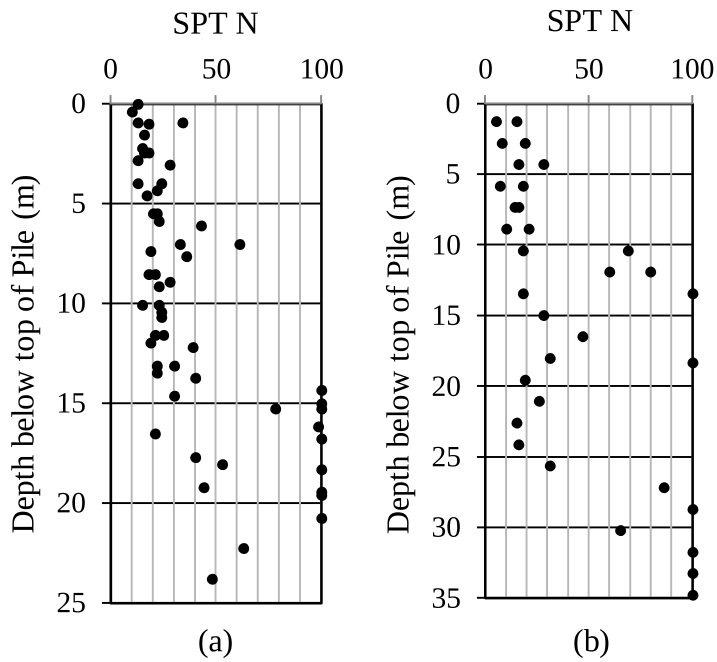

Figure 3. SPT N-value vs. depth below top of test pile: (a) CB-1 and b) CB-2.

TEST PILE INSTALLATION

To justify use of ACIP piles for the bridge on this project, satisfactory load deflection performance had to be demonstrated on two instrumented non-production test piles. To demonstrate constructability in the variable ground profile, four non-production 'indicator piles' were also required (see I-8 through I-11 on Figure 2). Two installation criteria were developed. Criterion 1 was defined as installation of the pile through residual soil and PWR, with refusal on competent rock. Criterion 2 was defined as embedment of the pile in PWR only, without bearing on competent rock. All piles (including production piles) were installed with measurement of drilling parameters using Automated Monitoring Equipment (AME). The AME logs provide near continuous records of penetration rate, rotation speed, hydraulic fluid pressure on the turntable driving the auger rotation (known as KDK pressure), auger withdrawal rate, and grout flow rate. Select output parameters from the AME records for the test piles are presented in Figures 4 and 5. Figure 4 shows that the penetration rate of the auger reduces below a depth of 15 m in test pile CB-1, which corresponds to the top of the PWR. The KDK pressure increases in the PWR. The penetration rate reduces significantly at the termination depth, indicative of refusal on competent rock (Criterion 1). In contrast, Figure 5 shows a steady KDK pressure through most of the drilling at test pile CB-2. The penetration rate declines steadily without a sharp break, consistent with refusal Criterion 2. A summary of the installation parameters for the two test piles and four indicator piles is presented in Table 1.

Table 1. Summary of test pile and indicator pile construction.

Pile ID	Drilling Time (mins)	Pile Length (m)	Refusal Criterion	Grout Factor (%)	Grouting Time (mins)
CB-1	8	22.4	1	126	7
CB-2	12	33.5	2	123	10
I-8	18	45.7	2	122	15
I-9	18	45.0	2	122	12
I-10	17	33.7	1	122	11
I-11	26	45.8	2	129	13

Figures 4 and 5 also show the grout flow rates. From these data, incremental grout factors were computed for each 1.5 m depth in the piles. The grout factor is the ratio of grout volume injected to the theoretical cylinder volume of the pile. Fleming (1995) recommended that a grout factor in the order of 120 percent be achieved, and this is supported by Brown *et al*. (2007). This overall grout factor was achieved in all the test piles and indicator piles (Table 1), although some incremental grout factors per 1.5 m depth interval were below 120 %. This prompted adjustments to the grouting procedures for the production piles. The grout used for the test piles was designed to achieve a minimum unconfined compressive strength of 45 N/mm^2 at 28 days and it included additives to enhance flow and reduce bleed.

STATIC LOAD TESTS

Test pile CB-1 was loaded in accordance with “Procedure A: Quick Test” per ASTM D1143 with a maximum test load of 8900 kN (Table 2 and Figure 6a). Given constraints with the adjacent soldier pile and lagging wall supporting Mangum Street, the reaction pile system at CB-2 could not achieve the desired maximum test load. Test pile CB-2 was therefore loaded to twice the design load and then held in accordance with “Procedure B: Maintained Test”. An unload and reload loop was then performed with a subsequent overload to a maximum test load of 8001 kN, in accordance with “Procedure C: Loading in Excess of Maintained Test” (Table 2 and Figure 6b). As well as verifying the axial load capacity of the tests piles, confirmation of the load transfer mechanism was achieved using arrays of sister bar strain gauges fixed to the center reinforcing bars at intervals throughout both test piles.

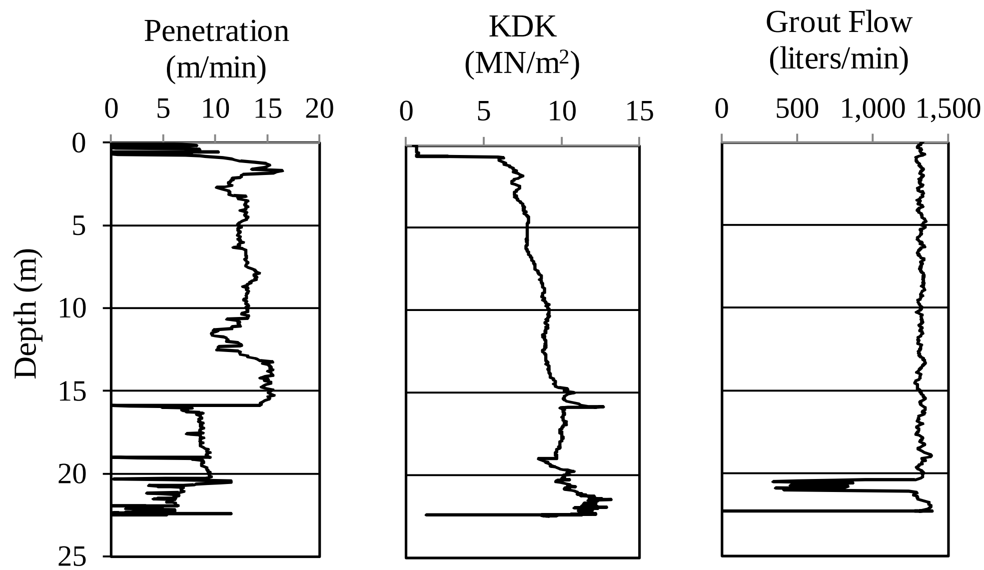

Figure 4. AME data for test pile CB-1 at Bent 1.

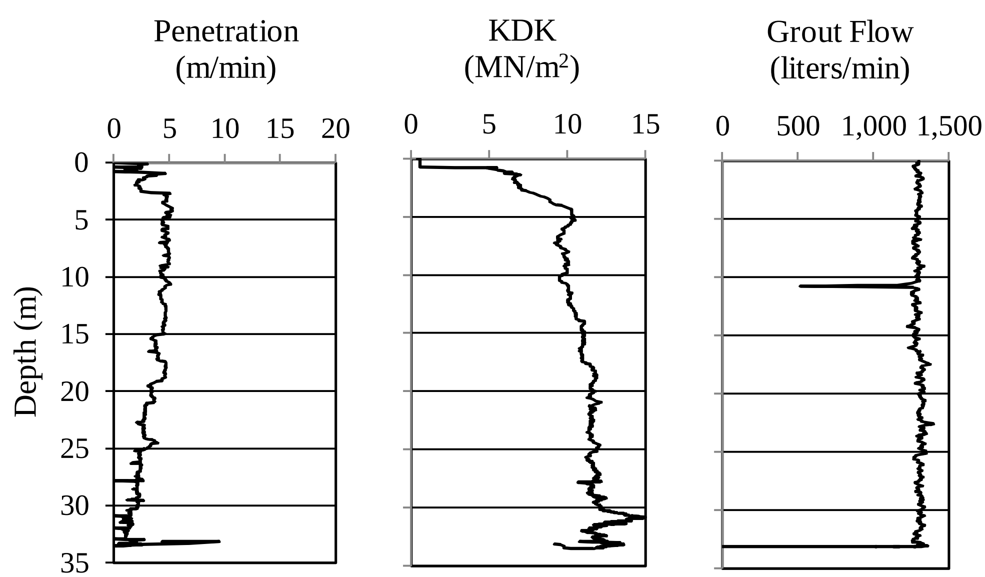

Figure 5. AME data for test pile CB-2 at Bent 2.

Table 2. Summary of pile head vertical defection

Pile ID	Design Load (kN)	Max. Test Load (kN)	Pile Head Deflection (mm) at Load DL	2DL	MTL
CB-1	3338	8900	4.5	11.9	18.2
CB-2	3338	8001	5.9	19.7	26.8

DL = design load; MTL = maximum test load.

The Geotechnical Ultimate Limit State (GULS) of an ACIP pile is typically defined as a pile head deflection equivalent to 5 % of the pile diameter (Brown et al. 2007). For 610 mm diameter ACIP piles, the GULS is therefore defined as a pile head deflection of 30.5 mm. Table 3 and Figure 6 show neither test pile reached this condition, although CB-2 approached it with a deflection of 4.4 % of the pile diameter. At the design load (DL), both test piles exhibited satisfactory behavior with less than 6 mm of pile head deflection. Reduction of the strain gauge data is presented in Figure 7. These data demonstrate that the load is carried principally on the pile shafts in side resistance, irrespective of the refusal criterion. At the maximum test load (Table 2) the proportion of load carried on the shafts, evaluated from the strain gauges, was 73 % and 90 % for test piles CB-1 and CB-2 respectively.

Given that test pile CB-2 approached a pile head deflection close to 5%, the strain gauge data from this pile are instructive, because it is reasonable to conclude that the shaft side resistance was approaching fully mobilized values within the shallower subsurface materials and mobilized values at deeper depths. Strain gauges SG1 through SG3 (Table 3) are located in a depth range commensurate with the residual soil. From Figure 7b, the proportion of the maximum test load carried over this depth interval (18.3 m) was 2.55 MN. This equates to a mobilized unit side resistance of 73 kN/m^2. Strain gauges SG4 through SG8 (Table 3) are located in a depth range commensurate with PWR. The proportion of the maximum test load carried on the shaft over this depth interval (12.2 m) was 4.65 MN. This equates to a mobilized unit side resistance value of 199 kN/m^2.

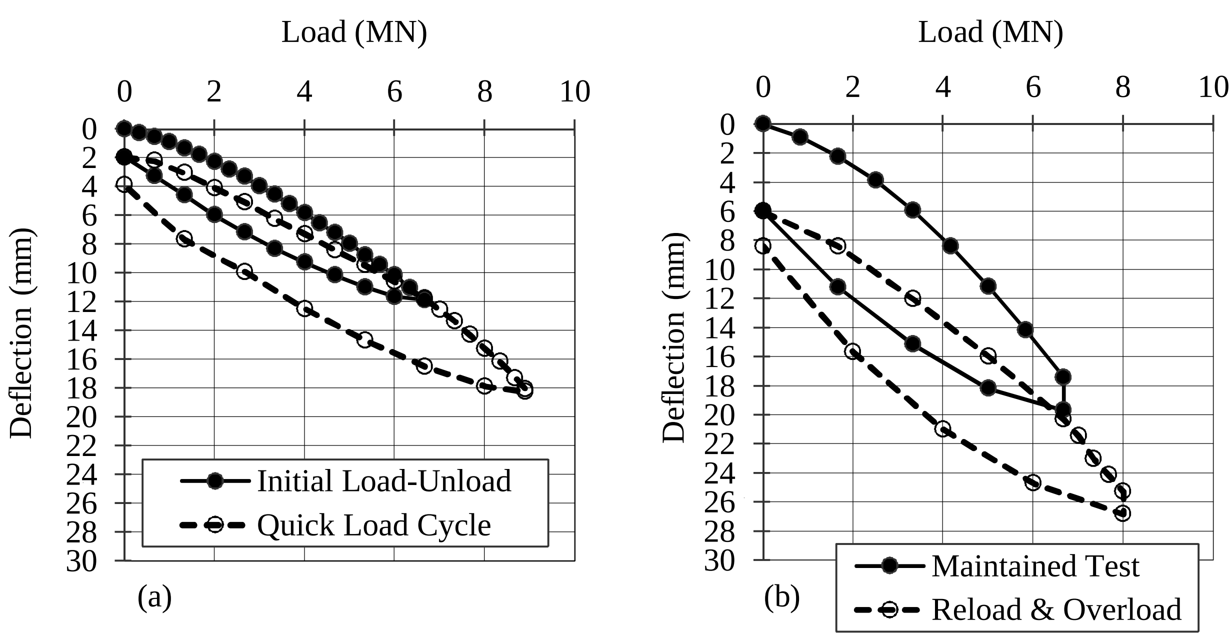

Figure 6. Load-deflection plot for compression load tests: (a) CB-1 and (b) CB-2.

The load from SG3 in pile CB-2 appears lower than it might actually be. The pile is transitioning from residual soil to PWR at this depth, with a possibility of a small change in diameter. A change in diameter will impact the magnitude of load derived from the strain gauge.

The mobilized unit side resistance values compare favorably with those found for conventional drilled shafts constructed in Piedmont soils under bentonite and polymer fluids (see Brown 2002). Mayne and Harris (1993) reported unit side resistance for a drilled shaft (test shaft C2) constructed at the Georgia Institute of Technology campus. At failure, the unit side resistance was 66 kN/m^2. Maximum unit side resistance values for another drilled shaft (test shaft C1) drilled at the campus did not reach failure, but a maximum unit side resistance value of 73 kN/m^2 was reported for the residual soil, and 234 kN/m^2 for the PWR which are similar to the values found for the ACIP test pile CB-2 at the bridge. Casing was not used for either of the test shafts C1 and C2. The campus site is about 1.6 miles north of the bridge.

Table 3. Depths of strain gauges below top of test pile.

Strain Gauge ID	CB-1 Depth below Top of Pile (m)	CB-2 Depth below Top of Pile (m)
SG1	1.52	1.52
SG2	10.06	9.14
SG3	15.24	18.29
SG4	19.35	21.34
SG5	20.88	22.86
SG6	22.40	28.96
SG7	--	32.00
SG8	--	33.53

FOUNDATION CONSTRUCTION

The results of the test program enabled production piling to begin, that followed industry practice (DFI 2003, 2010, Brown *et al*. 2007). AME records for production piles showed auger penetration rates in the PWR of approximately 1.5 to 3 m/min. The required minimum compressive strength of the grout was 41 N/mm^2. Reinforcement cages 10.5 m in length were lowered into the piles after grouting, and consisted of nine 25 mm diameter bars for longitudinal reinforcement plus a 35 mm diameter center bar inserted to full depth as a quality control measure. Shear steel comprised 16 mm diameter bars. Pile cap construction required thermal controls, with hoses routed through the pile cap steel to enable circulation of cooling water during concrete hydration and curing.

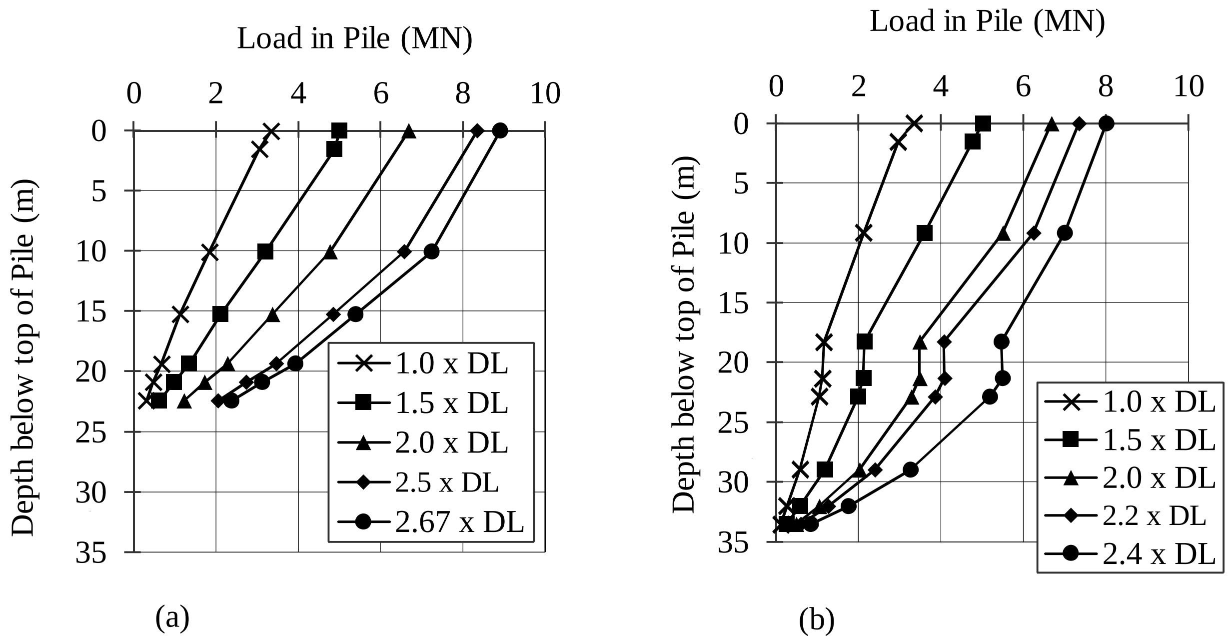

Figure 7. Load transfer from strain gauges in test piles: (a) CB-1 and (b) CB-2.

CONCLUSIONS

The data presented in this paper, together with the observations made during production piling leads to the following conclusions:

1. The ACIP piles in Piedmont residual soil and PWR are predominantly shaft controlled at the capacity and lengths tested for this project, with most of the load carried in side resistance.
2. Mobilized unit side resistance values in Piedmont residual soil and PWR were 73 kN/m^2 and 199 kN/m^2 respectively.
3. With careful application of AME during construction, coupled with static load testing, ACIP piles provide an alternative to conventional driven H-piles and drilled shafts for bridge foundations in the Piedmont geology. Where noise and vibration concerns occur in urban environments, ACIP piles offer benefits over driven piles, and their speed of construction provides advantages over drilled shafts.

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