Large-scale dynamic high-strain load testing of a bridge pier foundations

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ABSTRACT: The dramatic failure of a drilled shaft foundation resulted in the structural collapse of two bridge spans during the construction of a 5.2 km long viaduct in Central Florida, USA; shaking confidence in the 204 bridge pier support shafts already installed before the failure. A team of engineers conducting an independent forensic and remedial engineering study dynamically tested twelve completed bridge piers representing the varied geotechnical conditions along the bridge alignment. The 5 to 18 m tall structural piers were each supported by a single, 1.8 m diameter drilled shaft ranging in length between 13 and 30 m. The team performed high-strain dynamic load tests on top of the completed piers using a hydraulic hammer system with a ram weight of 534 kN and drop heights up to 1.9 m. Dynamic test instrumentation consisted of eight strain transducers and eight accelerometers, four each affixed near the pier bottom and the shaft top. Field data acquisition and initial processing were performed by means of two Pile Driving Analyzer® (PDA) systems, one each for the pier and shaft instrumentation. Pier movement under each hammer blow was independently measured. The CAPWAP® program was then used to evaluate the shaft load bearing capacity and static load-movement relationship. This paper presents the results of two of the pier-shaft tests as an illustration of the large-scale testing procedure utilized for this project. It describes the field testing and data analysis procedures, and the evaluation of data quality and consistency of results.

1 INTRODUCTION

The phenomenal urban growth of the West-Central Florida metropolitan area resulted in heavy annual commuter traffic of more than 30 million vehicles along the Lee Roy Selmon Crosstown Expressway between downtown Tampa and the eastern suburb of Brandon. In order to help relieve traffic congestion, the Tampa-Hillsborough County Expressway Authority (THCEA) developed plans for building a 14 km long state-of-the-art reversible three-lane expressway, at an initial estimated cost of approximately US$300 million. The project included three post-tensioned concrete box segmental bridges with a total length of 8.3 km, including a 5.2-kilometer-long viaduct. Due to the very limited right-of-way along the project length, it was necessary to construct many of the elevated structures overhanging the existing in-service highway upon slender piers that fit within the narrow median. Fig. 1 presents a general view of a section of the project showing a portion of the viaduct and support piers. The engineering challenges of an innovative superstructure, non-redundant foundation design, constrained construction area, varied geotechnical conditions, drilled shaft failures, and associated remedial work made this project unique in many respects.

The rectangular concrete bridge piers measure 1.5 by 1.8 m in cross-section at their base, and range in height between approximately 5 and 18 m. The foundation supporting each pier consisted of a single drilled shaft, 1.8 or 2.4 m in diameter ranging between approximately 13 and 30 m in length. The drilled shafts were constructed using conventional methods utilizing temporary steel casing and full-length steel cage reinforcement. Each non-redundant shaft foundation was designed for Pier design compression loads ranging from 7 to 24 MN. The subsurface conditions generally consisted of an overburden of sands, silts and clays over weathered limestone, with the intended bearing layer being the Florida Formation Limestone located at depths varying from 5 to 28 m below grade.

In April 2004, the dramatic failure of one of the foundation shafts (Brennan, 2004) and the resulting localized bridge collapse, combined with unacceptable foundation settlement observed at other locations, instigated a comprehensive examination of the entire project. As part of the review and remedial engineering investigations, twelve representative piers were subjected to high-strain dynamic load testing (DLT).

Figure 1. General view of project.

These locations were selected considering a number of factors to assess aspects of the foundation design assumptions. Typically, the tests included 3 to 6 individual impacts from a hydraulic hammer (534 kN ram and drop heights of up to 1.9 m), dynamic measurements of force and velocity utilizing eight strain transducers and eight accelerometers (four of each affixed to the bottom of the pier and the excavated top of the supporting shaft), and the independent measurement of movement for each blow. This paper presents the results of testing and data analyses of two shafts. The testing procedures were similar in both cases, but with markedly different results. High-strain dynamic load testing and related CAPWAP analyses were important tools in evaluating the in-place foundation conditions, developing remedial solutions, and restoring confidence in this important toll-road facility.

2 DYNAMIC LOAD TESTING

Likins et al. (2000) estimated that high-strain dynamic load pile and shaft testing is used at several thousand sites annually worldwide. The engineering literature contains numerous references to the use of dynamic testing and related data analyses methods of cast-in-place shaft foundations. For example, the proceedings of this and previous conferences on The Application of Stresswave Theory to Piles contain many papers on the topic.

Dynamic high-strain load tests measure and record the shaft-top strain and acceleration caused by each impact of a relatively large falling mass (i.e., hammer ram), along with the independent measurement of the permanent shaft-top penetration (set). The Pile Driving Analyzer (PDA) field instrumentation system and CAPWAP computer analysis program from Pile Dynamics, Inc. are commonly used for data acquisition and numerical analyses. Hussein et al. (1996) provide recommendations for sizing the loading system, i.e., hammer weight, drop height, and shaft top cushion. A commonly used Rule-of-Thumb suggests that a drop hammer allows the mobilization of capacities 50 to 100 times its weight. Tests are performed after the shaft concrete attains adequate strength to safely resist the applied dynamic stresses, and transmit sufficient load to overcome soil resistance forces. Typically, three to five hammer impacts are applied for each test.

The special circumstances of this project included large-scale dynamic load testing of drilled shafts under existing bridge piers, relatively high capacities, and a heavy loading system. A total of twelve 1.8-meter diameter shaft/piers were tested, ranging in lengths between 13 and 30 m, supporting piers ranging in height between 5 and 13 m. The loading system was an American Piledriving Equipment APE-750U hydraulic hammer (Heller, 2004). It has a 534 kN (60 tons) ram and a maximum drop height 1.9 m (6.25 ft). Based on the above rule-of-thumb, the hammer would be expected to mobilize capacities up to between 27 and 54 MN. Fig. 2 shows a typical view of the hammer on top of a pier.

Engineers used the GRLWEAP wave equation program to evaluate the dynamic compatibility of the hammer-pier-shaft system, design a hammer cushion, striker plate, and pier-top cushion to control the tension and compression stresses induced in the pier and its shaft foundation. The hammer cushion consisted of a nylon disk with a thickness of 150 mm and a diameter of 1.14 m. It rested on a 400 kN steel striker plate, 910 mm m

Figure 2. View of hammer on pier.

thick and a diameter of 2.67 m. The plywood cushion used to protect the pier top had a thickness of 150 mm with an area of 2.09 m^2 sized to fit just inside the pier's steel reinforcement. A special pier-top straddle was constructed using steel I-beams that allowed the safe use of the hammer without leads for testing. The following discussion focuses on the results of two particular shaft/piers, designated here as Shaft A and Shaft B.

The shafts had nominal diameters of 1.83 m, and the piers had a nearly rectangular cross-sectional size of 1.52 by 1.83 m. An approximately 2 by 2 m square cap with a thickness of 0.6 m was cast between the top of each shaft and bottom of pier at the ground surface. An approximately 2 m deep excavation was dug around each shaft to provide access for visual inspection and instrumentation. Dynamic testing instrumentation consisted of eight strain transducers and eight accelerometers, four each affixed to the four sides of the pier at 1.5 m above the top of the cap, and four each affixed to equidistant locations around the circumference of the shaft top 1.5 m below the bottom of the cap. The pier and shaft instrumentation are shown in Figs. 3 and 4, respectively. Two PDA units were employed, one each for the pier and shaft gages. Dynamic stresses in the pier were monitored closely during the tests to check bending stresses and preserve the integrity of the pier concrete. Displacement following each hammer impact was measured near the bottom of the pier by using surveyors' instruments aiming at two faces of the pier, a wireline with mirror and scale, and a simple laser-and-target system. The pier response was also recorded by video camera.

3 DISCUSSION OF TEST RESULTS

Shaft A had a length of 22.5 m and supported a 6 m high pier. Measurement of shaft circumference at the gages location, i.e., 1.5 m below shaft top at bottom of cap, indicated a cross-sectional area of 2.85 m^2 (i.e., 1.9 m diameter). Shaft B was 18.8 m long supporting an 11 m high pier. Measurement of shaft circumference at the gages location, i.e., 1.5 m

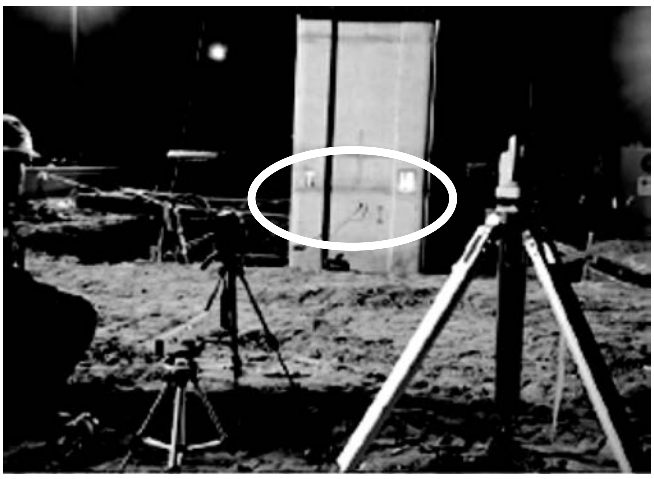

Figure 3. Pier instrumentation.

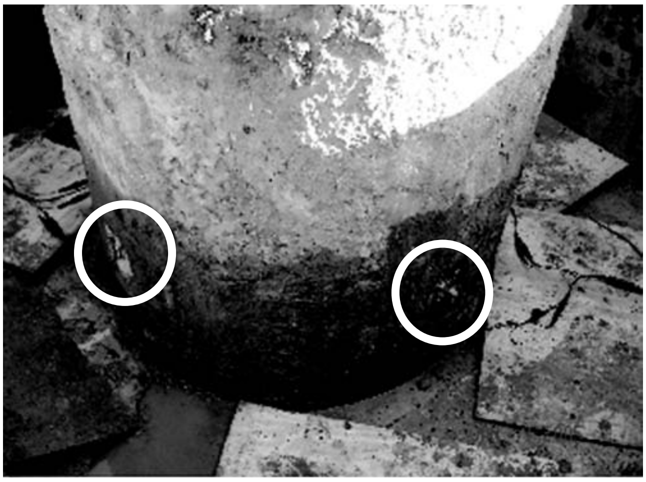

Figure 4. Shaft instrumentation.

below shaft top, indicated a cross-sectional area of 2.86 m^2 (i.e., 1.9 m diameter). In both cases, the cross-sectional area at the pier gage locations was 2.62 m^2, 9% smaller than the shaft top but different in shape. A stresswave speed value of 3800 m/s, with corresponding elastic modulus of 32 GPa, and a material unit weight of 23.6 kN/m^3 were used in the processing and analysis of the dynamic test data.

Dynamic load tests of Shafts A and B were performed in the same manner, each test consisting of three hammer impacts with drop heights of 0.6, 1.2 and 1.9 m. Fig. 5 presents plots of the averaged force and velocity test records from the Shaft A test blows for both the pier and the shaft transducers. Similarly, Fig. 6 presents plots of the test records from the pier and shaft for the Shaft B test blows. The characteristics of the test records indicate that the shaft records have better quality data than the pier records as far as evaluation of shaft load bearing capacity is concerned. The apparent lack of proportionality, and slight time shift, between force and velocity

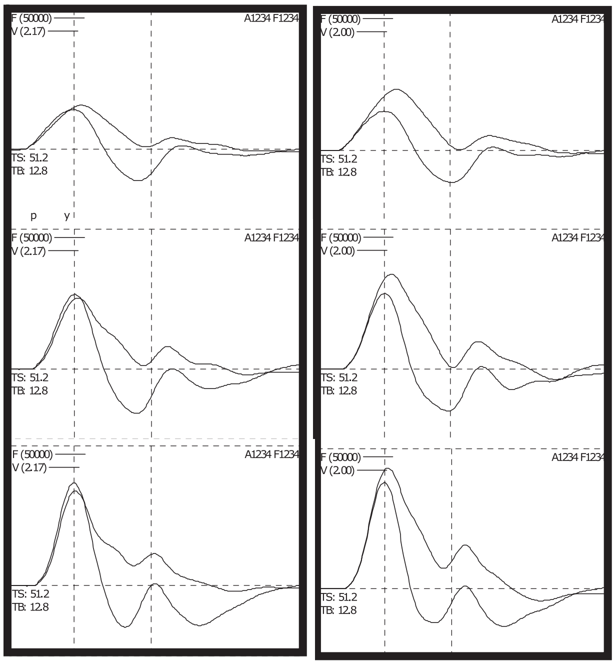

Figure 5. Plots of test records for Pier (left) and Shaft (right) A.

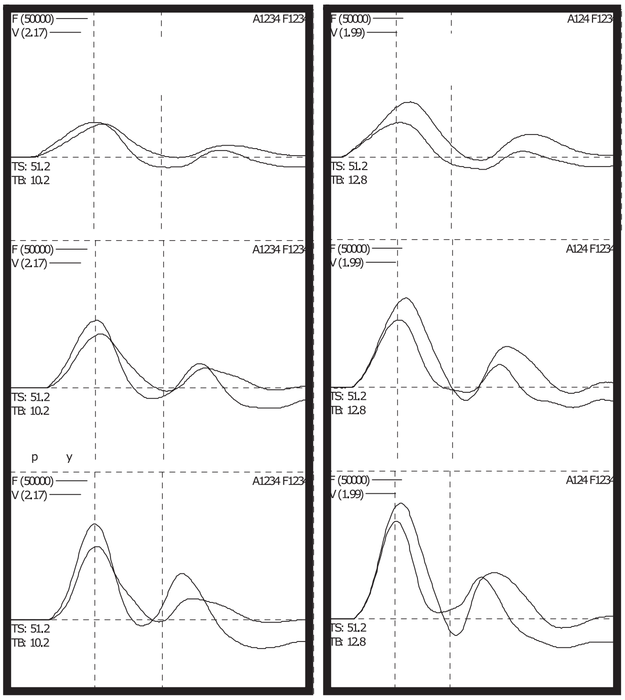

Figure 6. Plots of test records for Pier (left) and Shaft (right) B.

pier data could be the result of the cap non-uniformity present between the pier bottom and the shaft top just below the pier gages location. The shaft-top test records exhibit excellent data quality, perhaps in part due to the full pier-height distance between the hammer impact and the shaft gages locations which tends to result in uniform strain and motion distributions in the shaft cross-sectional area at the measuring location. This paper focuses on the test results from the shafts data.

Figs. 7 and 8 present plots of force, velocity, energy and displacement records obtained under each of the three test hammer blows from the shaft-top measurements for Shafts A and B, respectively. The dynamic load testing procedures for Shafts A and B were similar to each other. Tables 1 and 2 present a summary of the testing results.

Permanent set was taken as the averaged value of the various reported readings. The measured set values under hammer Blows 1, 2, and 3 respectively were 0.6, 1.0, and 1.5 mm for Shaft A and 1.8, 3.3 and 4.2 mm for Shaft B. Shaft B experienced permanent sets approximately three times greater than Shaft A under similar hammer impacts.

A comparison of the test records for the two shafts indicates comparable results for the three similar drop height hammer test blows. The maximum impact force (FMX) generally ranged between 20 and 40 MN, with Shaft A values less than 10% higher than Shaft B. The corresponding maximum impact stresses ranged between approximately 7 and 15 MPa. Shaft-top maximum loading velocities ranged between 0.5 and 1.5 m/s under the three hammer impacts. Maximum shaft-top transferred energies (EMX) under hammer Blows 1, 2, and 3 for Shaft A were 72, 182, and 301 kN-m; and for Shaft B they were 65, 180, and 307 kN-m, respectively. Maximum shaft-top

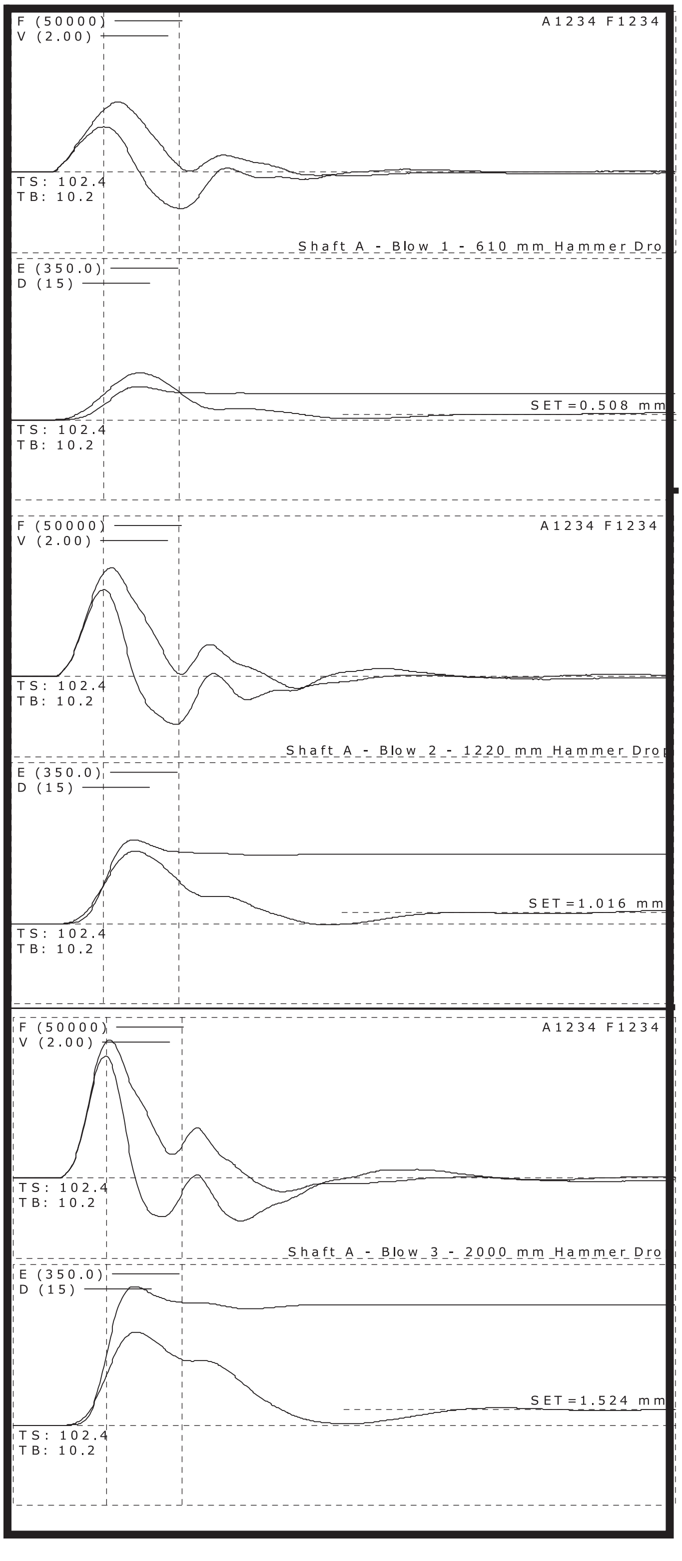

Figure 7. Plots of Shaft A test records for the three blows.

displacements (DMX) under hammer Blows 1, 2, and 3 for Shaft A were 4, 7, and 9 mm; and for Shaft B they were 5, 8, and 13 mm, respectively.

For load bearing capacity evaluations, dynamic shaft-top PDA data obtained under the third hammer blow of each test were analyzed with CAPWAP. Figs. 9 and 10 present plots of the data from Blow 3 for the Shaft A and Shaft B tests, respectively. Each figure includes: individual forces and velocities from each of the four strain transducers and accelerometers, averaged force, velocity, displacement, transferred energy, wave-down, and wave-up records. Force and velocity test records

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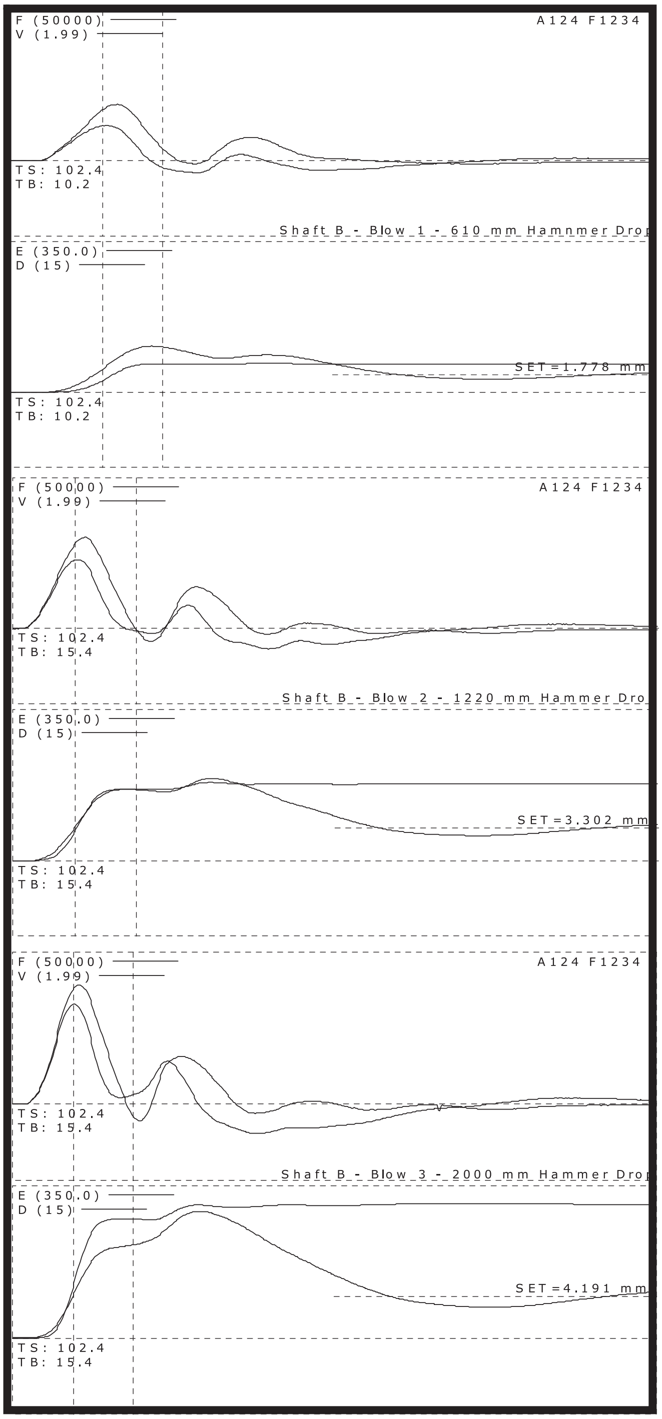

Figure 8. Plots of Shaft B test records for the three hammer test blows.

Table 1. Dynamic test summary – Shaft A

Blow no.	1	2	3
Hammer Drop, m	0.6	1.2	1.9
Set, mm	0.6	1.0	1.5
FMX, MN	21.8	33.9	42.8
CSX, MPA	7.7	11.9	15.1
VMX, m/s	0.6	1.1	1.5
EMX, kN-m	72	182	301
DMX, mm	4	7	9
R, MN	19.3	28.9	34.7

obtained with the various gages indicate consistent data with good quality, except for one of the velocity records from Shaft B. As shown in Fig. 10, one accelerometer signal was unusual and therefore was

Table 2. Dynamic test summary – Shaft B

Blow no.	1	2	3
Hammer Drop, m	0.6	1.2	1.9
Set, mm	1.8	3.3	4.2
FMX, MN	18.7	30.2	39.1
CSX, MPA	6.6	10.6	13.7
VMX, m/s	0.5	0.9	1.3
EMX, kN-m	65	180	307
DMX, mm	5	8	13
R, MN	14.5	18.7	21.1

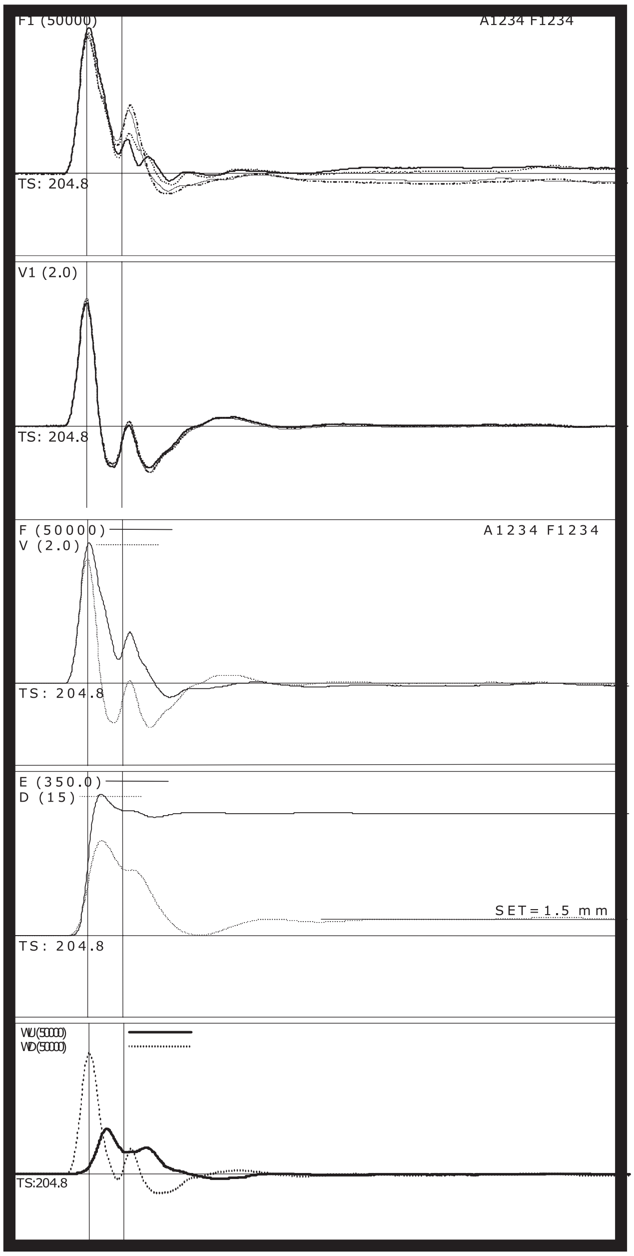

Figure 9. PDA data under blow 3 – Shaft A.

excluded from record averaging. Table 3 summarizes CAPWAP analyses results.

Comparison of results for Blow 3 (drop height of 1.9 m) indicate that the Shaft A data shows 10% higher maximum impact force, 15% higher maximum velocity, 2% lower maximum transferred energy, 31% lower maximum displacement, and 64% lower permanent set than Shaft B.

Modelling of the shafts for CAPWAP analysis utilized the measured area at the PDA gage location for 1.8 m of shaft length below the gages, and then the

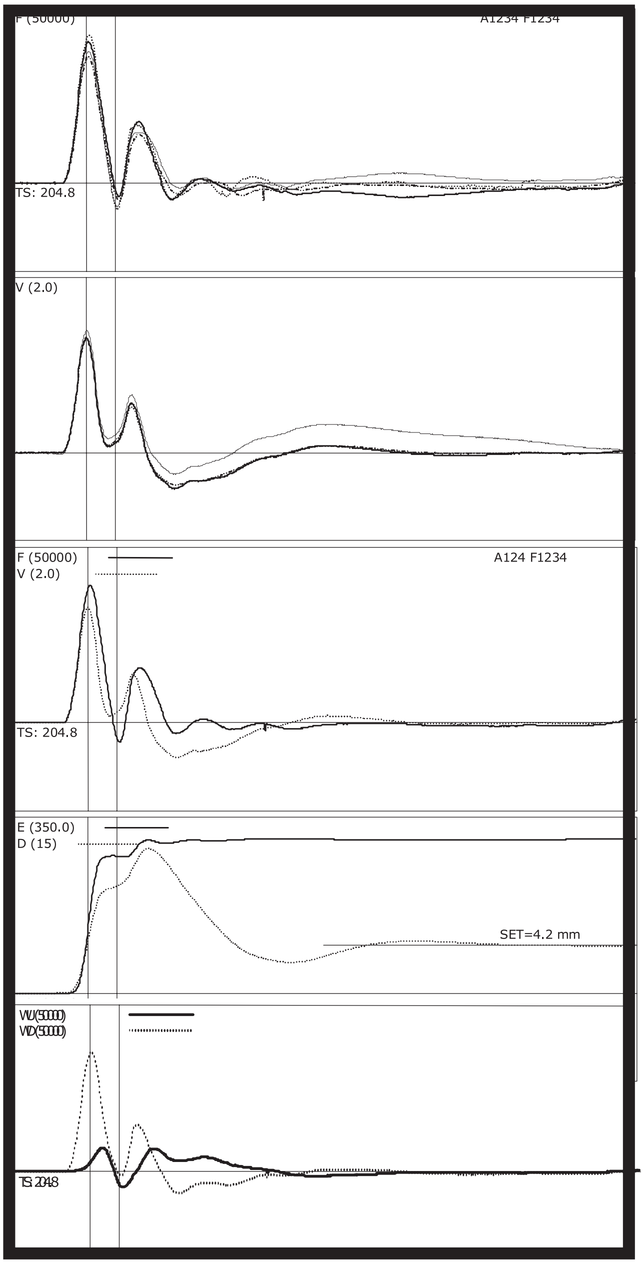

Figure 10. PDA data under blow 3 – Shaft B.

Table 3. CAPWAP analyses results, blow 3

		Shaft A	Shaft B
Capacity- total	MN	35	21
- shaft	MN	19	16
- toe	MN	16	5
Damping –shaft	s/m	1.9	0.73
-toe	s/m	1.0	0.76
Quakes - shaft	mm	1.8	2.3
-toe	mm	4.6	10.9

area corresponding to the nominal shaft design diameter for the rest of the shaft length. During analyses, the shaft impedance values were adjusted as part of the signal-matching process.

Figs. 11 and 12 present the CAPWAP analyses results showing the shaft impedance profiles. The CAPWAP analyses indicate capacity values of 35 and 21 MN for Shaft A and Shaft B, respectively. Shaft A has a load bearing capacity 64% higher than that of Shaft B. The side shear values were 19 and 16 MN, and end bearing values were 16 and 5 MN

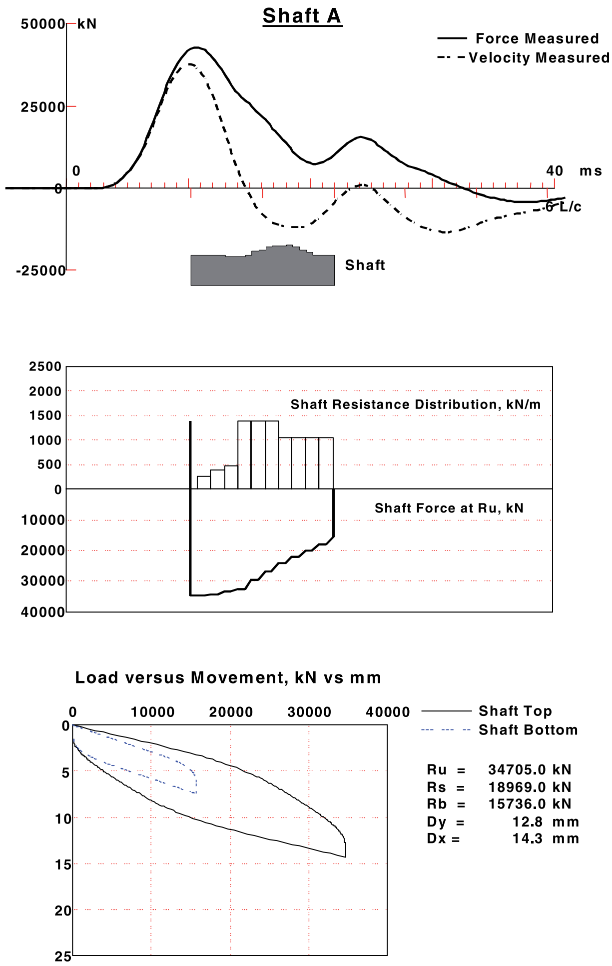

Figure 11. CAPWAP results – Shaft A.

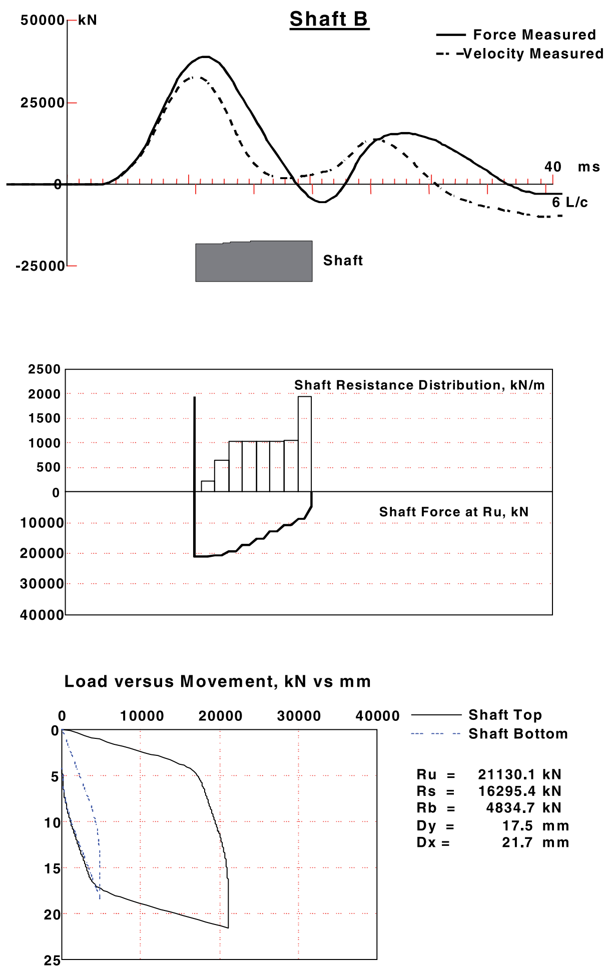

Figure 12. CAPWAP results – Shaft B.

for Shafts A and B, respectively. While the averaged unit side shear values were somewhat similar (156 kPa for Shaft A and 165 kPa for Shaft A), the unit end bearing values were very different from each other (6.0 MPa for Shaft A and 1.8 MPa for Shaft B).

Smith skin damping factors were 1.87 and 0.73 s/m, and toe damping factors were 1.02 and 0.76 s/m for Shaft A and Shaft B, respectively. Skin quake values were somewhat similar (1.8 mm for Shaft A and 2.3 mm for Shaft B), while toe quakes were very different (4.6 mm for Shaft A and 10.9 mm for Shaft B). The characteristics of the dynamic test records and the data analyses indicate that the major difference in load bearing capacities of the two shafts results from different end bearing behavior. Figs. 11 and 12 also show the expected static load versus movement relationships at the shaft top and bottom, calculated from the elastic properties, quakes, and static capacity. Shaft A exhibits load-movement behavior similar to Shaft B up to a load of approximately 17 MN (i.e., approximately the shaft resistance value), but much stronger and stiffer over all and end-bearing responses.

4 REMEDIATION AND PROJECT CONCLUSION

The results of the overall foundation study required remediation of 154 of the 218 pier shafts (Anderson and McGillivray, 2006). The foundations were expanded to include two additional 1.2 meter-diameter shafts at 67 piers and up to ten 250-mm diameter micro-piles at the 87 piers locations. Following a year of remedial work (Powers, 2005), the project opened to the traveling public in July 2006 (Florida Transportation Monthly, 2006) without additional incident and has received multiple awards including the 2007 Toll Excellence Award from the International Bridge, Tunnel and Turnpike Association.

5 SUMMARY

The expansion of the Lee Roy Selmon Crosstown Expressway in West-Central Florida, USA, necessitated the employment of innovative engineering and construction solutions for a multitude of challenging technical and site conditions. The US$ 300 million, 13 km long, project included three state-of-the-art bridges with a total length of 8.3 km, mostly carried on single piers elevated from the middle of the narrow median of the existing in-service highway. Most of the 218 piers for the 5.2 km main viaduct were supported by one drilled shaft, having a diameter of 1.8 m, and ranging in length between 13 and 30 m with the intended bearing layer of Florida Limestone located at depths varying from 5 to 28 m below grade.

The dramatic failure of one of the foundation shafts and associated localized structural collapse, combined with the unacceptable settlement of other shafts, required a comprehensive and critical examination of the entire project, ultimately resulting in an extensive remediation effort. Twelve representative piers were subjected to high-strain dynamic load testing. Testing was performed utilizing individual impacts of a large hydraulic hammer (534 kN ram with drop height 1.9 m), dynamic measurements of force and velocity utilizing eight strain transducers and eight accelerometers (four each on the pier and on the supporting shaft), and the independent measurement of movement under each blow. Using similar test procedures the two shafts discussed in this paper showed a markedly different response. Under a 1.9 m hammer drop height impact, Shaft A indicated a static load capacity of 35 MN with 1.5 mm set, while Shaft B had 4.2 mm set and a capacity of only 21 MN, the difference attributed mainly to lower end bearing. In summary, the 534 kN hammer activated as much capacity as could be expected, with safe dynamic stresses. High-strain dynamic load testing and related CAPWAP data analyses were important tools in evaluating the project foundations in-place conditions at the tested shafts, developing data for use in the design of remedial solution, and restoring confidence in this very important toll-road facility.

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