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16. Abstract

This report presents the results of the Treasure Island Liquefaction Test (TILT), a joint project carried out by University of California, San Diego, and Brigham Young University. To improve our understanding of the lateral load behavior of deep foundations in liquefied soil, a series of full-scale lateral load tests were been performed at the National Geotechnical Experimentation Site (NGES) at Treasure Island in San Francisco, California. The ultimate goal of the (TILT) project was to develop lateral load-displacement relationships for a variety of individual piles and pile groups in liquefied sand under full-scale conditions. The tests were carried out using a high-speed hydraulic loading system after the sand surrounding the piles was liquefied using blasting techniques. This report presents back-calculated *p-y* curves for a single pipe piles, pile groups, and Cast-in-Steel Shell piles before and during liquefaction, as well as through dissipation of excess pore water pressures. In addition, recommendations with regard to design of deep foundations in liquefied soil are presented.

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Hayward Baker and Geotechnics America/Mustang Construction provided additional project funding as well as donated services for installation of ground improvement measures. Condon–Johnson & Associates and Foundation Constructors donated foundation installation services. Additional services were also donated by Subsurface Consultants and Kleinfelder & Associates. Their support is gratefully acknowledged.

We would like to thank Thomas Weaver and Dusty Lane, the graduate research assistants responsible for the full-scale testing. Travis Gerber, a graduate research assistant, has also contributed significantly to this work. Their contribution to the testing program and this report is greatly appreciated.

Finally, the project could never have taken place without the full cooperation of the United States Navy and the City of San Francisco. Their cooperation is greatly appreciated.

EXECUTIVE SUMMARY

This report presents the results of the Treasure Island Liquefaction Test (TILT), a joint project carried out by University of California, San Diego, and Brigham Young University. To improve our understanding of the lateral load behavior of deep foundations in liquefied soil, a series of full-scale lateral load tests were been performed at the National Geotechnical Experimentation Site (NGES) at Treasure Island in San Francisco, California. The ultimate goal of the TILT project was to develop lateral load-displacement relationships for a variety of individual piles and pile groups in liquefied sand under full-scale conditions. The tests were carried out using a high-speed hydraulic loading system after the sand surrounding the piles was liquefied using blasting techniques. This report presents back-calculated *p-y* curves for single piles, pile groups, and Cast-in-Steel Shell piles before and during liquefaction, as well as through dissipation of excess pore water pressures. Furthermore, the results of studies on the use of stone columns and synthetic earthquake drains as liquefaction mitigation are also presented.

The primary conclusions and recommendations that will have a significant impact on design practice are:

1. The shape of the back-calculated *p-y* curves for liquefied sand are significantly different than those currently recommended for design, using either the p-multiplier approach or the residual strength approach. Specifically, the *p-y* curves for liquefied sand obtained from this study show a concave up shape, rather than a concave down shape typical of clay and non-liquefied sand.

2. Alternative methods for developing *p-y* curves in liquefied sand (i.e. p-multipliers or residual strength curves) may result in computed displacements and bending moments that adequately capture the measured response over only a limited range of depth and load, and may result in significant error elsewhere. If these alternative *p-y* curves are used, the effect of their shape on the computed foundation and superstructure response should be considered for the anticipated loading conditions.

3. Based the limitations of the current p-y analysis procedures mentioned above, it is recommended that analysis procedures be developed that capture the dilational response of the liquefied soil that result in this concave up shape for the p-y curves.

4. Group effects appear to be relatively inconsequential for pile groups in liquefied sand. For both the 4-pile group and the 9-pile group, the load-displacement curves for the individual piles within a group were essentially the same. In addition, the load-displacement curve for the single pile was very similar to that for the piles in the 9-pile group.

5. For design purposes, p-multipliers for group effects can be taken as 1.0 for liquefied sand. However, as the average r_u decreases and the frictional resistance increases, group effects will also become important and will need to be accounted for with appropriate p-multipliers.

6. Installation of stone columns significantly limited the excess pore pressure increase resulting from controlled blasting and significantly increased the rate of excess pore pressure dissipation after blasting, thus effectively mitigating the liquefaction hazard. In addition, the stone columns increased the stiffness of the foundation system 2.5 to 3.5 times that of the system in the liquefied soil. Even in the pre-blast, non-liquefied testing, the stone columns increased the stiffness of the foundation system by approximately 25 to 45 percent.

7. Pushover analyses should only be used if kinematic loads are not expected to contribute significantly to the foundation and superstructure response. Furthermore, when performing a pushover analysis, pile response should be assessed under lateral loads applied before and after the onset of liquefaction.

8. If a dynamic/time history analysis which implements *p-y* curves to model the soil is required, the effect of dilational soil response on the shape of *p-y* curves may have a significant effect on the structural response. Current procedures only account for pore pressure changes due to free-field soil strains and neglect the effect of pore pressure changes resulting from interaction with the foundation.

The results obtained from the full-scale pile testing at Treasure Island compare favorably with centrifuge test results under similar conditions. However, the TILT results are only for sands of relative density of approximately 50 percent. Current research is underway to combine the TILT results with centrifuge test results and other field studies to develop comprehensive design recommendations for deep foundations in liquefied sand.

In summary, the TILT project has resulted in several direct benefits to the project sponsors for application in bridge design. The testing and subsequent analyses have greatly increased our understanding of soil-foundation-structure interaction in liquefied soil, resulting in specific recommendations that can be immediately implemented into design practice, as well as general conclusions and recommendations that will be useful in assessing our current design methodology. Combined with the results of ongoing research, these also can be developed into specific design recommendations. In addition, the general agreement between the TILT full-scale testing and the smaller scale centrifuge test results increases our confidence in the numerous centrifuge studies already completed elsewhere. Finally, the TILT project provides some of the first full-scale quantitative data on two different ground improvement techniques, providing insight into our design procedures and increasing our confidence level in the ability of these methods to mitigate liquefaction hazards.

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1 INTRODUCTION

The Treasure Island Liquefaction Test (TILT) is a research project that consisted of full-scale lateral load pile tests in non-liquefied and liquefied ground at Treasure Island in the San Francisco Bay. This project is a joint venture between the University of California, San Diego and Brigham Young University, in close interaction with research personnel from the California Department of Transportation (Caltrans) and several other Departments of Transportation. Participants in this pooled fund study include the states of Alaska, California, Missouri, New York, Oregon, Utah, and Washington. Several other organizations collaborated on the project. The University of Michigan carried out visual-cone testing with funding from the National Science Foundation. The United Stated Geological Survey conducted ground-penetrating radar testing at the site. Utah State University conducted shear wave velocity tests at the site also with funding from the National Science Foundation.

1.1 Background and Research Significance

The lateral load capacity of deep foundations is critically important in the design of bridge structures in seismically active regions; however, there is currently little information to guide engineers in the design of piles embedded in liquefiable soils. Typical design procedures for piles in liquefiable soils are conservative and assume that the liquefied soil will provide little or no resistance to lateral movement. However, recent laboratory studies suggest that while the resistance may be reduced from the non-liquefied state, it is not zero and may be substantial (e.g. Wilson et al., 2000; Dobry et al., 1996).

Ongoing centrifuge studies using small-scale models (e.g. Wilson et al., 2000; Dobry et al., 1996) are providing valuable insight on soil-pile interaction in liquefied soil. However, full-scale tests are necessary to verify/calibrate these models and provide ground truth information. To improve our understanding of the lateral load behavior of deep foundations in liquefied soil, we conducted a series of lateral load tests on a full-scale single pile, pile groups, and Cast-in-Steel-Shell (CISS) piles. The testing was conducted near the National Geotechnical Experimentation Site on Treasure Island in San Francisco Bay where liquefaction has been observed in past earthquakes. Both static and cyclic tests were conducted in non-liquefied soil, and then cyclic tests were conducted after a surface layer over 4 m thick was liquefied using controlled blasting techniques.

1.2 Project Objectives

The overall objective of this research is to develop recommendations for soil parameters to use in the design and analysis of deep foundations subjected to lateral loading in liquefied soil based on the results of the proposed full-scale testing, in conjunction with available results of related centrifuge tests. Specifically, our objectives are to:

1. Characterize the proposed site through an extensive field exploration program.

2. Determine optimum charge size, delay and layout, as well as instrumentation layout by conducting a pilot blast program at the site.

3. Determine the lateral load capacity and distribution of load in a full-scale pile, pile group, and CISS pile in non-liquefied sand under static and cyclic loading conditions.

4. Determine the lateral load capacity and distribution of load in a full-scale pile, pile group, and CISS pile in liquefied sand under cyclic loading conditions.

5. Back-calculate p-y curves to model the resistance provided by liquefied sand.

6. Develop appropriate p-multipliers to account for reduction in capacity due to group effects.

1.3 Overview of Testing

Eleven full-scale field tests were conducted at Treasure Island. The testing consisted of two pilot liquefaction tests, two single pile tests, two 4-pile group/0.6-m CISS pile tests, two nine-pile group/0.9-m CISS pile tests, an earthquake drain (E~Quake Drain) test, a pre-stressed concrete pile test, and a concrete filled fiber reinforced polymer (FRP) pile test. The pile load tests consisted of lateral loads being applied to the piles in the free-head condition (no rotational restraint). Down-hole explosives were used to induce liquefaction in all except for the pre-stressed and FRP pile tests. Prior to the second 4-pile group/0.6-m CISS pile test, stone columns were installed to improve soil conditions. A list of field tests conducted and the date of testing is presented in Table 1.1. The tests were conducted in five different excavations within the test site. A site plan of the TILT project is shown in Figure 1.1.

Data was collected from each test using various instrumentation. Pore pressure transducers were used to measure in-situ water pressures. All piles except for the pre-stressed and FRP piles were instrumented with strain gages. Linear potentiometers were used to measure pile head displacement and rotation. Load cells on the hydraulic actuator and individual piles within the pile groups measured the applied lateral force. A slope inclinometer was used with the 0.6-m and 0.9-m CISS piles at the end of testing to obtain the final displaced shape. Peak particle velocity was measured using portable seismographs and settlement measurements were made using driven stakes and a surveyor’s rod and level. A summary of data collected from in-situ soil tests as well as lateral load tests is presented in Table 1.2.

The UCSD Mobile Structural Testing (MoST) laboratory, which houses a high speed data acquisition system capable of acquiring over 200 channels of data, and a high speed data acquisition system from BYU were used to obtain data from the load cells, linear potentiometers, strain gages, and pore pressure transducers. The UCSD system collected the strain gage data while the BYU system collected the pore pressure and displacement data. Both systems acquired the lateral force applied by the actuator in order to synchronize the data.

This report contains a comprehensive collection of data obtained from the in-situ testing and full-scale field tests as reported in the Structural Systems Research Report titled "Full-Scale Behavior of Laterally Loaded Deep Foundations in Liquefied Sand: Test Results" as well as p-y curves back-calculated from the full-scale testing. The test results were included in this report to provide a single complete document that can be referenced for both measured test results as well as the back-calculated p-y curves. The report has been organized to present data from each test site in separate chapters. Data from the pilot liquefaction study will be presented first followed by results from the single pile tests, 4-pile group/0.6-m CISS pile tests, nine- pile group/0.9-m CISS pile and EQ Drain tests. The back-calculated p-y curve are presented for the single pile, 0.6-m and 0.9-m CISS pile, and the 4-Pile and 9-pile group. Conclusions and recommendations based upon the back-calculated p-y curves are presented to assist in pile design at sites where liquefaction is a concern.

Table 1.1 List of Field Tests

Test	Date Performed
Pilot Liquefaction Study Blast 1	10/23/98
Pilot Liquefaction Study Blast 2	10/26/98
Single Pile Blast 1	1/20/99
Single Pile Blast 2	1/28/99
Four Pile Group/0.6-m CISS Pile Blast 1	2/4/99
Four Pile Group/0.6-m CISS Pile Blast 2 (with Stone Columns)	2/27/99
Nine Pile Group/0.9-m CISS Pile Blast 1	2/18/99
Nine Pile Group/0.9-m CISS Pile Blast 2	2/19/99
E~Quake Drain Test	2/25/99
Pre-Stressed Concrete Pile	3/8/99
Concrete Filled FRP Pile	3/8/99

Table 1.2 Summary of Data Collected

	Insitu Tests							Pile Tests							Concrete Tests		
	Nuclear Density	Radar	Soil Boring	CPT	Vs	Pressuremeter	Dilitometer	Load	Displacement	Strain	Inclinometer	Particler Velocity	Pore Pressure	Settlement	Concrete UC	Gamma Logging	Rebar Tension
Pilot Liquefaction Study																	
Field Exploration			x	x	x												
Blast No.1				x								x	x	x			
Blast No.2				x								x	x	x			
Blast No.3				x								x	x	x			
Single Pile Tests																	
Field Exploration			x	x	x	x	x										
Blast No.1								x	x	x		x	x	x			
Blast No.2								x	x	x		x	x	x			
Composite & Pre-stressed Pile Tests								x	x								
0.6-m CISS / 4 Pile Group																	
Field Exploration	x	x	x	x	x										x	x	x
Blast No.1								x	x	x	x	x	x	x			
After Stone Column Installation		x		x	x												
Blast No.2					x			x	x	x	x	x	x	x			
0.9-m CISS / 9 Pile Group																	
Field Exploration		x	x	x	x										x	x	x
Blast No.1					x			x	x	x	x	x	x	x			
Blast No.2					x			x	x	x	x	x	x	x			
Drain Test																	
Field Exploration	x			x													
Blast No.1												x	x	x			

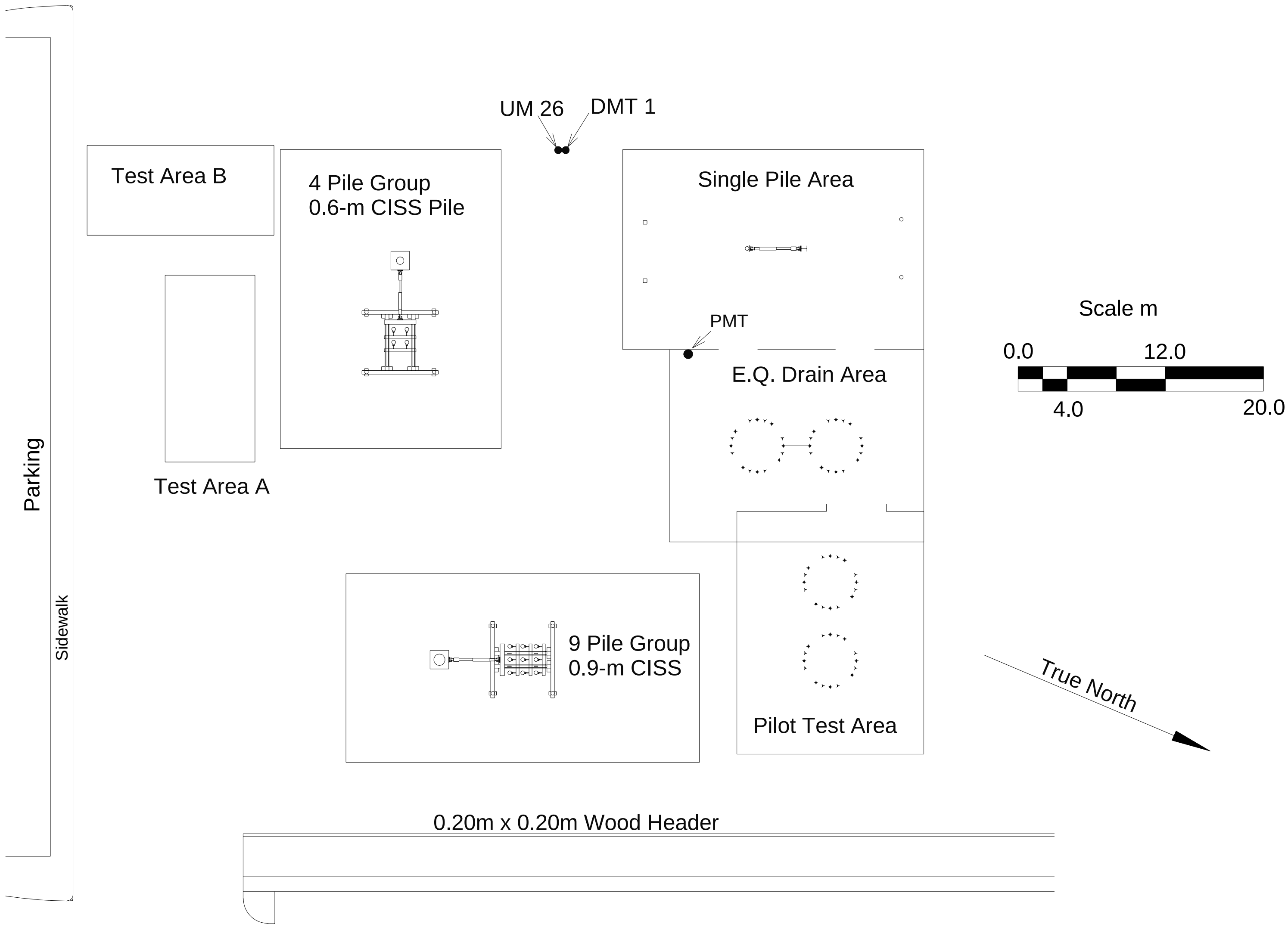


Figure 1.1 Treasure Island Liquefaction Test Site Plan

Figure 1.2 Aerial Photograph of Treasure Island Liquefaction Test Site

2 PILOT LIQUEFACTION STUDY

An area adjacent to the pile test sites was used to conduct a pilot liquefaction study prior to foundation testing. The pilot liquefaction study consisted of two small trial blasts and two pilot blasts. Two trial test blasts were performed in an effort to assess pore-pressure transducer capabilities, while the pilot blasts verified the required charge weight, delay and pattern to induce liquefaction. Although blast densification has been used successfully over the last 50 years in a variety of soil and site conditions, site-specific studies are generally recommended (Narin van Court and Mitchell, 1995). This site-specific pilot study was carried out in order to verify that the controlled blasting was a viable technique to use for this project, prior to any foundations being installed. This chapter presents data from the subsurface investigation and in-situ testing at the pilot study site, followed by plots of excess pore pressure ratios, peak particle velocity as a function of scaled distance and settlement contours resulting from blasting.

The test set-ups for Trial Blasts A and B are shown in Figures 2.1 and 2.2, respectively. We found that the vibrating wire transducers used in Trial Blast A were not capable of withstanding the peak pressure caused by the initial blast. Trial Blast B utilized piezoresistive transducers, which performed well, and were used for the remainder of testing at Treasure Island.

2.1 Pilot Study Site Characterization

Prior to and after blasting, a series of in-situ tests were performed to characterize the soil at the pilot site. Figure 2.3 shows the location and type of in-situ tests performed for this effort. The soil boring log BH-1 is shown in Figure 2.4. This log shows the water table at approximately 1.5 m below the original ground surface. The low standard penetration test (SPT) blow counts in the sand indicate liquefaction susceptibility of the deposit. For the SPT procedures used in all the soil borings, the measured hammer energy effeciency was approximately 45 to 50 percent. Figures 2.5 through 2.10 show results from six CPT's performed within the excavated test area before blasting. Thirteen additional CPT's were performed after the first pilot blast and are shown in Figures 2.11 through 2.23. Shear and compression wave velocity testing was also carried out at the pilot study site prior to any blasting. A seismic cone test at the location of CPT-3 measured shear wave velocity and a suspension logger was used in the borehole to measure shear and compression wave velocities. The velocity profile is shown in Figure 2.24. Correlation between the measured shear wave velocities using the seismic cone and suspension logger is good. Shear wave velocities in the profile generally vary between 100 m/s and 200 m/s.

2.2 Pilot Study Test Results

The initial pilot liquefaction blast occurred on October 23, 1998, with the second pilot blast occurring three days later on October 26, 1998. A map showing the location and depth of the pore-pressure transducers (PPT's) and location of the down-hole explosives for both pilot blasts is presented in Figure 2.25. The PPT's were placed at depths ranging from 1 to 6 m, and the explosives were placed at a depth of approximately 3 m below the excavated ground surface. Pore pressure transducers remained at the location shown in Figure 2.25 for the second blast while the down-hole explosives were rotated clockwise 0.3 m at the same radial distance and depth used for the first blast. The water table for the pilot study was approximately 0.5 m below the excavated ground surface.

2.2.1 Excess Pore Pressure Ratio's

Excess pore pressure ratios from the first blast are presented in Figures 2.26 through 2.35. The excess pore pressure ratio plots are grouped according to location, starting at the west end of the excavation and moving east. All transducers below a depth of 1 m show excess pore pressure ratios reaching or exceeding a value of 80% initially. Many transducers show excess pore pressure ratios reaching 100%. Approximately five minutes after blasting, the excess pore pressure ratios dropped below 80%. The excess pore pressure ratios generally ranged between 10% and 20% one hour after the blast. In addition to the pore pressure transducers indicating excess pore pressure ratios of 100%, the presence of sand boils after blasting provided evidence that liquefaction had occurred. Sand boils first appeared approximately 5 minutes after the blast, and continued for over 20 minutes.

The second pilot blast was performed on October 26, 1998. The objective of the second blast was to verify that liquefaction could be induced more than once at the same site with the same charge weight and pattern. Again, excess pore pressure ratios below 1 m typically exceeded 80% with many transducers showing values of 100% for the first five minutes after blasting. After one hour, excess pore pressure ratios had dropped to between 10% and 20%. Excess pore pressure ratios for the second blast are shown in Figures 2.36 through 2.45. Again, sand boils provided further evidence that liquefaction had occurred as shown in Figure 2.46.

2.2.2 Settlement

Figure 2.47 shows how we performed an elevation survey to measure settlement due to blasting. Contours of blast induced settlement are presented in Figures 2.48 and 2.49 for the first and second blast respectively. The total settlement resulting from both blasts is presented in Figure 2.50. The contours of settlement are in millimeters, while the coordinates from the edge of the excavation are in meters.

2.2.3 Peak Particle Velocity

Portable seismographs were placed at various distances from the blasts to monitor peak particle velocity to verify that vibration levels were low enough to not disturb residents or cause damage to adjacent structures. The measured peak particle velocity is plotted against scaled distance in Figure 2.51. A general equation for estimating peak radial velocity for saturated soils has been given by Charlie and Abt (1985). The peak radial velocity in mm/sec is given as

$$V_p = 12000\left(\frac{R}{W^{1/3}}\right)^{-1.5} \tag{2.1}$$

where V_p is the peak radial velocity, R is the distance from the charge in meters, and W is the charge mass in kilograms. Results from this equation predict velocities significantly greater than those measured at Treasure Island in the loose sand deposit. A best fit of the recorded data can be approximated using following equation.

$$V_p = 2400\left(\frac{R}{W^{1/3}}\right)^{-1.5} \tag{2.2}$$

Three components of velocity were measured at each seismograph: vertical, longitudinal and transverse. The vertical velocity was the largest for each test.

2.3 Summary

The pilot liquefaction study was quite successful. The trial blasts showed that piezoresistive pore pressure transducers were necessary to resist the initial shock wave produced by the blasting, yet still able to record pore pressures following the blast with sufficient accuracy and precision. The charge size, delay and pattern were found sufficient to liquefy the sand inside the ring of charges. Using the same charge size, delay and pattern, the site was liquefied a second time, thus verifying the repeatability of the test. We were also able to develop a site-specific relationship correlating peak particle velocity and scaled distance. This site-specific relationship may be useful in predicting peak particle velocities at other sites where blast densification is used to mitigate liquefaction hazards. Based on the success of this pilot study, a series of full-scale foundation tests were carried out as presented in the following chapters.

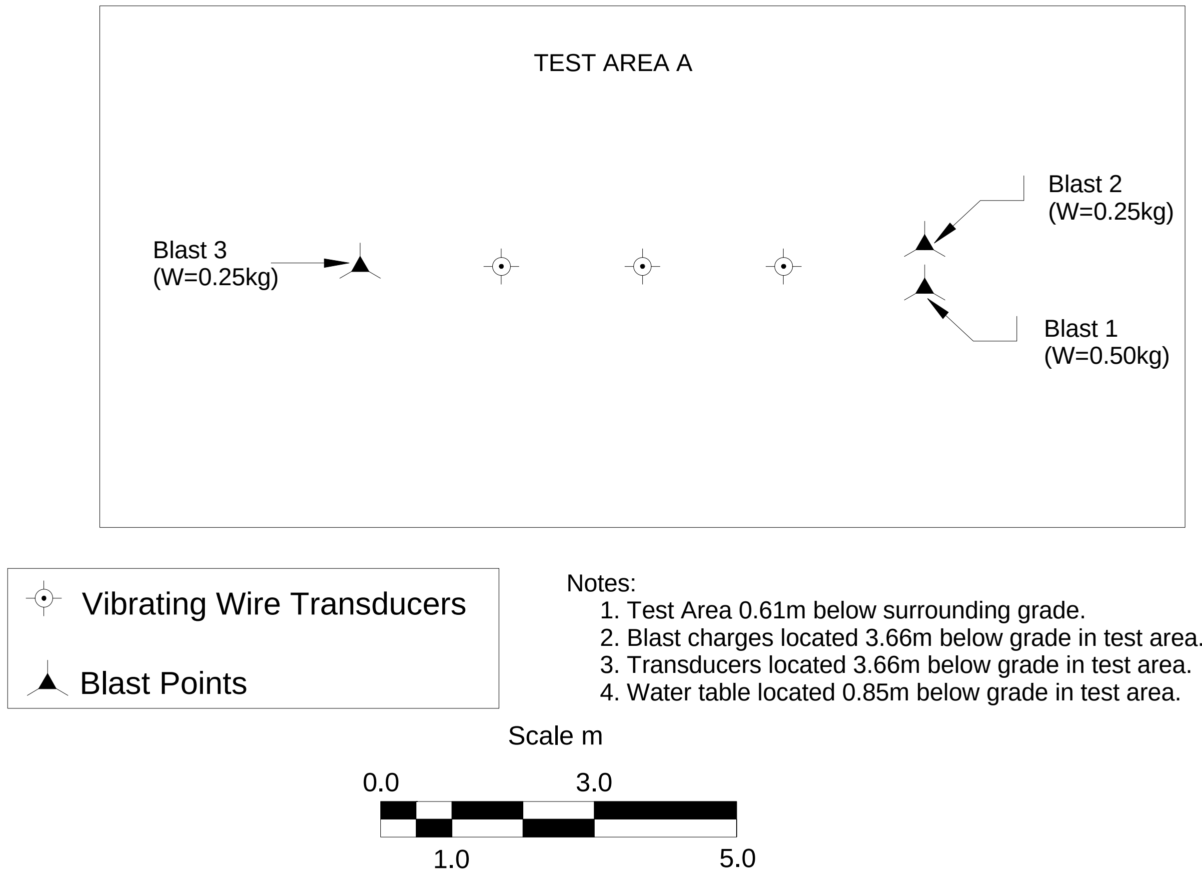


Figure 2.1 Location of Pore Pressure Transducers and Explosives for Trial Blast A

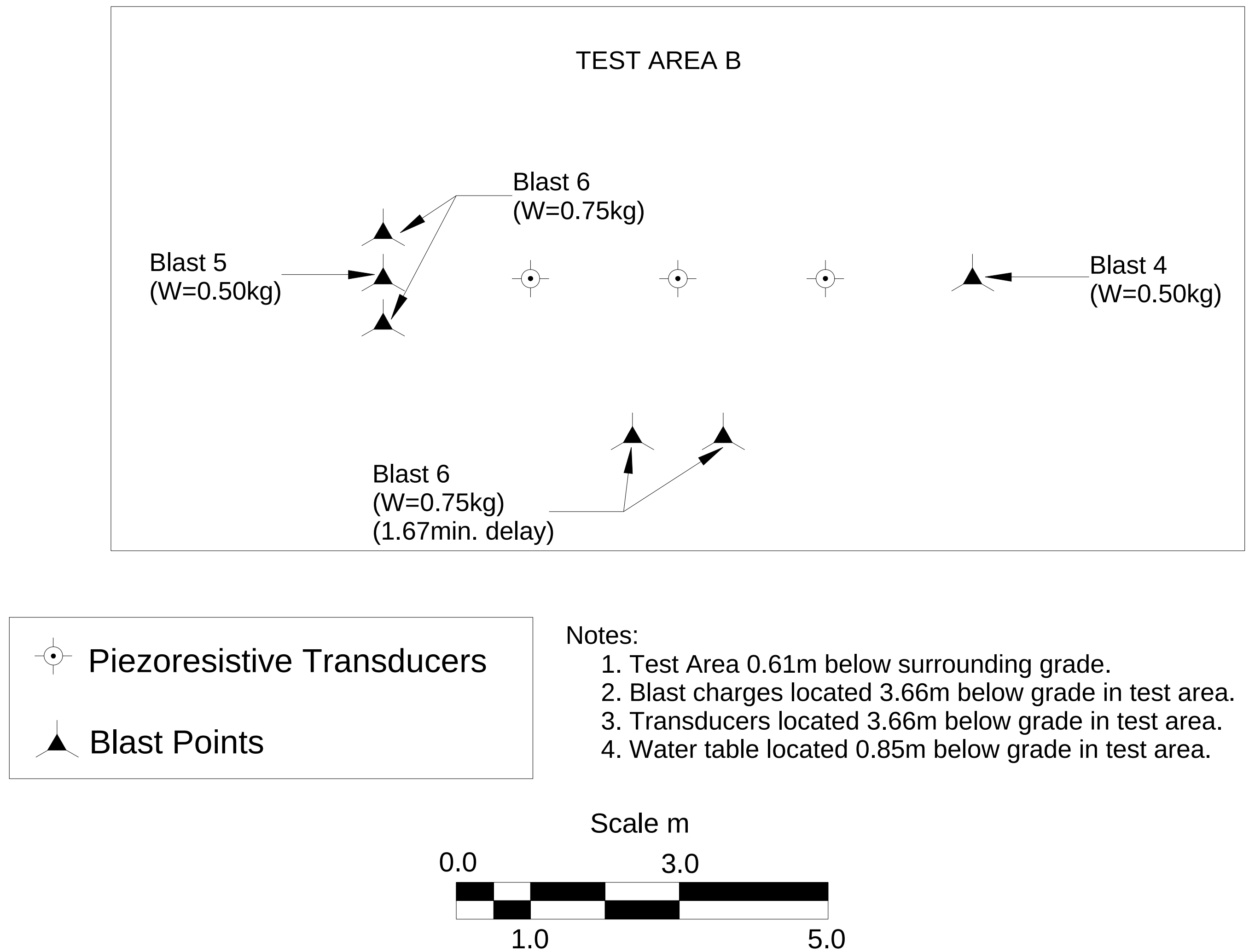


Figure 2.2 Location of Pore Pressure Transducers and Explosives for Trial Blast B

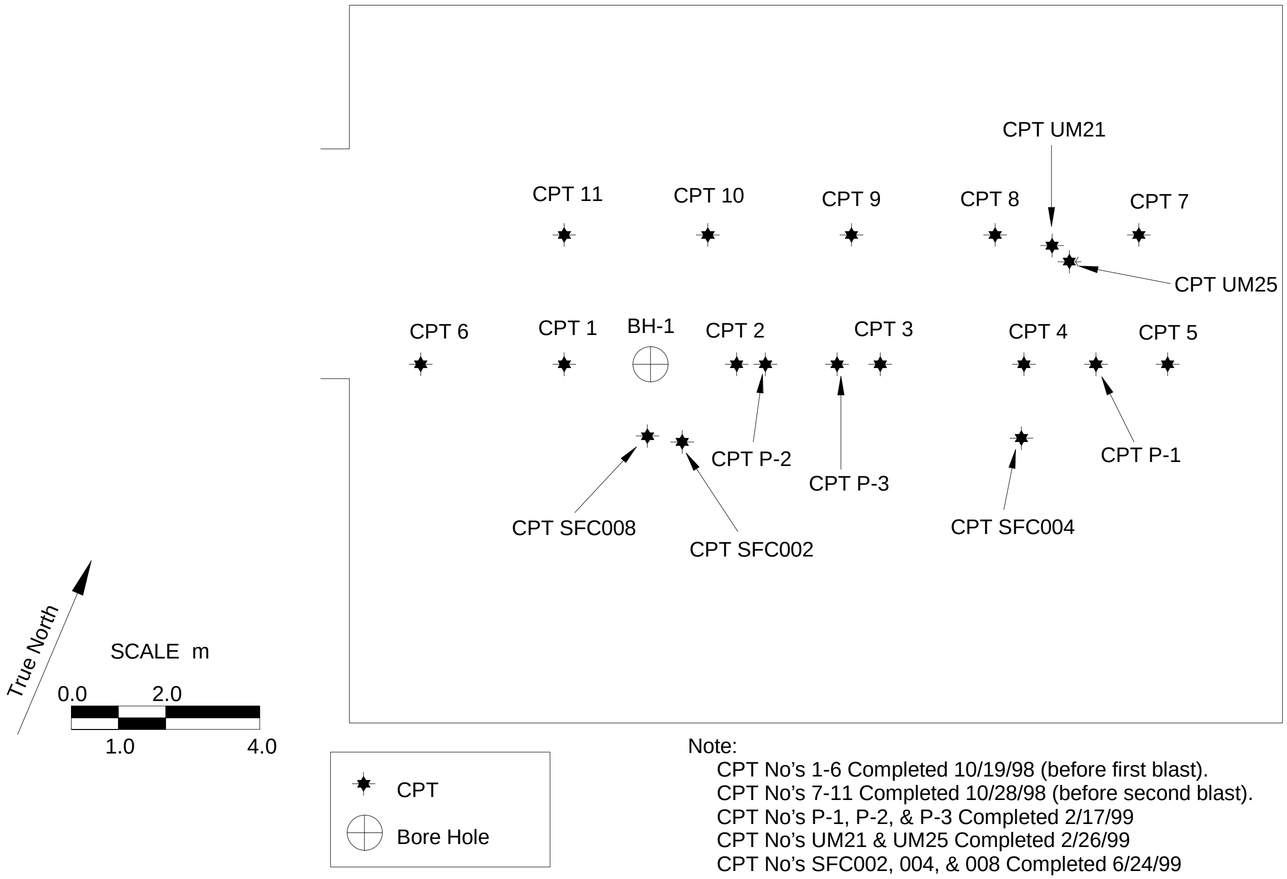


Figure 2.3 Location Map of In-Situ Tests for Pilot Liquefaction Study Area

Brigham Young University & University of California - San Diego Telephone: (801) 378-6334 Fax: (801) 378-4449	Boring No: BH-1 Logged By: tw Date Started: 9/17/1998 Date Finished: 9/17/1998	Elevation (m): 3.4 Drill Contractor: Pitcher Drilling Method: Rotary Wash	Sheet 1 of 2	TREASURE ISLAND SAN FRANCISCO BAY CALIFORNIA

Location: Pilot Liquefaction Study

Material Description	Water Level	Depth (m)	Location	Number	Type	Recovery (cm/cm)	Blows/15 cm	$(N_1)_{60}$ Blows/30 cm	USCS	AASHTO	S_u (kPa)	Torvane (kPa)	Pocket Pentometer (kPa)	Dry Density (kN/m3)	Moisture Content %	LL	PL	PI	Passing #200 Sieve (%)	OTHER DATA AND REMARKS
Sand, SP, brown, very moist, medium dense, fine grained		1	☒	S-1	SPT	46/46	5-8-10		SP											Top Has Shells, No Shells at Bottom
	▼	2	☒	S-2	SPT	46/46	6-9-10		SP										3	
		3	☒	S-3	SPT	46/46	4-5-6		SP											
		4	☒	S-4	SPT	46/46	5-4-2		SP										5	
		5	☒	S-5	SPT	46/46	4-5-5		SP											
			☒	S-6	SPT	46/46	4-3-3		SP										7	Encountered Bay Mud @ 19 Feet
Clay, grey, saturated, loose		6																		
Sand Lense																				
Clay, grey, saturated, loose			☒	S-7	SPT	46/46	0-1-1		CL											
Sand, grey, loose, fine grained		7																		
		8	☒	S-8	SPT	30/46	1-2-1		SP										13	Clay In Bit
Clay, grey, soft		9	☒	S-9	SPT	46/46	0-1-3		CL											Clay - First 16", Sand - Last 2"
Sand, black & grey, saturated, loose		10																		

LEGEND AND NOTES
- ☒ SPT = Standard Penetration Test, Split Spoon Sample
- ■ US = Undisturbed Shelby Tube, Pushed
- ○ Ring = Dames & Moore Sampler
- ▨ Bulk = Bulk Sample From Cuttings

GROUNDWATER DEPTHS:
- ∇ = Initial
- ▼ = End of Drilling

COMMENTS:

LOG OF BORING METRIC SANFRAN.GPJ LANE.GDT 9/14/00

Figure 2.4a Soil Boring Log for Pilot Liquefaction Study Area

Brigham Young University
&
University of California - San Diego
Telephone: (801) 378-6334
Fax: (801) 378-4449

Boring No: BH-1 | Sheet 2 of 2
Logged By: tw | Elevation (m): 3.4
Date Started: 9/17/1998 | Drill Contractor: Pitcher
Date Finished: 9/17/1998 | Drilling Method: Rotary Wash

TREASURE ISLAND
SAN FRANCISCO BAY
CALIFORNIA

Location: Pilot Liquefaction Study

Material Description	Water Level	Depth (m)	Sample Data: Location	Number	Type	Recovery (cm/cm)	Blows/15 cm	$(N_1)_{60}$ Blows/30 cm	Soil Classification: USCS	AASHTO	S_u (kPa)	Torvane (kPa)	Pocket Pentometer (kPa)	Dry Density (kN/m3)	Moisture Content %	Atterberg Limits: Liquid Limits LL	Plastic Limits PL	Plasticity Index PI	Passing #200 Sieve (%)	OTHER DATA AND REMARKS
		11	X	S-10	SPT	46/46	1-1-1		SP										17	
		12	X	S-11	SPT	46/46	1-1-2		SC										23	Hit More Firm Layer @ 37.5'
Clay, grey, moist/saturated, soft, w/sand grains in clay		13																		
		14	X	S-12	SPT	46/46	1-0-1		CL											More Clay In Sand
		15	X	S-13	SPT	46/46	1-2-1		CL											Less Sand
		16																		
		17	X	S-14	SPT	46/46	0-1-1		CL											
		18																		
		19	X	S-15	SPT	46/46	2-0-1		CL											
Bottom of Bore Hole		20																		

LEGEND AND NOTES
SPT = Standard Penetration Test, Split Spoon Sample
US = Undisturbed Shelby Tube, Pushed
Ring = Dames & Moore Sampler
Bulk = Bulk Sample From Cuttings

GROUNDWATER DEPTHS:
▽ = Initial
▼ = End of Drilling

COMMENTS:

LOG_OF_BORING_METRIC SANFRAN.GPJ LANE.GDT 9/14/00

Figure 2.4b Soil Boring Log for Pilot Liquefaction Study Area

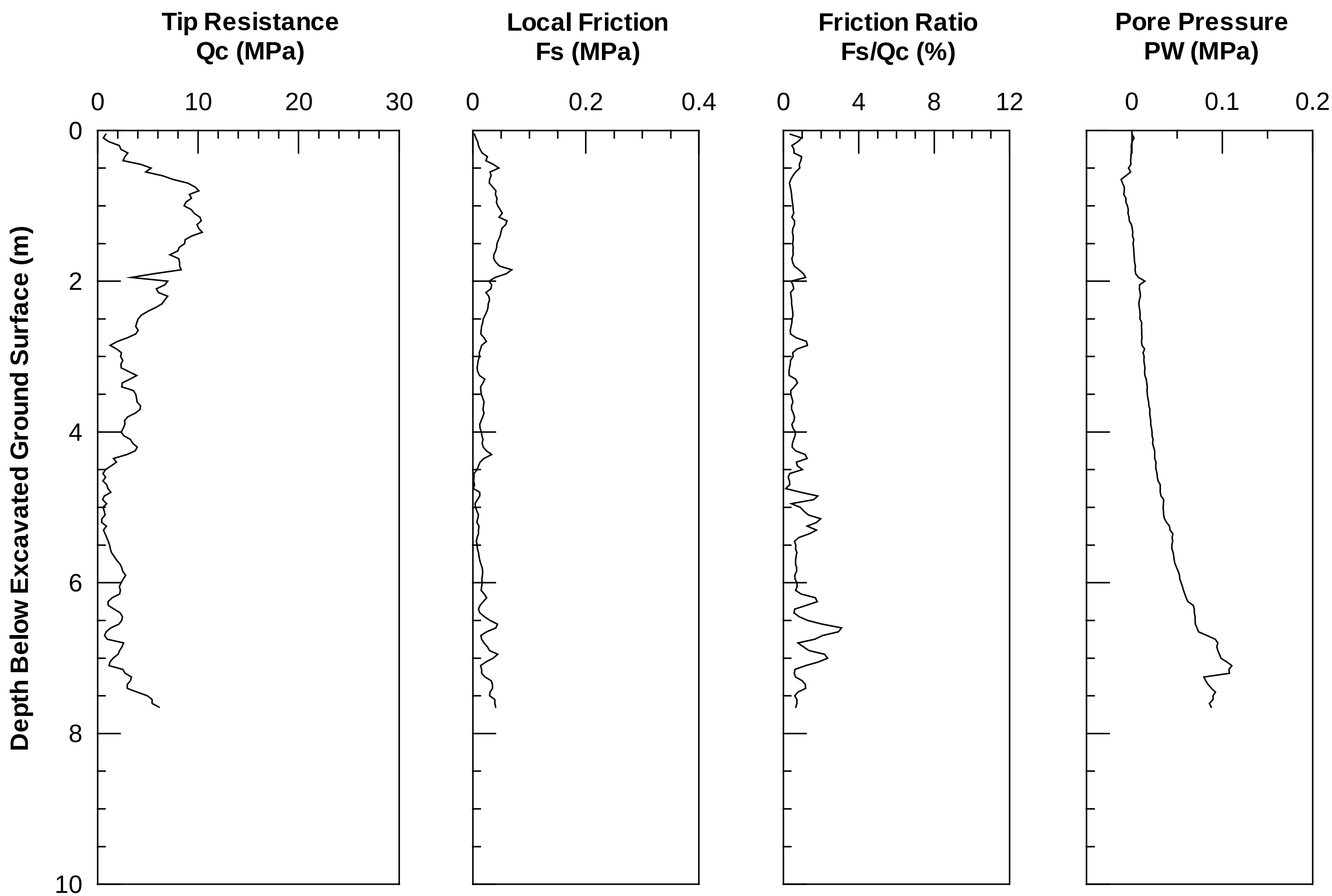


Figure 2.5 CPT-1 Logs for the Pilot Liquefaction Study Area (10-19-98)

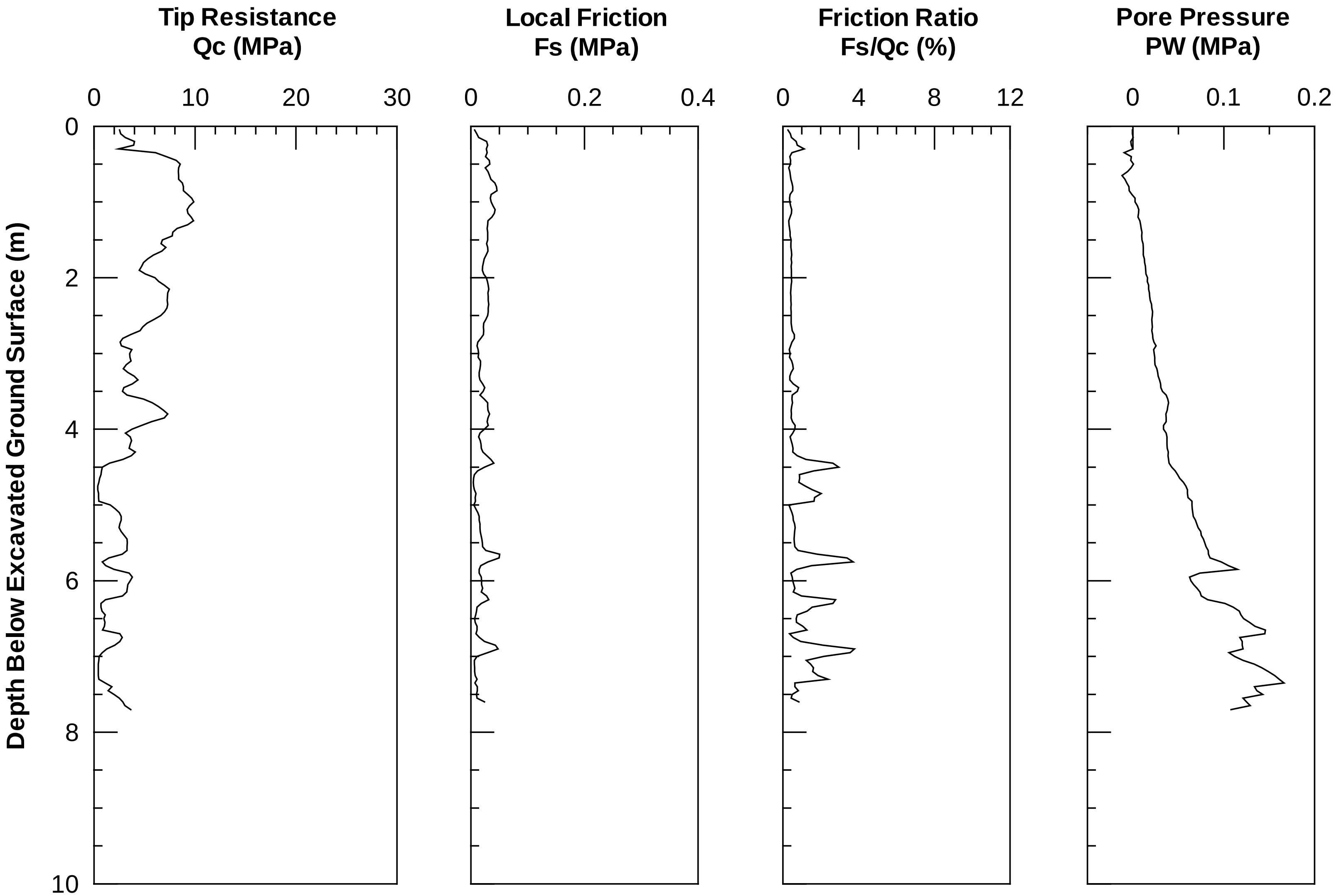


Figure 2.6 CPT-2 Logs for Pilot Liquefaction Study Area (10-19-98)

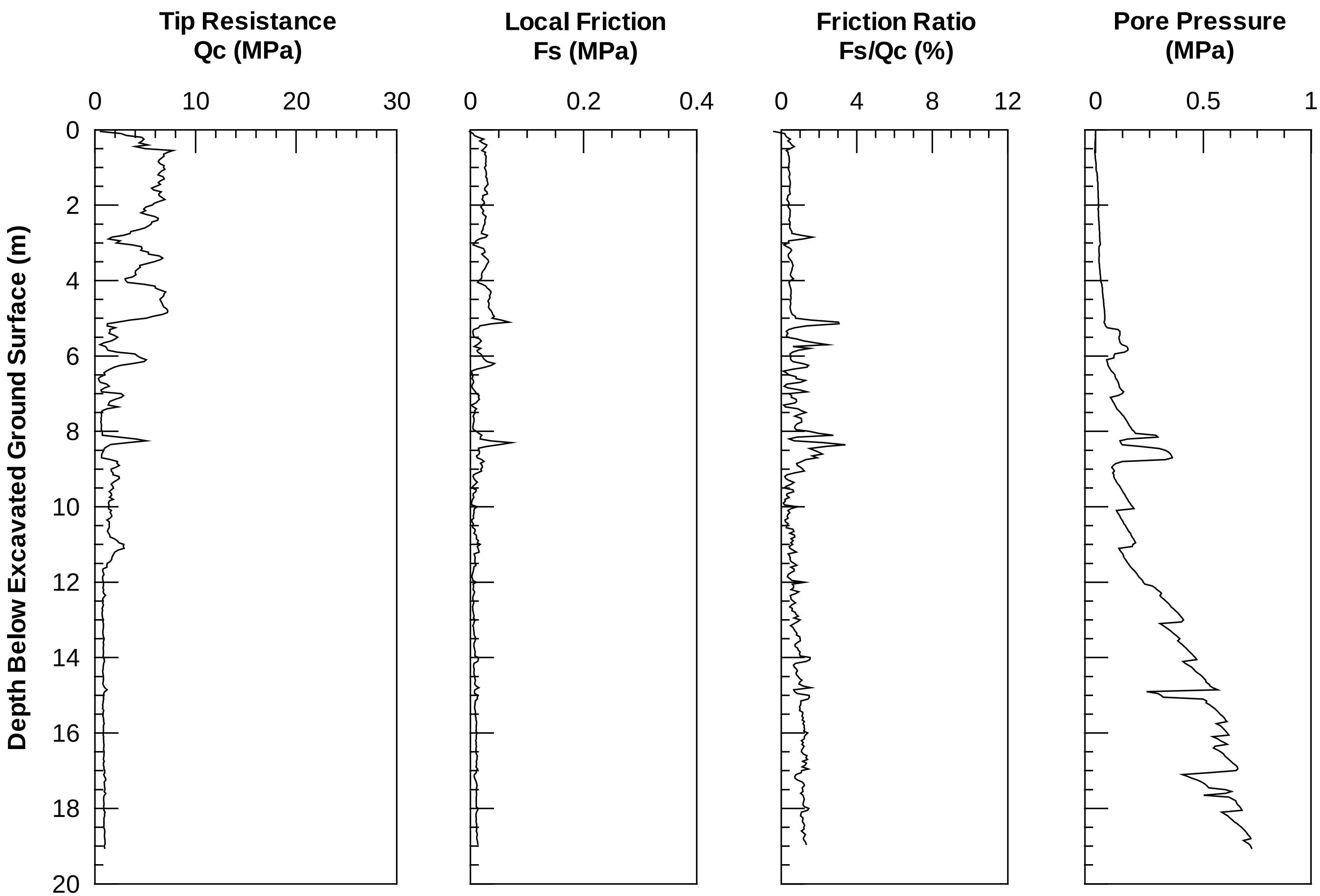


Figure 2.7 CPT-3 Logs for Pilot Liquefaction Study Area (10-19-98)

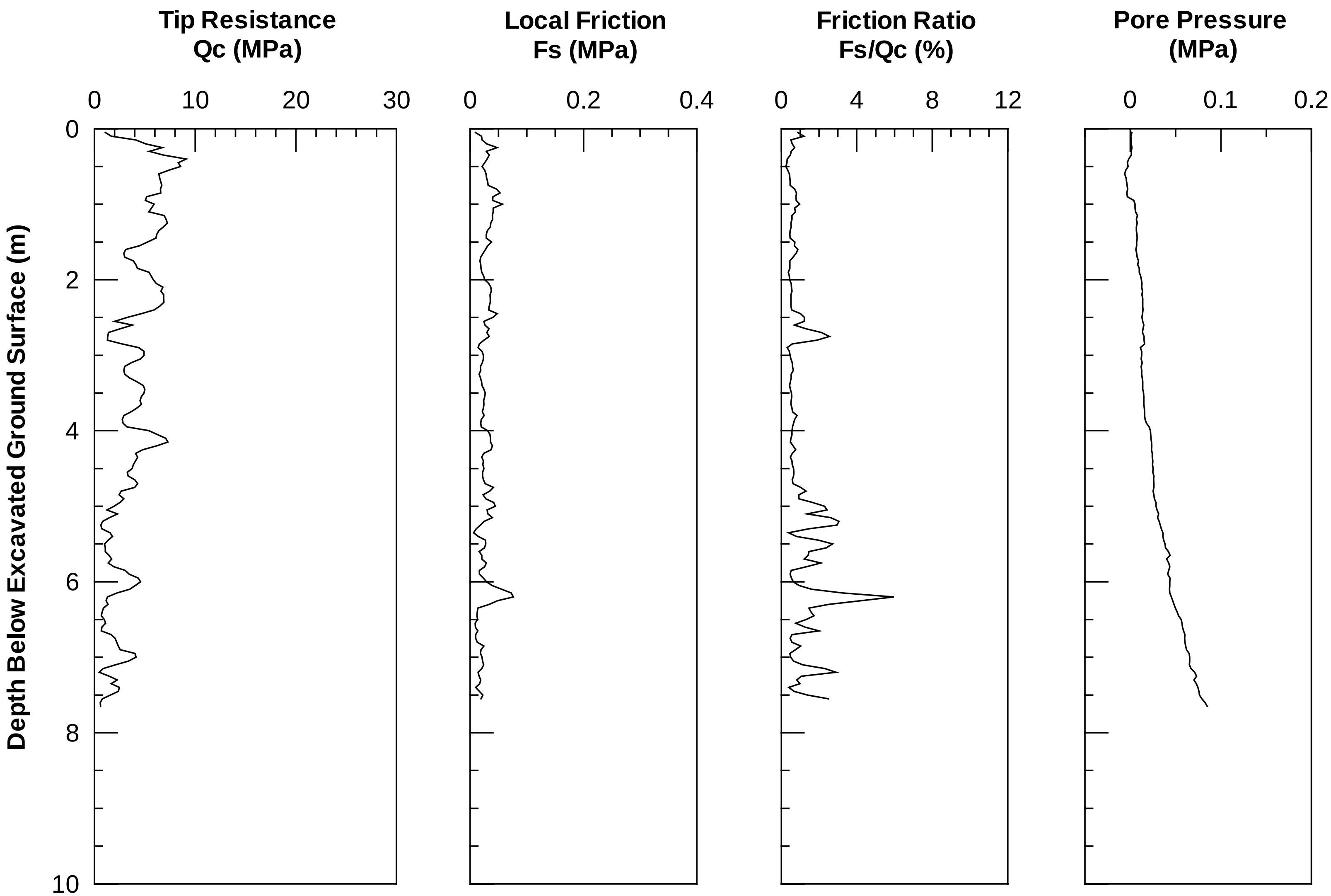


Figure 2.8 CPT-4 Logs for Pilot Liquefaction Study Area (10-19-98)

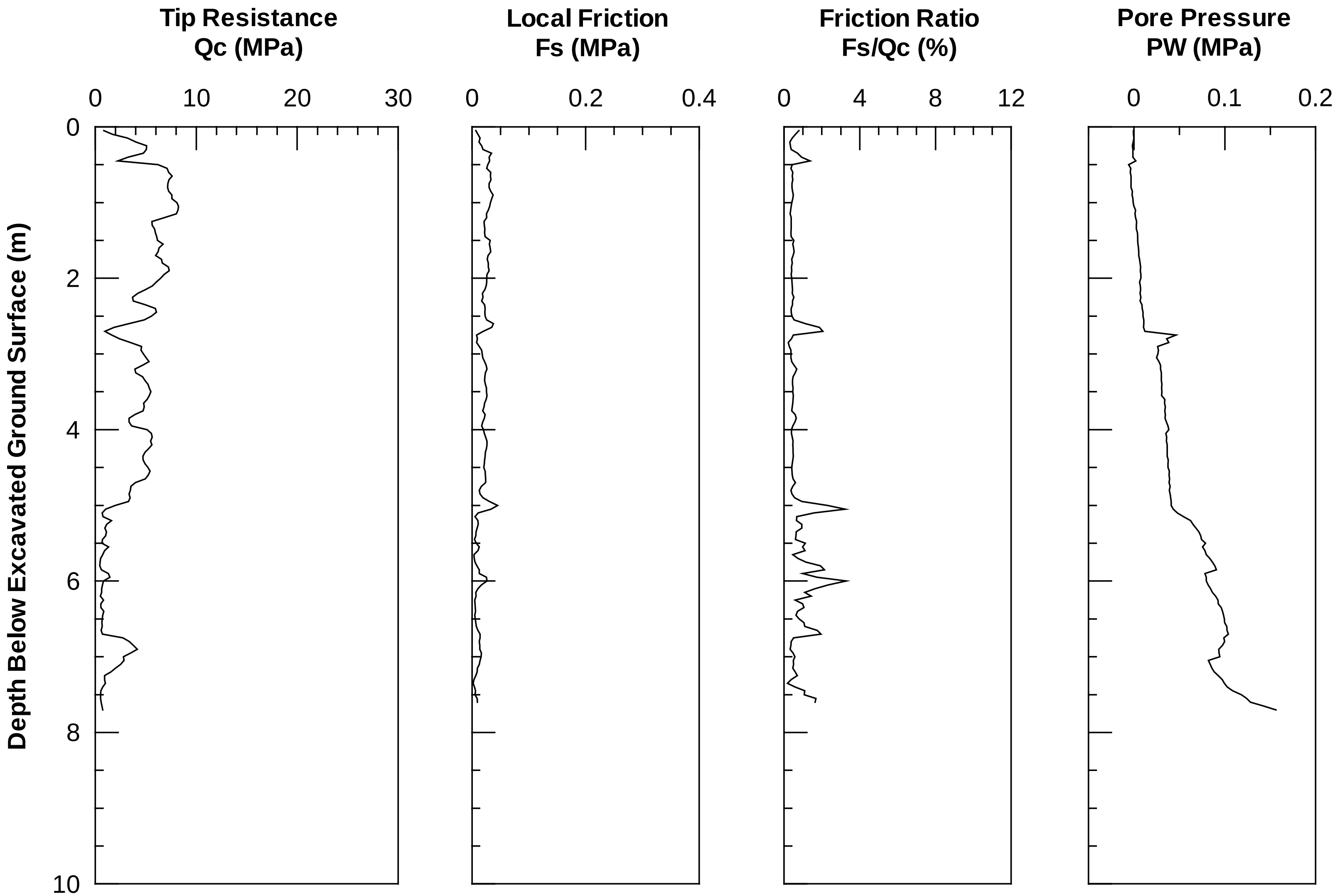


Figure 2.9 CPT-5 Logs for Pilot Liquefaction Study Area (10-19-98)

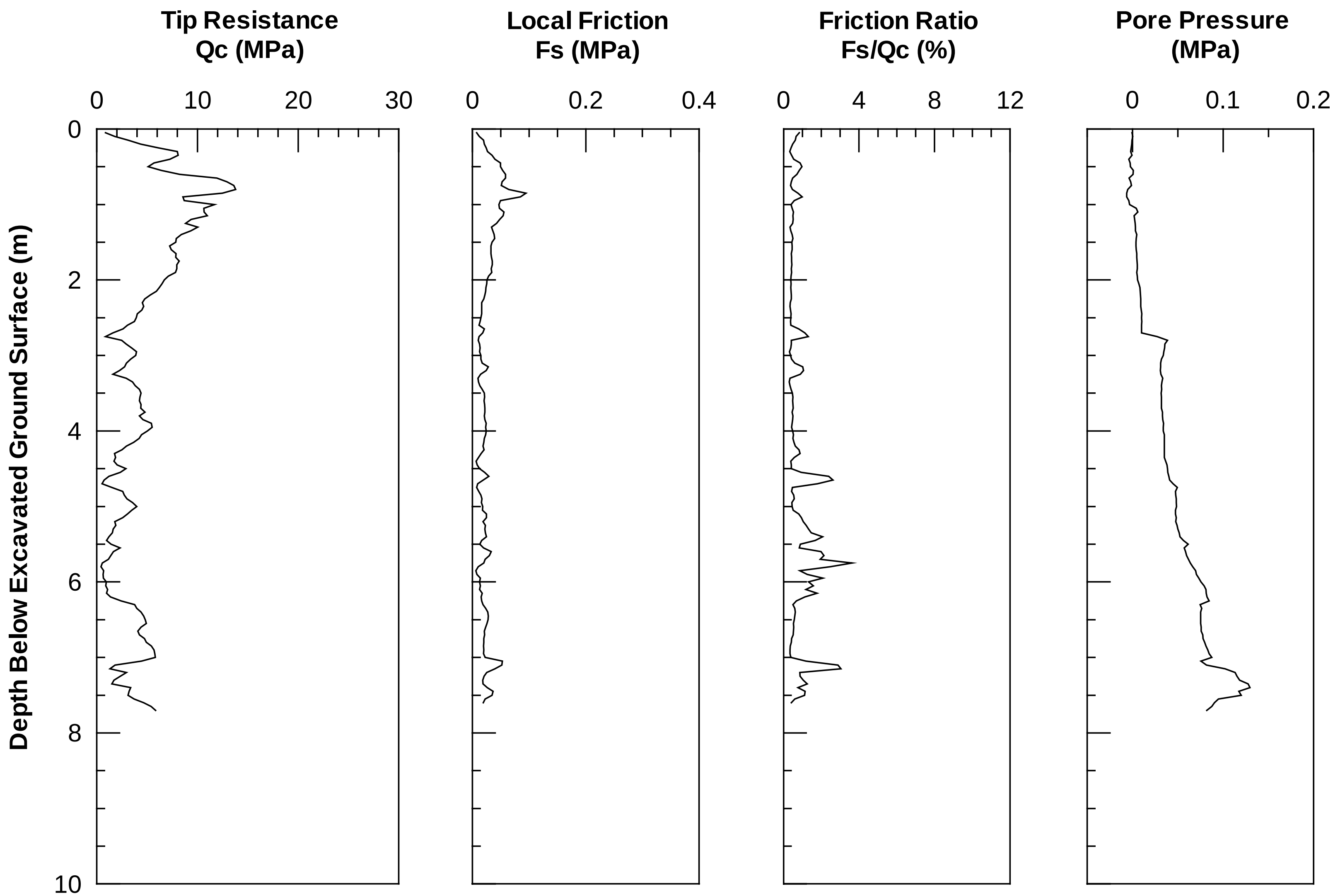


Figure 2.10 CPT-6 Logs for Pilot Liquefaction Study Area (10-19-98)

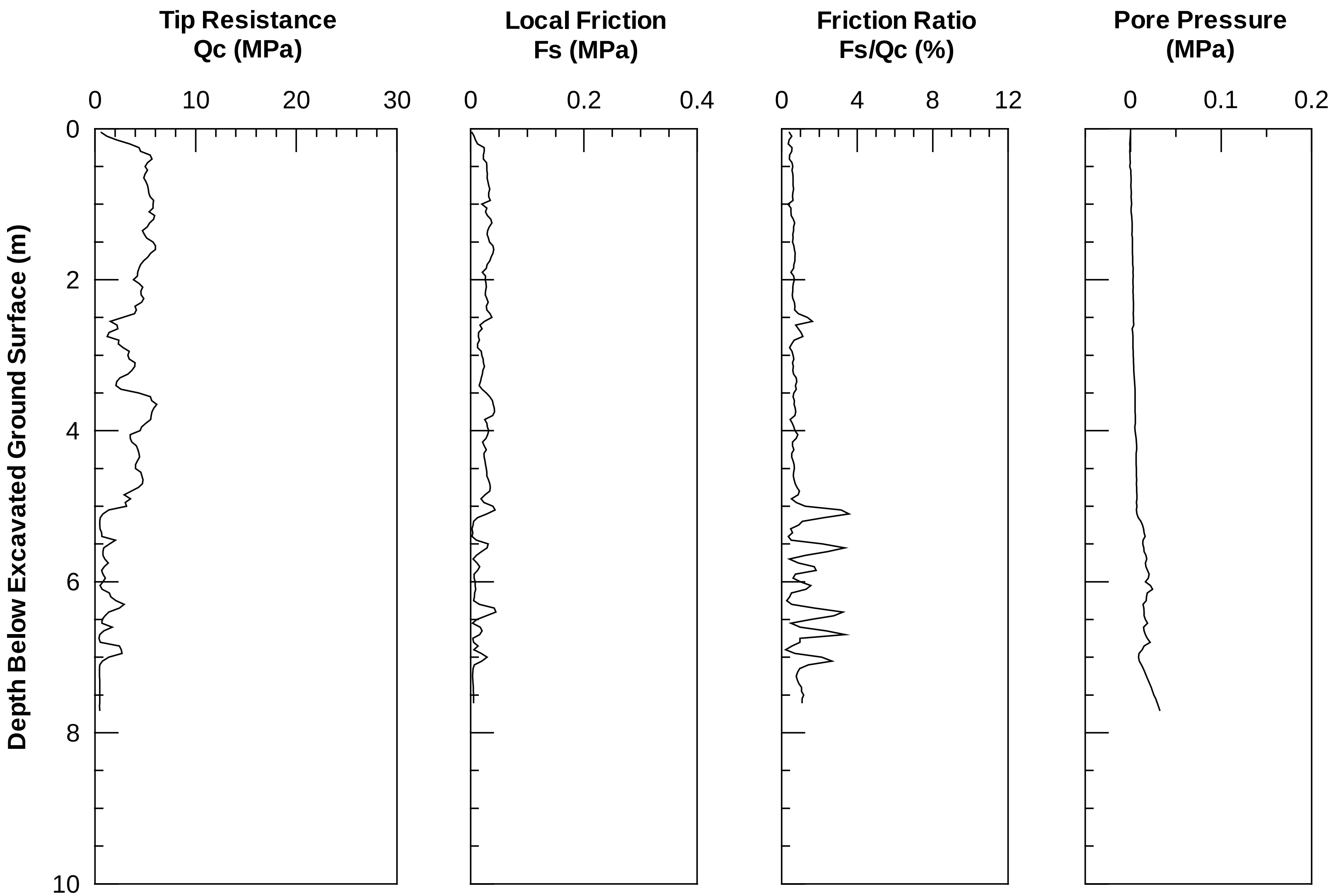


Figure 2.11 CPT-7 Logs for Pilot Liquefaction Study Area (10-28-98)

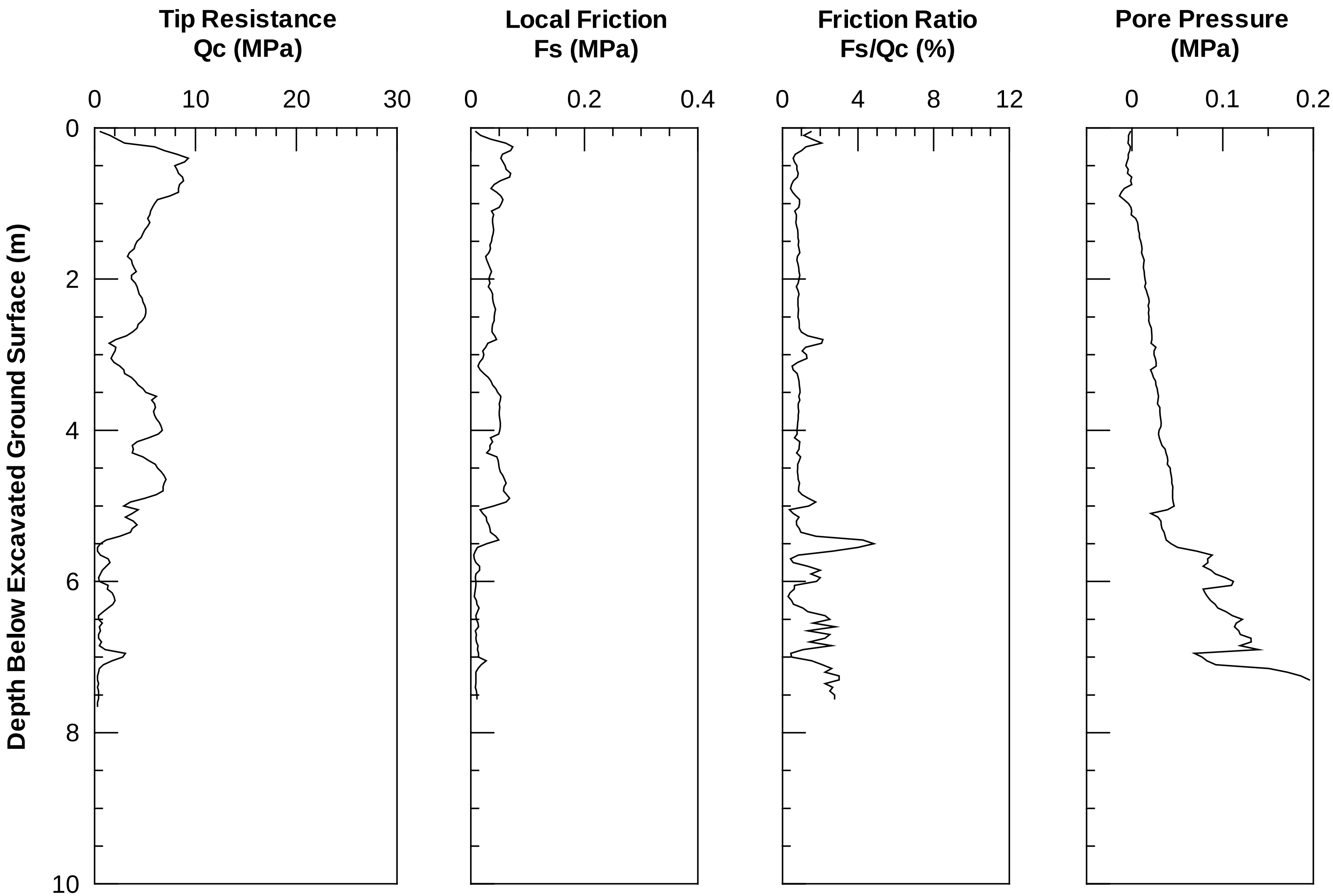


Figure 2.12 CPT-8 Logs for Pilot Liquefaction Study Area (10-28-98)

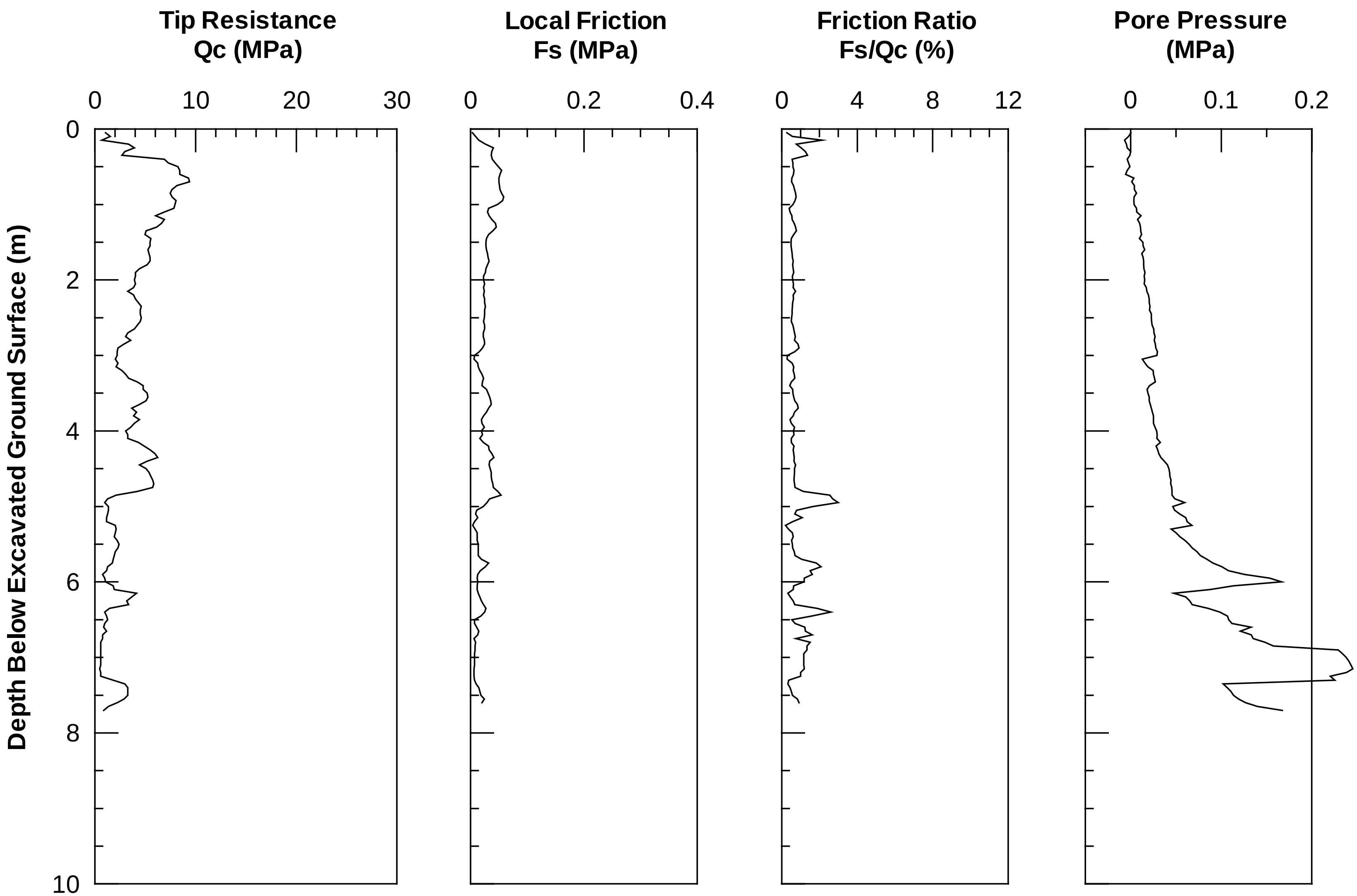


Figure 2.13 CPT-9 Logs for Pilot Liquefaction Study Area (10-28-98)

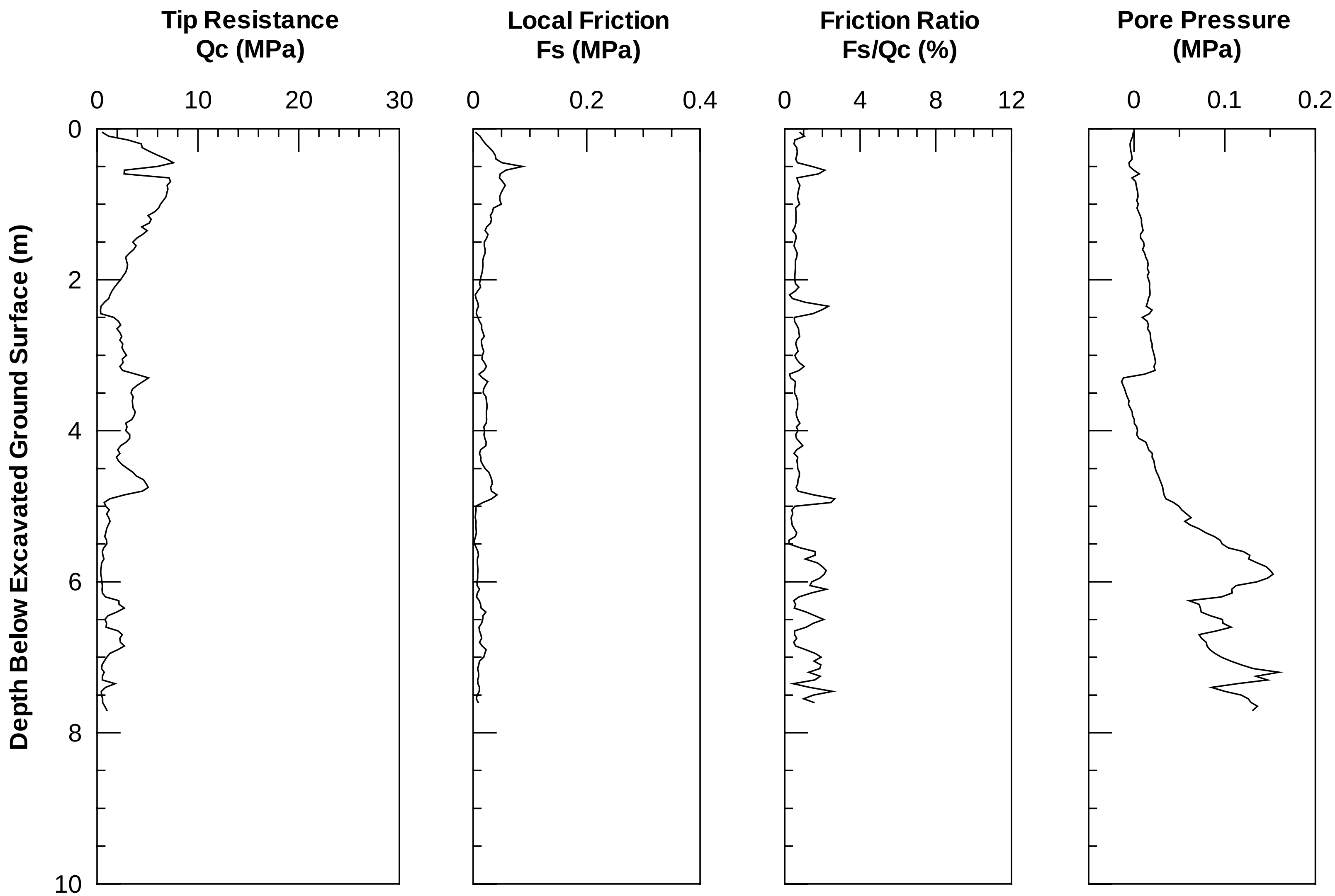


Figure 2.14 CPT-10 Logs for Pilot Liquefaction Study Area (10-28-98)

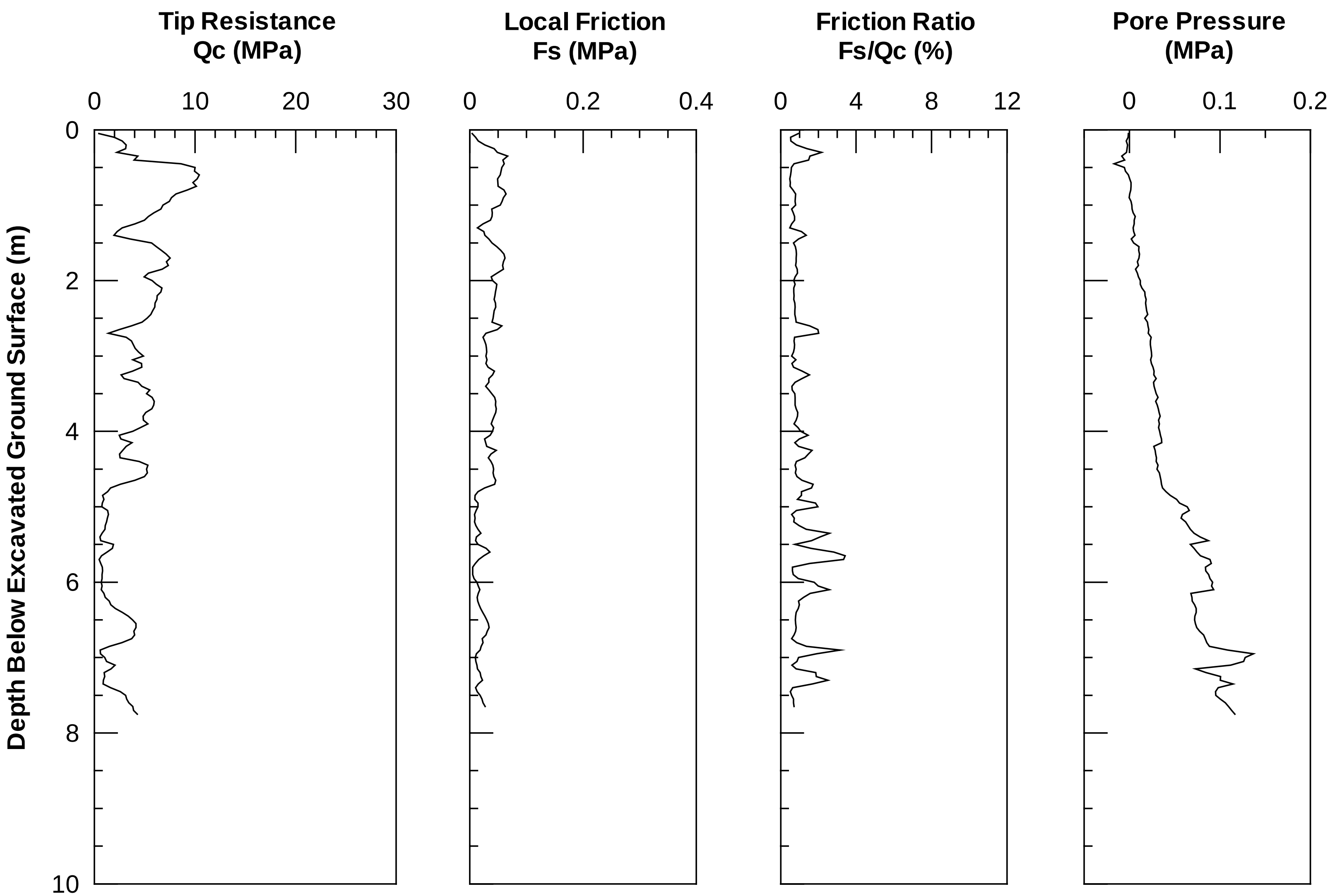


Figure 2.15 CPT-11 Logs for Pilot Liquefaction Study Area (10-28-98)

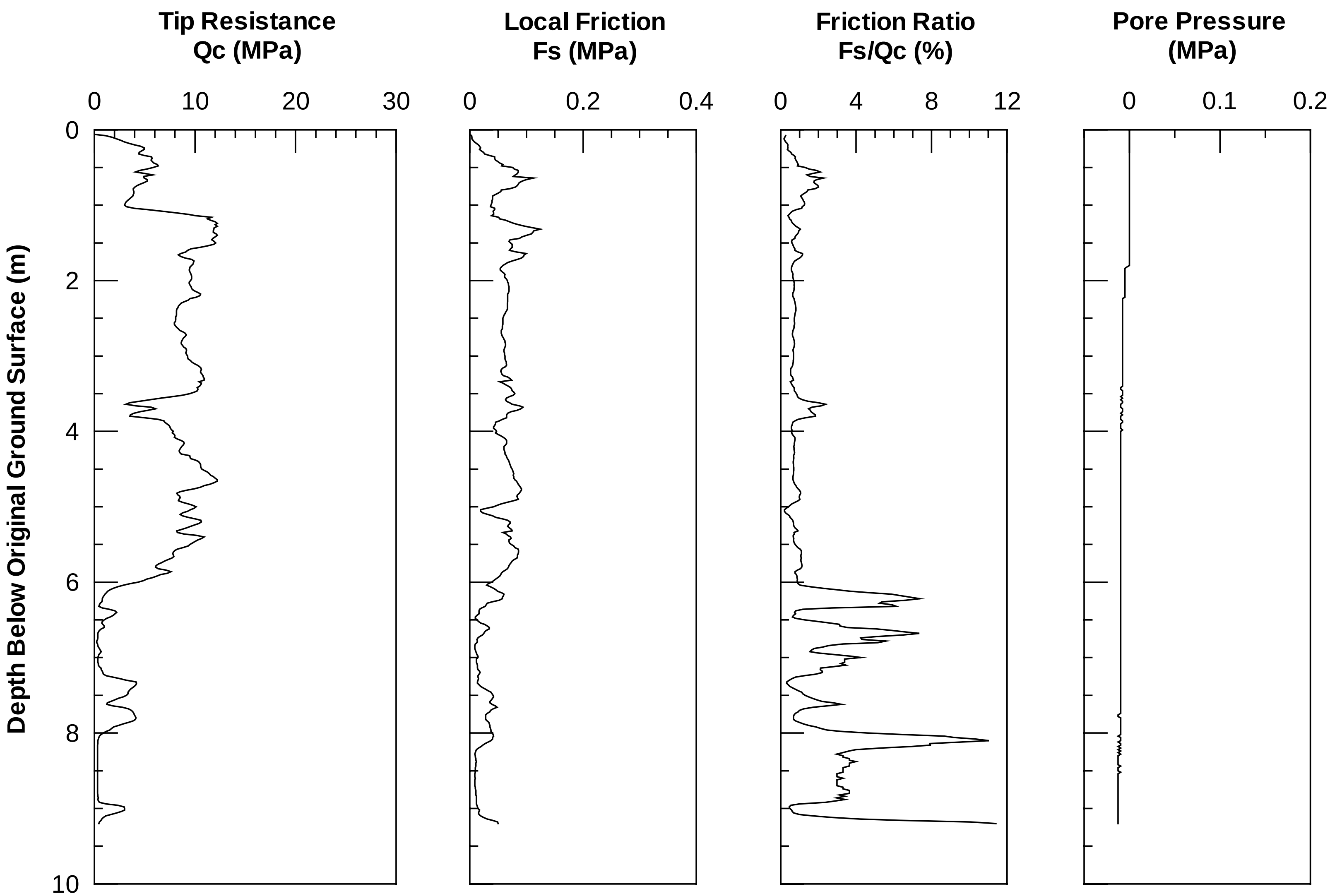


Figure 2.16 CPT P-1 Logs for Pilot Liquefaction Study Area (2-17-99)

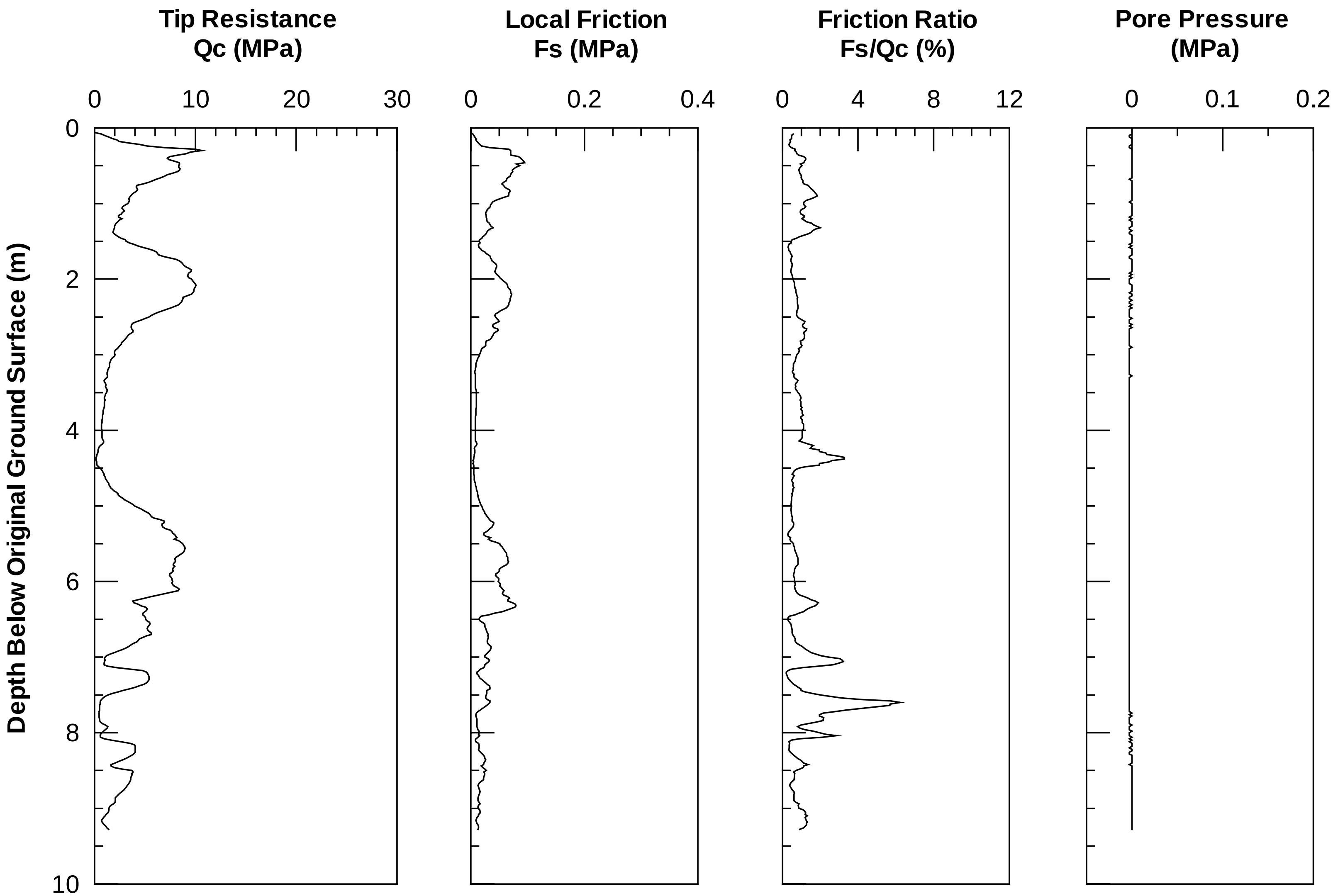


Figure 2.17 CPT P-2 Logs for Pilot Liquefaction Study Area (2-17-99)

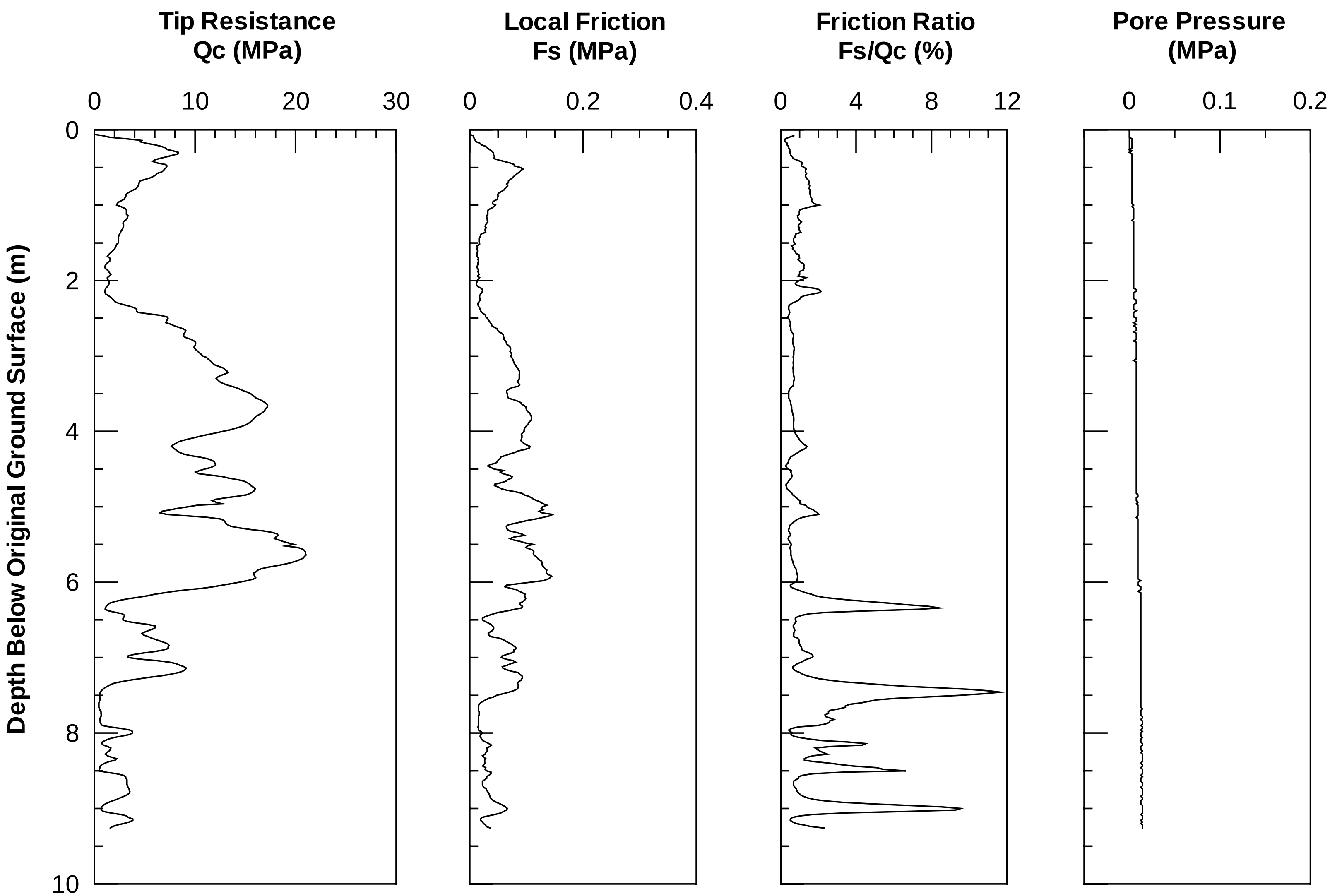


Figure 2.18 CPT P-3 Logs for Pilot Liquefaction Study Area (2-17-99)

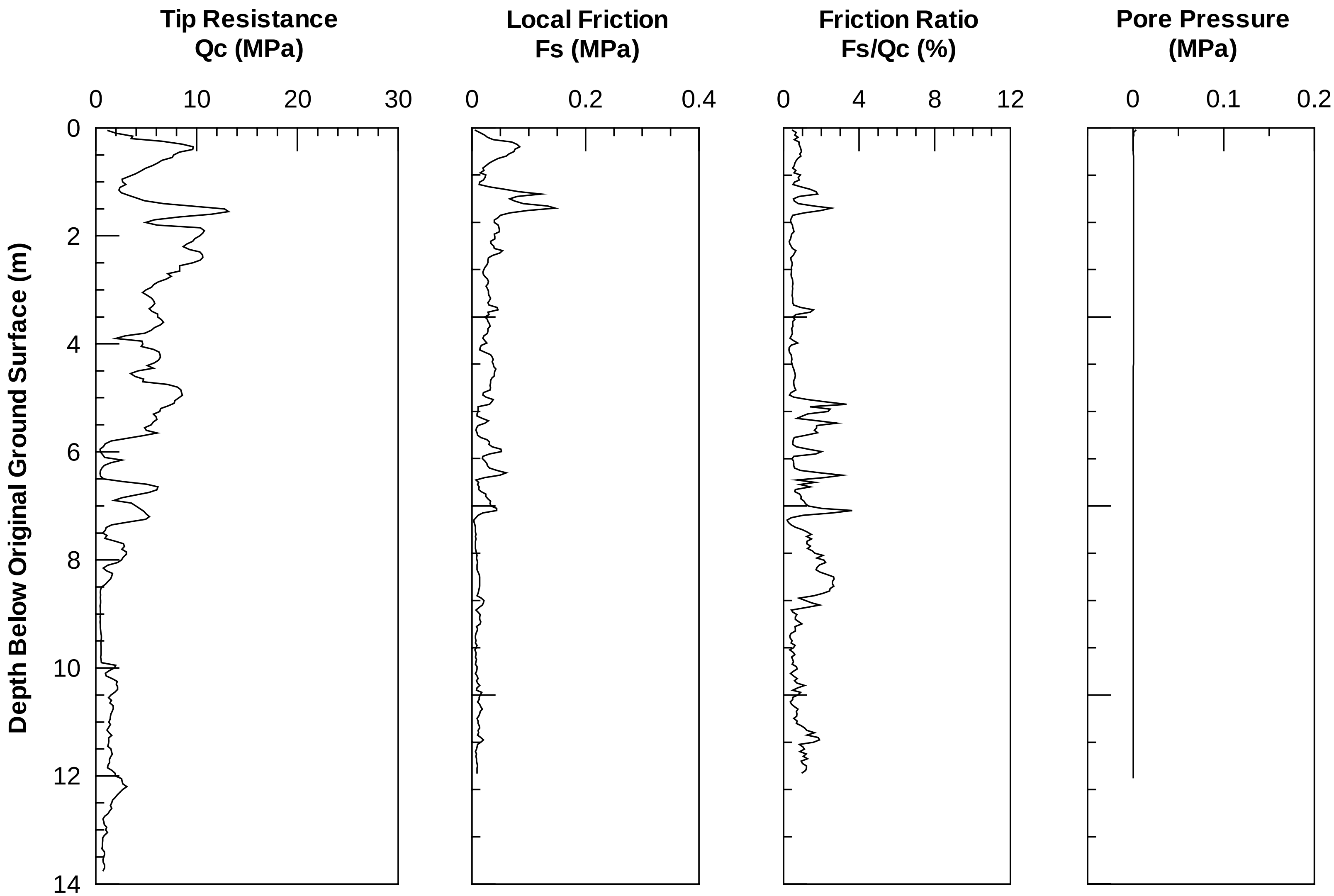


Figure 2.19 CPT UM21 Logs for Pilot Liquefaction Study Area (2-26-99)

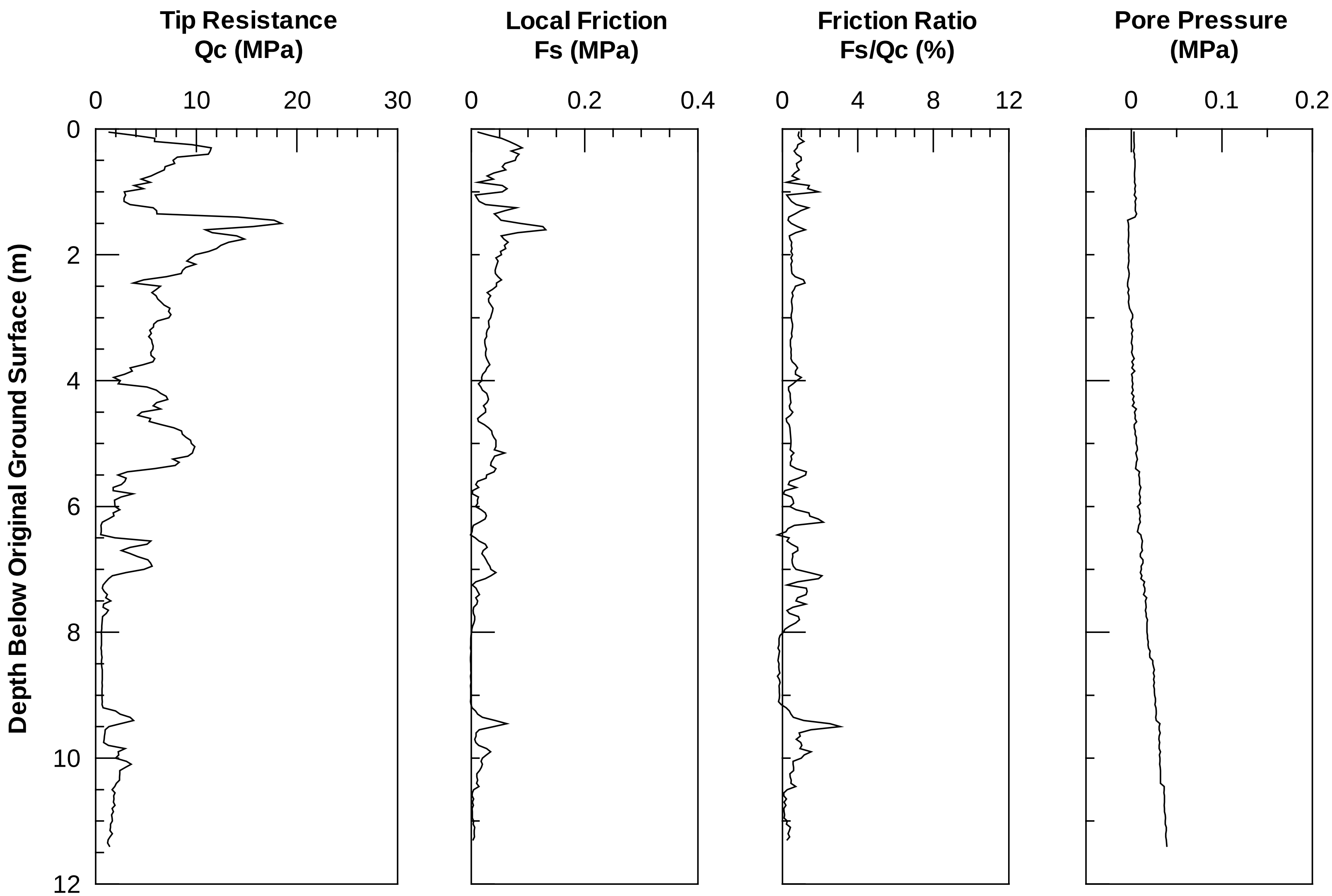


Figure 2.20 CPT UM25 Logs for Pilot Liquefaction Study Area (2-26-99)

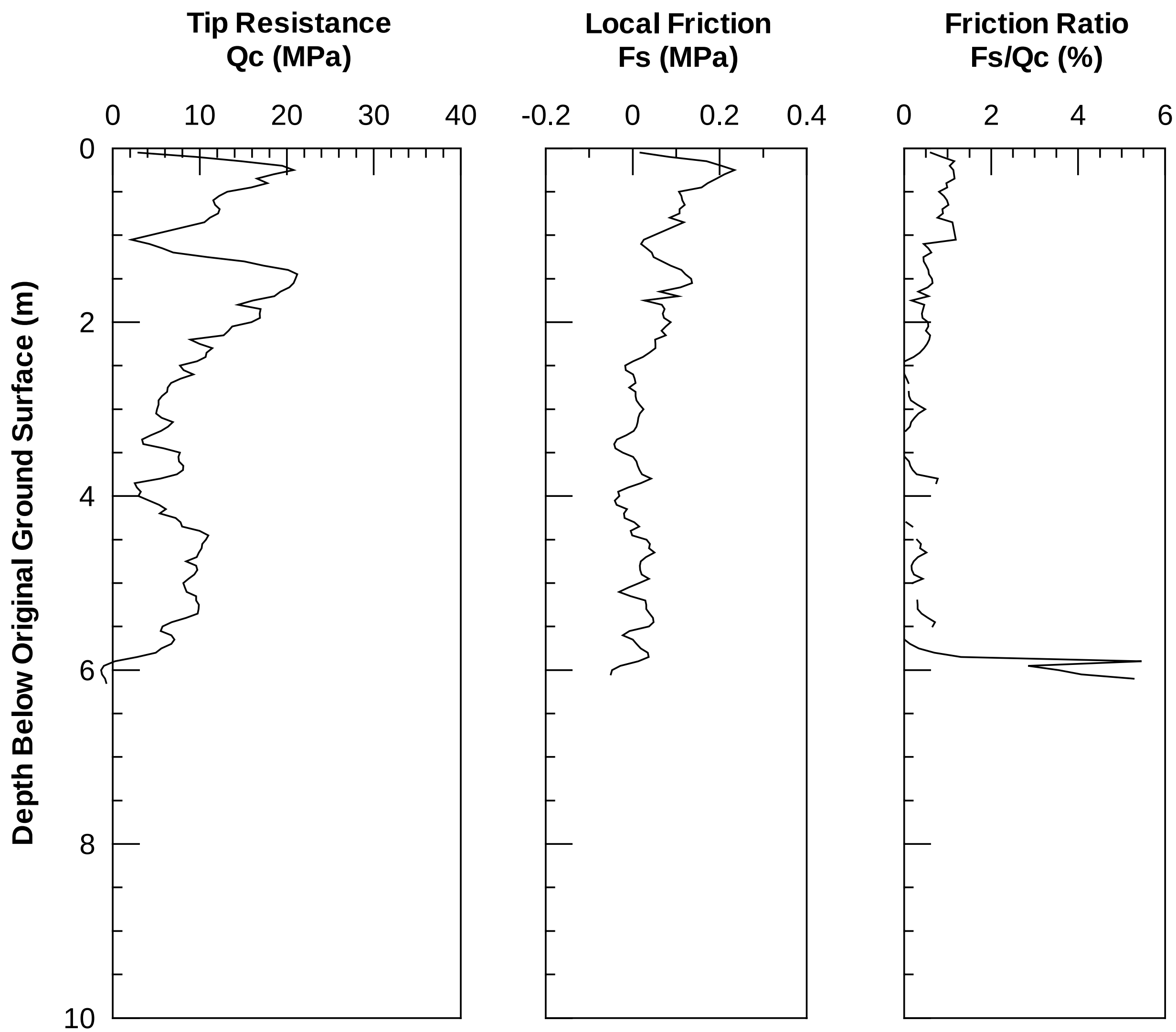


Figure 2.21 CPT SFC002 Logs for Pilot Liquefaction Study Area (6-24-99)

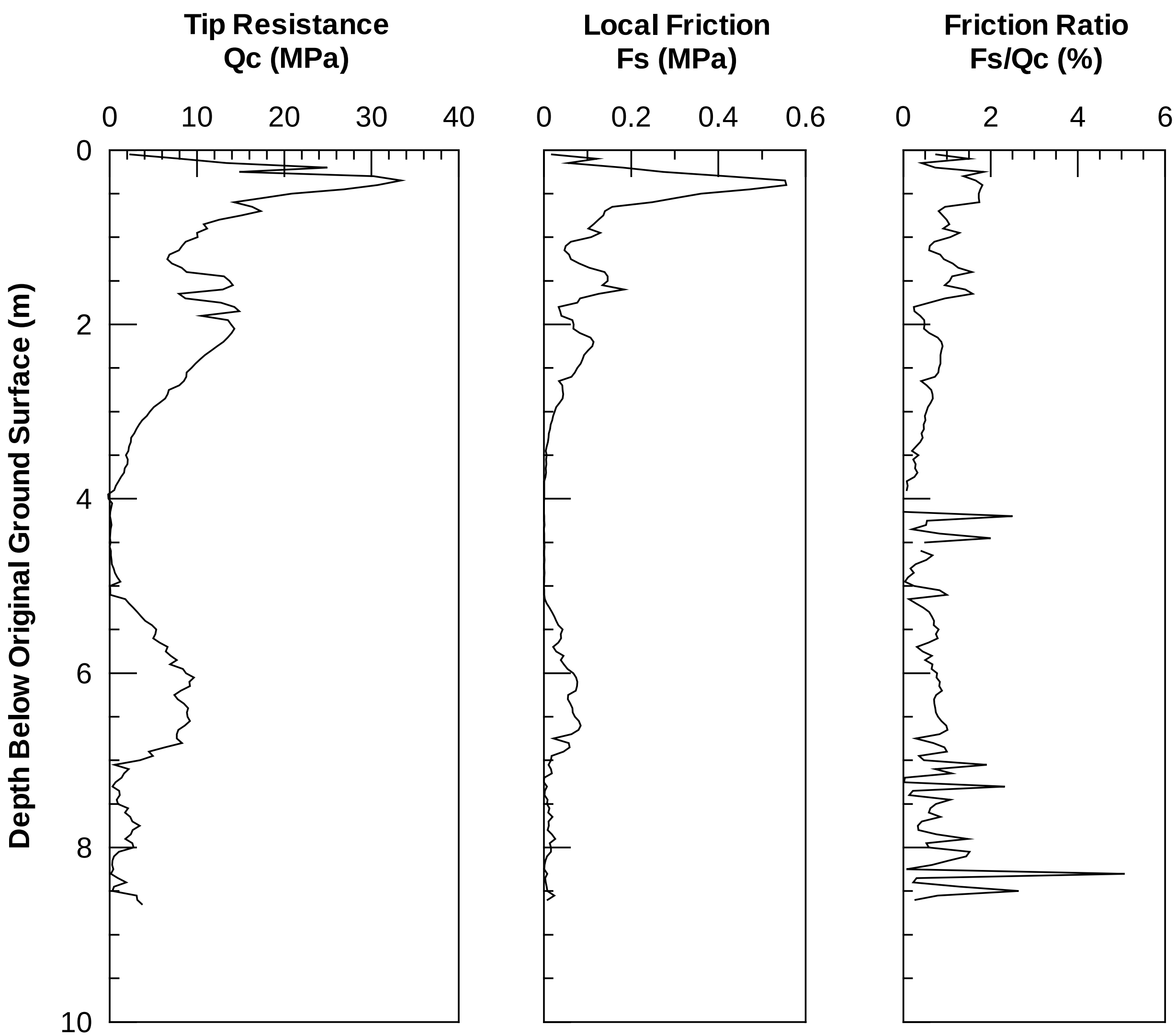


Figure 2.22 CPT SFC004 Logs for Pilot Liquefaction Study Area (6-24-99)

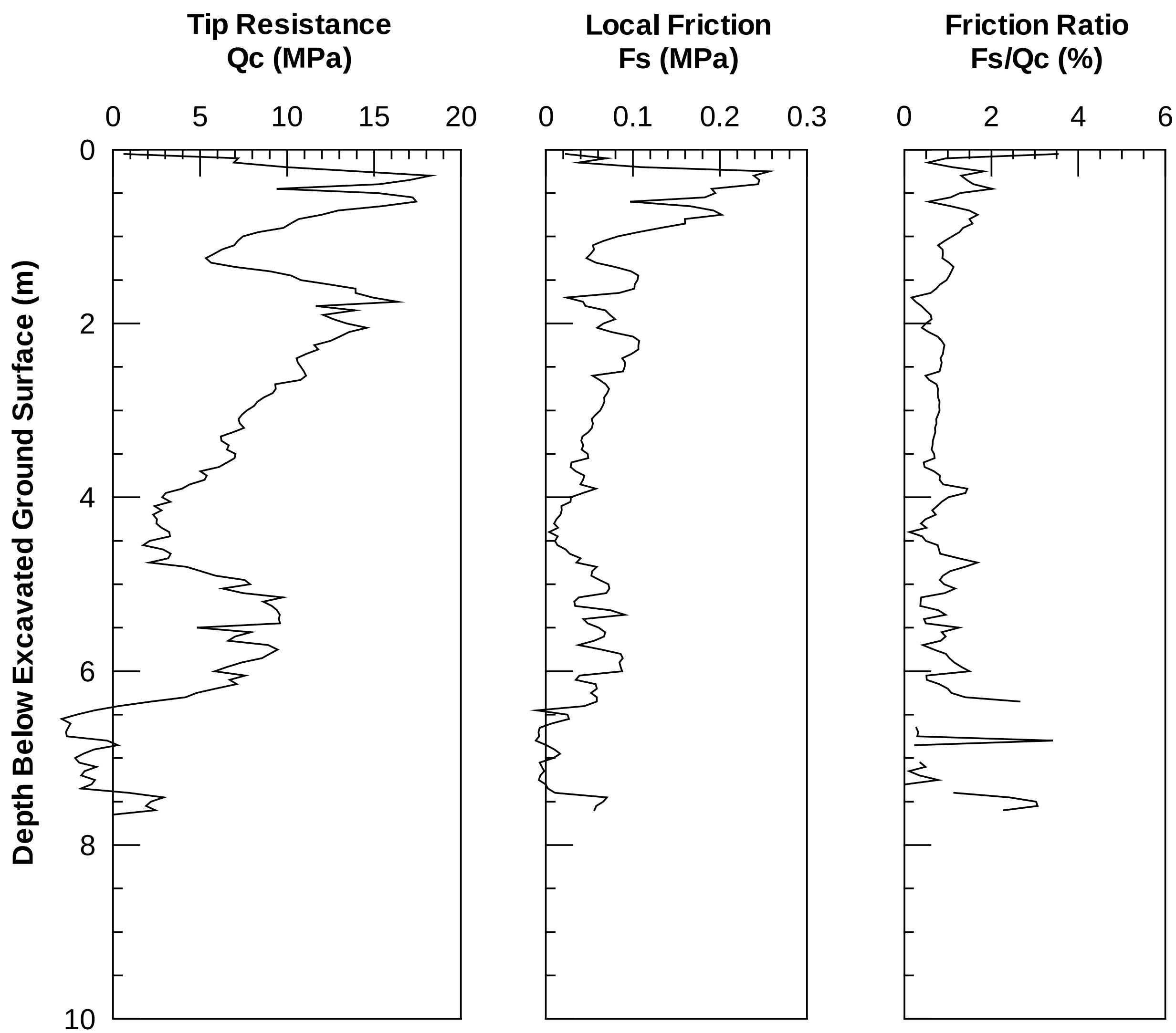


Figure 2.23 CPT SFC008 Logs for Single Pile Test Area 5 Months After 2nd Blast (6-24-99)

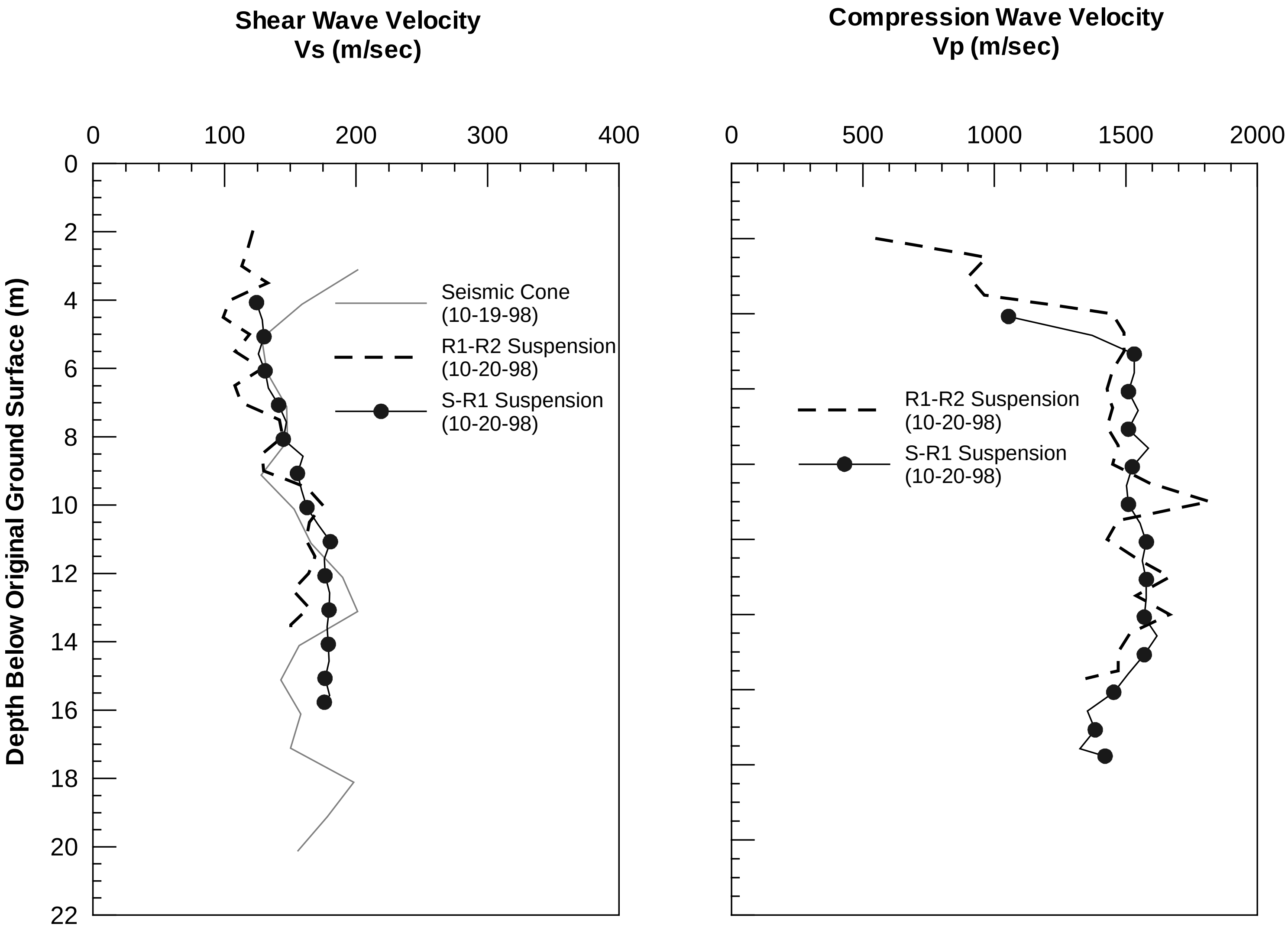


Figure 2.24 Pre-Blast Velocity Profile at Pilot Liquefaction Site a) Shear Wave b) Compression Wave

PPT	Depth
00	2.7m
01	0.9m
02	3.7m
03	4.6m
04	2.7m
05	0.9m
06	2.7m
87	4.6m
88	0.9m
89	2.7m
90	1.8m
91	5.5m
92	2.7m
93	2.7m
94	0.9m
95	0.9m
96	2.7m
97	2.7m
98	4.6m
99	2.7m

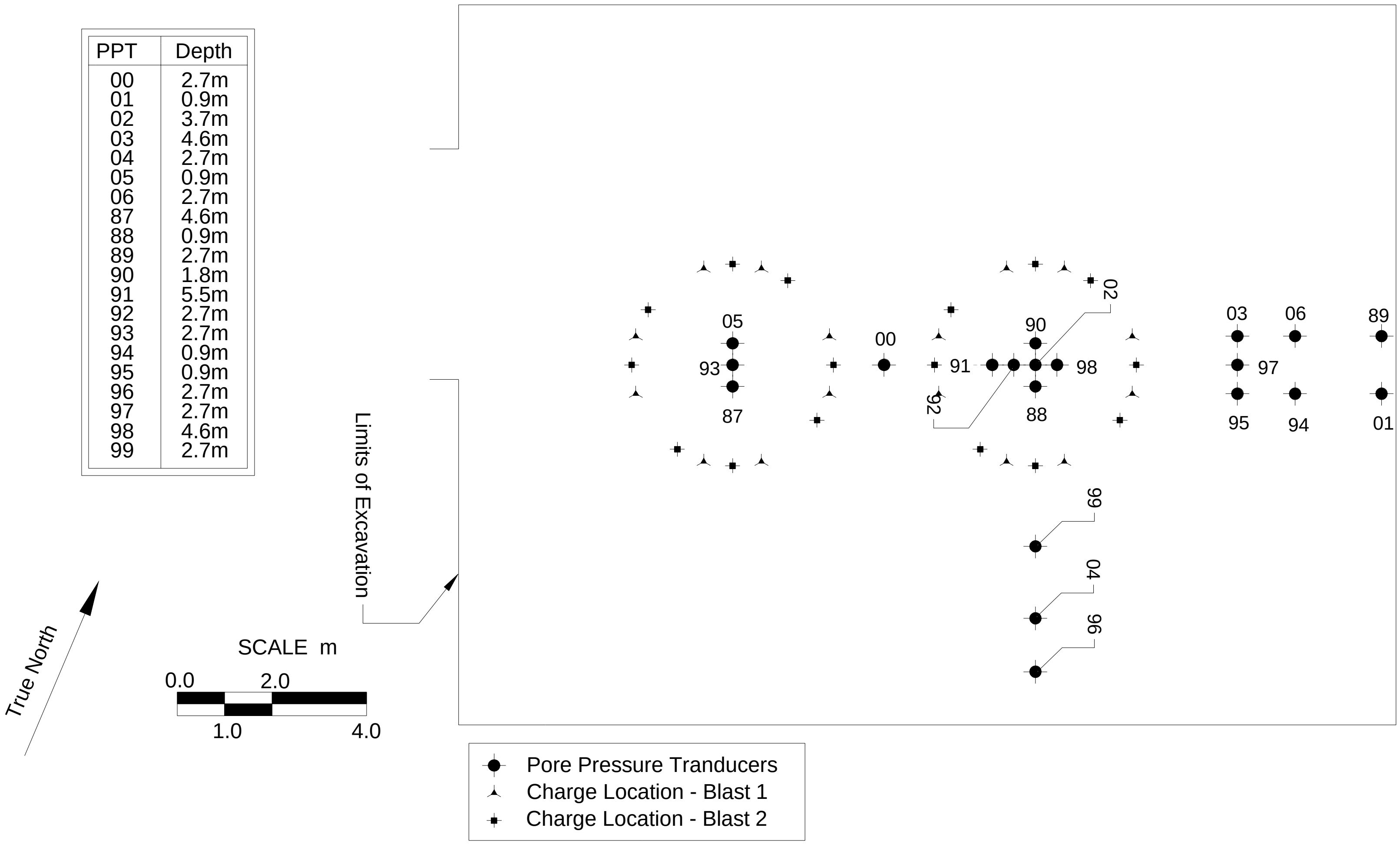


Figure 2.25 Location of Pore Pressure Transducers and Explosives for Pilot Liquefaction Test

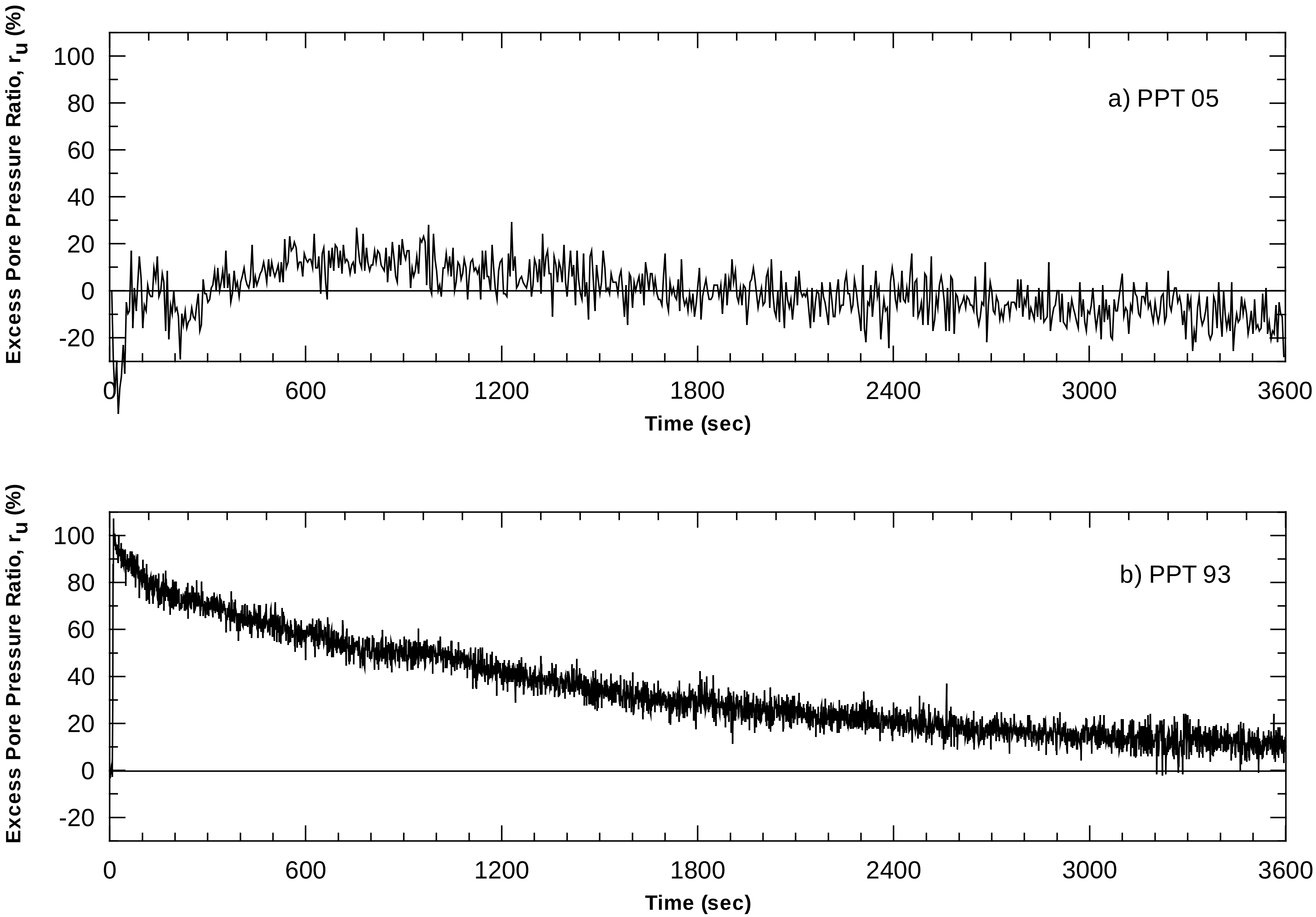


Figure 2.26 Excess Pore Pressure Ratio for Pilot Liquefacti on Test 1st Blast a) PPT 05 and b) PPT 93

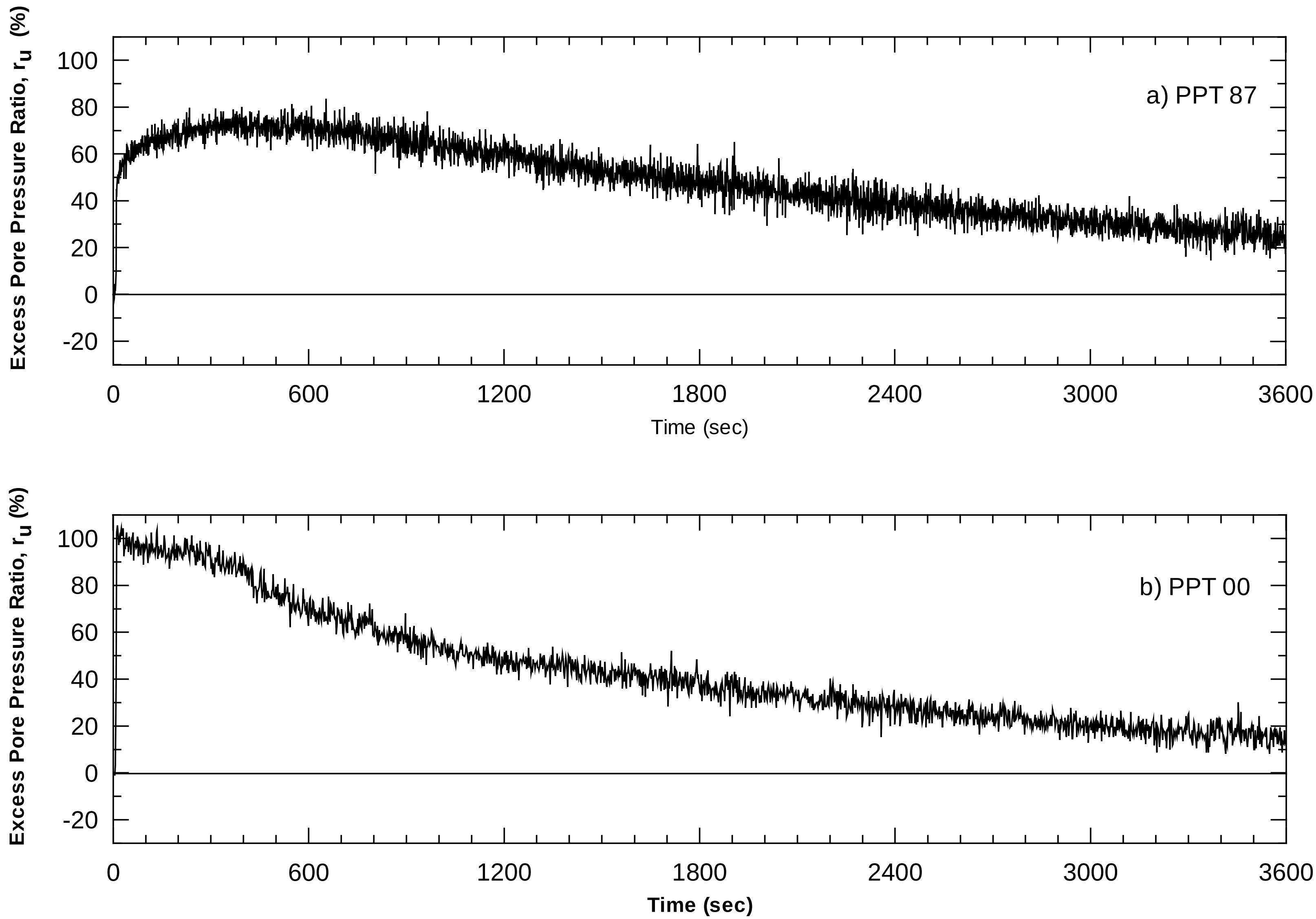


Figure 2.27 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 87 and b) PPT 00

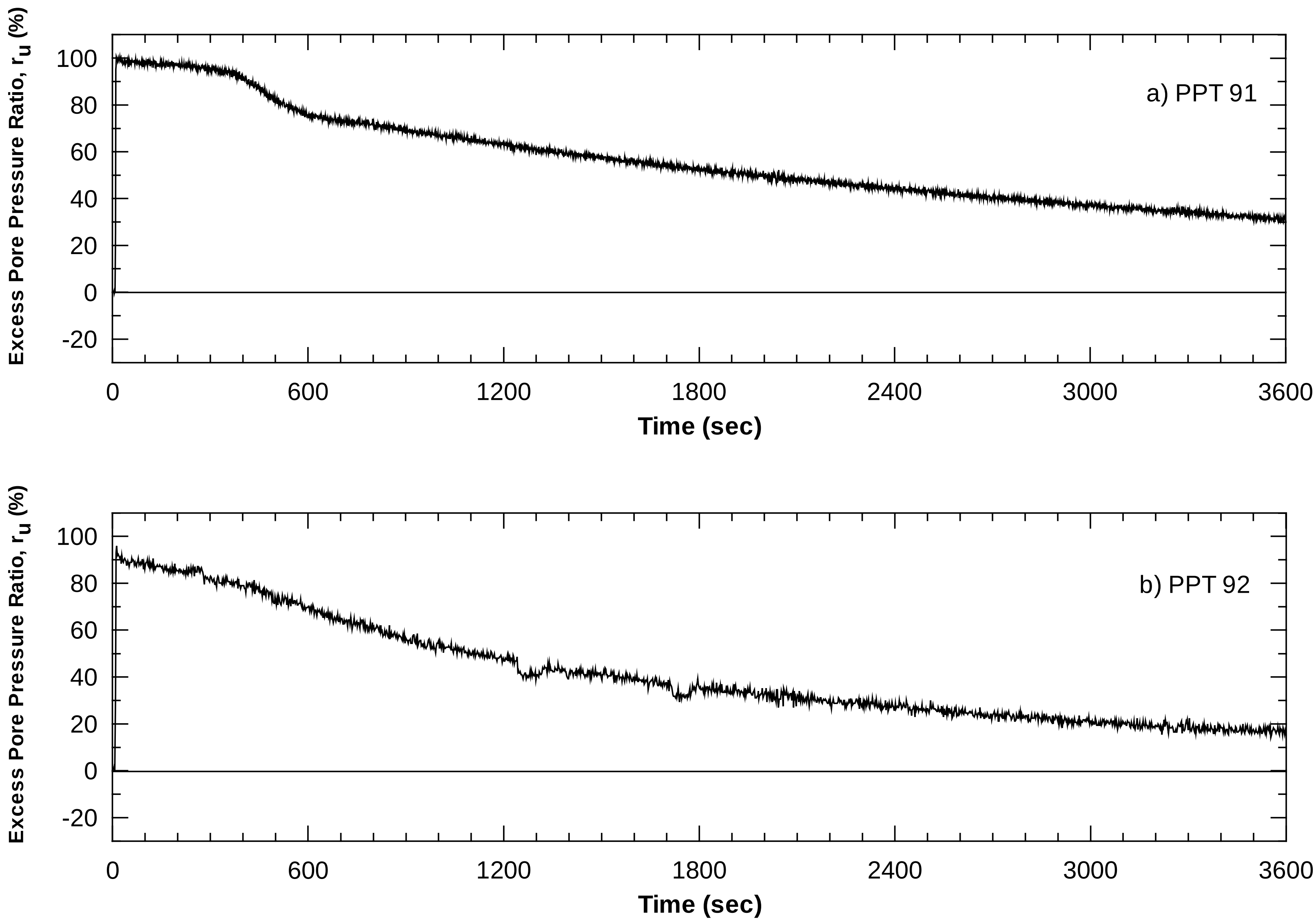


Figure 2.28 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 91 and b) PPT 9 2

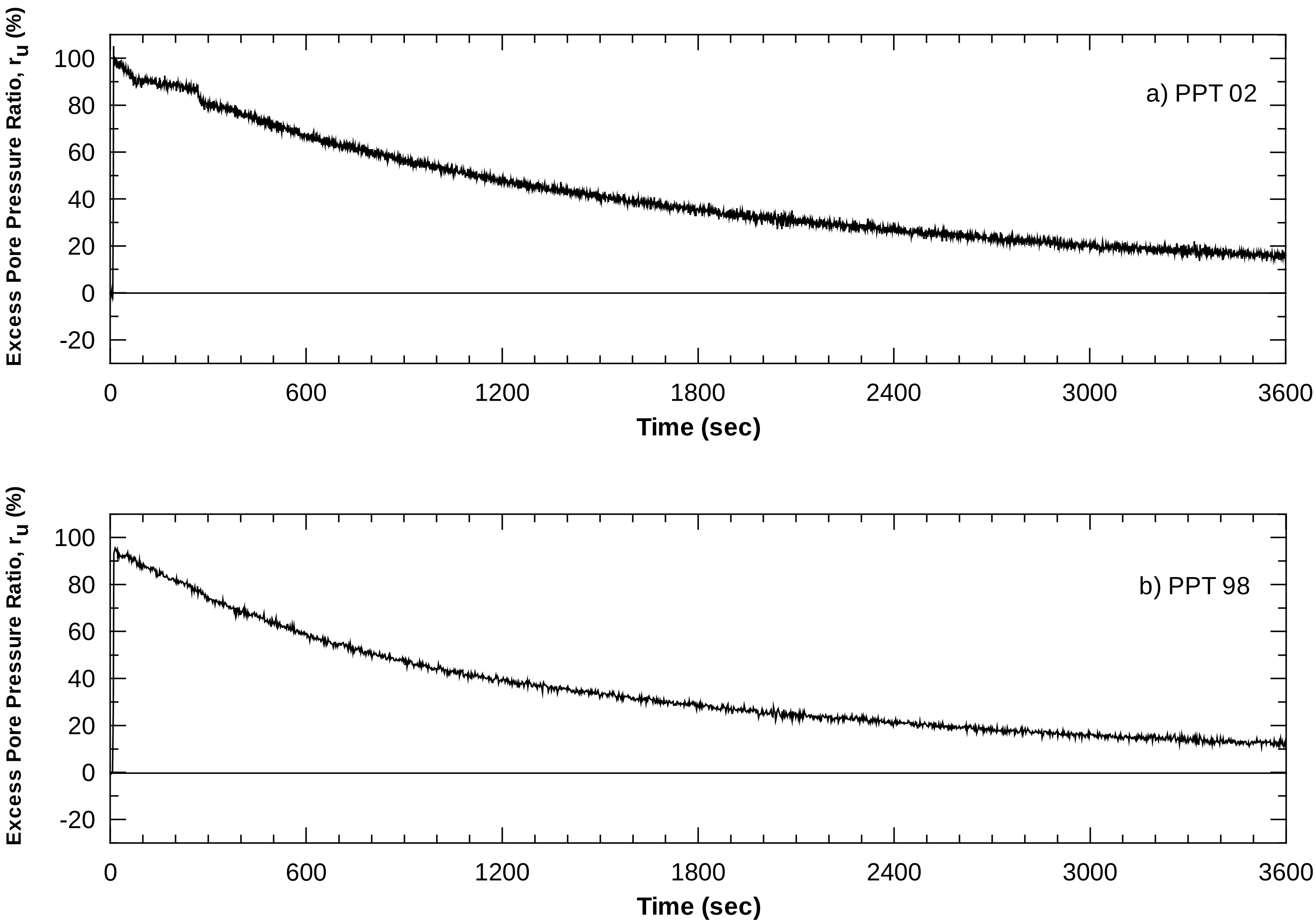


Figure 2.29 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 02 and b) PPT 98

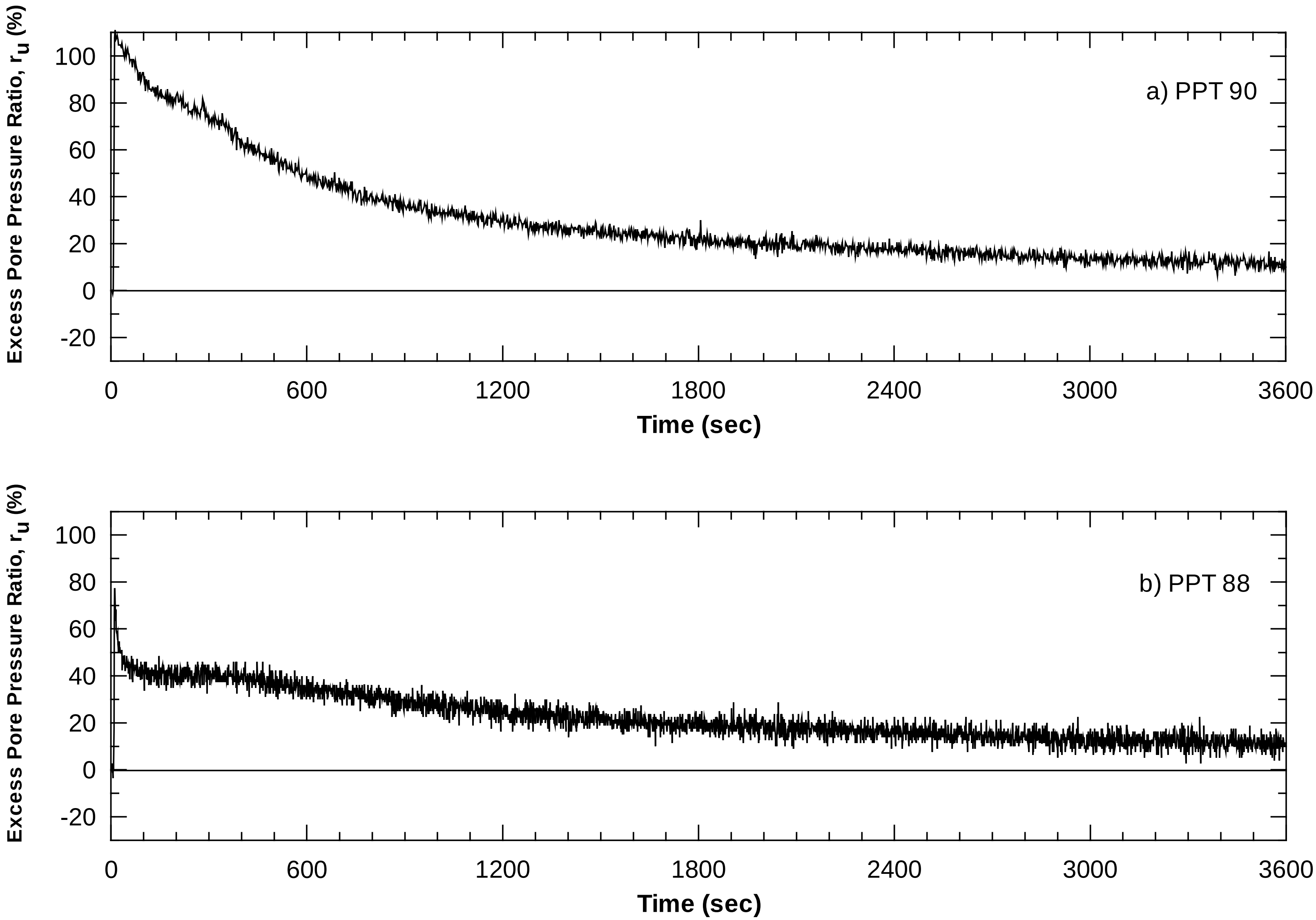


Figure 2.30 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 90 and b) PPT 88

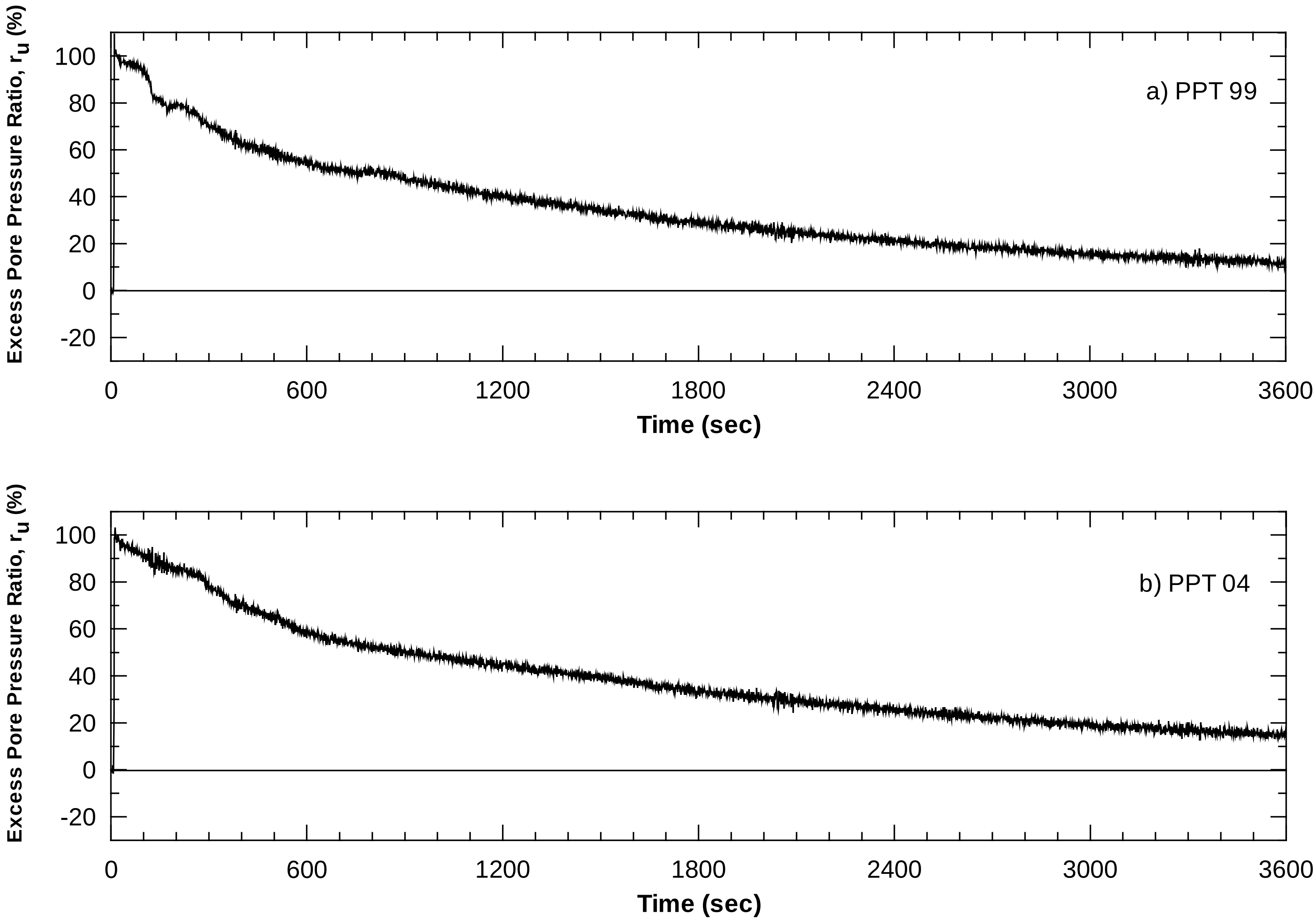


Figure 2.31 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 99 and b) PPT 04

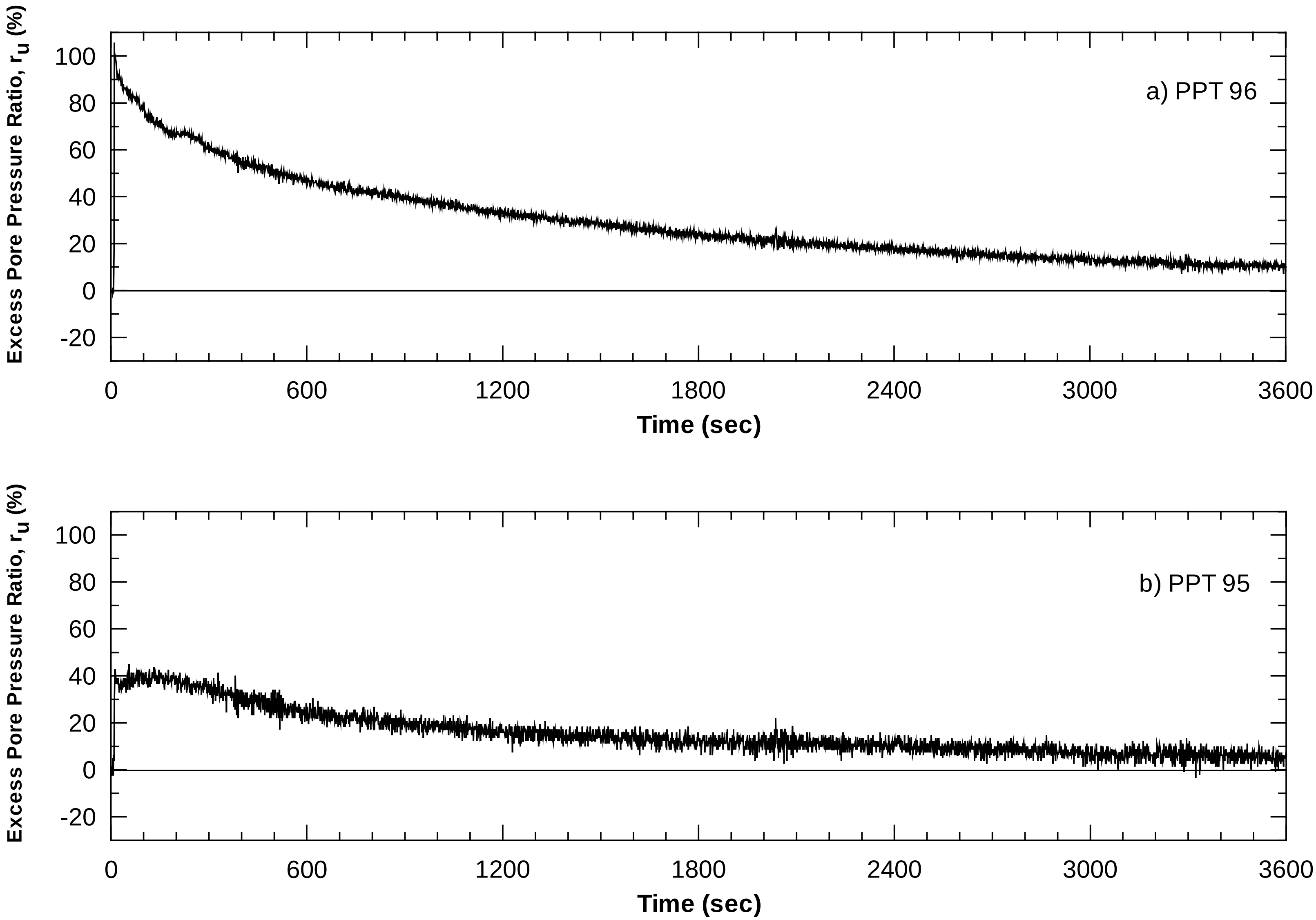


Figure 2.32 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 96 and b) PPT 95

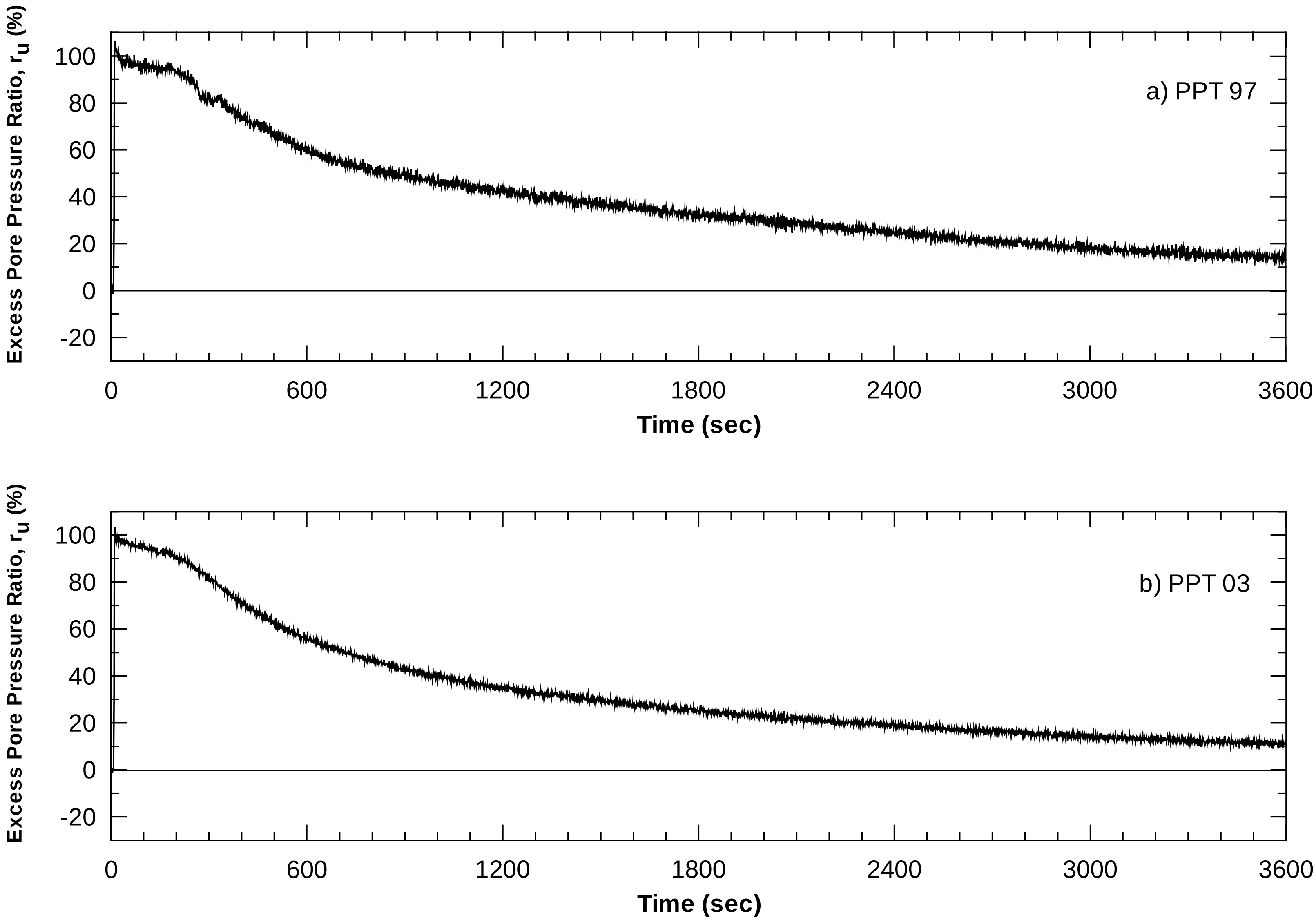


Figure 2.33 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 97 and b) PPT 03

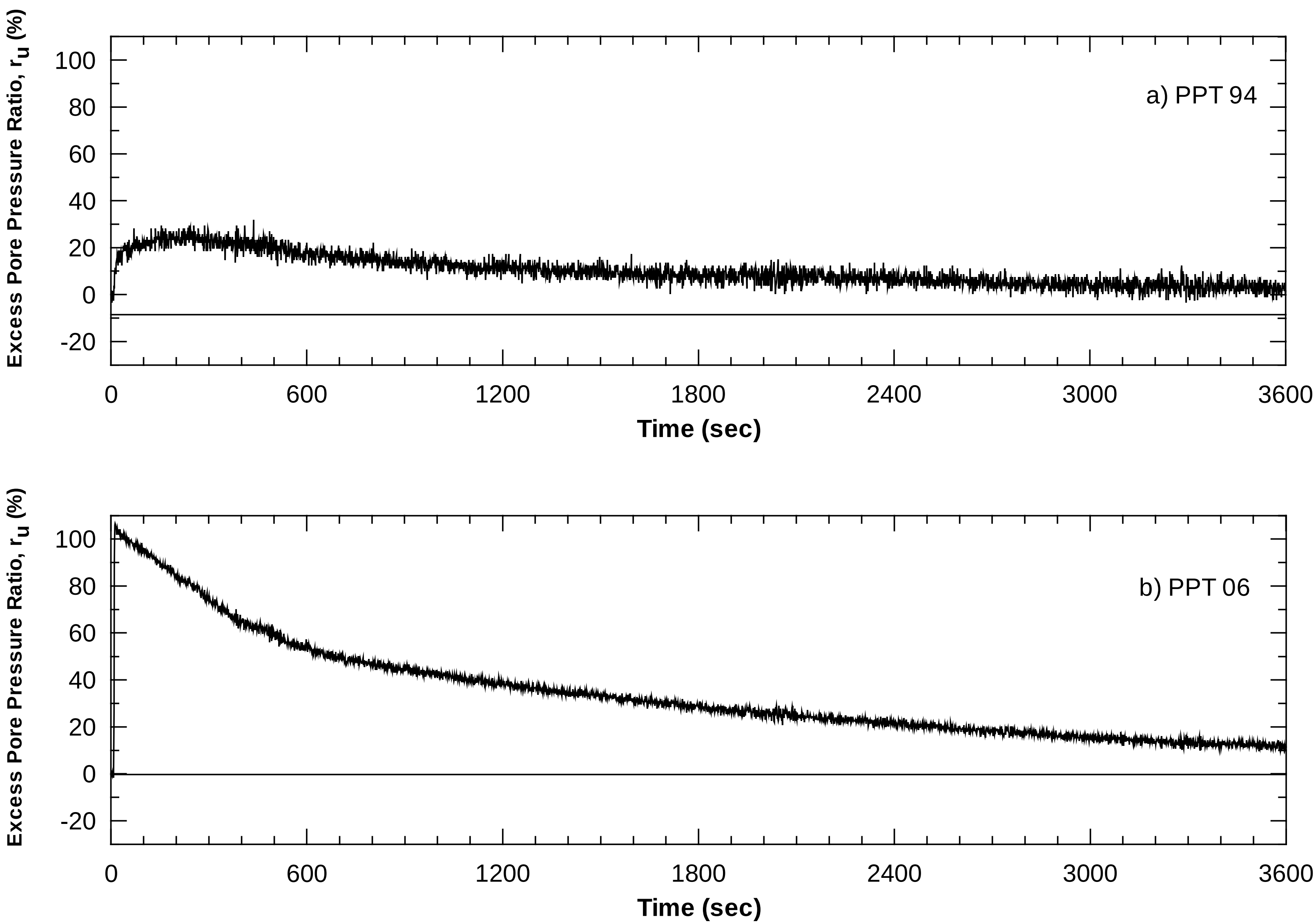


Figure 2.34 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 94 and b) PPT 06

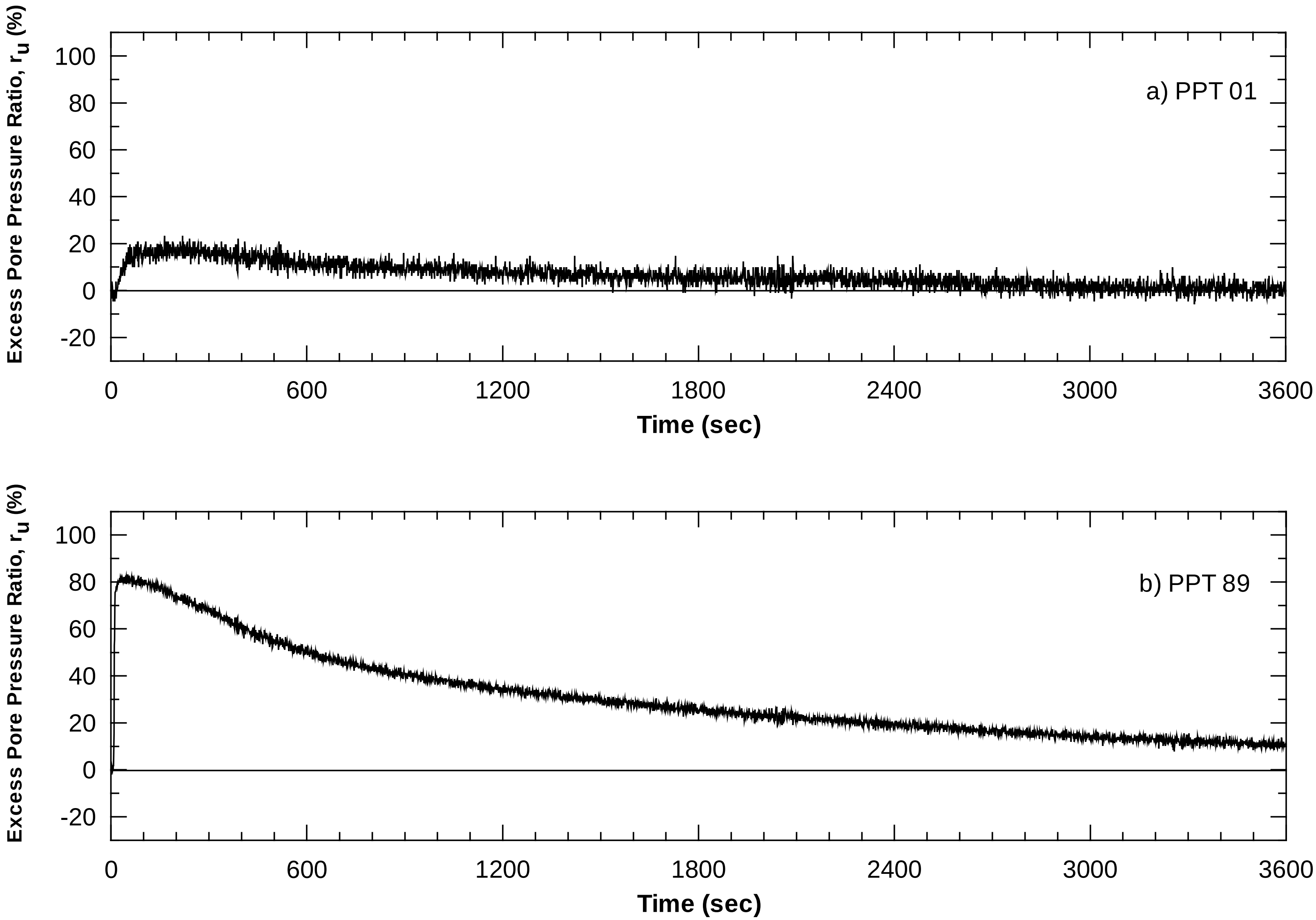


Figure 2.35 Excess Pore Pressure Ratio for Pilot Liquefaction Test 1st Blast a) PPT 01 and b) PPT 89

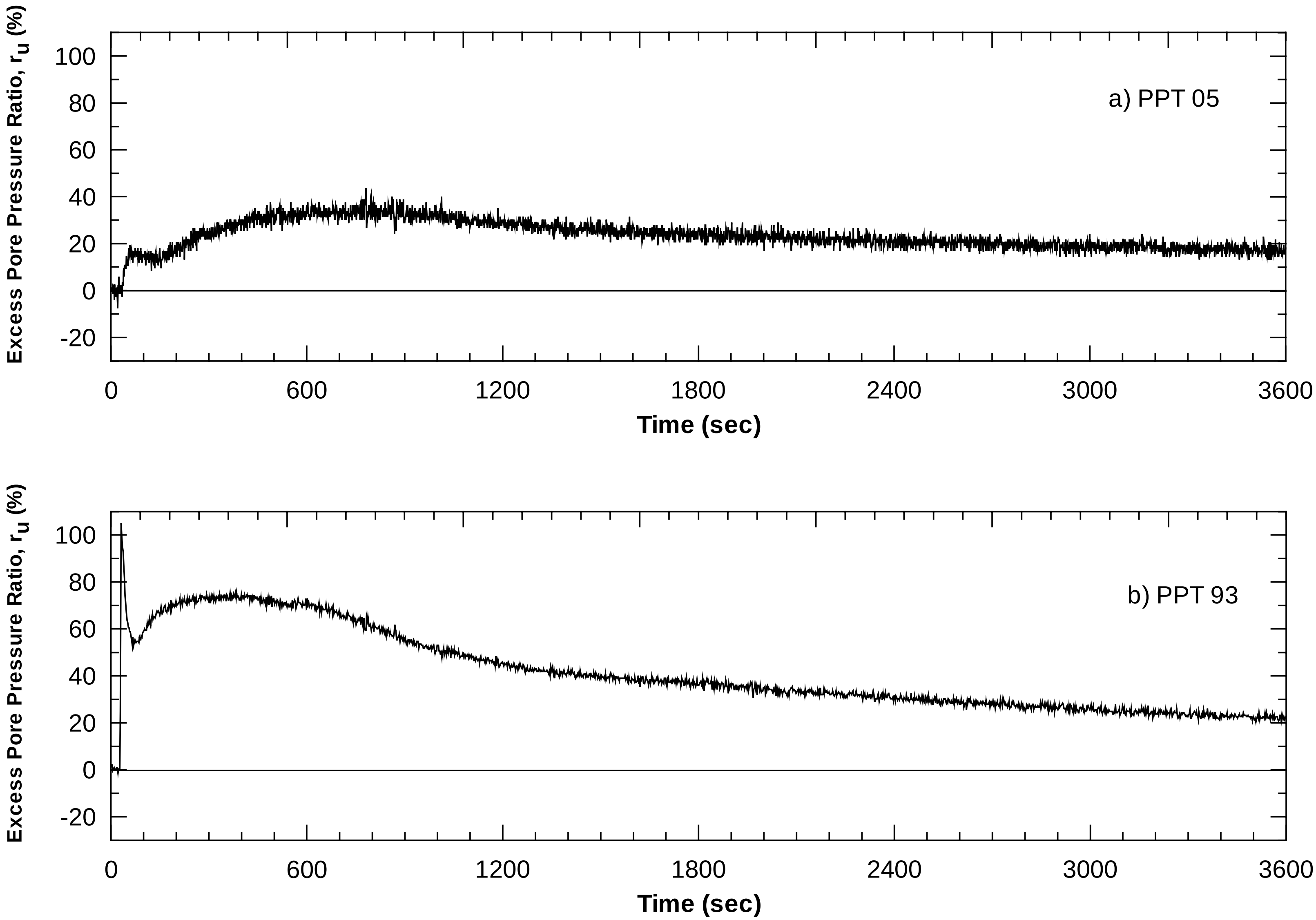


Figure 2.36 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 05 and b) PPT 93

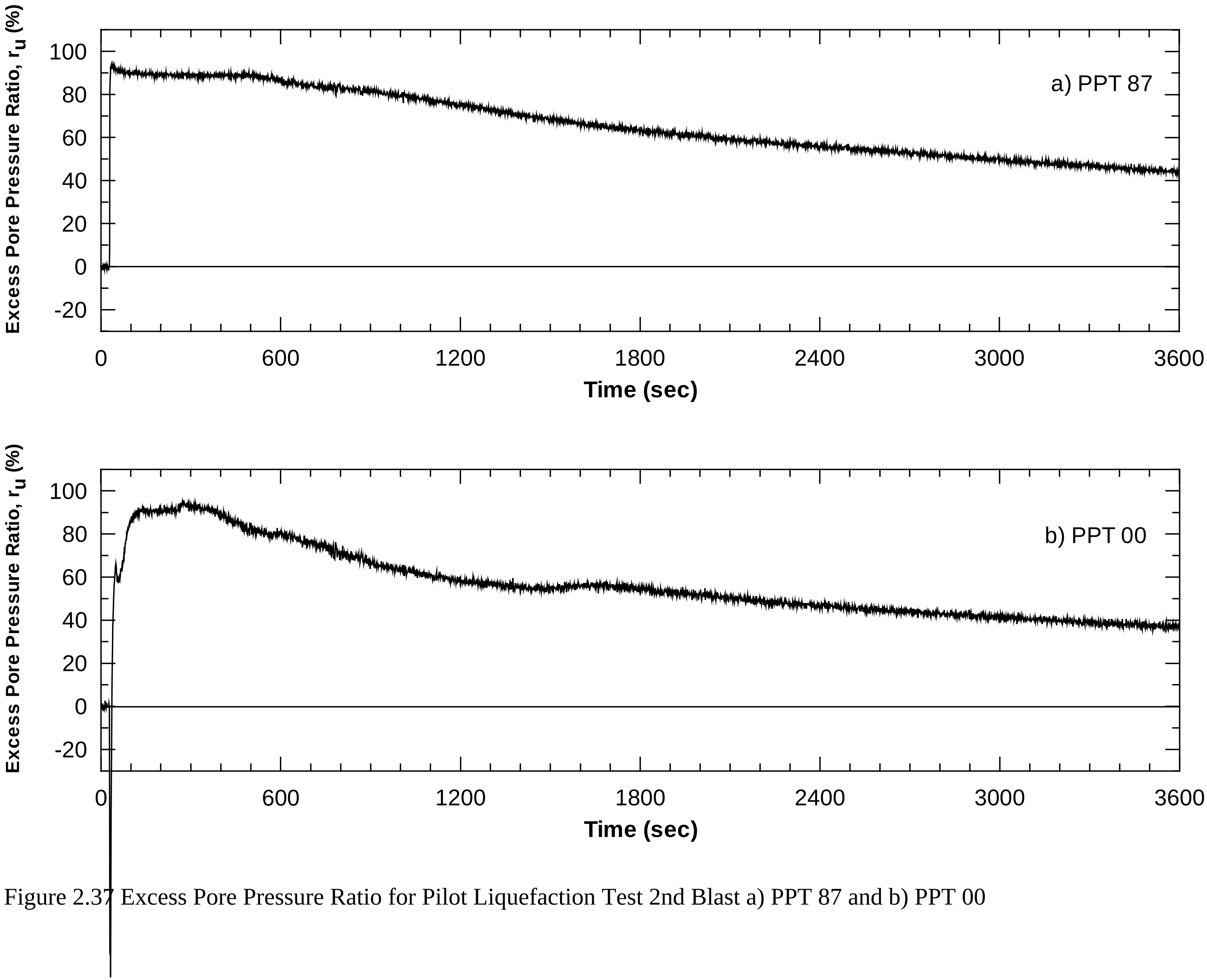


Figure 2.37 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 87 and b) PPT 00

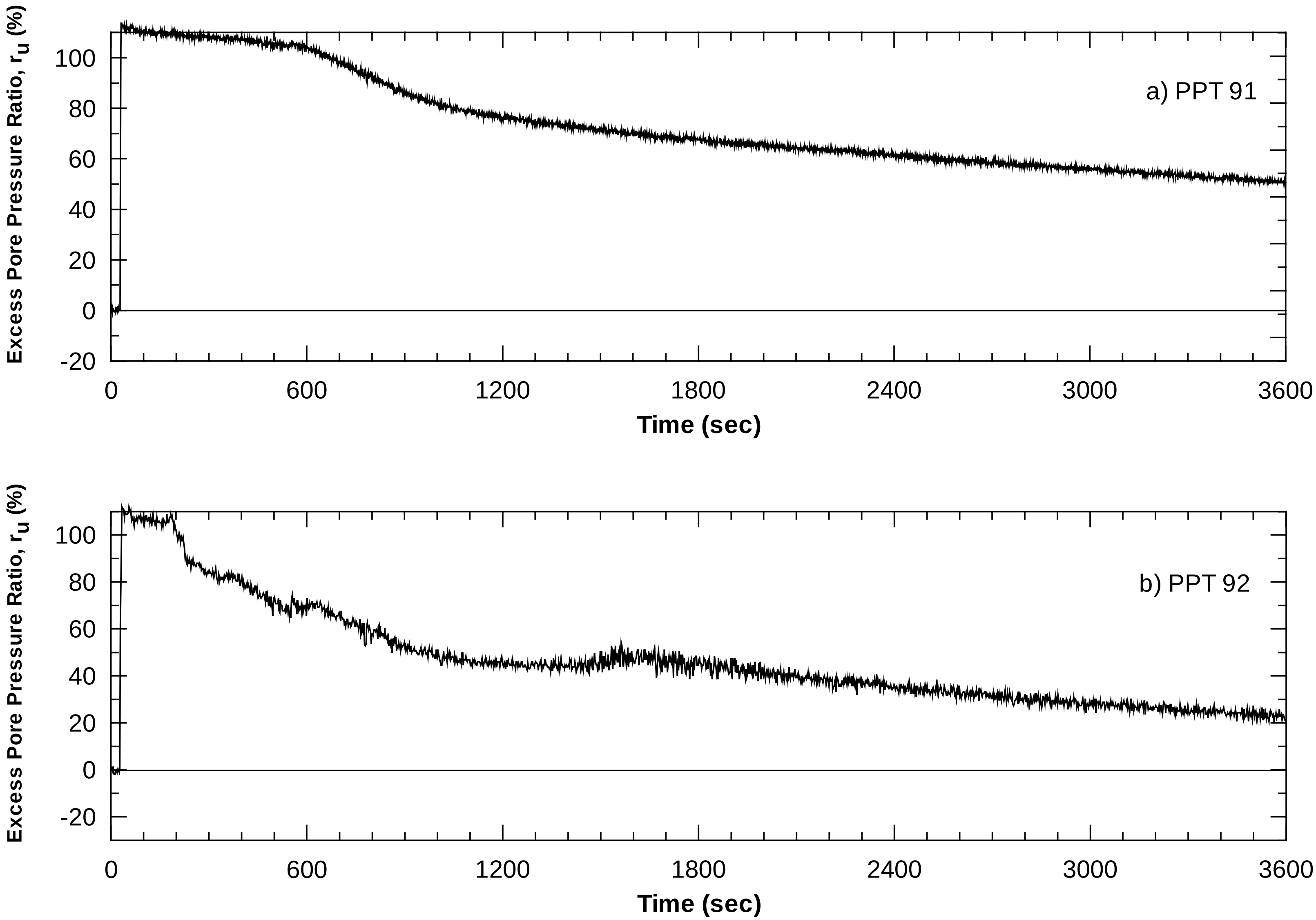


Figure 2.38 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 91 and b) PPT 92

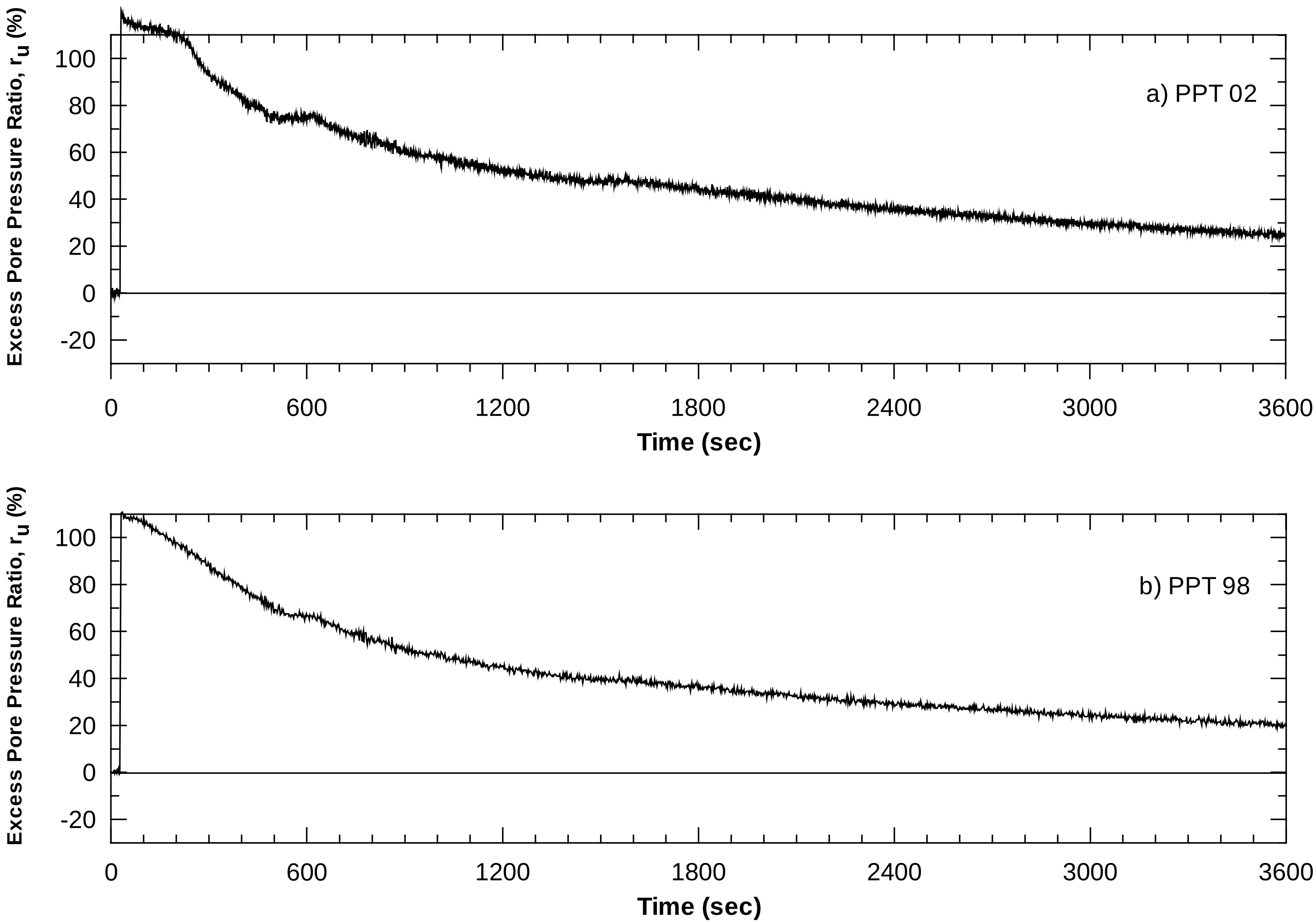


Figure 2.39 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 02 and b) PPT 98

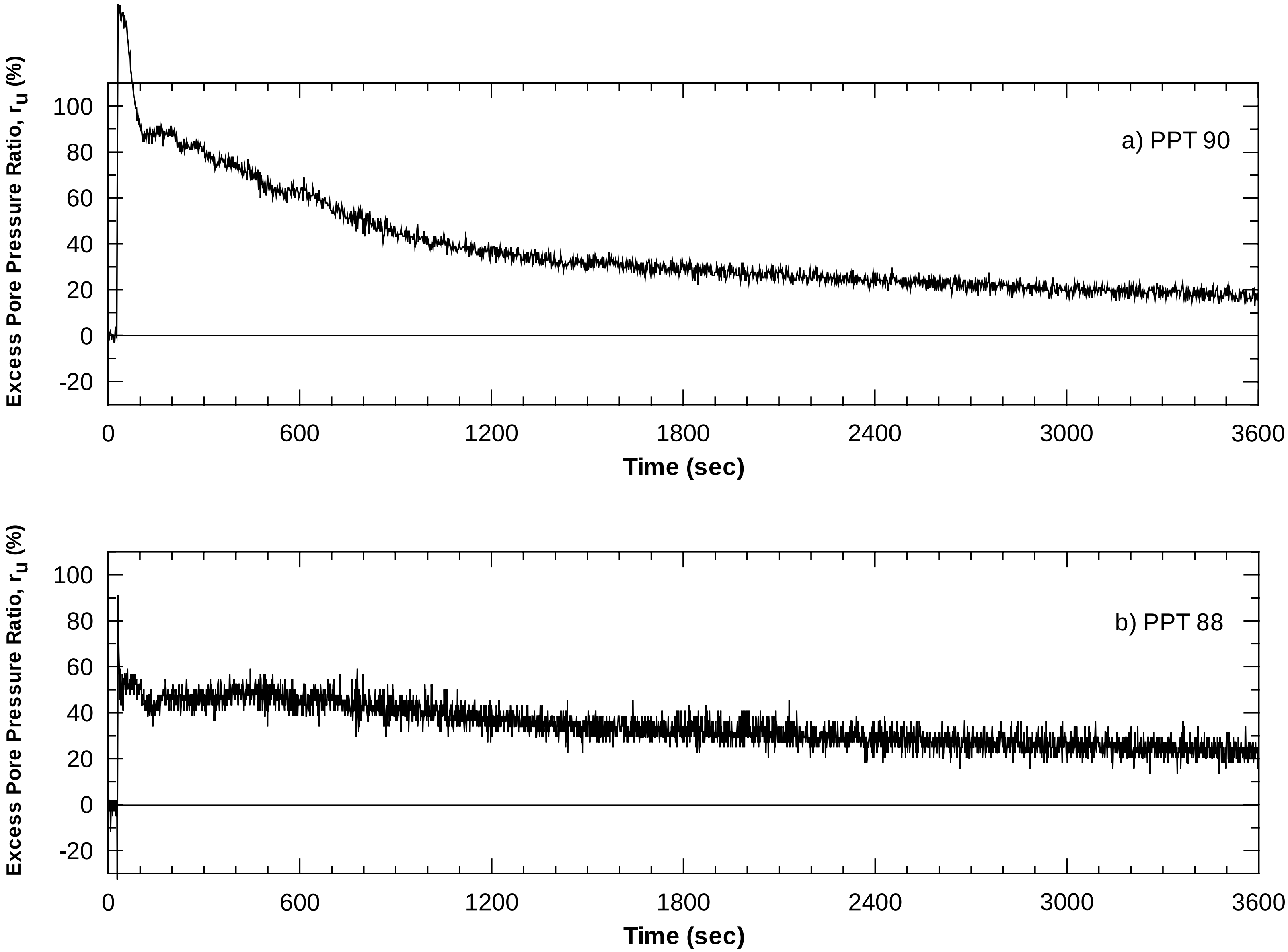


Figure 2.40 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 90 and b) PPT 88

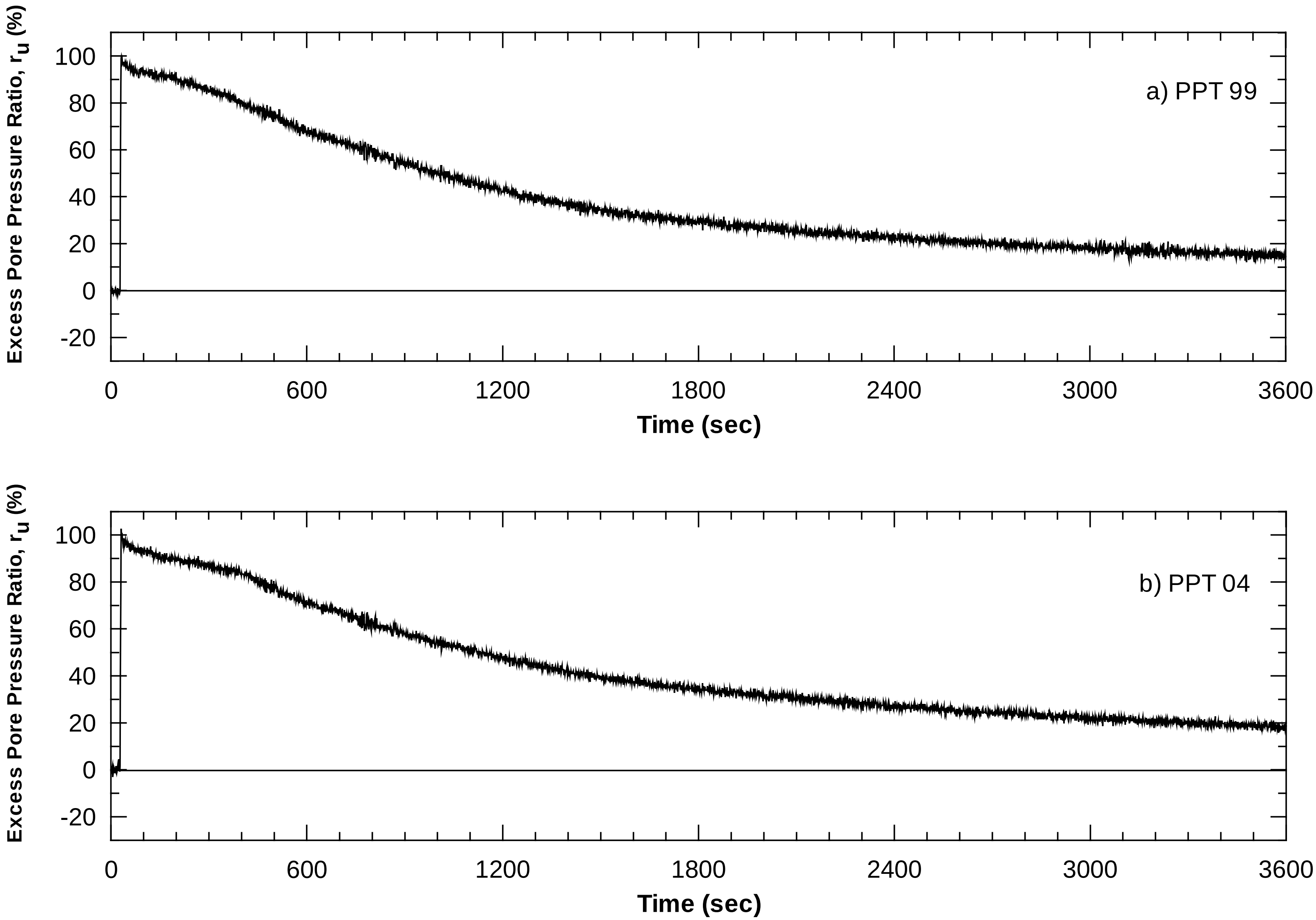


Figure 2.41 Excess Pore PressureRatio for Pilot Liquefaction Test 2nd Blast a) PPT 99 and b) PPT 04

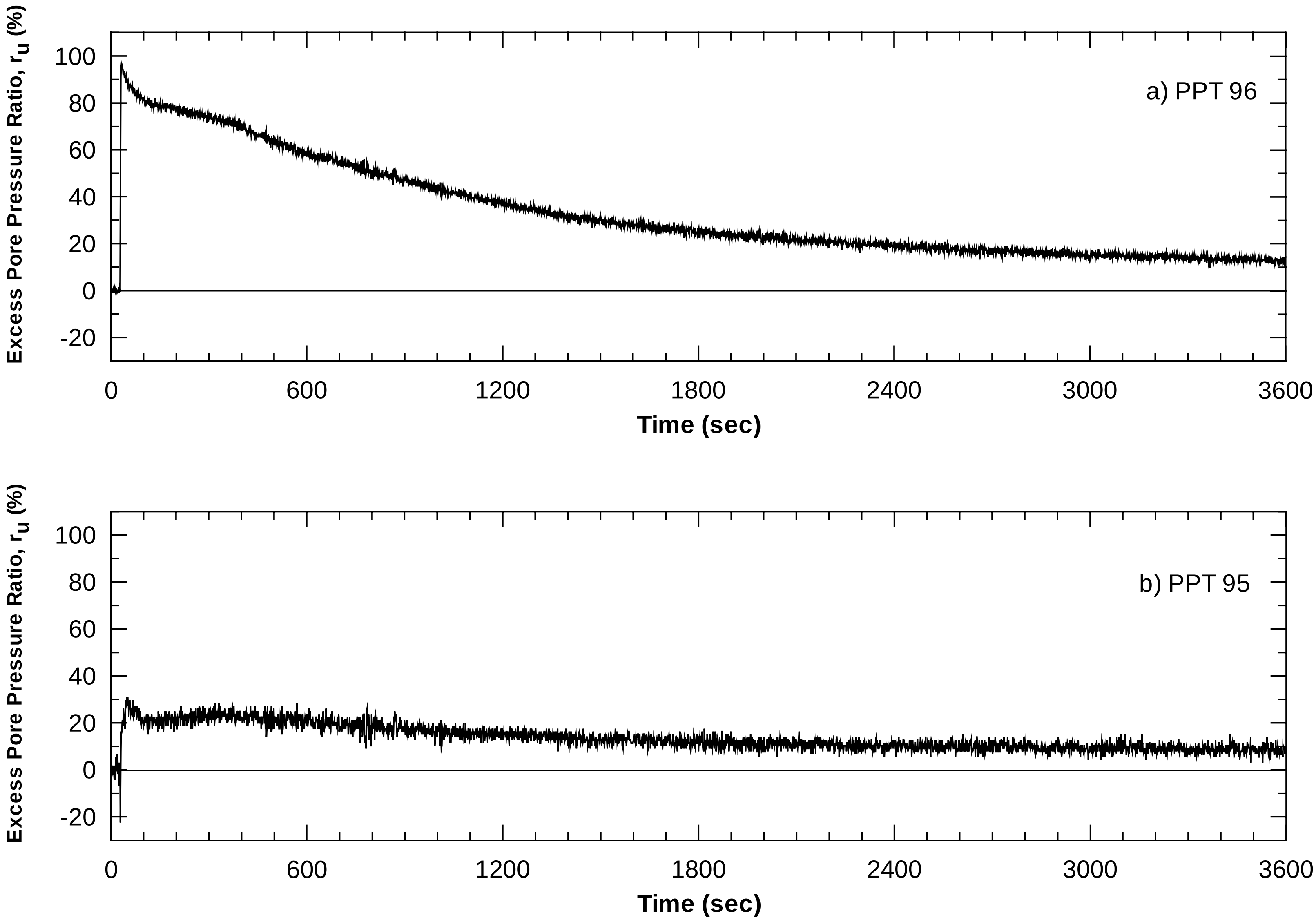


Figure 2.42 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 96 and b) PPT 95

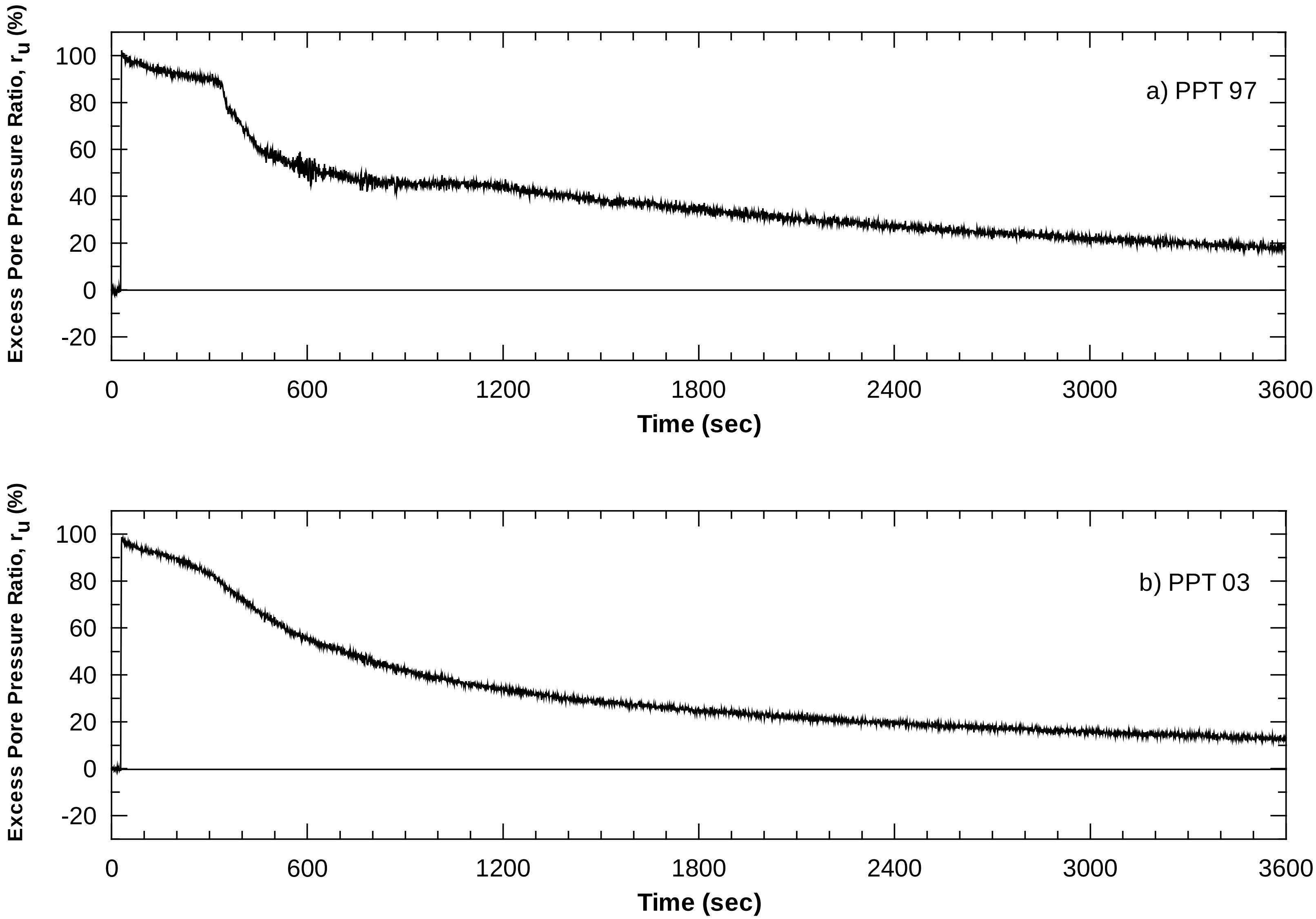


Figure 2.43 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 97 and b) PPT 03

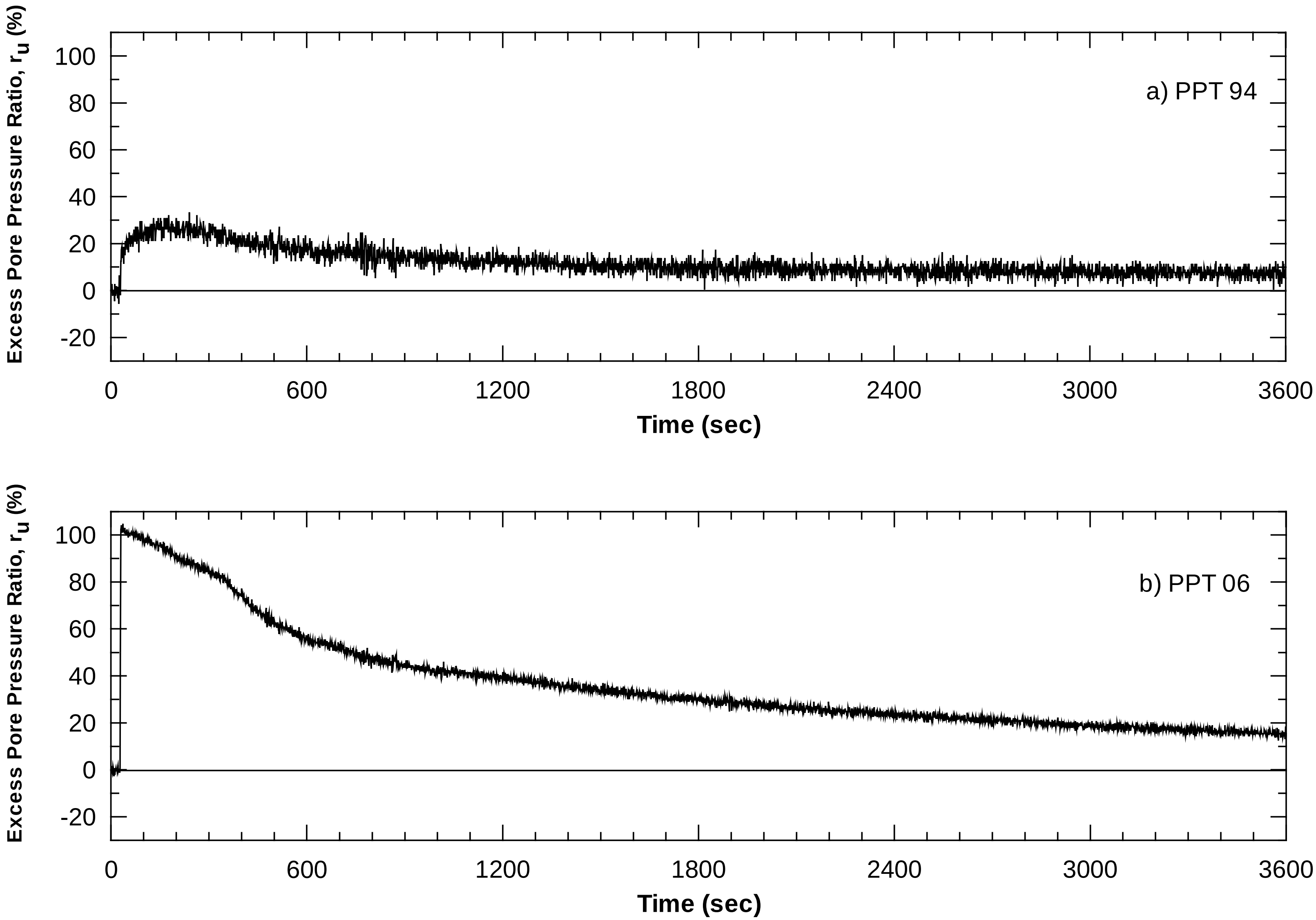


Figure 2.44 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 94 and b) PPT 06

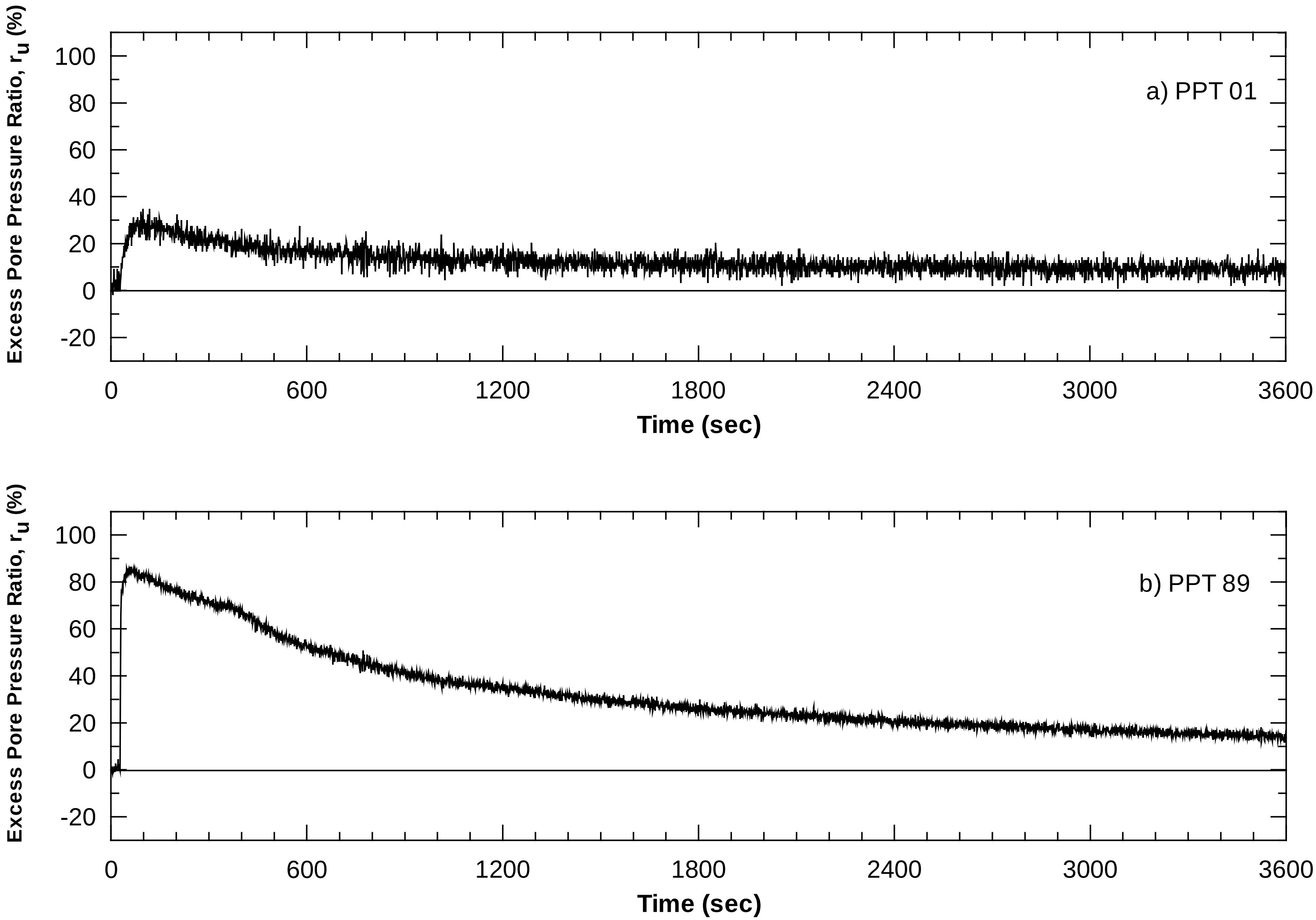


Figure 2.45 Excess Pore Pressure Ratio for Pilot Liquefaction Test 2nd Blast a) PPT 01 and b) PPT 89

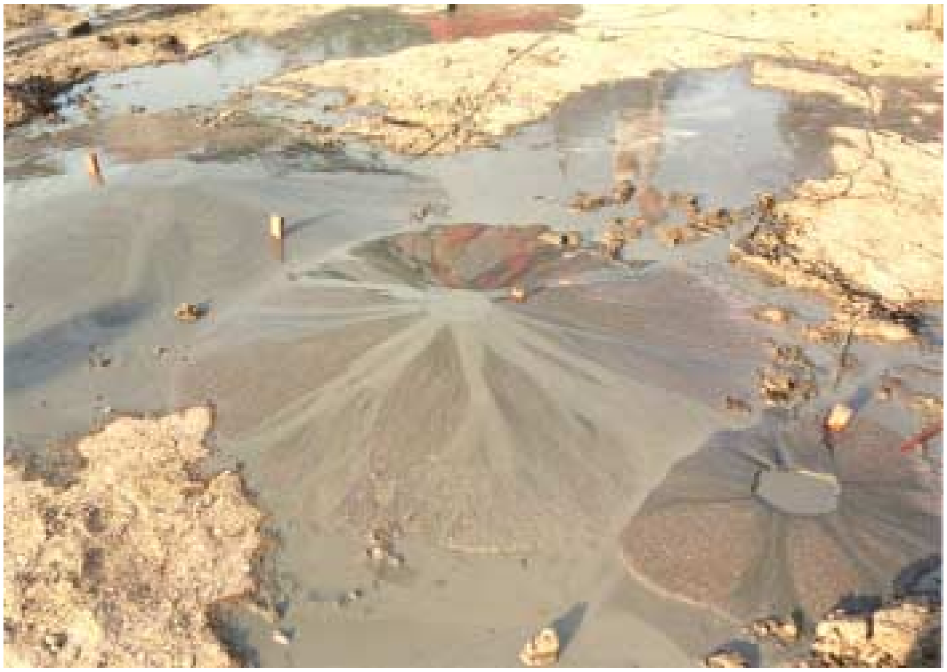


Figure 2.46 Formation of Sand Boils During Pilot Liquefaction Study

Figure 2.47 Settlement Survey after Pilot Liquefaction Test

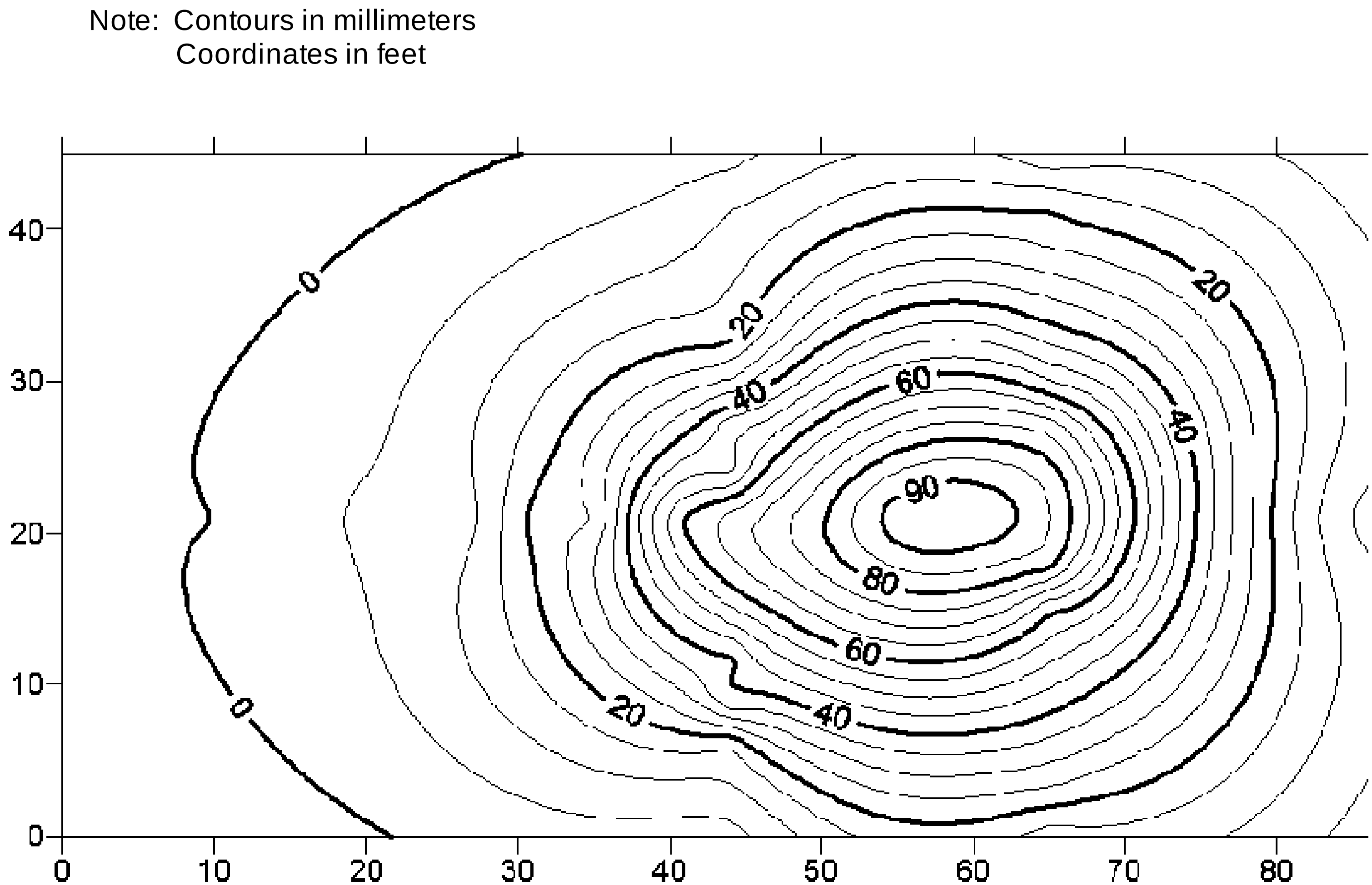


Figure 2.48 Settlement t Contours for 1st Blast of Pilot Liquefaction Study

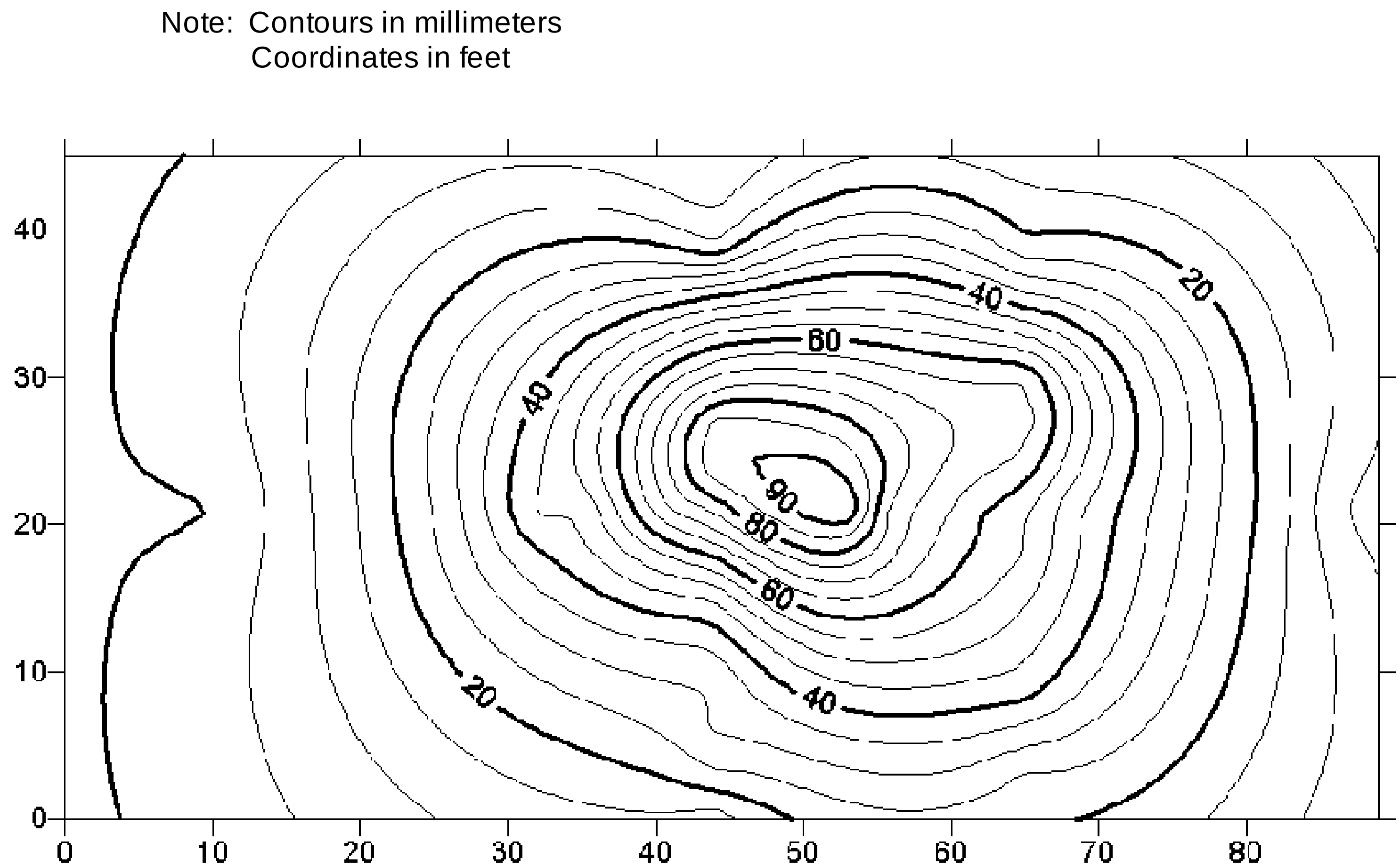


Figure 2.49 Settlement Contours for 2nd Blast of Pilot Liquefaction Study

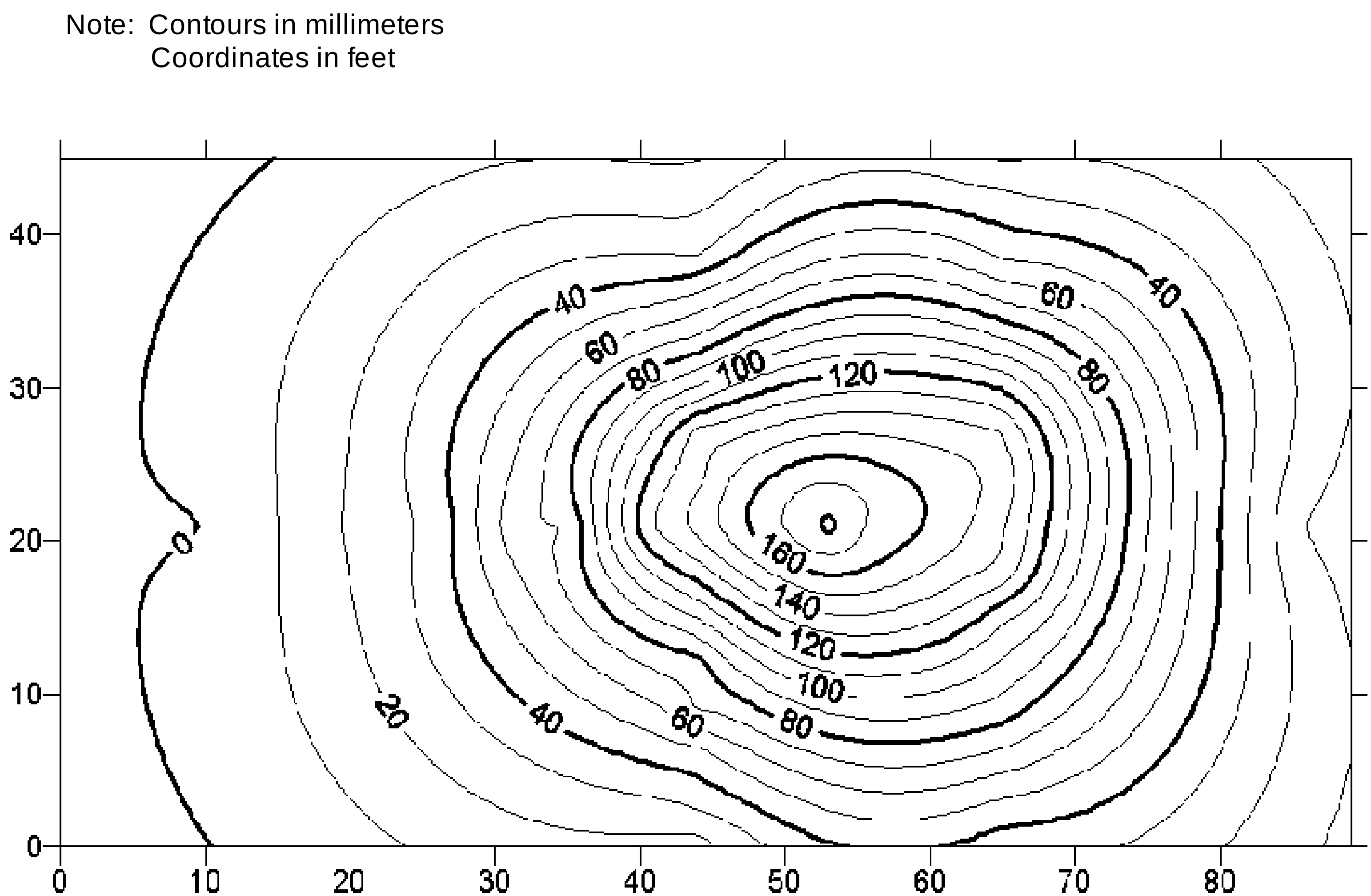


Figure 2.50 Combined Settlement Contours for 1st and 2nd Blast of Pilot Liquefaction Study

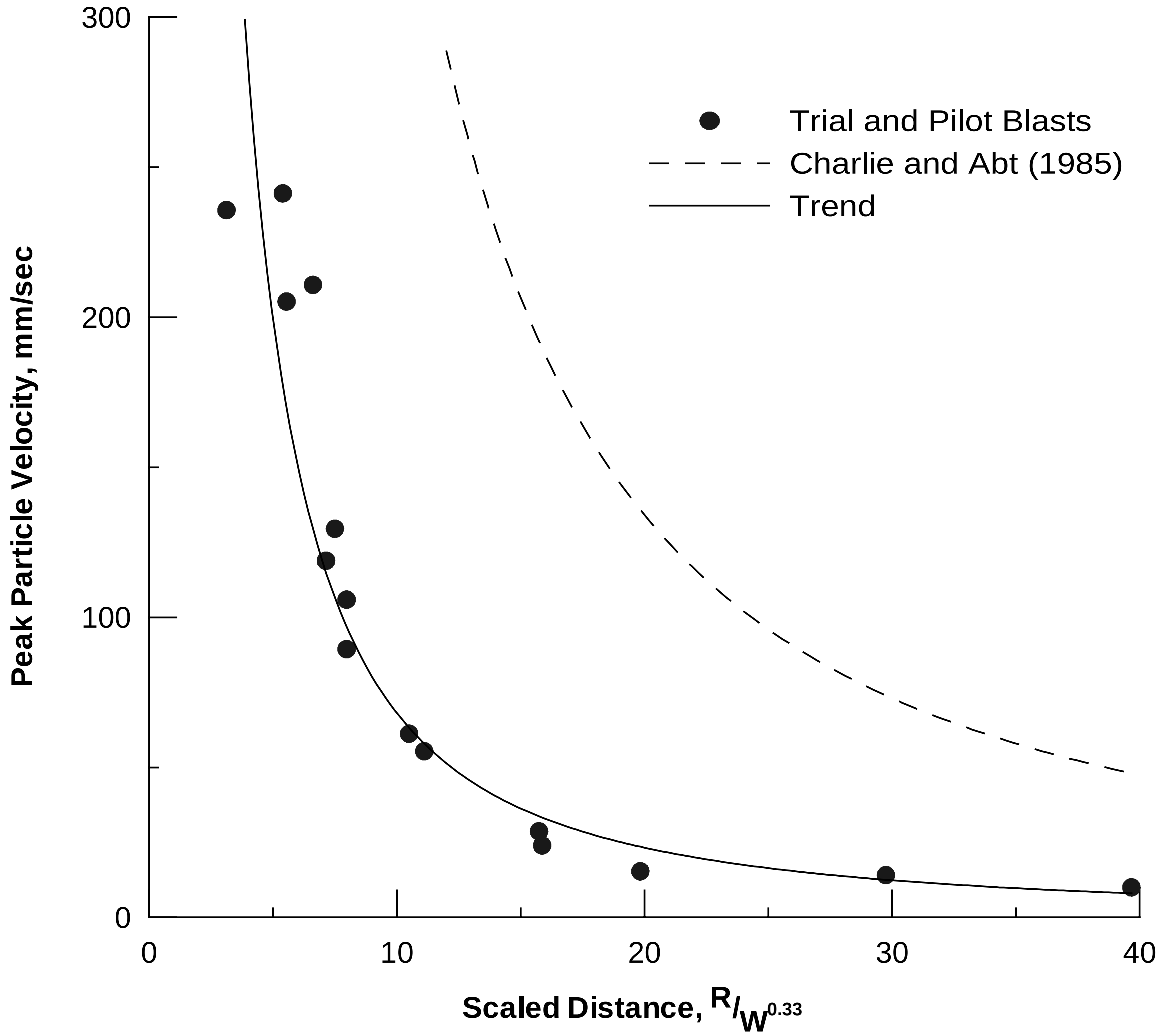


Figure 2.51 Peak Particle Velocity vs. Scaled Distance

3 SINGLE PILE TESTS

The single pile tests investigated the lateral response of a steel pipe pile (0.32-m O.D.) and an H-Pile (HP 12x53). All piles were driven through the sand layer and well into the underlying clay layer to ensure that they performed as “long piles”. Though they would have been driven deeper to a bearing layer on a real bridge, vertical capacity was not a concern for the lateral load testing. In all cases, driving was very easy. The single pile tests were the first ever full-scale lateral load field tests of piles in liquefied soil. Results of the single pile test will be valuable in verifying the validity of model tests performed in the laboratory. The single pile tests also provided baseline information needed to assess the performance of the pile groups and large diameter CISS piles tested at Treasure Island. In this chapter, data from the subsurface investigation and in-situ testing for the single pile tests are presented along with load and displacement test results, excess pore pressure ratios, and settlement contours resulting from blasting.

3.1 Single Pile Site Characterization

A comprehensive field exploration was carried out before any foundation testing was performed. Figure 3.1 shows the location of in-situ tests at the single pile test site. The soil boring log for BH-4 is presented in Figure 3.2. The boring logs from the single pile site show the subsurface conditions as consisting of relatively loose, poorly graded sand from the ground surface to a depth of 9.1 meters. A soft clay layer was observed at a depth of 9.1 meters and extended to the bottom of the boring at 18.3 meters. The water table at the time of the boring was 1.7 meters below the original ground surface. Low

SPT blow counts indicated that the sand below the water table was liquefiable. Six CPT's performed before foundation testing are shown in Figures 3.3 through 3.8. Cone penetration logs generally confirmed SPT results, showing low tip resistance of between 5 and 10 MPa in the sand deposit indicating a relatively low density. An additional six cone penetration tests were conducted after the single pile tests were completed. CPT logs S-1, S-2, and S-3 in Figures 3.10 through 3.12 (20 days after testing) do not show any increase in tip resistance compared to cone logs prior to blasting. CPT logs SFC007 and SFC008 in Figures 3.13 and 3.14 (5 months after testing) show an increase in tip resistance when compared to cone logs prior to blasting and 20 days after blasting.

Shear wave velocity tests shown in Figure 3.15 were conducted before and after the first single pile test. Results of the down-hole test show a reduction in the shear wave velocity of approximately 12 m/sec (approximately 10%) in the upper 3.5 meters within hours of the first blast. After the blast, the shear wave velocity below 3.5 meters increased by 18 m/sec according to down-hole measurements.

3.2 Single Pile Test Results

The first of two single pile tests was conducted on January 20, 1999. The single piles were loaded using a 2200 kN capacity hydraulic actuator. Two linear potentiometers were attached to each pile to measure lateral displacement and rotation at the load point. The lateral load was applied 0.76 m above the excavated ground surface. The piles were also instrumented with strain gages along the depth of the pile. A plan view of the test set-up is shown in Figure 3.16.

3.2.1 Load-Displacement Results for Test 1

Three static tests were performed prior to blasting with a maximum displacement of 38 mm. Two of the static tests consisted of pulling the piles together and one test where the piles were pushed in opposite directions. After the explosives were detonated, the piles were cycled through a series of displacements. Due to the H-pile experiencing failure early in testing, displacement of the pipe pile was used to control cycling. The first series of cycles consisted of pushing the pipe pile to displacements of 76 mm and 152 mm once each. Then the pipe pile was cycled through ten 228 mm displacements. No control was placed on the displacement level of the H-pile during these cycles. As testing progressed, the H-pile became less stiff. As the H-pile reached a displacement of approximately 375 mm, the maximum stroke of the actuator was approached, preventing the pipe pile from being displaced to the target displacement of 228 mm. The pipe pile displaced between 150 and 178 mm upon reaching the maximum actuator stroke. Load-displacement results for the pipe pile are shown in Figures 3.17 and 3.18. Results for the H-pile are shown in Figures 3.19 and 3.20. A review of the pipe pile test results shows that as testing continued past the initial few post blast loading cycles, the soil pile system provided more resistance to lateral loads. This increase in lateral stiffness is attributed to the reduction of the excess pore pressure with time resulting in an increase in soil shear strength. Although the soil strength increased as pore pressures decreased, there was no significant increase in the H-pile lateral stiffness. The low stiffness provided by the H-pile is likely due to the formation of a plastic hinge approximately 1.9 m below the excavated ground surface.

3.2.2 Excess Pore Pressure Response for Test 1

Pore pressures within the soil were measured during the entire loading and for approximately one hour after loading ceased. The location of each pore pressure transducer and blast point is shown in Figure 3.21. The excess pore pressure ratio for each transducer along with a plot of loading with time is presented in Figures 3.22 through 3.33. The excess pore pressure ratio plots are grouped according to location, beginning with the transducers at the south end of the excavation. Excess pore pressure ratios near the piles were significantly affected as the pile was displaced through the soil. As the pile was pushed and pulled, the excess pore pressure ratios would decrease and increase as can be seen in the figures. Excess pore pressures ratios at a distance of 4.2 m and 6.4 m from the pile were minimally affected as the piles were displaced. Similar to the pilot study, sand boils began forming within approximately 5 minutes of the blast as shown in Figure 3.34, and continued for sometime.

3.2.3 Load-Displacement Results for Test 2

Eight days after the first single pile test, a second lateral load test was performed. A 1.5-m square and 0.6-m deep concrete block was placed at grade around the failed H-pile to provide a stiff reaction for the pipe pile. Linear potentiometers were only connected to the pipe pile to measure displacement and rotation. A static load test was again performed prior to detonation of the explosives. Applied load versus lateral displacement plots for the entire test are presented in Figures 3.35 through 3.37. The pipe pile was displaced 38 mm toward the H-pile during the static test. Approximately 10 seconds after detonation of the explosives, the pipe pile was cycled through eleven series of displacements. The first series consisted of a 76 mm, 152 mm and eleven 228 mm cycles. The cycles consisted of pushing the pile to the target displacement and then returning the pile to a zero displacement at the load point. During the first post blast series, the soil-pile system reduced in stiffness during each 228 mm cycle. Post blast Series 2 through 11 consisted of a 76 mm, 152 mm and four 228 mm cycles. The soil-pile system showed an incremental increase in stiffness from post blast Series 2 through 11. The increase in stiffness can be attributed to the reduction in excess pore pressures with time.

3.2.4 Excess Pore Pressure Response for Test 2

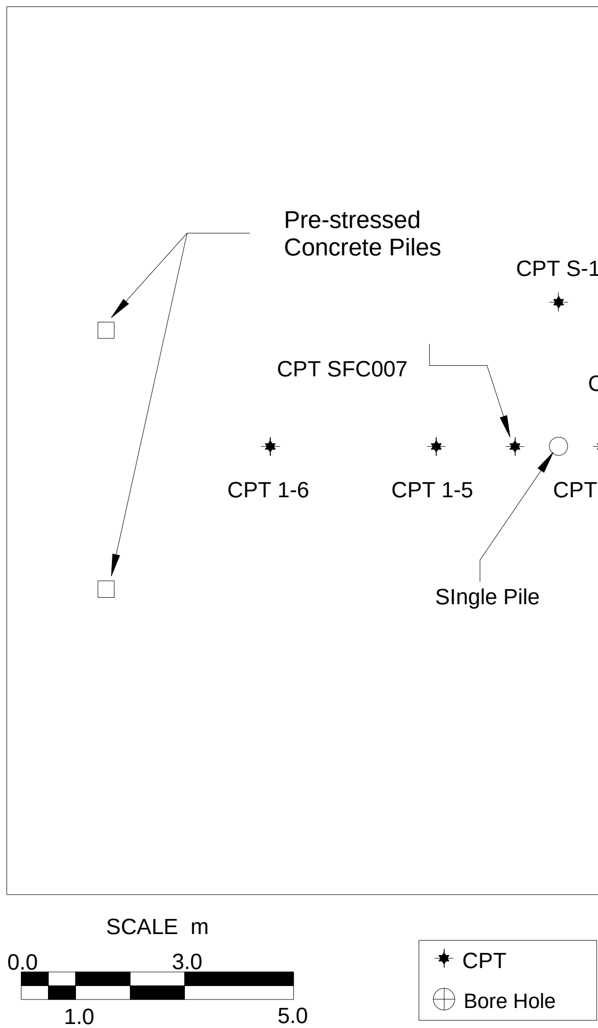
Pore pressure response for the second single pile test are similar to the first test, as can be seen from excess pore pressure ratios and load versus time plots in Figures 3.38 through 3.44. Excess pore pressure ratios at a distance of 4.2 m and 6.4 m from the pile were minimally affected as the pile was displaced. Excess pore pressures were affected the greatest at shallow depths near the pile. Only pore pressure transducers near the pipe pile were used during the second test. The formation of sand boils again provided evidence of liquefaction within the ring of charges.

3.2.5 Settlement

Immediately prior to and after each blast, an elevation survey was performed to measure settlement in the excavation. Figure 3.45 presents settlement contours as measured after the first blast. The contours of settlement are in millimeters, while the coordinates from the edge of the excavation are in meters. It can been seen from the figure that 200 mm of settlement was observed in the center of the test area, between the two piles.

3.3 Summary of Testing

The single pile test provided valuable results. CPT results showed an increase in soil density approximately 5 months after blasting. Load test results were consistent with previous laboratory testing showing a decrease in pile stiffness when the soil liquefies. As pore pressures decreased, the lateral stiffness of the soil-pile system increased. During the first ten minutes after detonation of the explosives, excess pore pressure ratios remained above 80%. This allowed for the piles to be displaced through a number of cycles while pore pressures were high. In addition to the value of the single pile test data alone, the single pile tests will be used as a reference for the larger diameter pile and pile group tests.



Note:
CPT No’s 1-1 through 1-6 Completed 11/06/98 (before first blast)
CPT No RBG1 Completed 2/05/99
CPT No’s S-1, S-2, & S-3 Completed 2/17/99
CPT No’s SFC006 & SFC007 Completed 6/24/99

Figure 3.1 Location Map of In-Situ Tests for Single Pile Area

Brigham Young University & University of California - San Diego Telephone: (801) 378-6334 Fax: (801) 378-4449	Boring No: BH-4 Logged By: jdl Date Started: 10/16/1998 Date Finished: 10/16/1998	Elevation (m): 3.4 Drill Contractor: Pitcher Drilling Method: Rotary Wash	Sheet 1 of 2	TREASURE ISLAND SAN FRANCISCO BAY CALIFORNIA

Location: Single Pile

Material Description	Water Level	Depth (m)	Sample Data: Number	Type	Recovery (cm/cm)	Blows/15 cm	$(N_1)_{60}$ Blows/30 cm	Soil Classification: USCS	AASHTO	S_u (kPa)	Torvane (kPa)	Pocket Pentometer (kPa)	Dry Density (kN/m3)	Moisture Content %	Atterberg Limits: Liquid Limits LL	Plastic Limits PL	Plasticity Index PI	Passing #200 Sieve (%)	OTHER DATA AND REMARKS
Brown, moist, Fined-Grained Sand w/Shells		1	S-1	SPT	46/46	8-11-11		SP											
	▼	2	S-2	SPT	30/46	4-4-7		SP											
Black, Saturated, Fine-Grained Silty Sand		3	S-3	SPT	23/46	4-5-5		SP-SM											
		4	S-4	SPT	30/46	3-3-5		SP-SM											
		5	S-5	SPT	33/46	5-7-7		SP-SM											
		6	S-6	SPT	46/46	4-5-2		SP-SM											
		7	S-7	SPT	43/46	2-2-1		SP-SM											
Dark, Clay, Saturated Fine-Medium Sand w/Some Silt		8	S-8	SPT	46/46	2-2-2		SP-SM											
		9	S-9	SPT	46/46	2-0-0		SM											
Saturated, Gray Clay		10	S-10	SPT	46/46			CL											

LEGEND AND NOTES
- SPT = Standard Penetration Test, Split Spoon Sample
- US = Undisturbed Shelby Tube, Pushed
- Ring = Dames & Moore Sampler
- Bulk = Bulk Sample From Cuttings

GROUNDWATER DEPTHS:
- ∇ = Initial
- ▼ = End of Drilling

COMMENTS:

LOG_OF_BORING_METRIC SANFRAN.GPJ LANE.GDT 9/14/00

Figure 3.2a Soil Boring Log for Single Pile Area

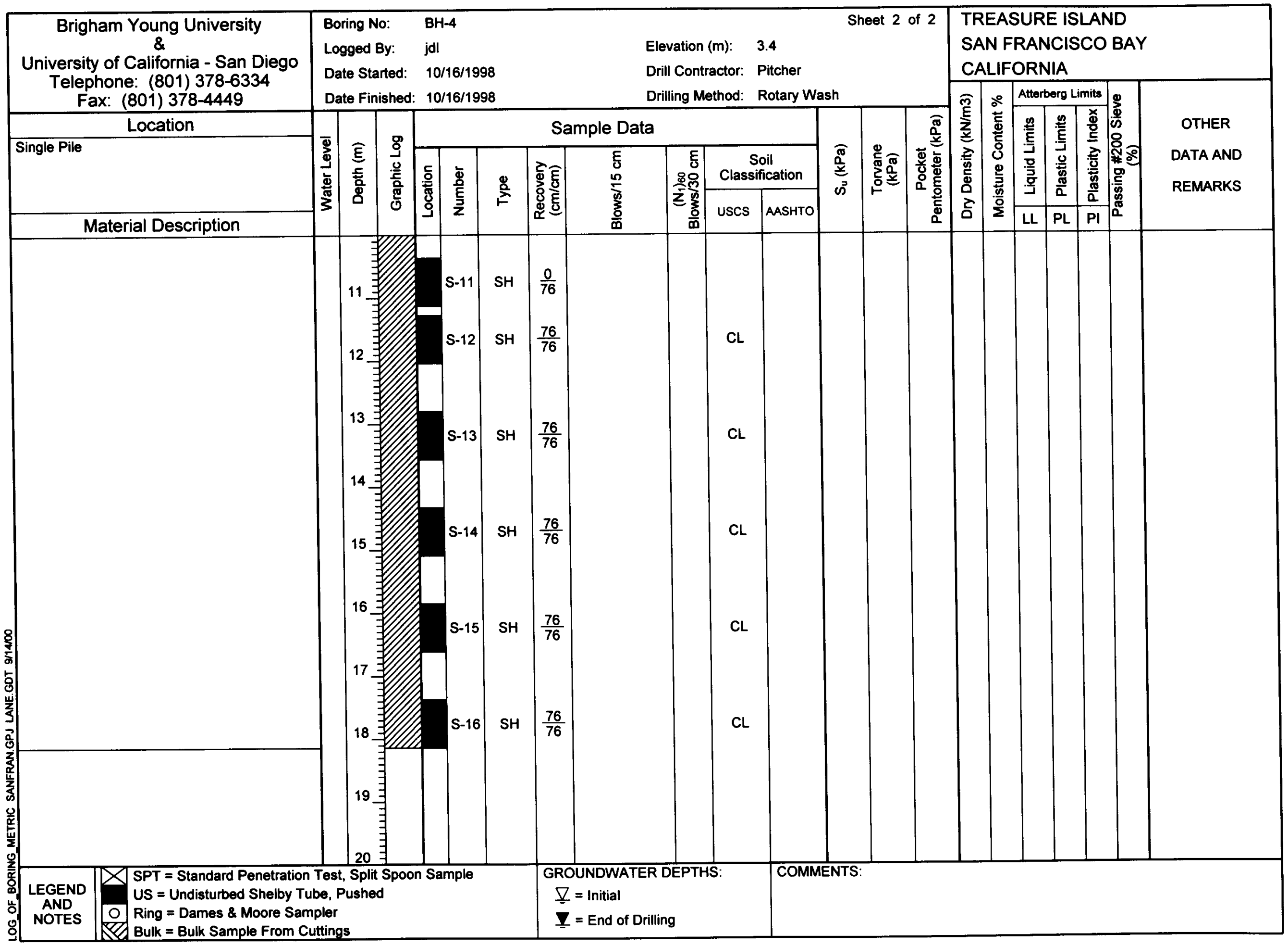


Figure 3.2b Soil Boring Log for Single Pile Area

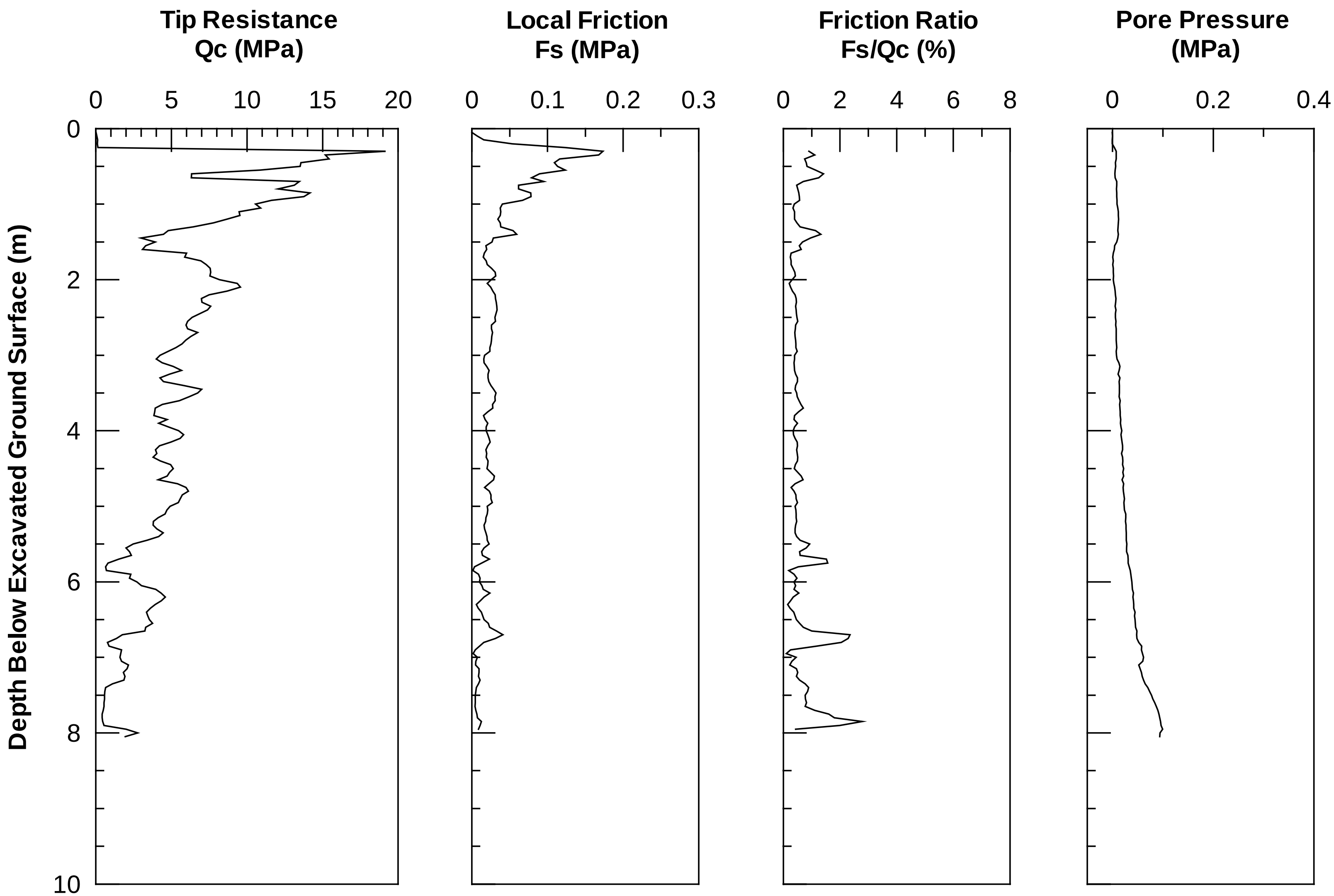


Figure 3.3 CPT1-1 Logs for Single Pile Test Area (10-30-98)

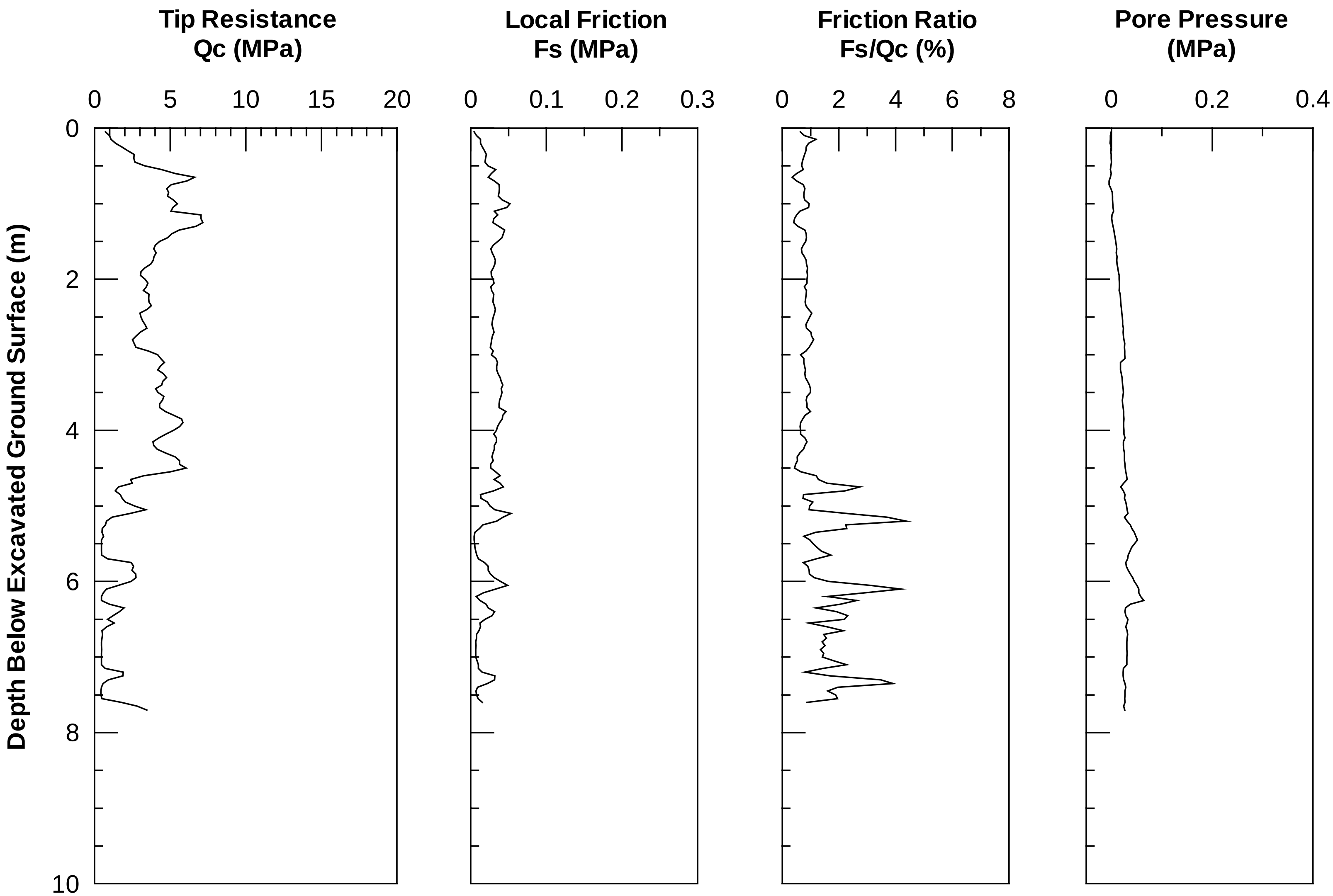


Figure 3.4 CPT1-2 Logs for Single Pile Test Area (11-06-98)

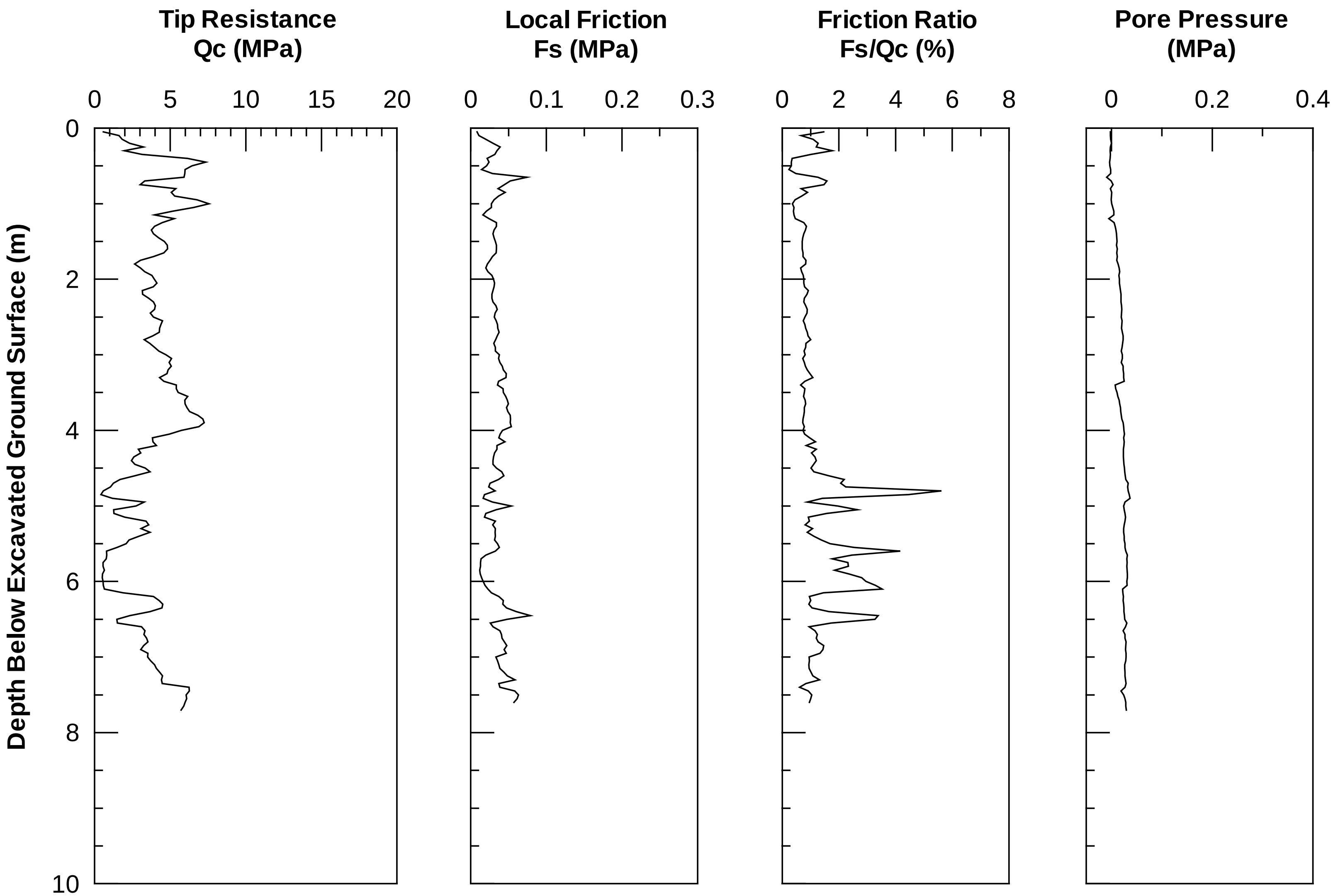


Figure 3.5 CPT1-3 Logs for Single Pile Test Area (11-06-98)

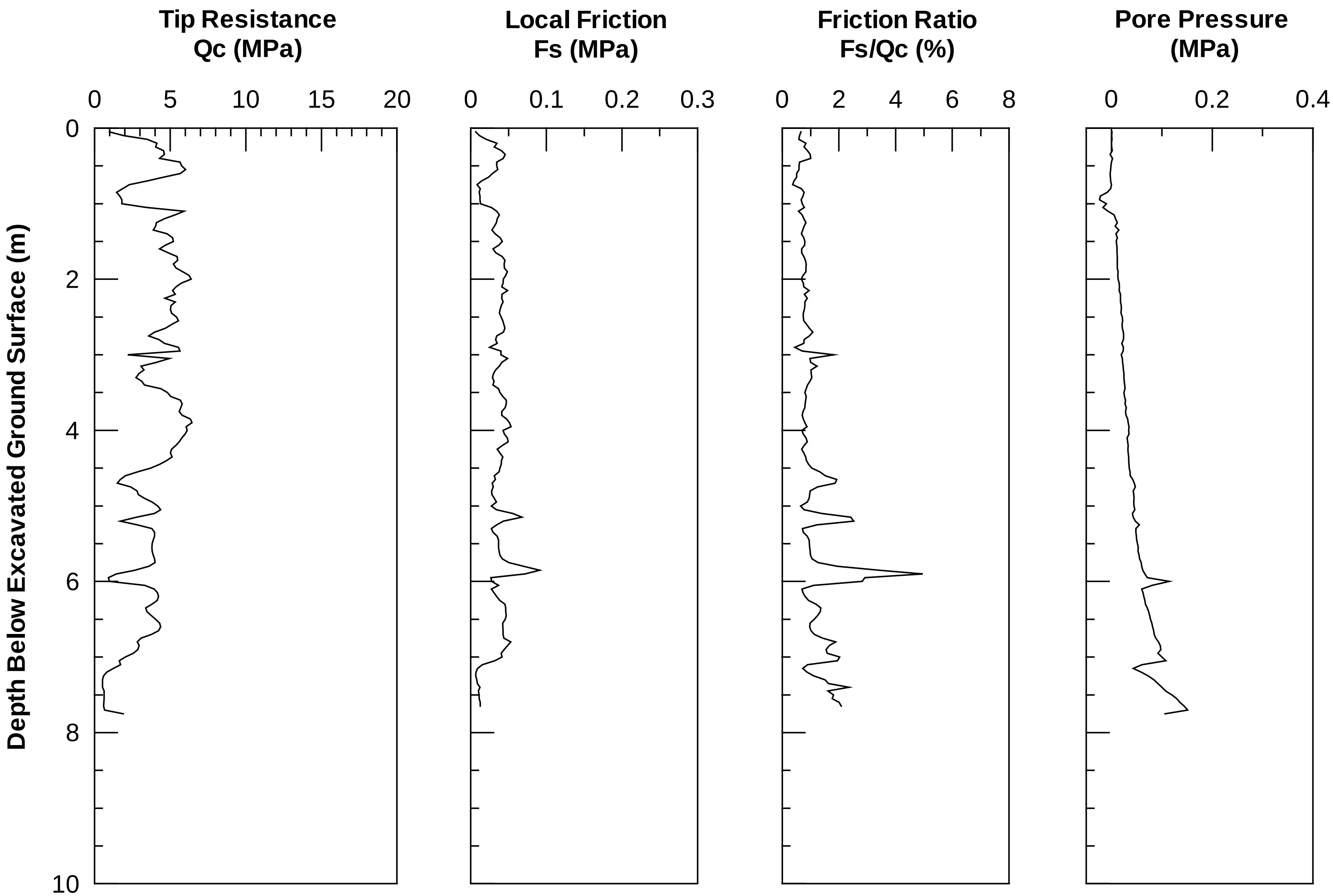


Figure 3.6 CPT1-4 Logs for Single Pile Test Area (11-06-98)

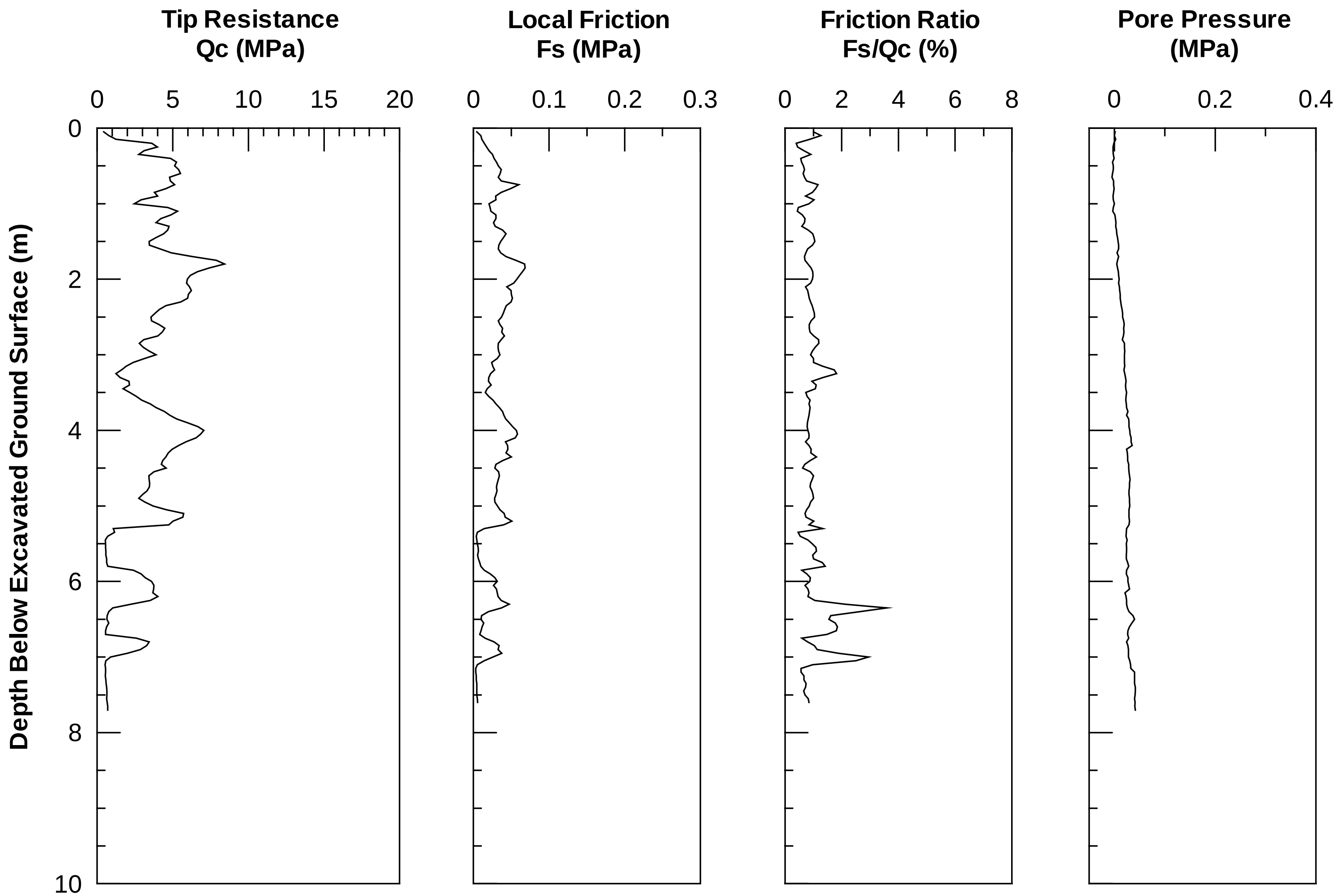


Figure 3.7 CPT1-5 Logs for Single Pile Test Area (11-06-98)

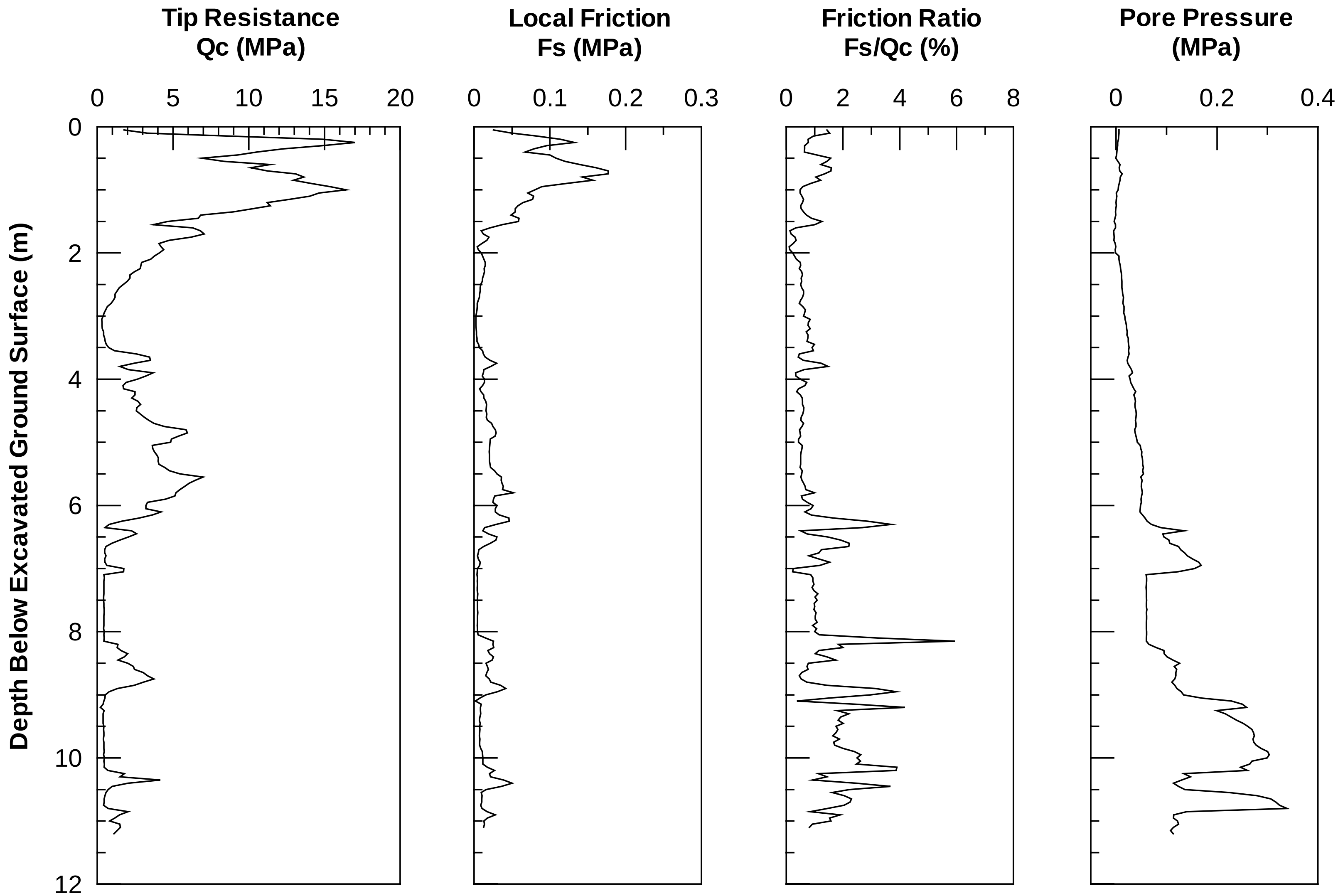


Figure 3.8 CPT1-6 Logs for Single Pile Test Area (10 -30-98)

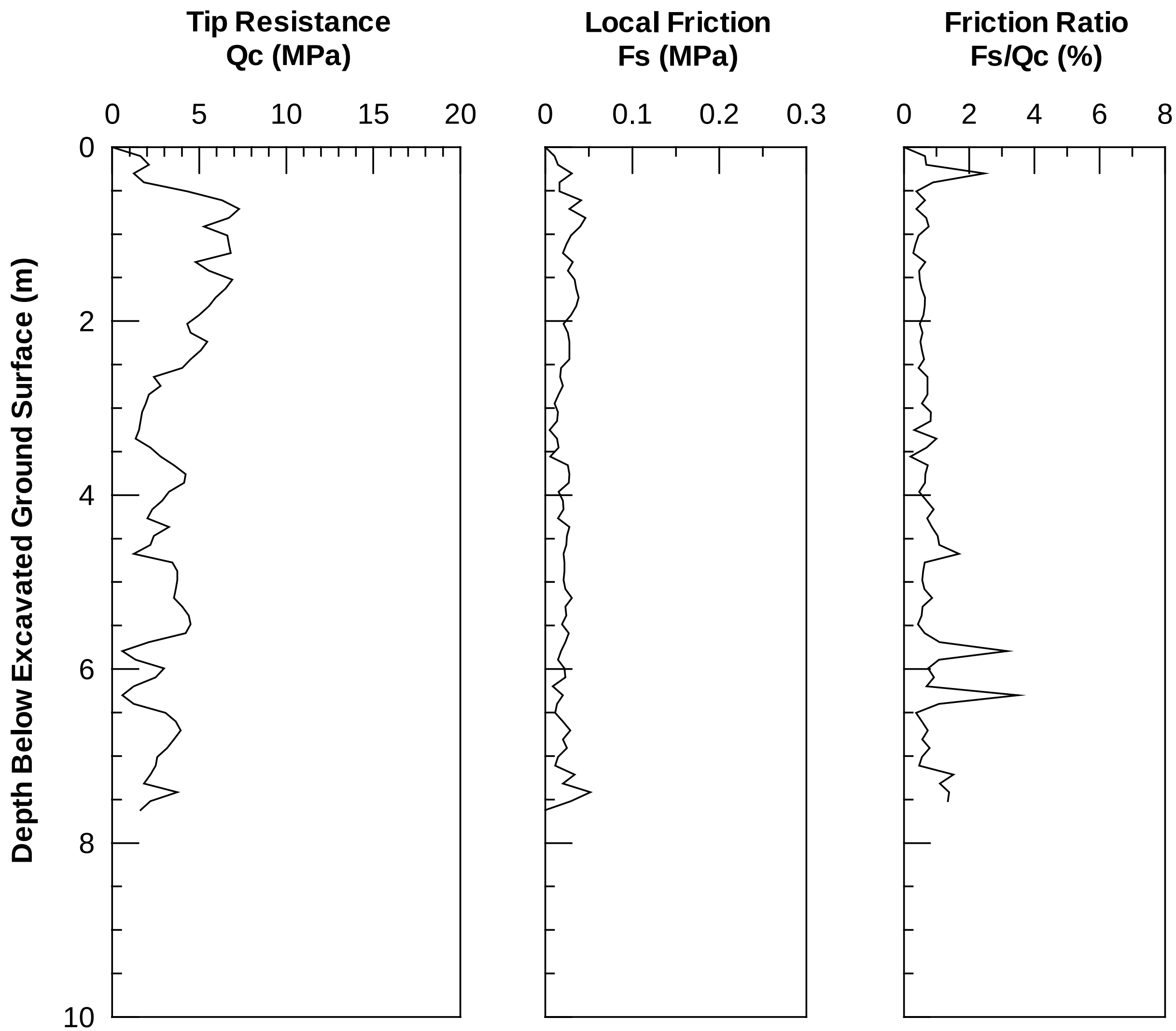


Figure 3.9 CPT RBG1 Logs for Single Pile Test Area (2-05-99)

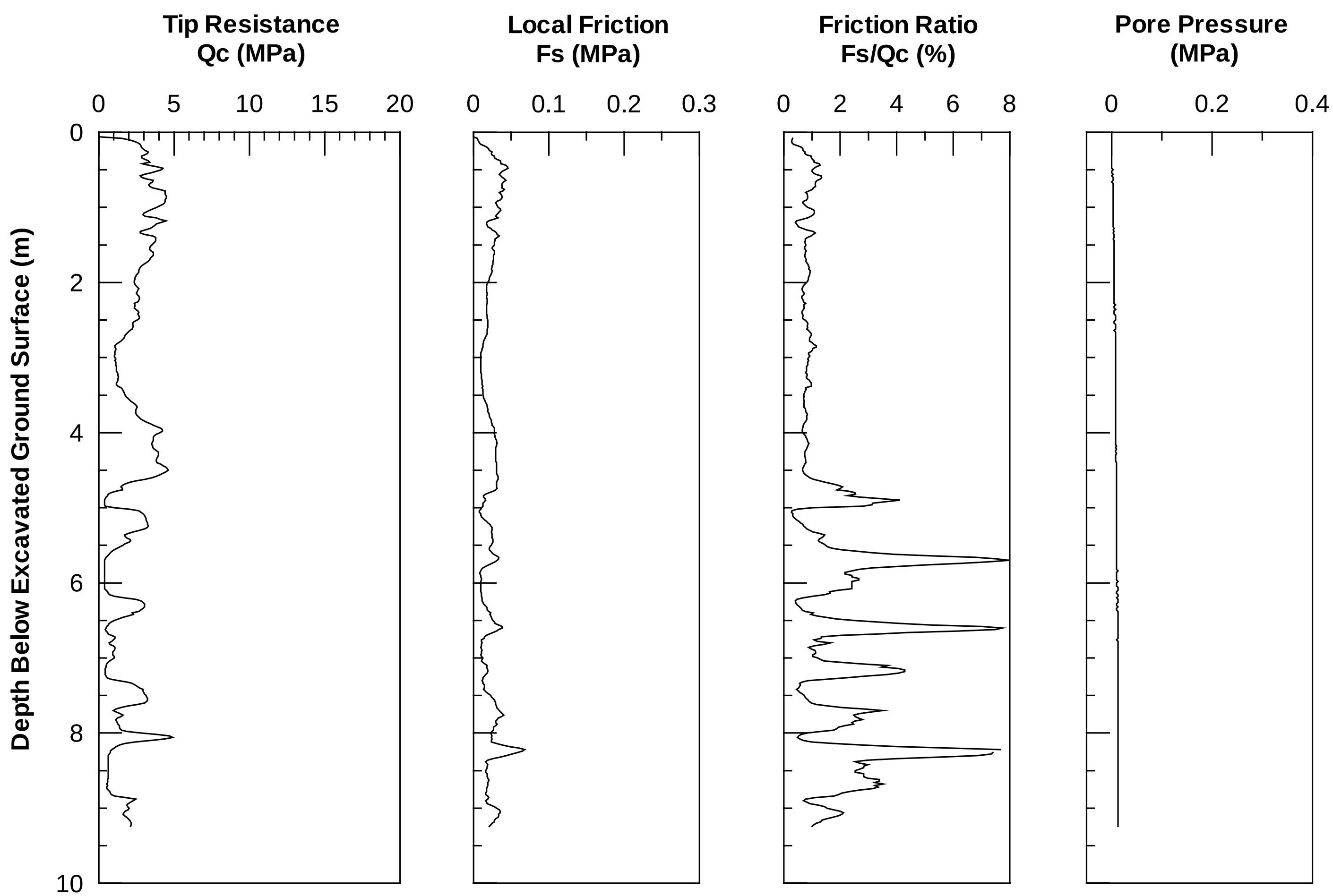


Figure 3.10 CPT S-1 Logs for Single Pile Test Area (2-17-99)

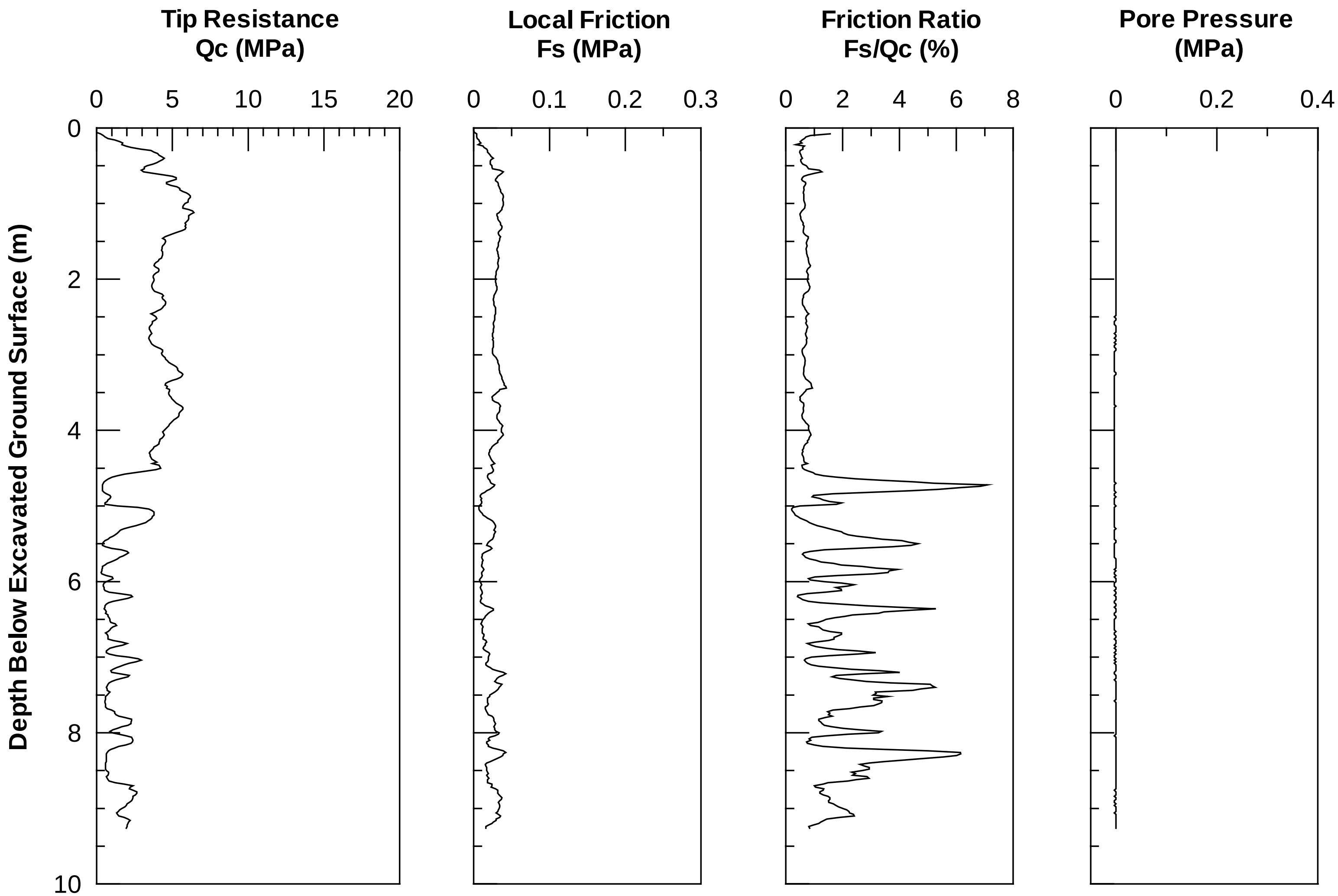


Figure 3.11 CPT S-2 Logs for Single Pile Test Area (2-17-99)

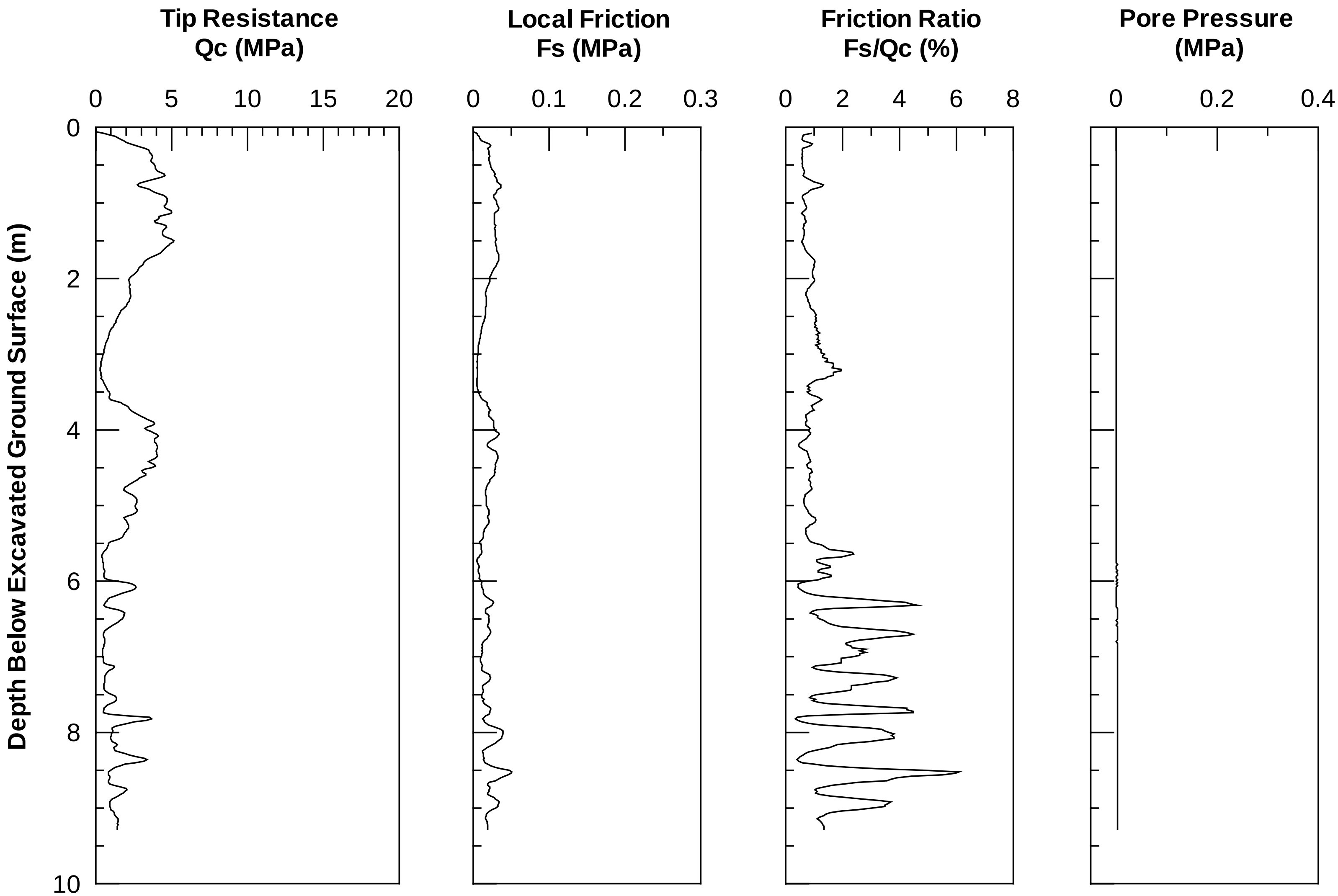


Figure 3.12 CPT S-3 Logs for Single Pile Test Area (2-17-99)

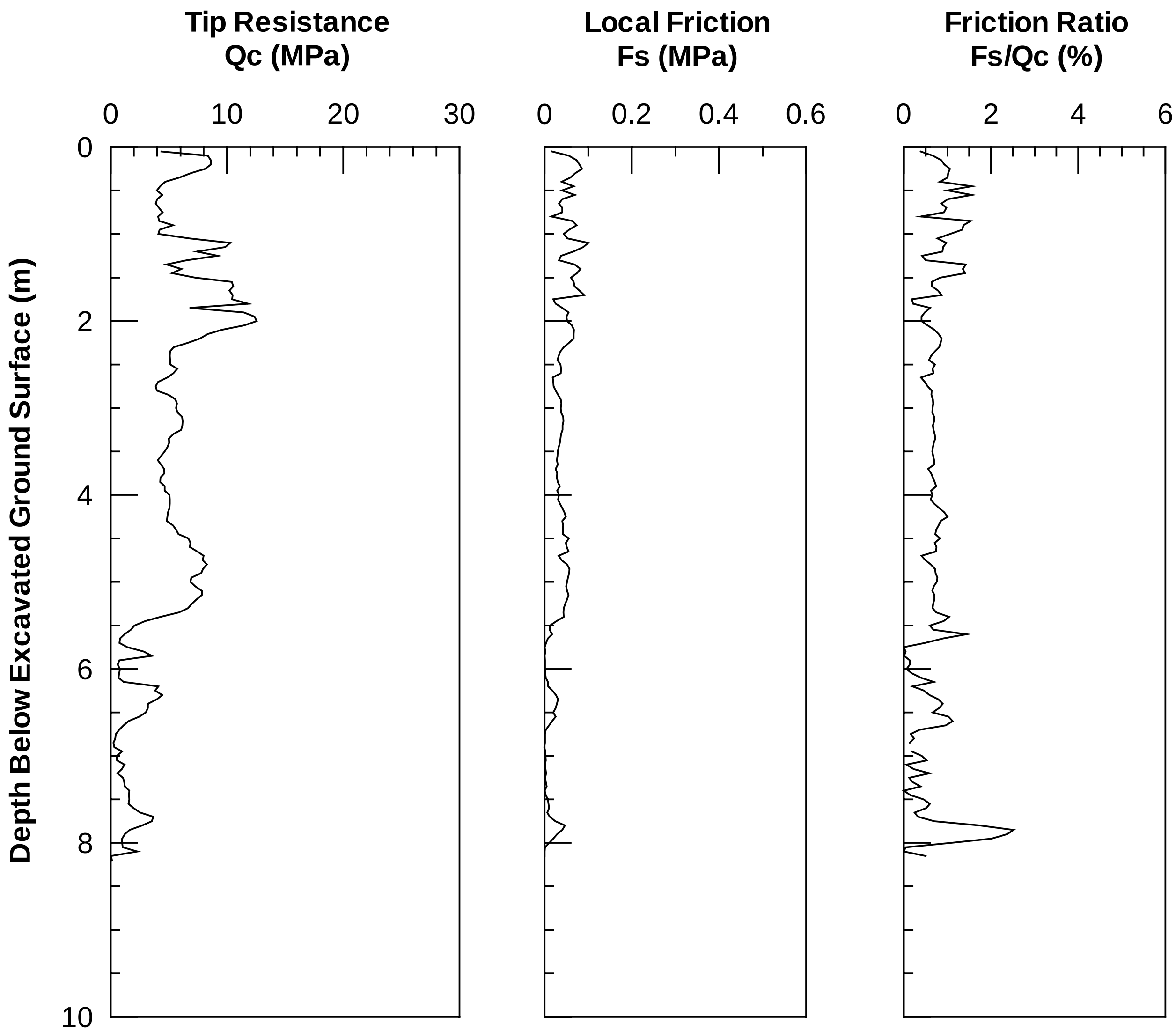


Figure 3.13 CPT SFC006 Logs for Single Pile Test Area (6-24-99)

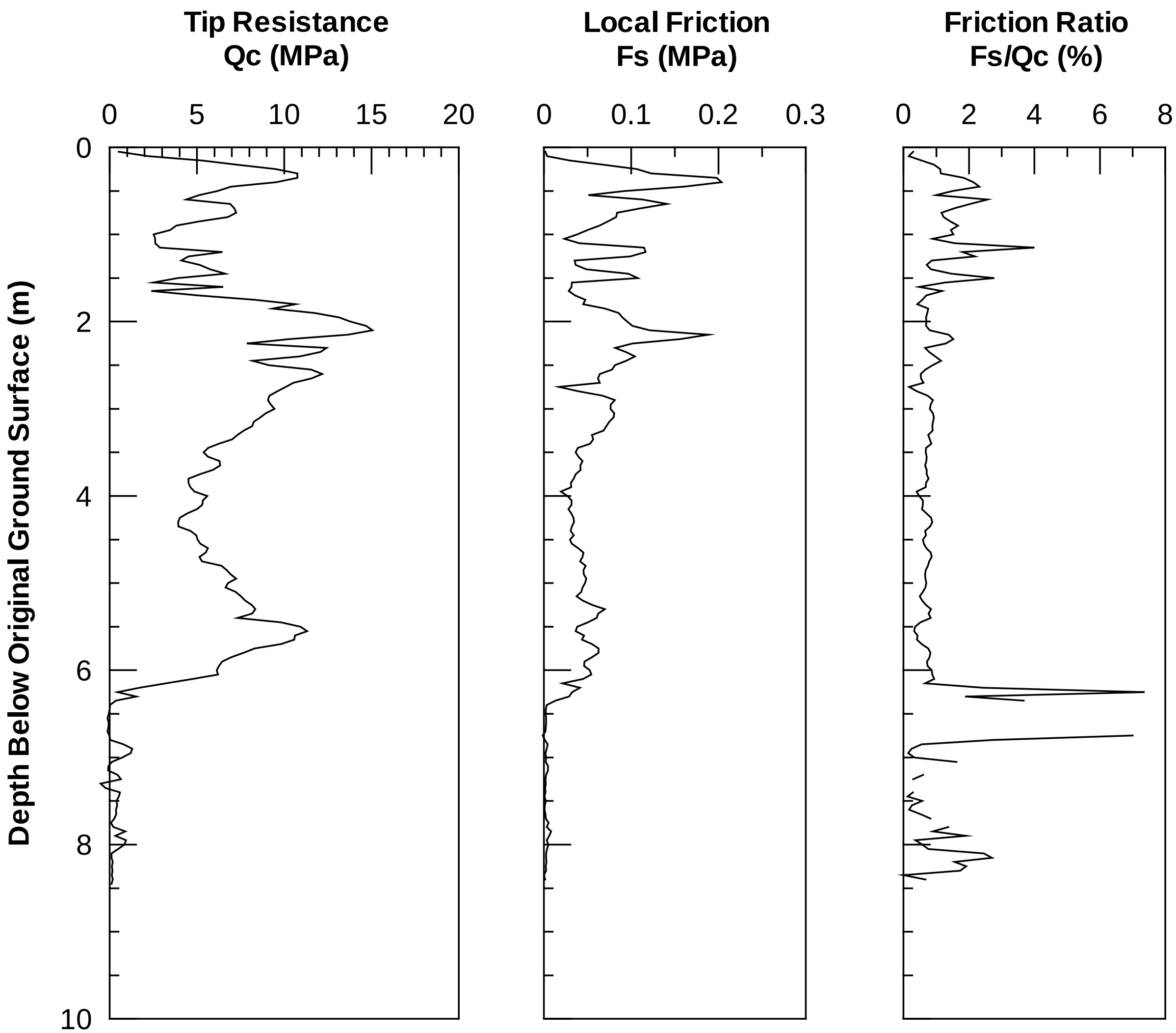


Figure 3.14 CPT SFC007 Logs for Single Pile Test Area (6-24-99)

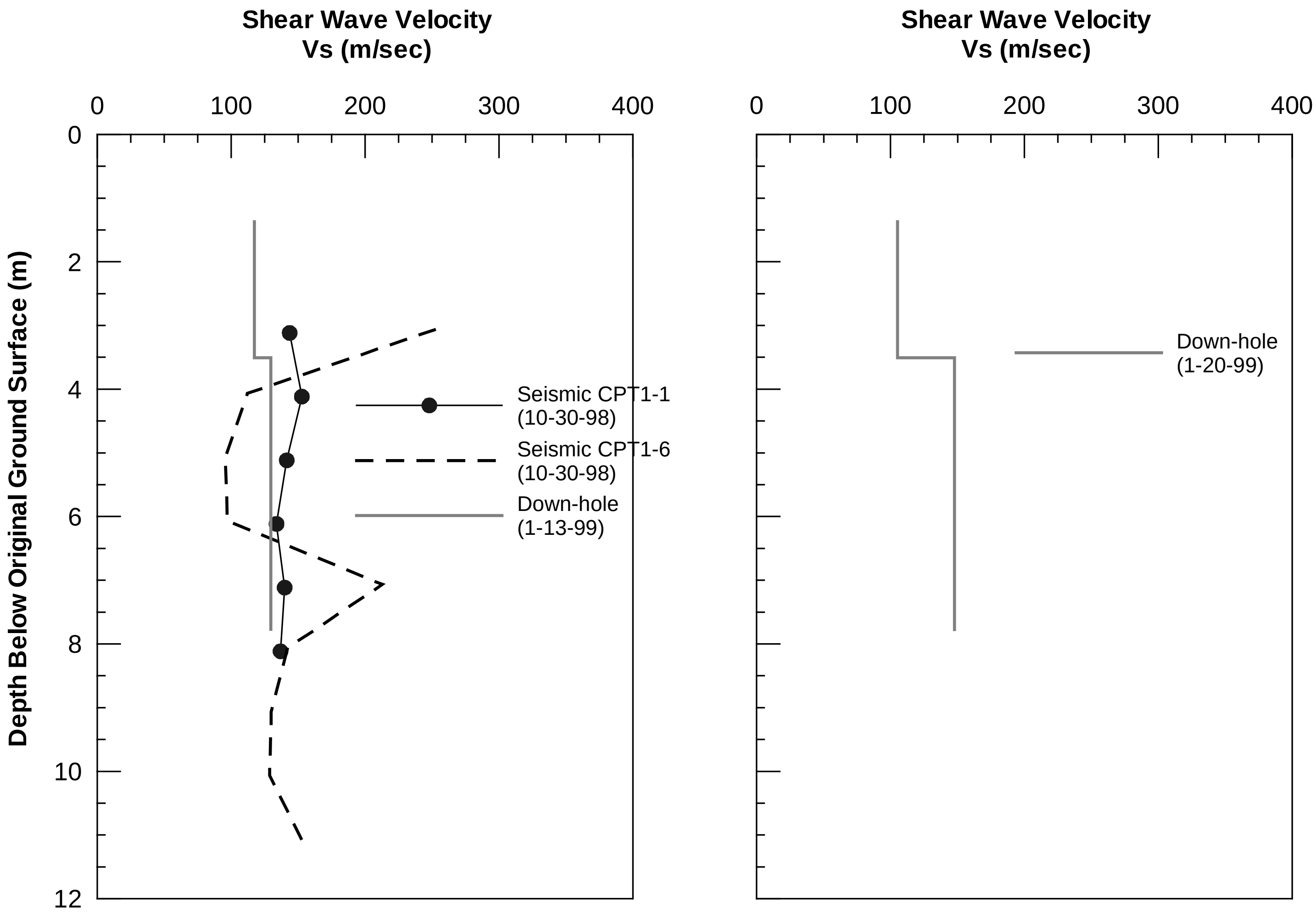


Figure 3.15 Shear Wave Velocity at Single Pile Test Area a) Pre-Blast Velocity b) Post 1st Blast Velocity

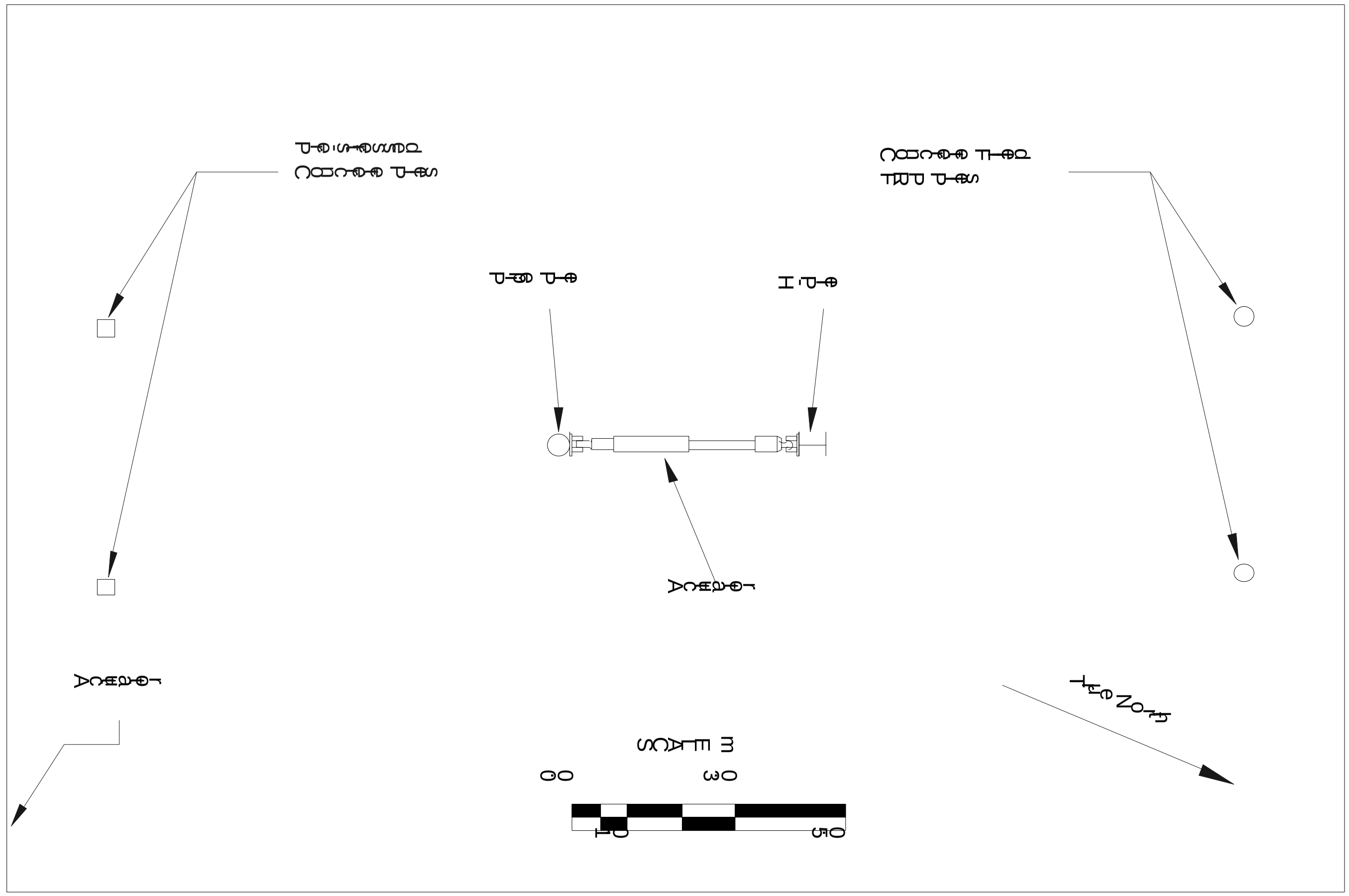


Figure 3.16 Plan View of Single Pilt Test Set-Up

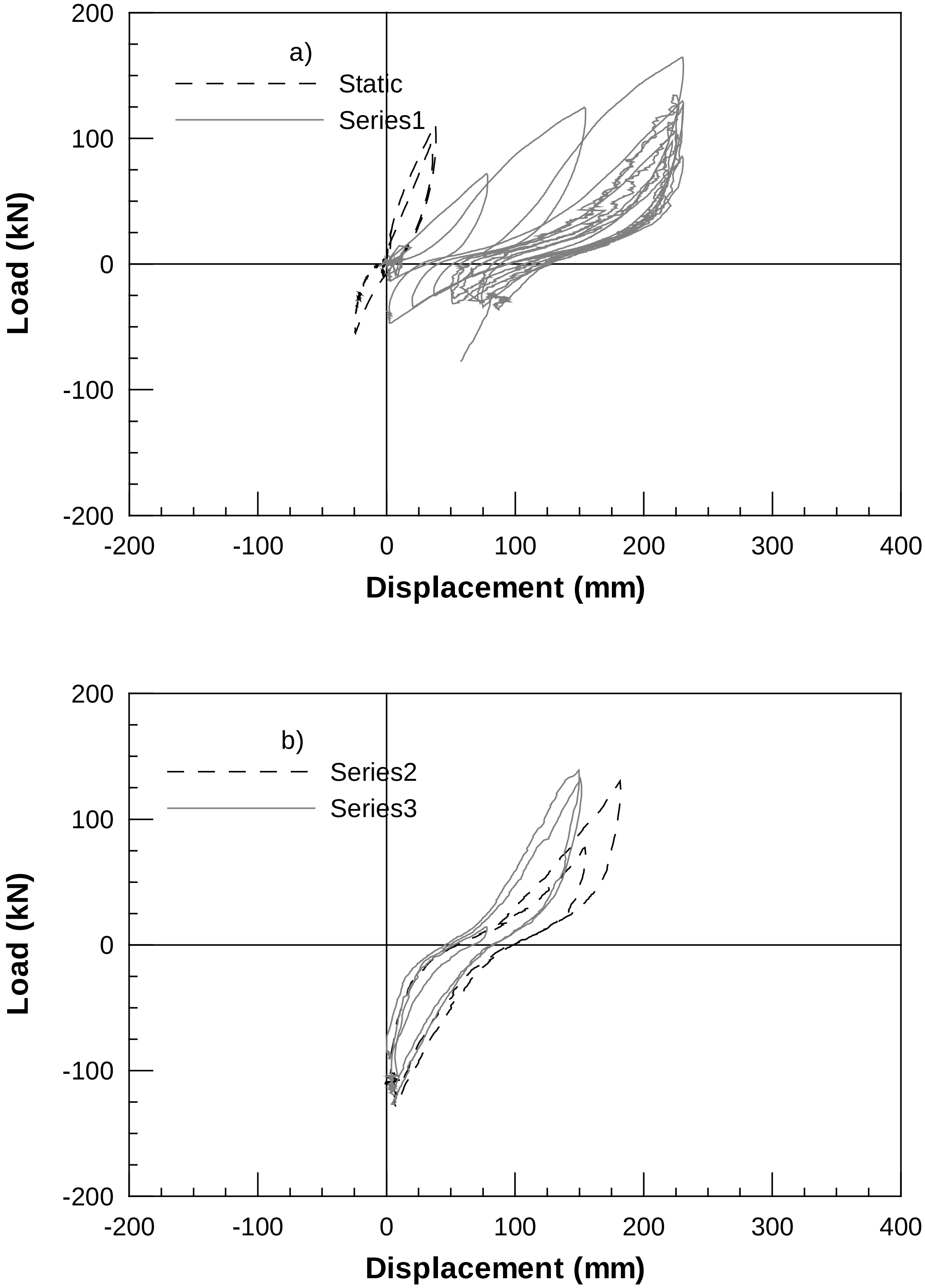


Figure 3.17 Load vs Displacement for Single Pipe Pile 1st Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3

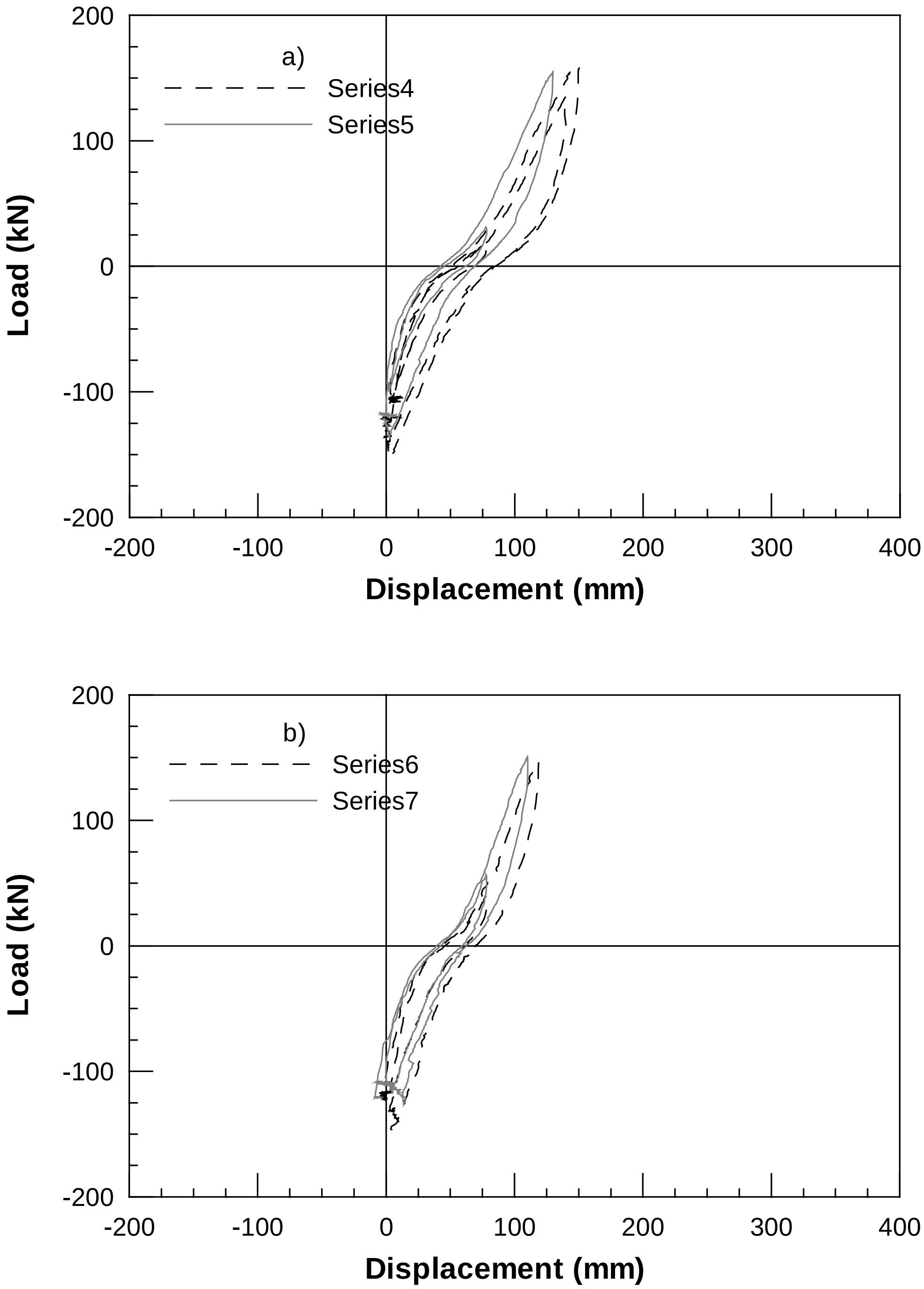


Figure 3.18 Load vs Displacement for Single Pipe Pile 1st Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

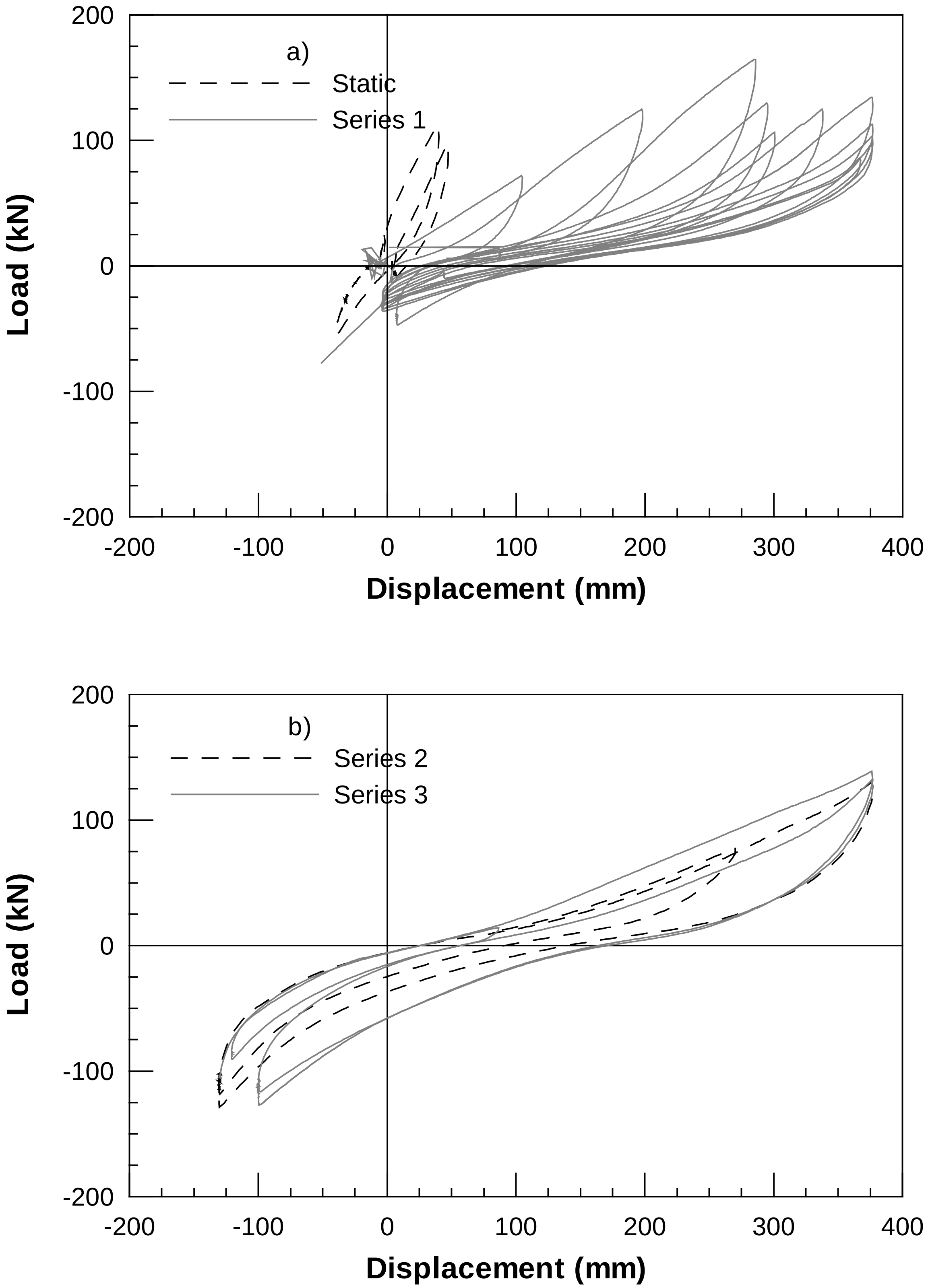


Figure 3.19 Load vs Displacement for H-Pile 1st Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3

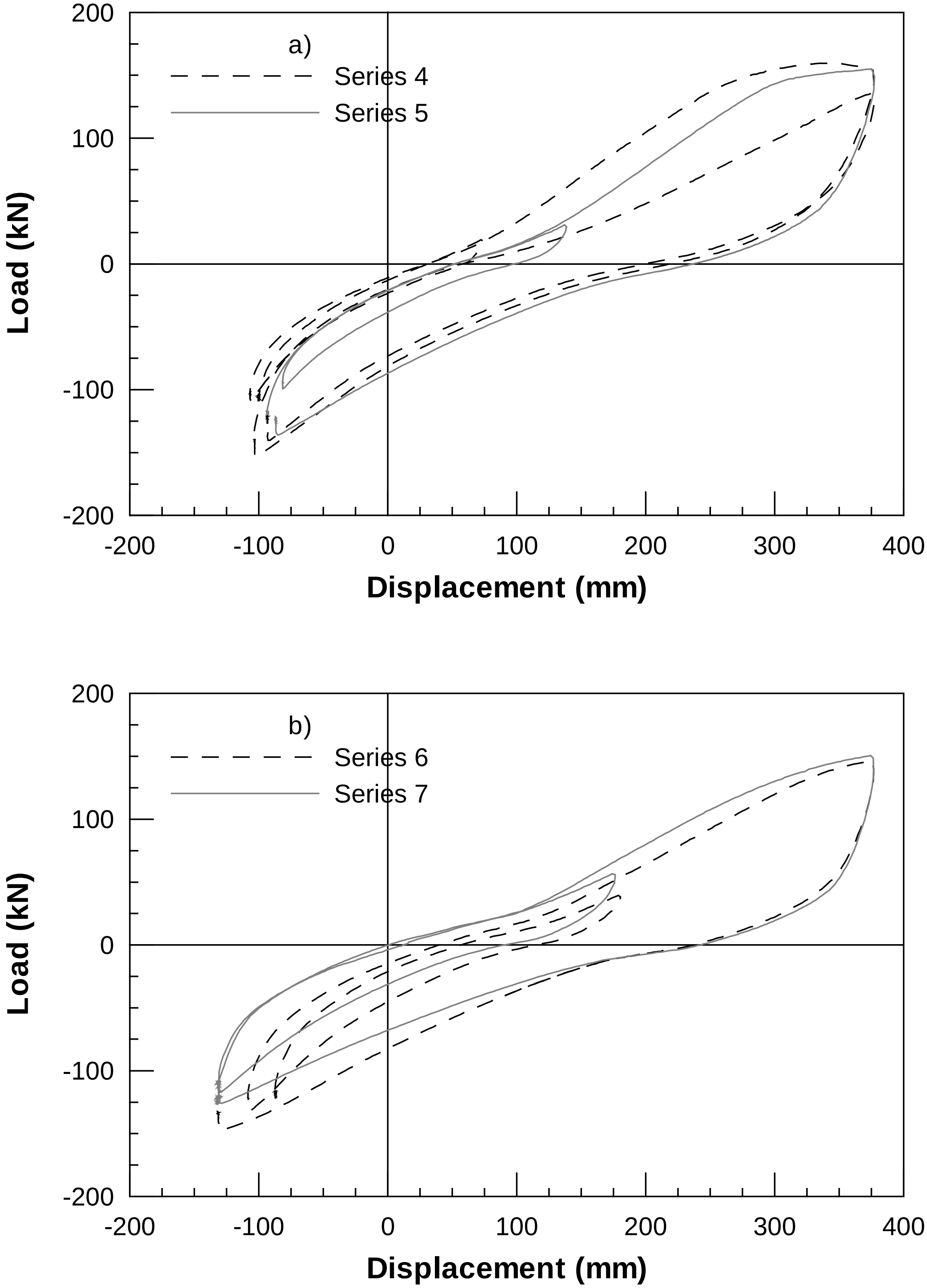


Figure 3.20 Load vs Displacement for H-Pile 1st Blast a) Post Blast Series 4 and 5
b) Post Blast Series 6 and 7

PPT	Depth
01	3.25m
02	3.25m
03	5.08m
04	5.08m
06	1.42m
10	2.34m
16	1.42m
24	1.42m
38	3.25m
39	2.34m
44	4.17m
47	4.17m
54	2.34m
55	3.25m
57	3.25m
87	3.25m
89	1.42m
90	6.00m
92	3.25m
93	2.34m
94	4.17m
95	3.25m
96	2.34m
97	4.17m
98	6.00m
99	3.25m

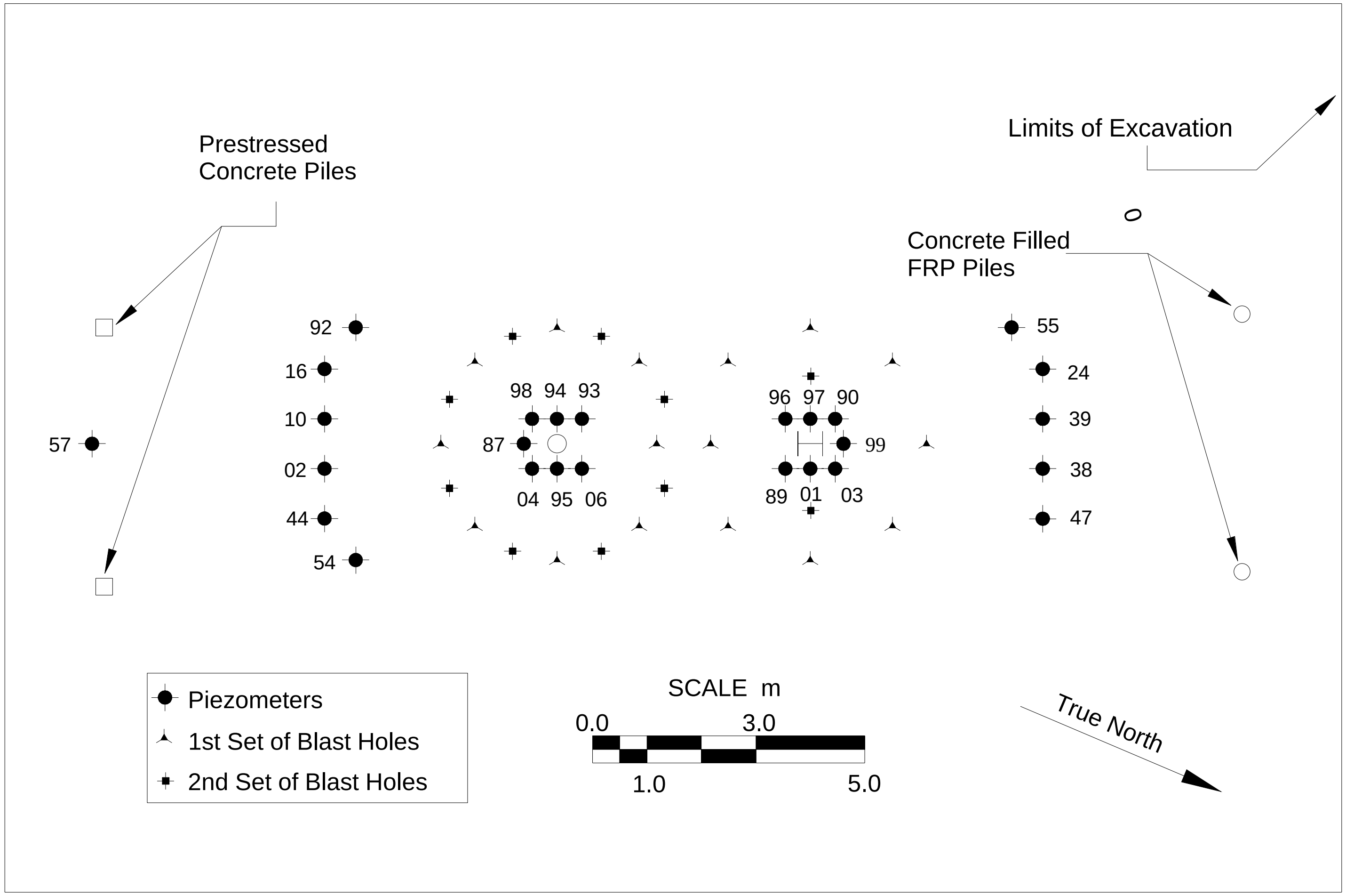


Figure 3.21 Location of Pore Pressure Transducers and Explosives for Single Pile Test

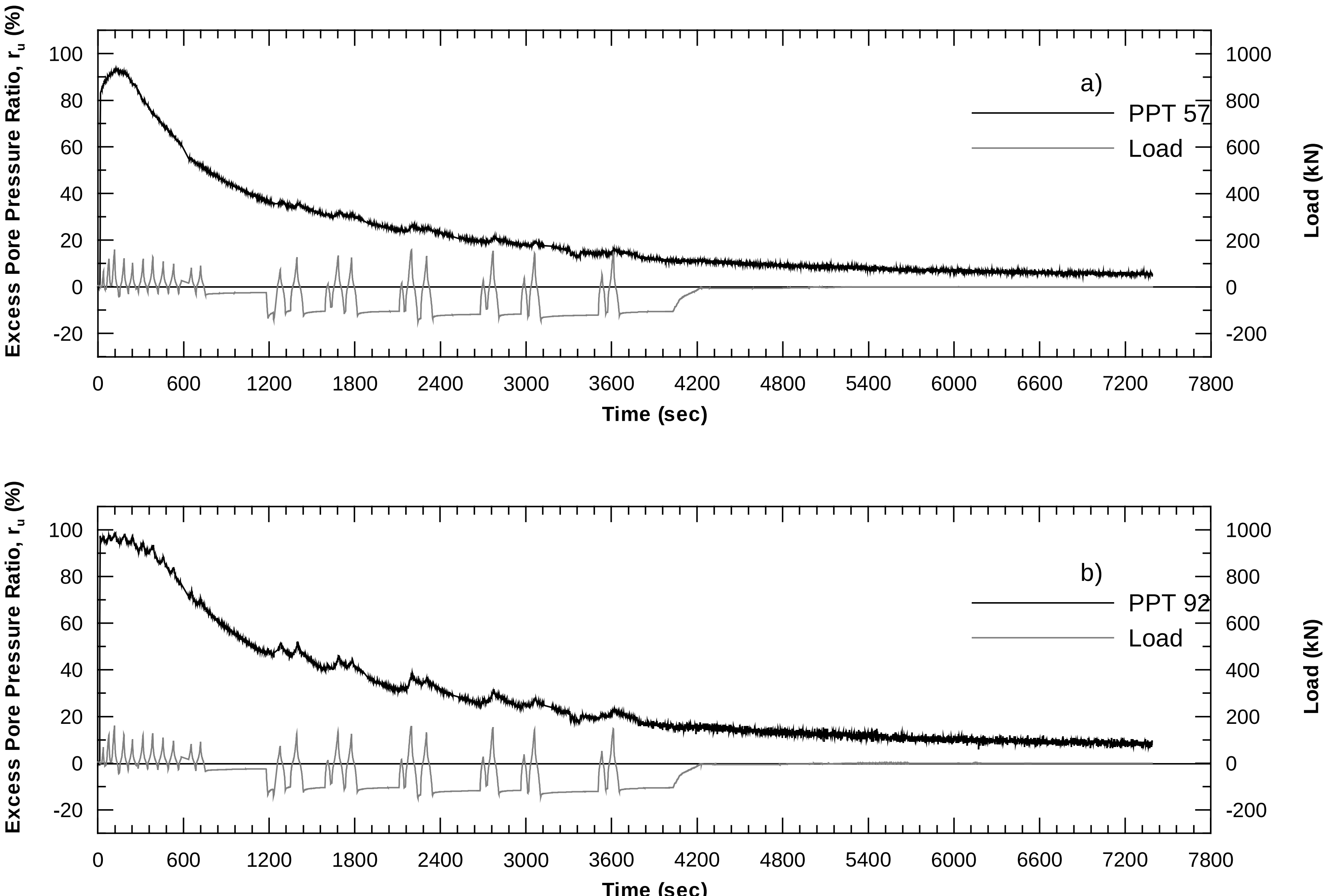


Figure 3.22 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a) PPT57 b) PPT92

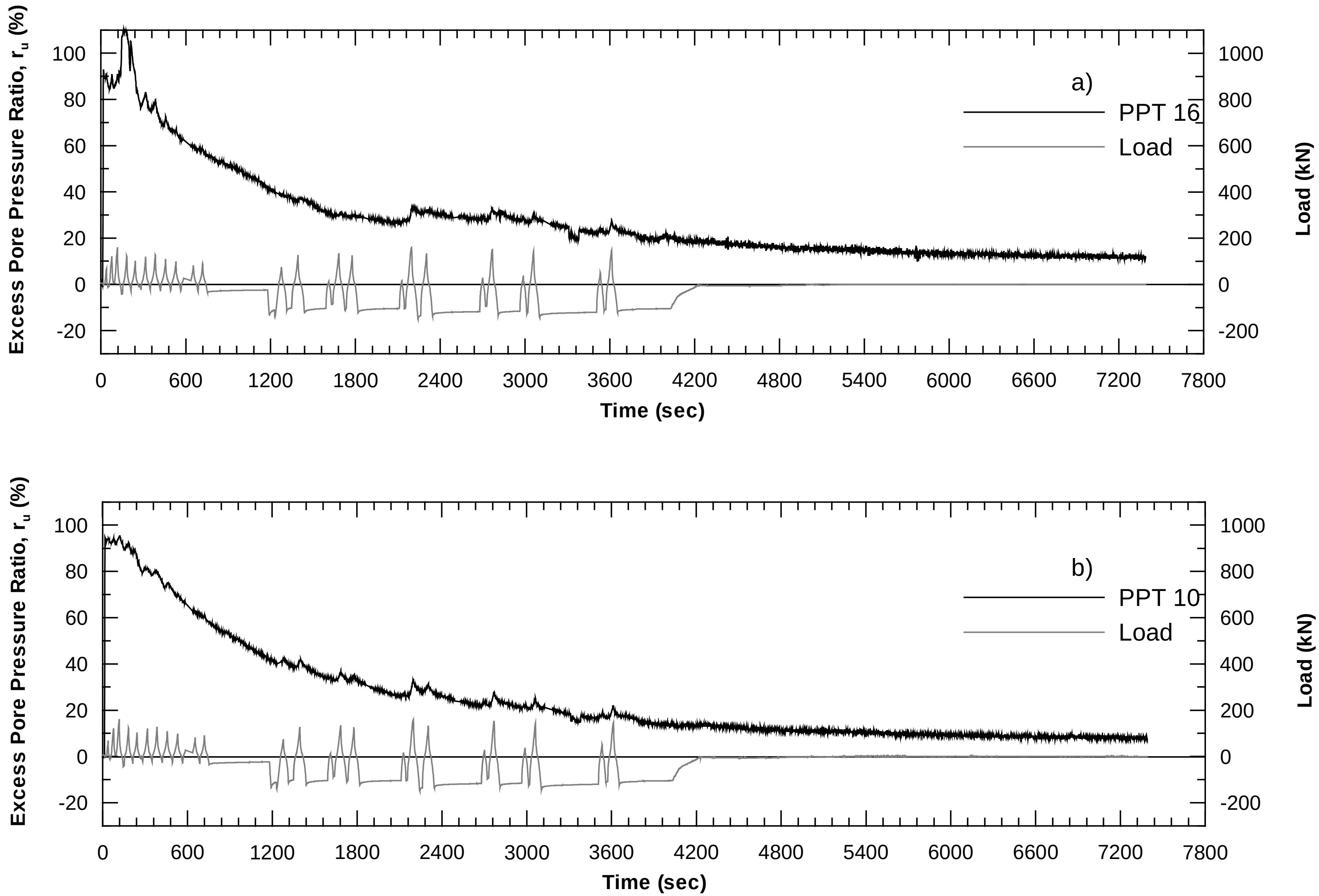


Figure 3.23 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a) PPT16 b) PPT10

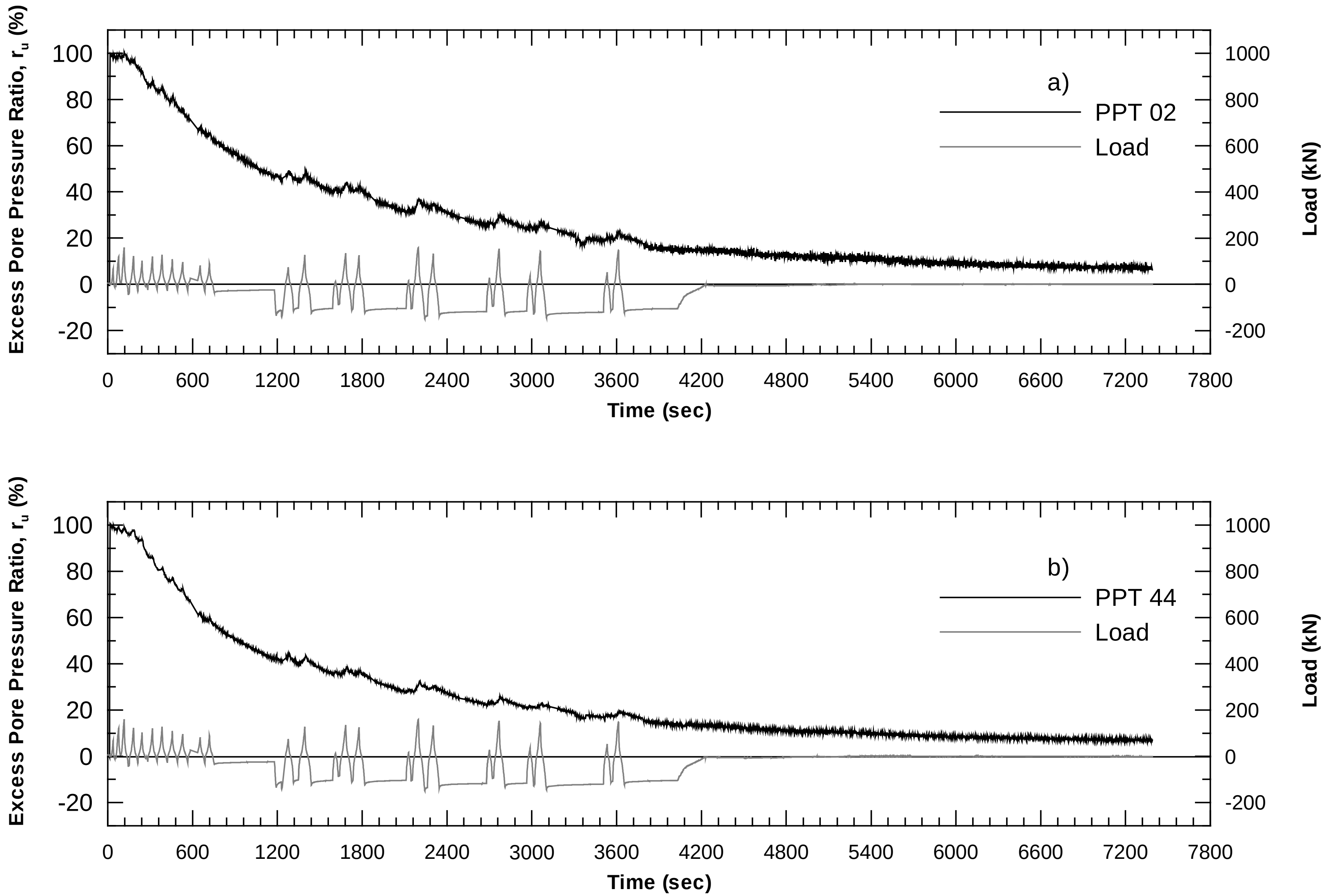


Figure 3.24 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a) PPT02 b) PPT44

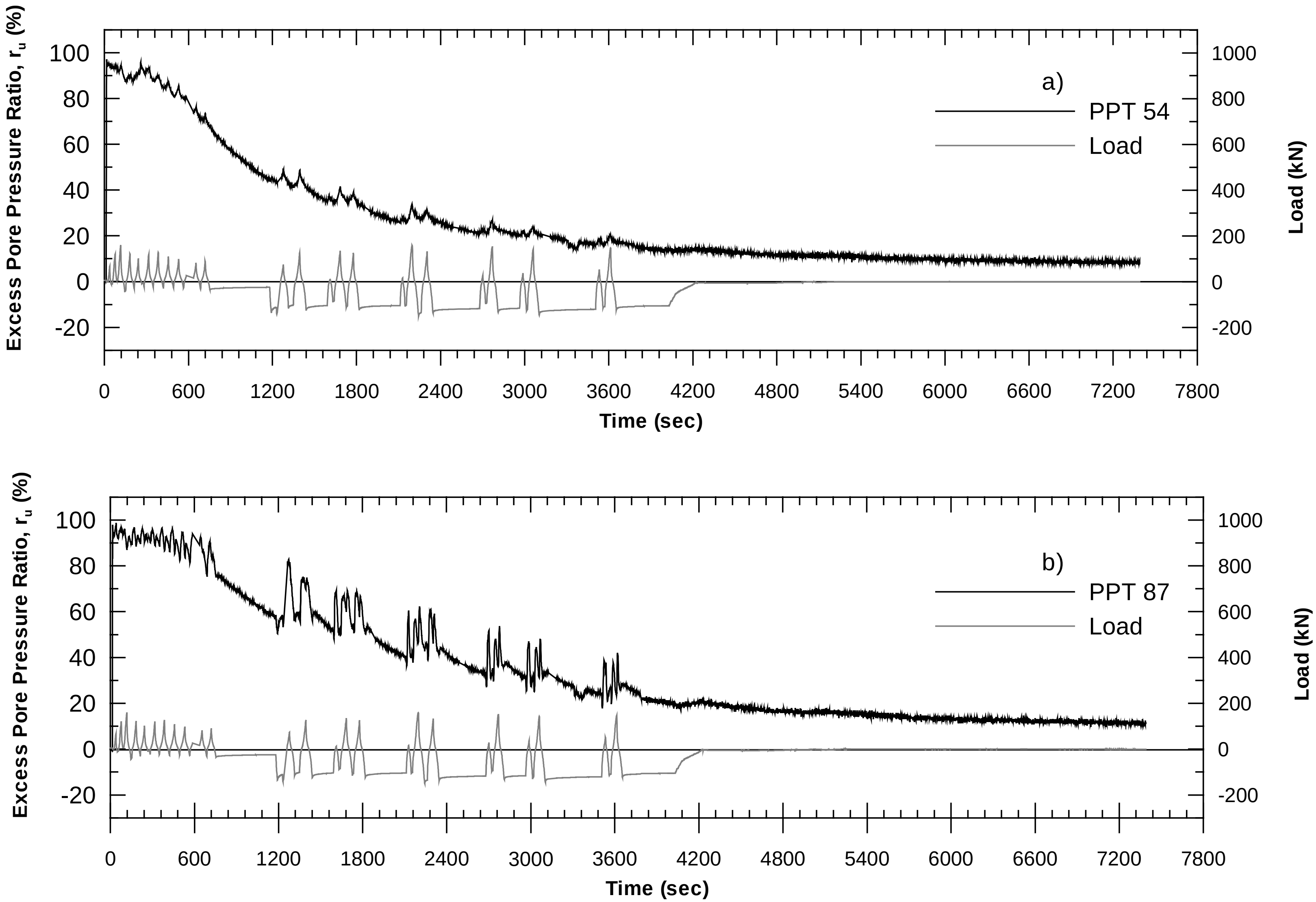


Figure 3.25 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a) PPT54 b) PPT87

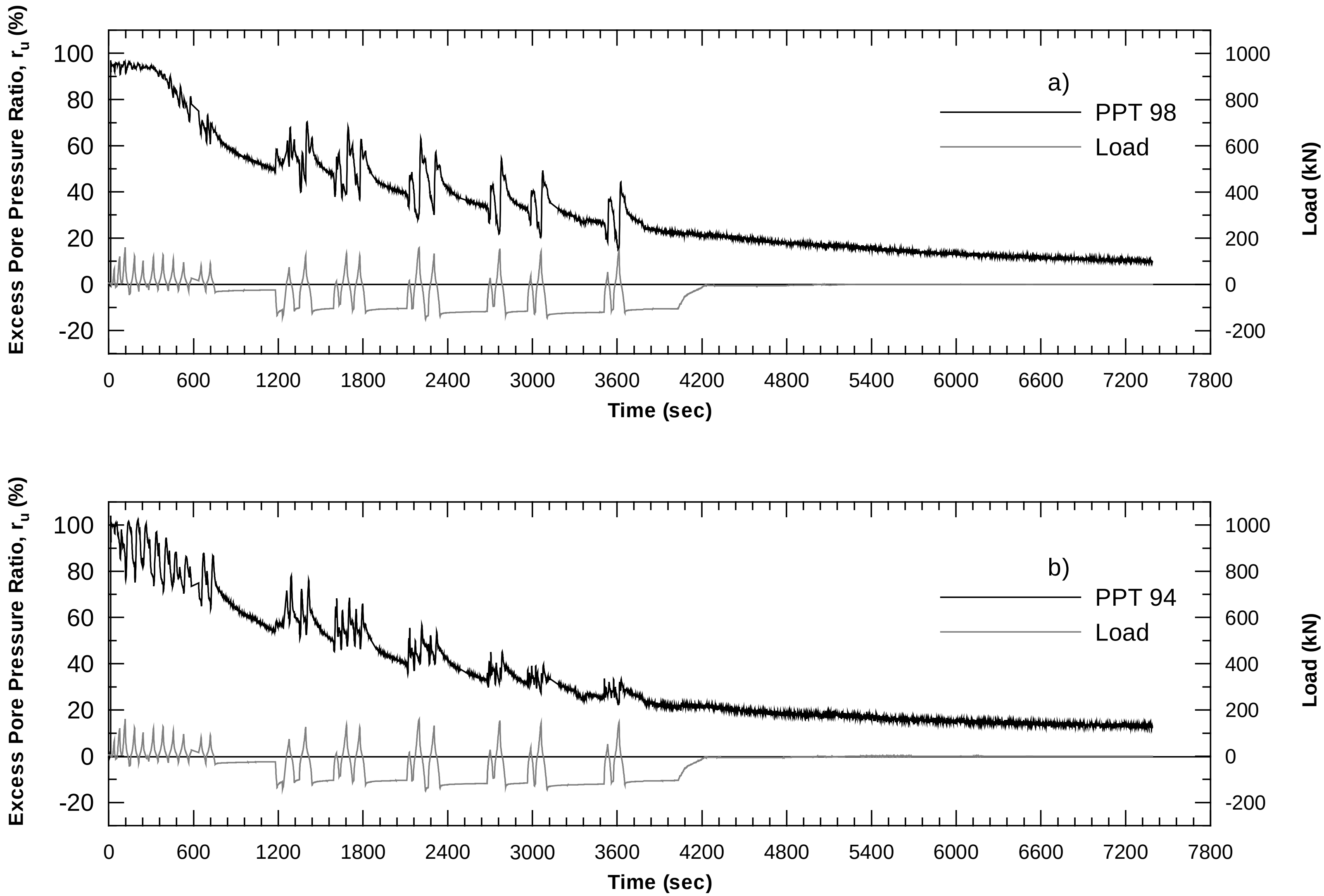


Figure 3.26 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a) PPT98 b) PPT94

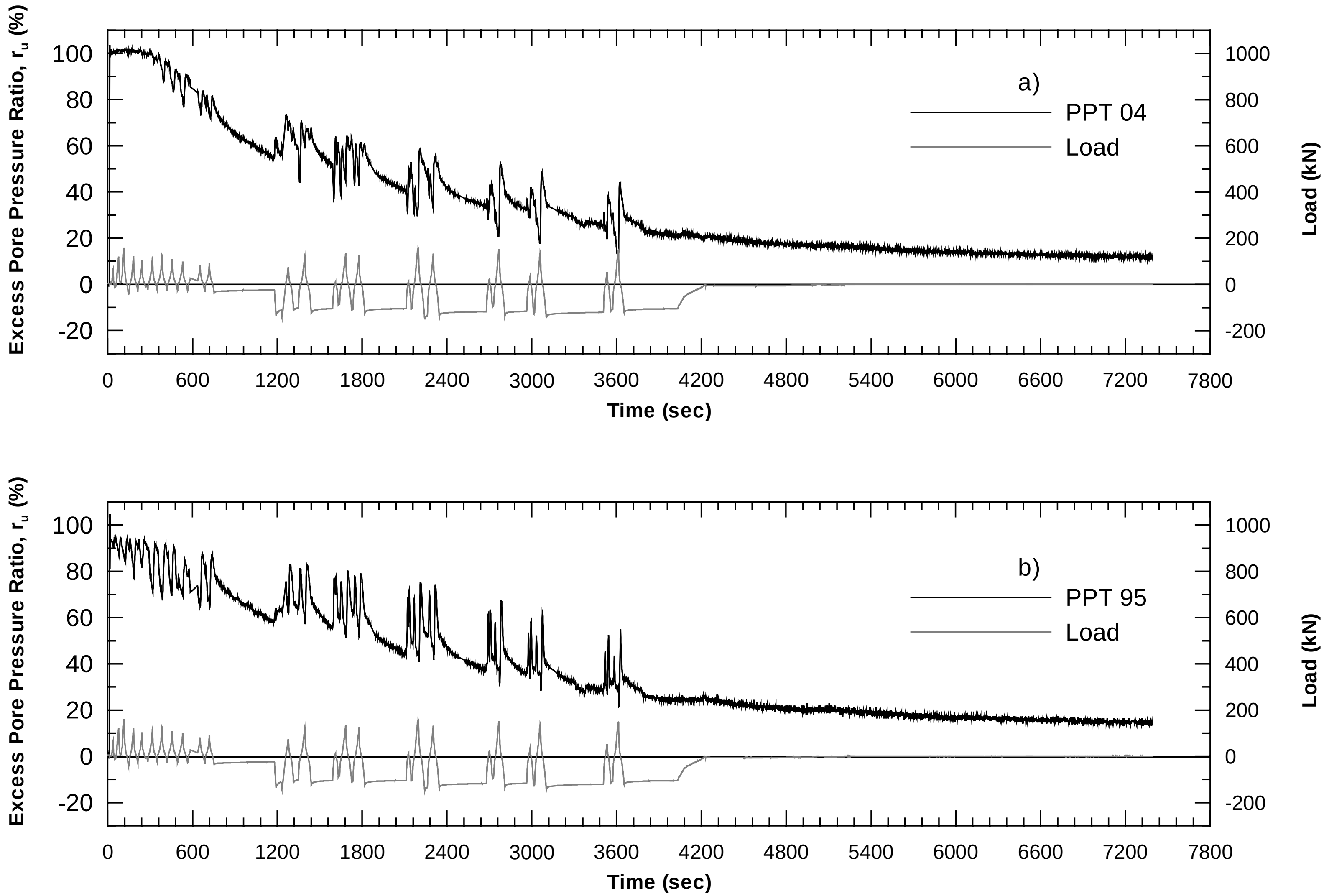


Figure 3.27 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a)PPT04 b)PPT95

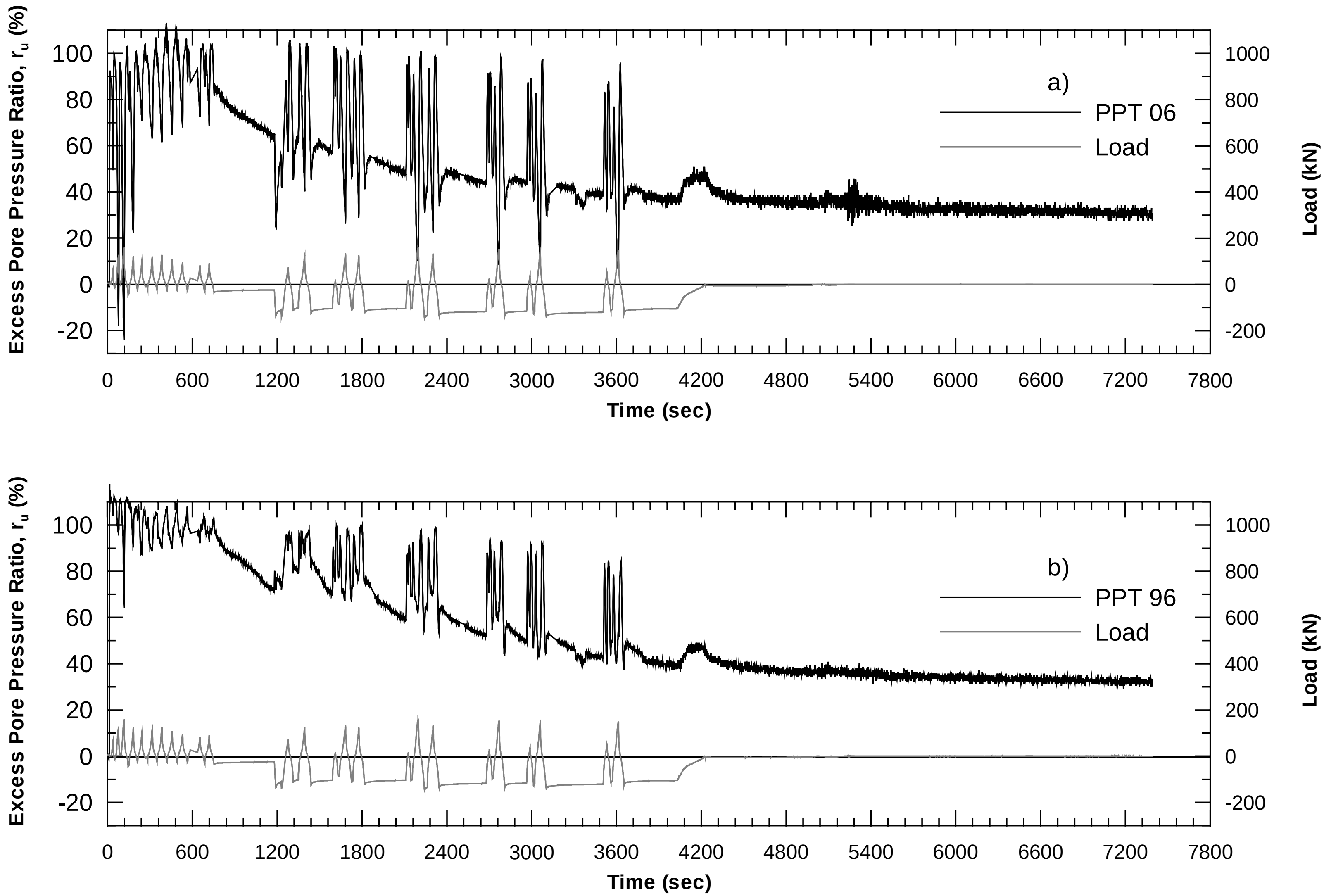


Figure 3.28 Excess Pore Pressure Ratio for Single Pipe Test 1st Blast a) PPT06 b) PPT96

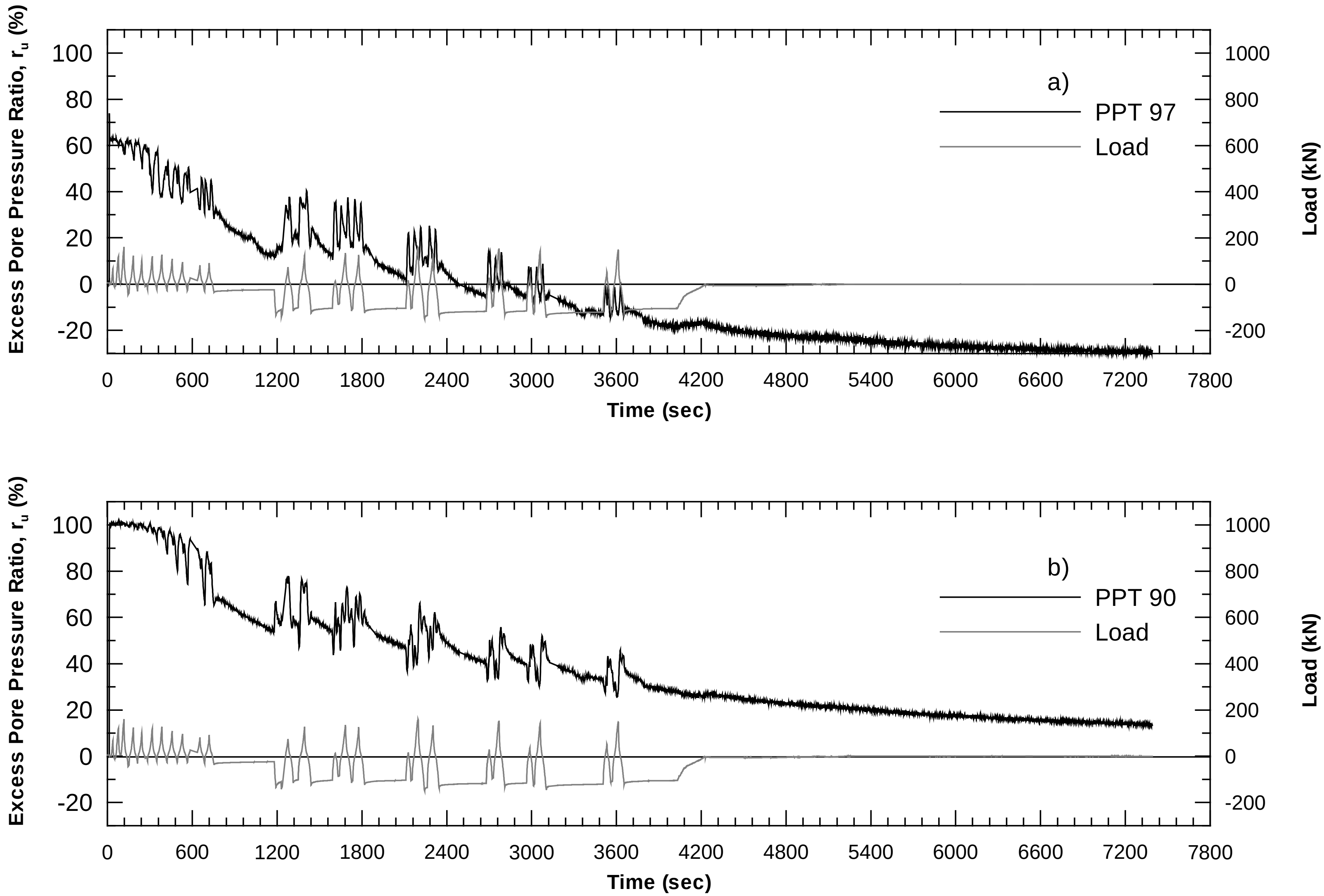


Figure 3.29 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a) PPT97 b)PPT90

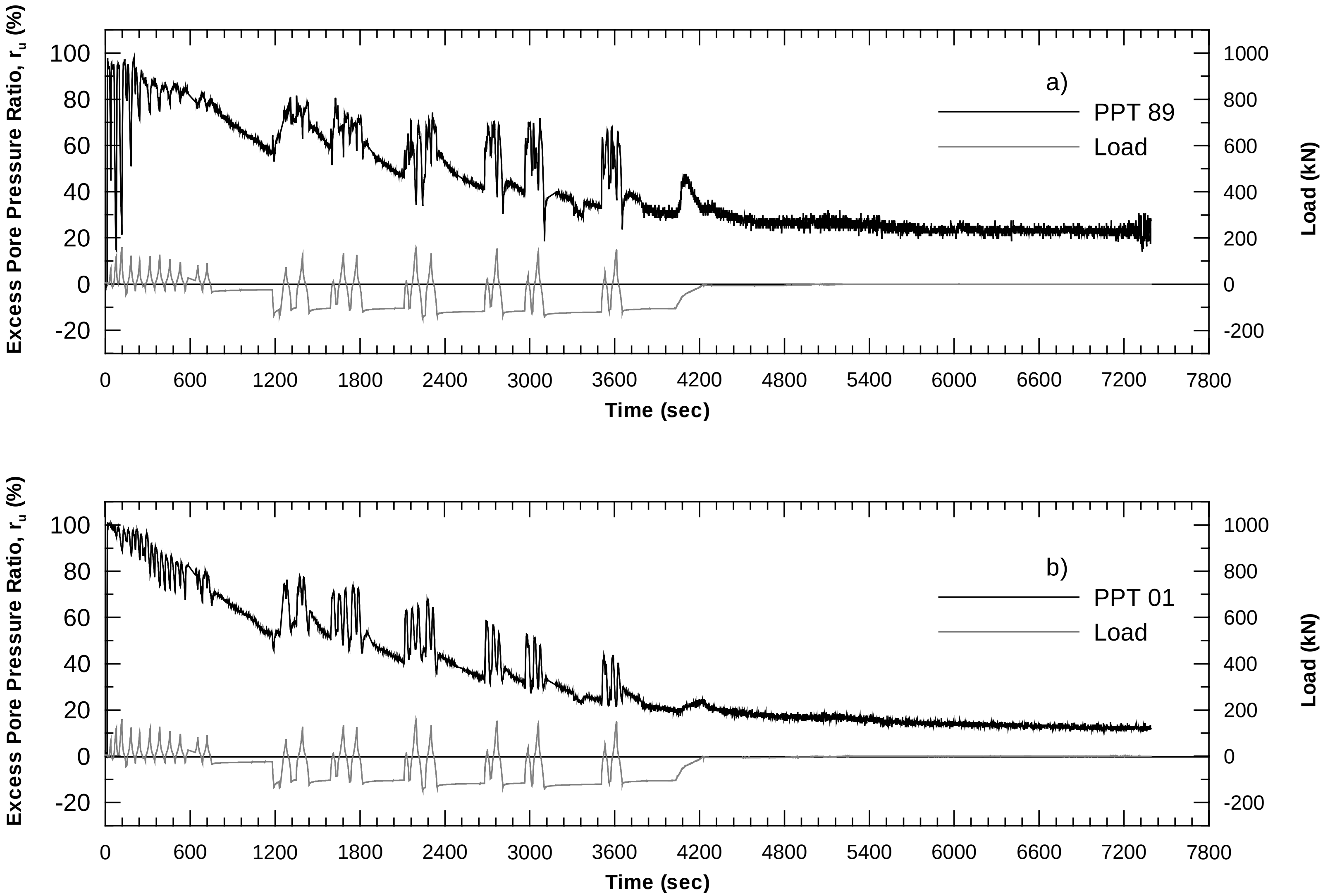
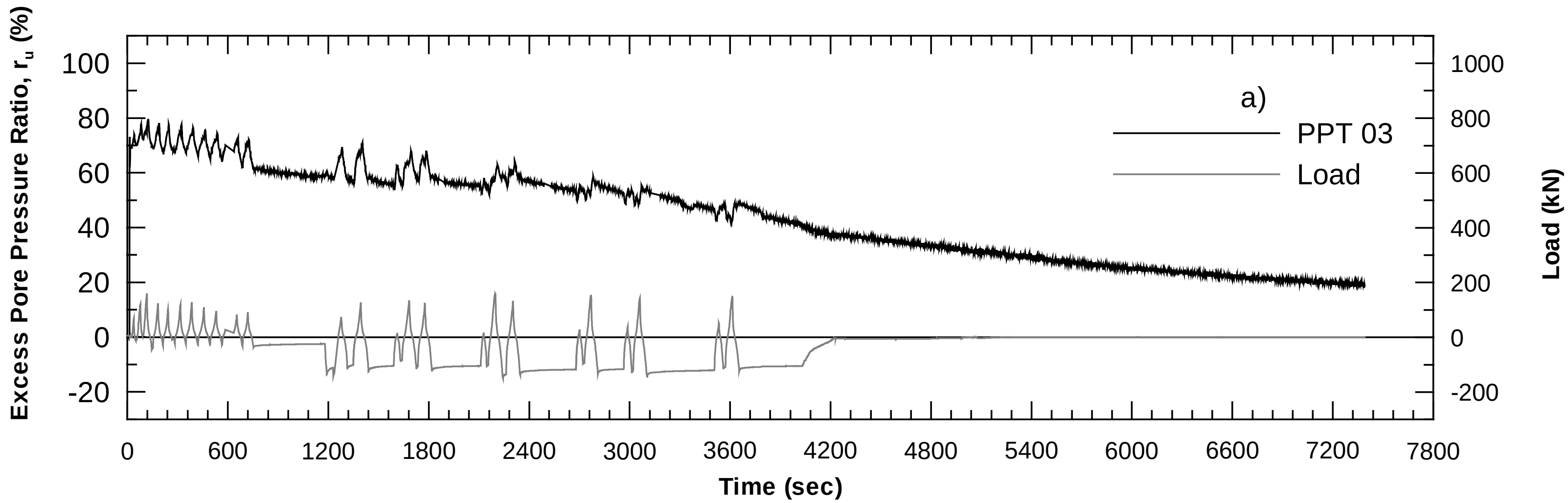


Figure 3.30 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a)PPT89 b)PPT01



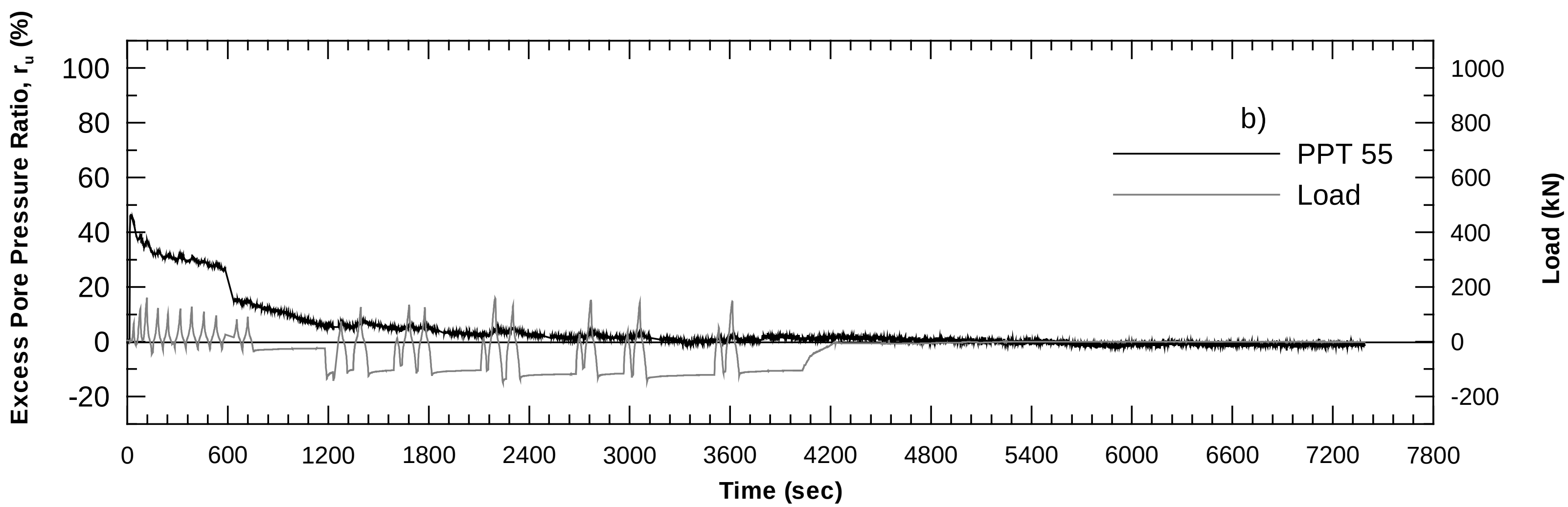


Figure 3.31 Excess Pores Pressure Ratio for Single Pile Test 1st Blast a)PPT03 b)PPT55

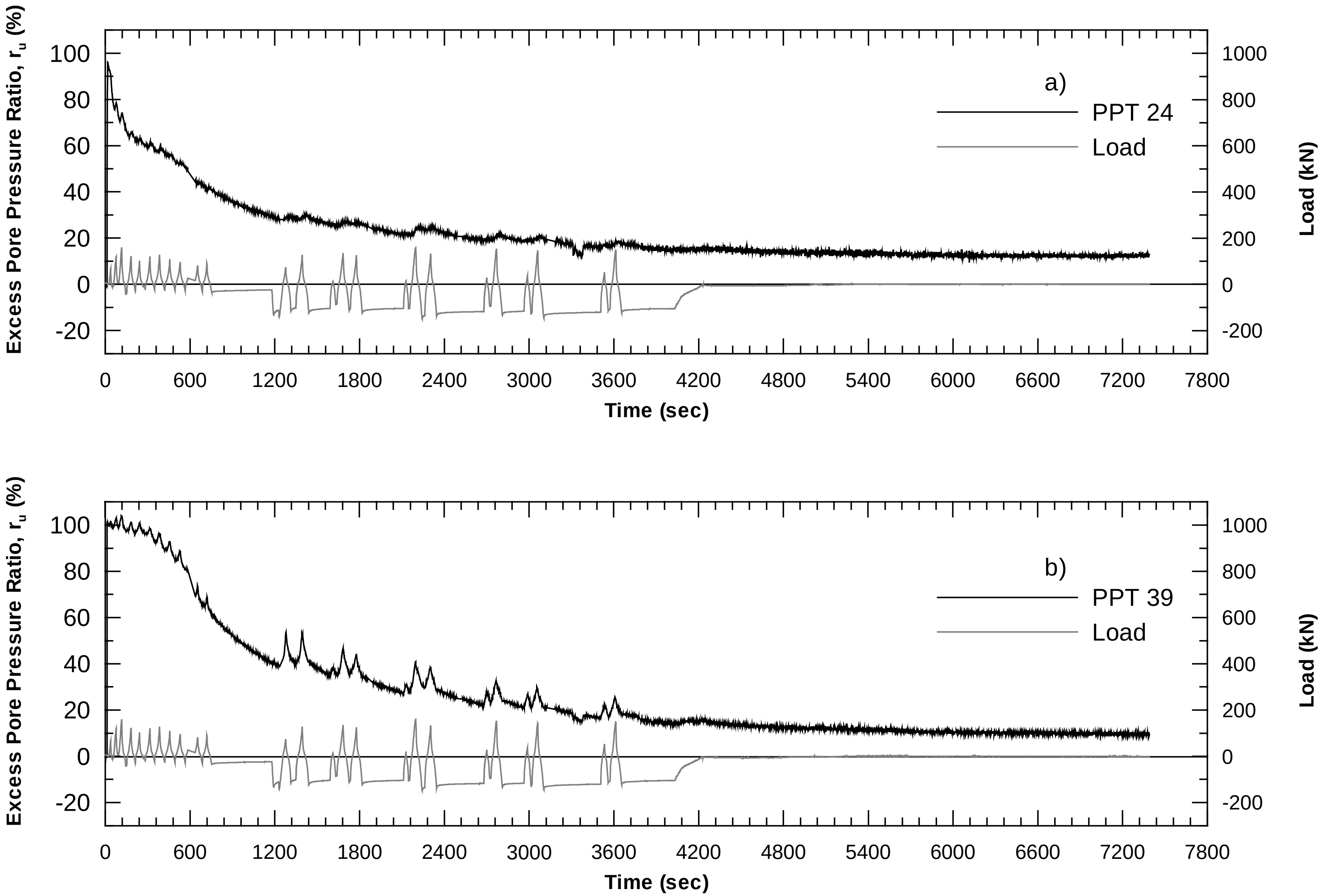


Figure 3.32 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a)PPT24 b)PPT39

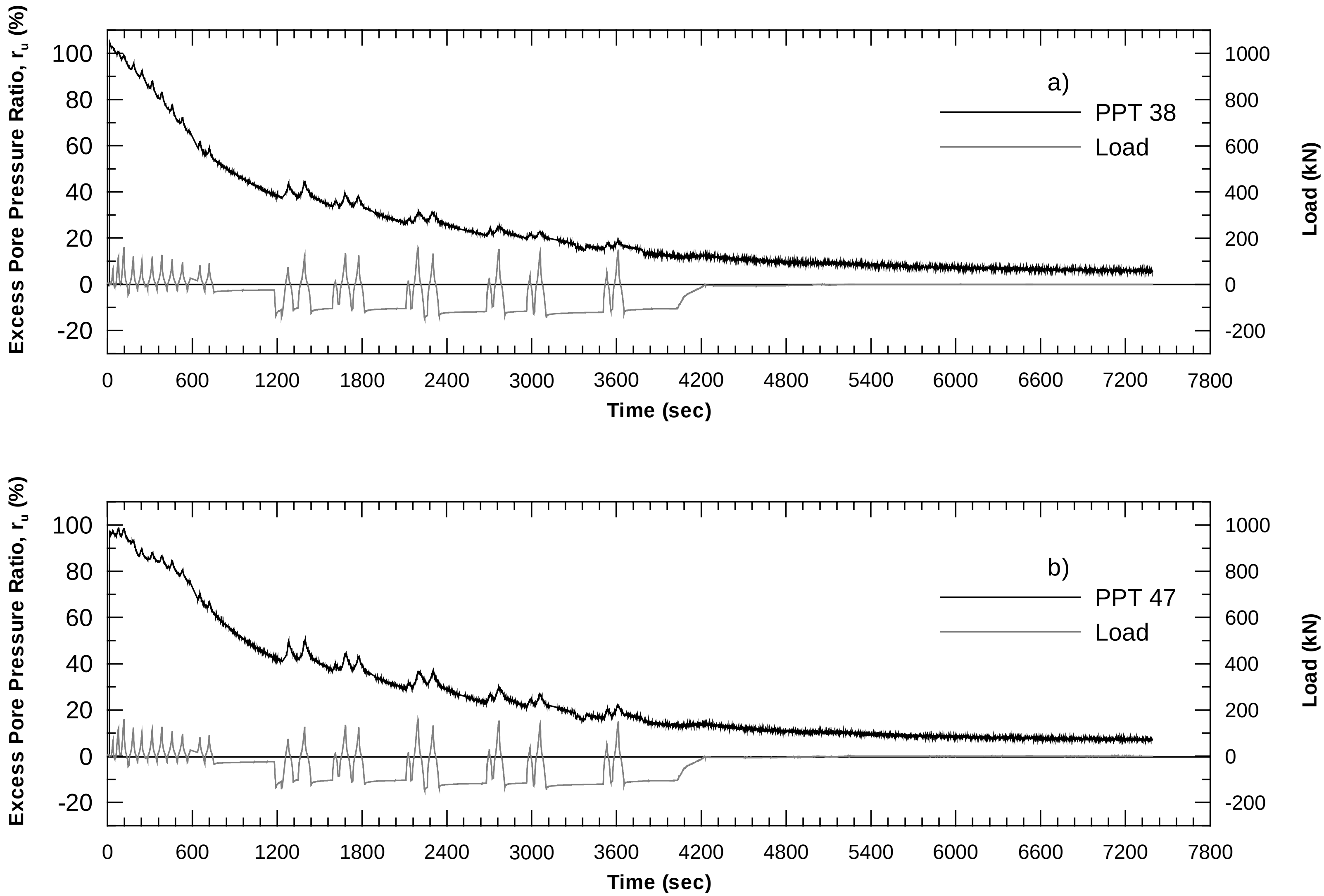


Figure 3.33 Excess Pore Pressure Ratio for Single Pile Test 1st Blast a)PPT38 b)PPT47

Figure 3.34 Sand Boils Near H-Pile During Single Pile Test

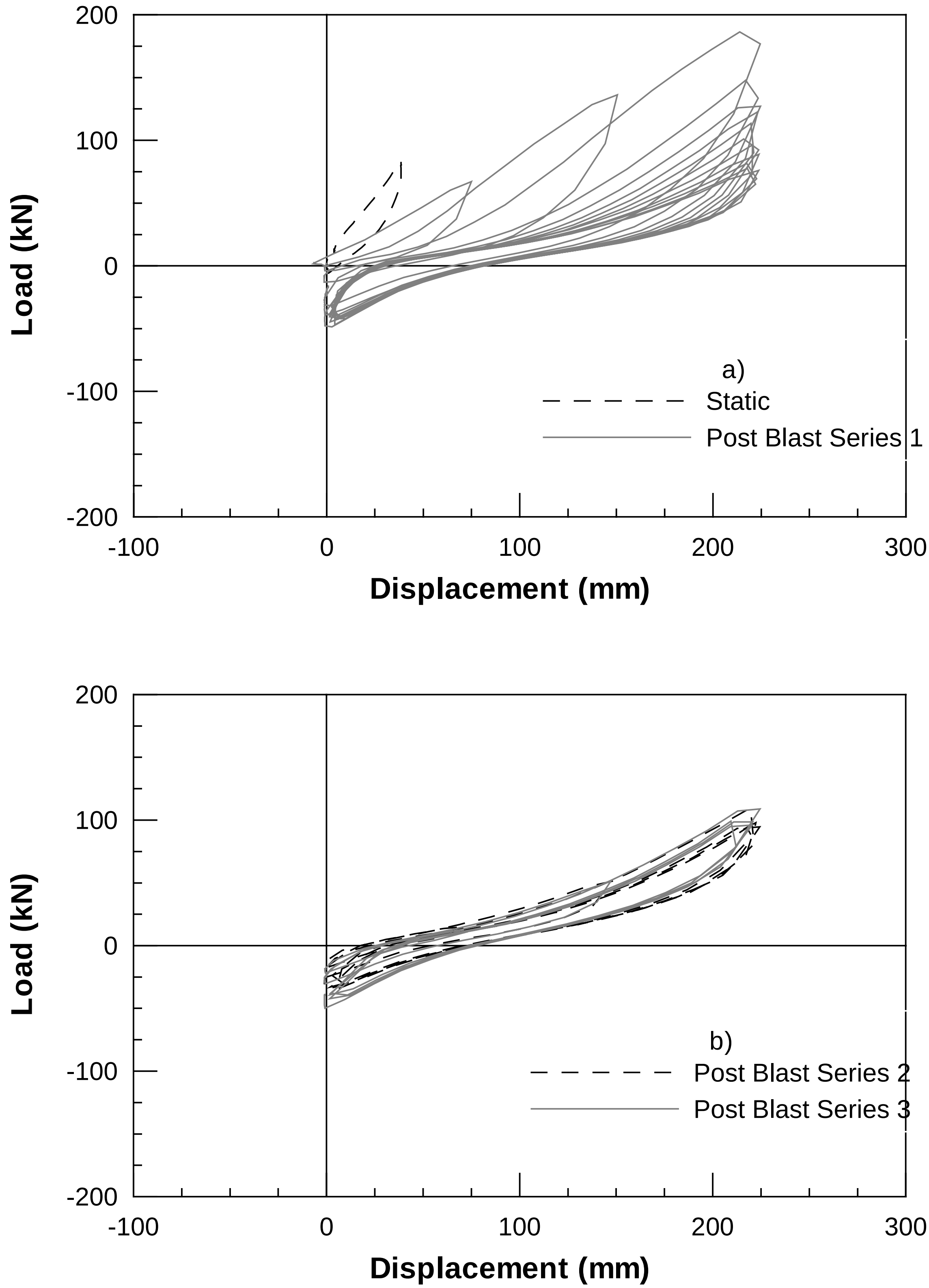
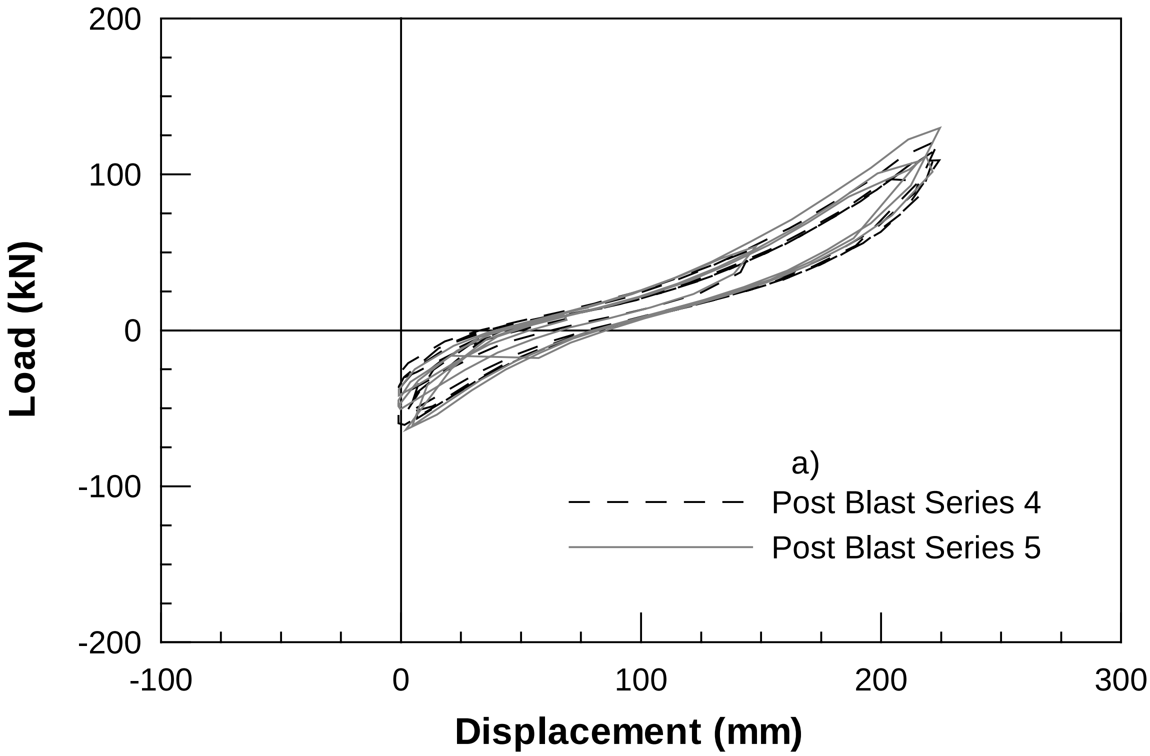


Figure 3.35 Load vs Displacement for Single Pipe Pile 2nd Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3



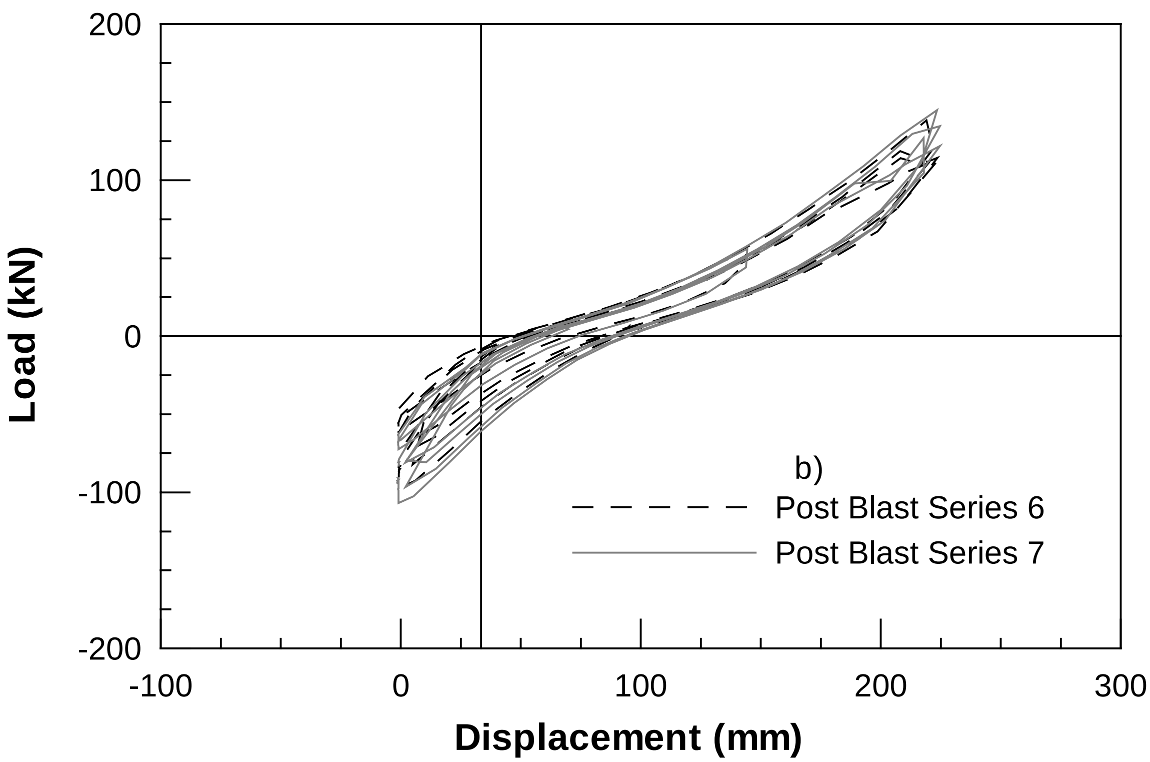


Figure 3.36 Load vs Displacement for Single Pipe Pile 2nd Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

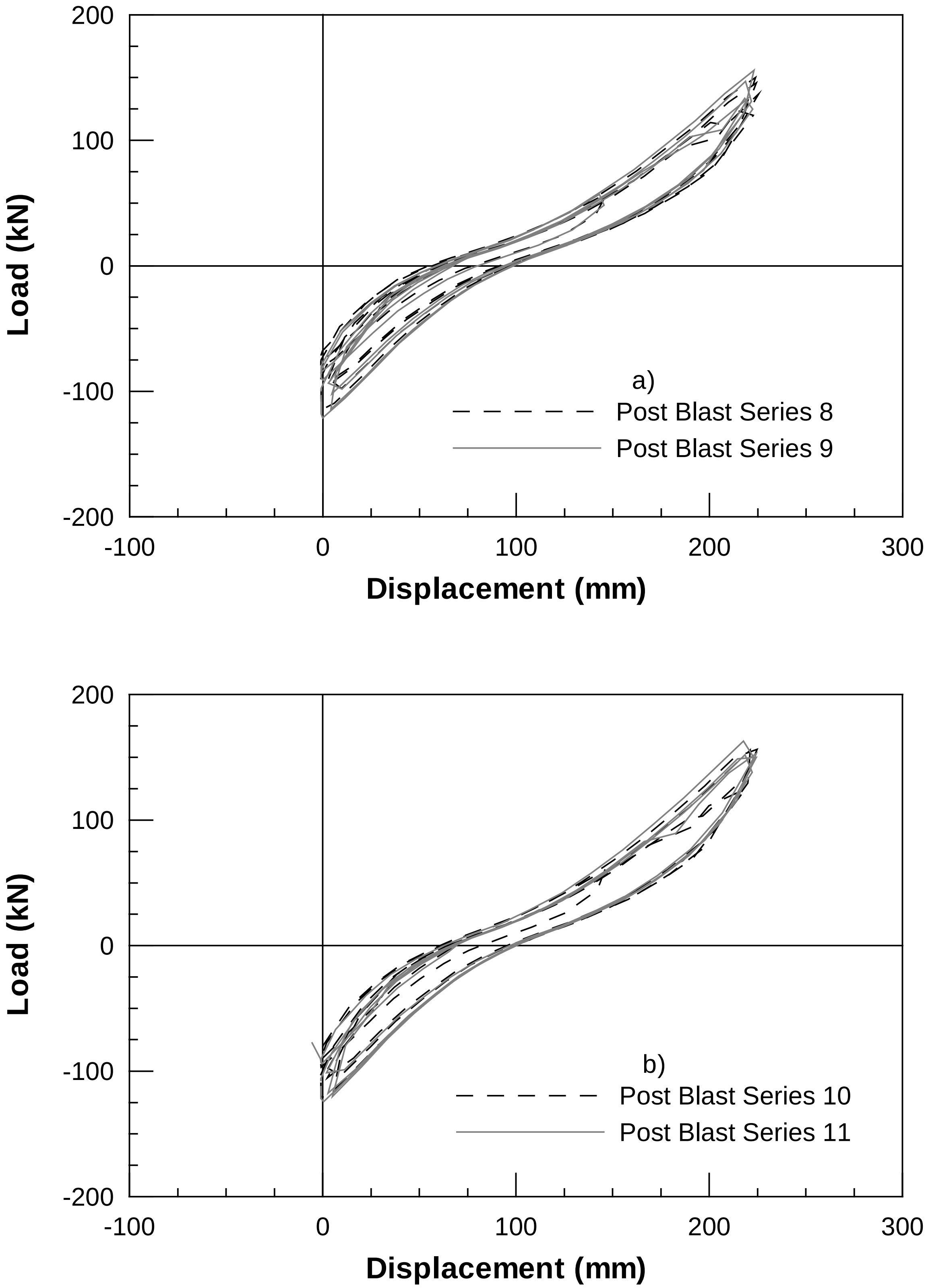


Figure 3.37 Load vs Displacement for Single Pipe Pile 2nd Blast a) Post Blast Series 8 and 9 b) Post Blast Series 10 and 11

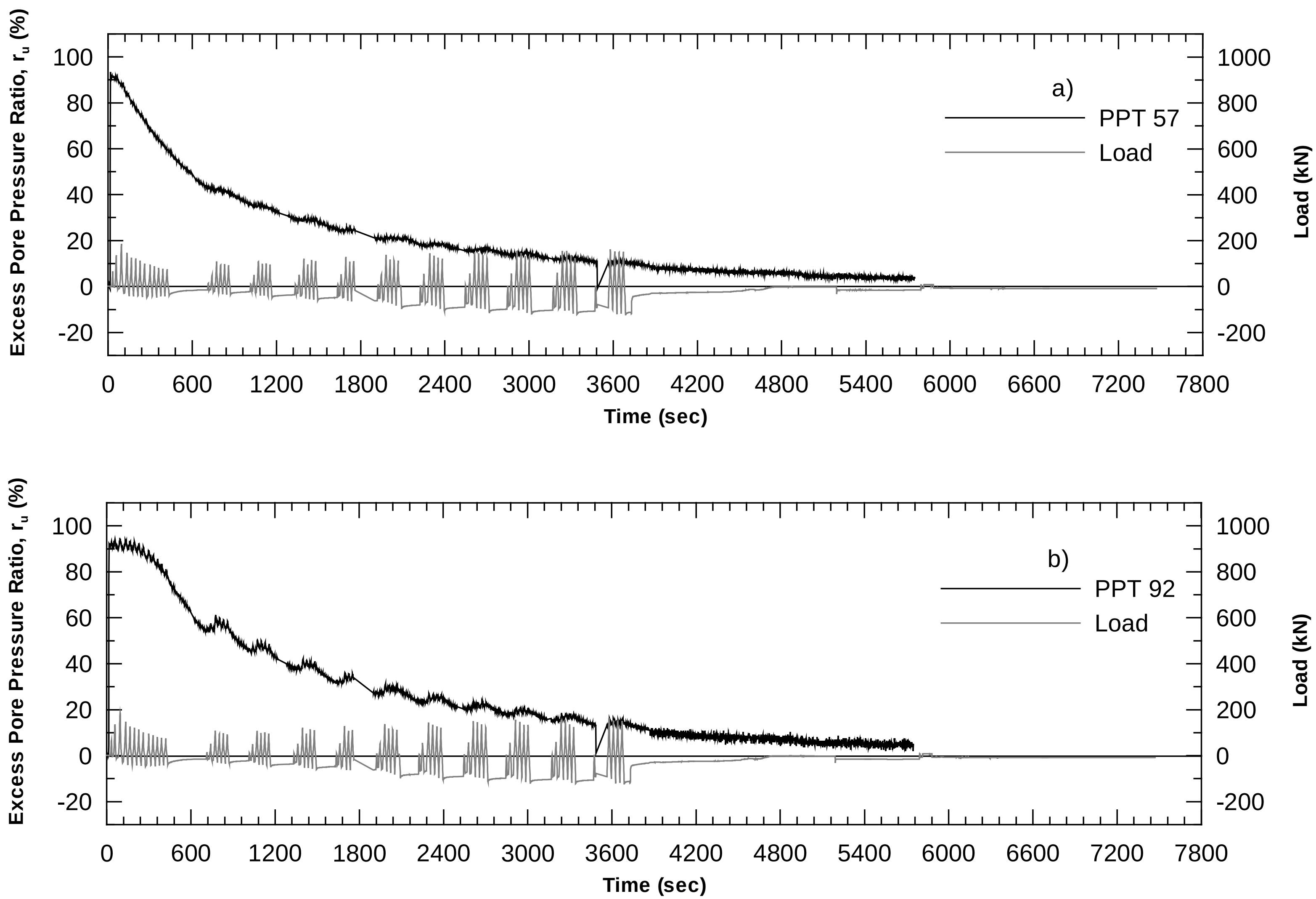


Figure 3.38 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast a)PPT57 b)PPT92

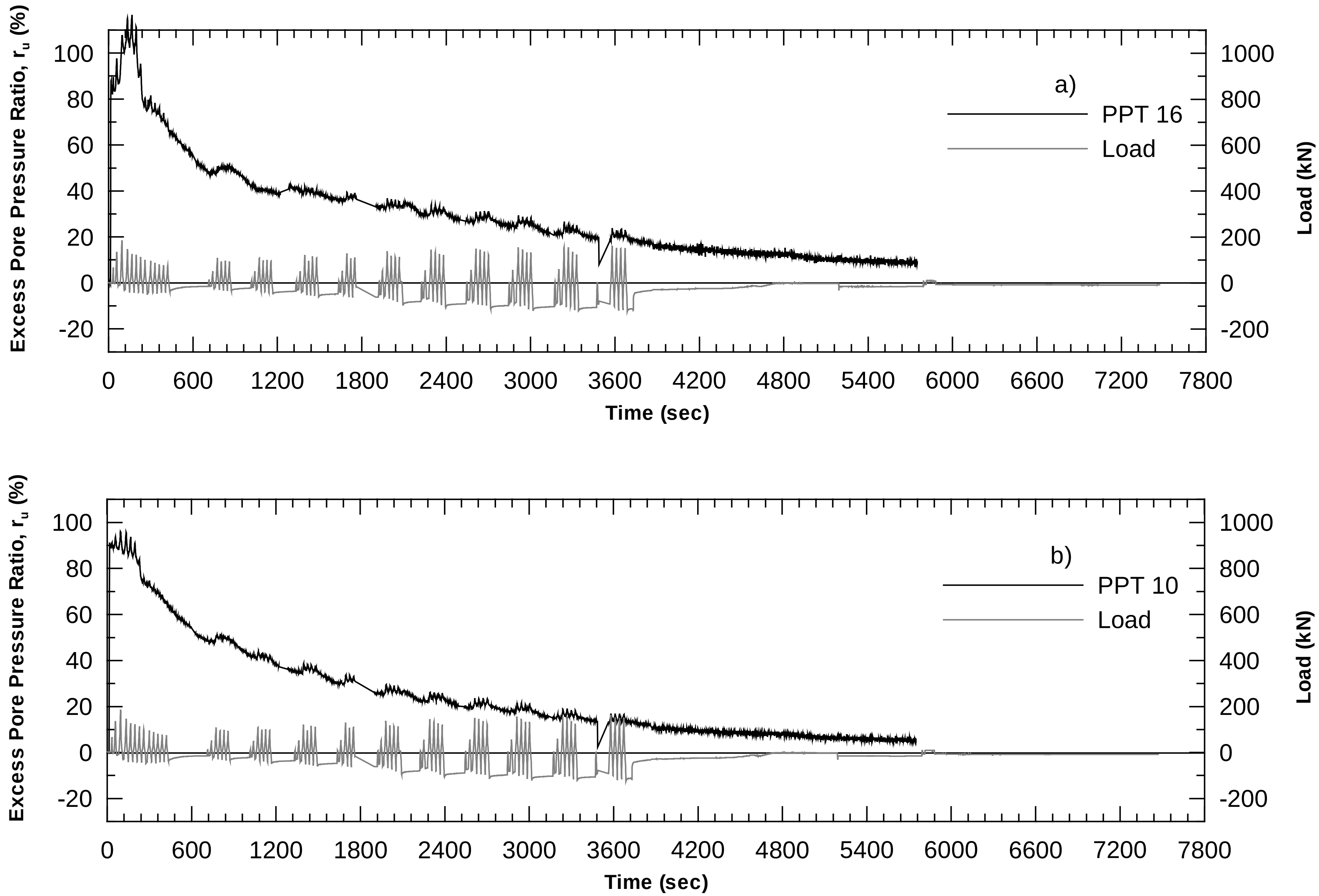


Figure 3.39 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast a)PPT16 b)PPT10

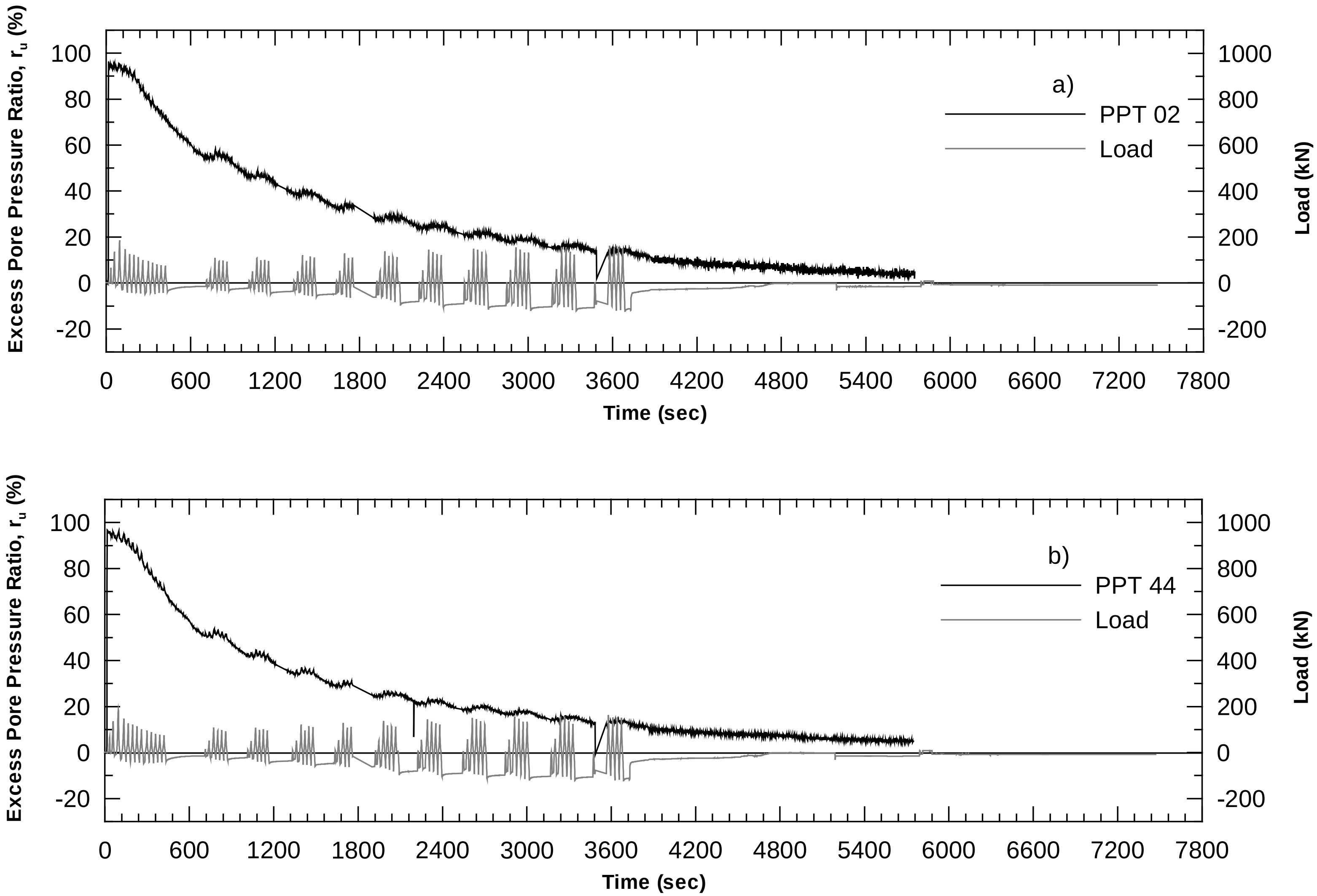


Figure 3.40 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast a)PPT02 b)PPT44

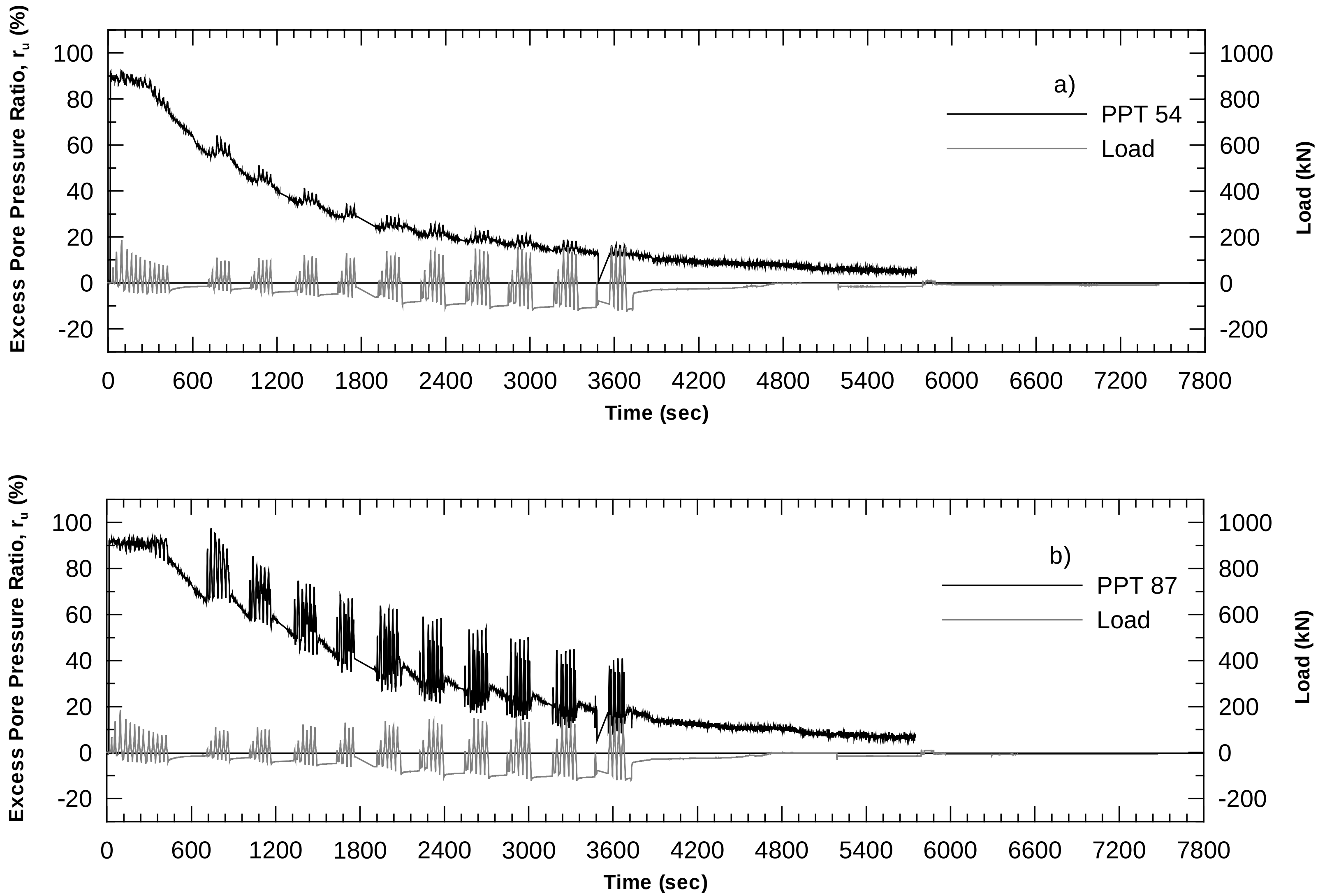


Figure 3.41 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast a)PPT54 b)PPT87

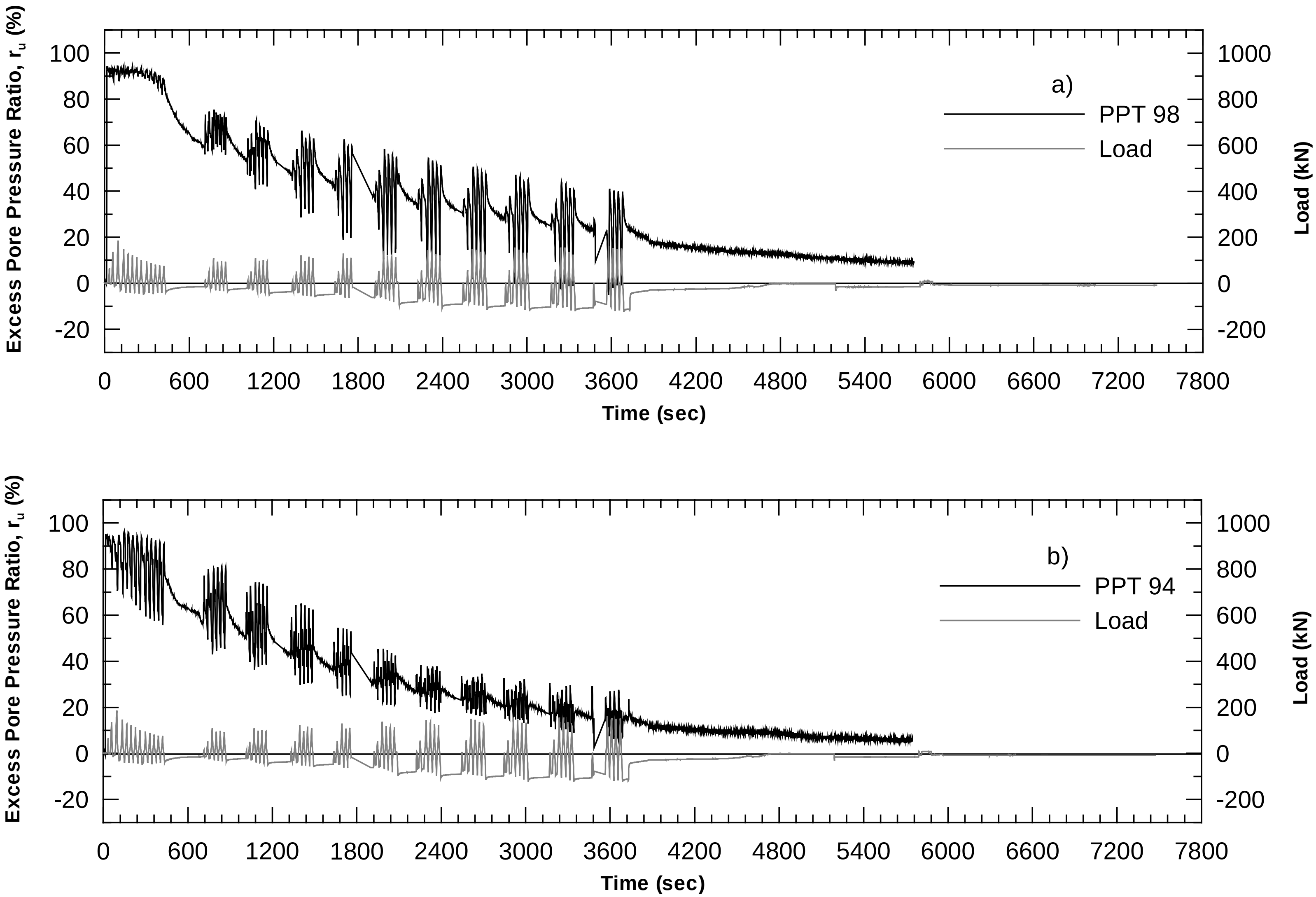


Figure 3.42 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast a)PPT98 b)PPT94

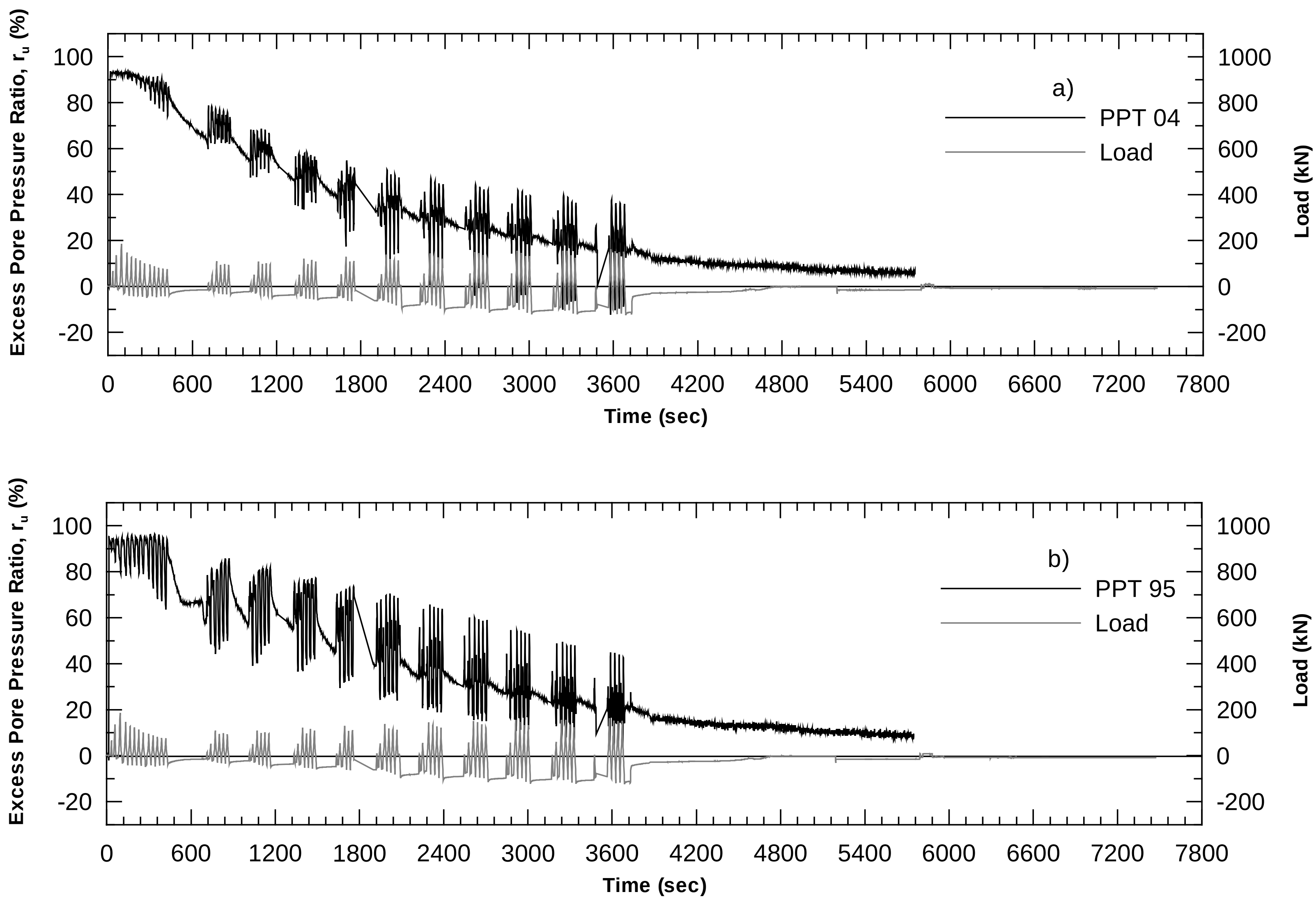


Figure 3.43 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast a)PPT04 b)PPT95

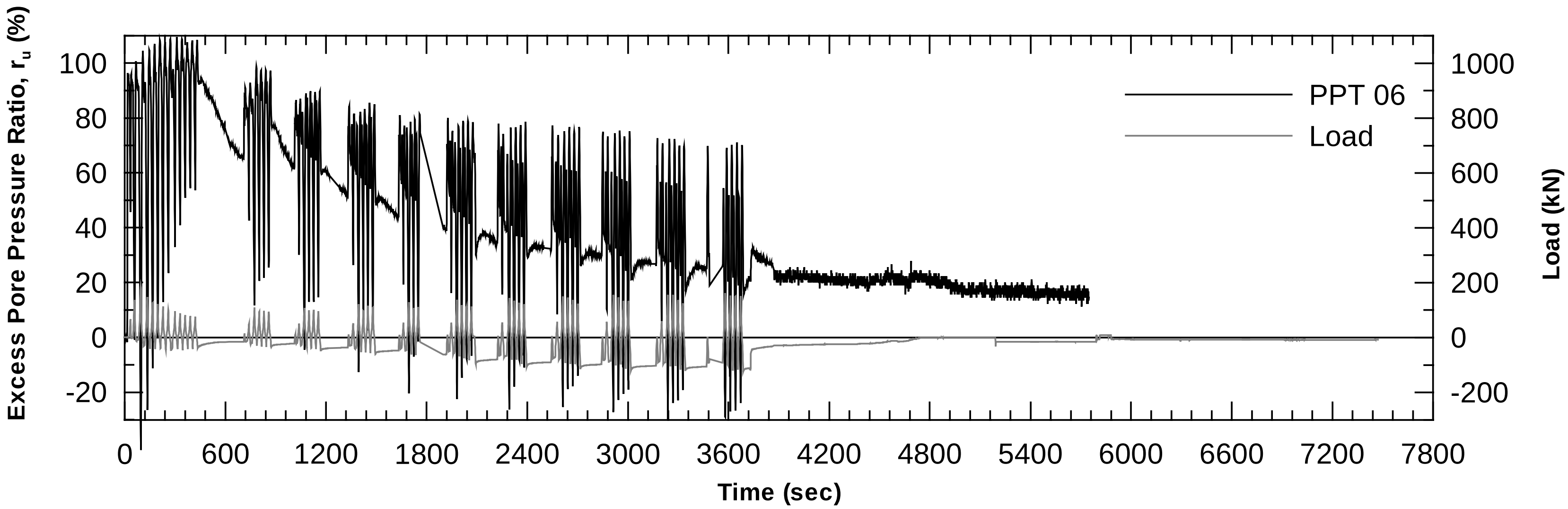


Figure 3.44 Excess Pore Pressure Ratio for Single Pile Test 2nd Blast PPT06

Note: Contours in millimeters
Coordinates in feet

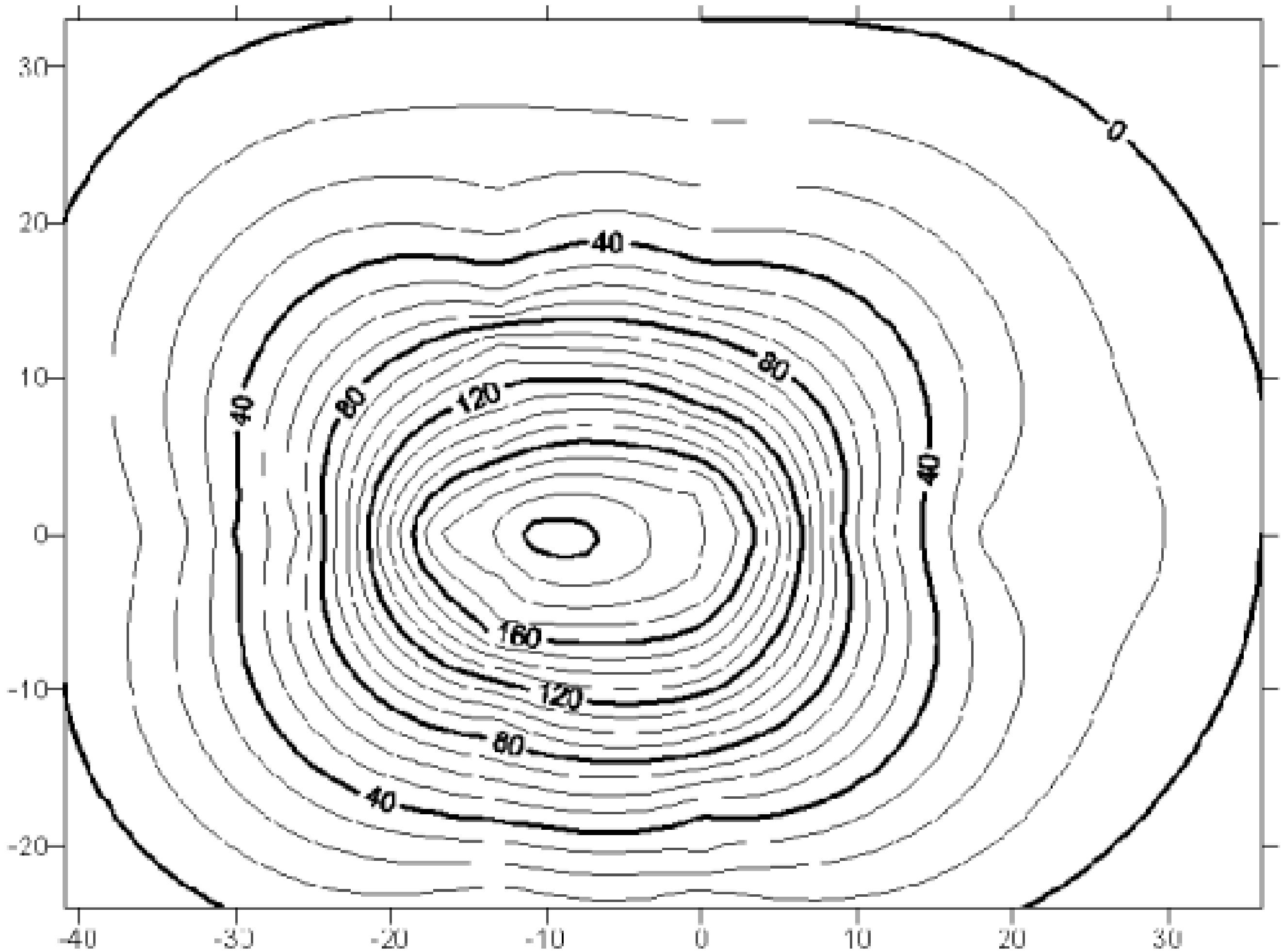


Figure 3.45 Contours of Settlement for Single Pile Test 1st Blast

4 4-PILE GROUP AND 0.6-m CISS PILE TESTS

The 4-pile group and 0.6-m CISS pile tests were performed to investigate effects of group behavior of closely spaced piles and large diameter piles in liquefied soils. The 4-pile group consisted of 0.32 m outside diameter pipe piles spaced at 3.5 pile diameters driven to a depth of 12 m. The 0.6-m CISS pile consisted of a 0.6-m outside diameter steel shell with a 13 mm wall thickness driven to a depth of 13.8 m. In all cases, driving was very easy. Soil inside the 0.6-m steel shell was augered out, and a rebar cage consisting of nine 28.7 mm (#9) bars spaced at 143 mm on center and a 9.5 mm (#3) spiral with a 114 mm spacing was then placed inside the steel shell. The steel shell was then filled with concrete. A 2.2 MN hydraulic actuator placed between the pile group and CISS pile applied lateral loads during testing. The test set-up is shown in Figure 4.1. Pile instrumentation consisted of strain gages along the depth of the pile and displacement transducers at the pile head. Pore pressure transducers were placed along the sides of the piles and at various distances in front of the piles. Data from the subsurface investigation and in-situ testing are presented along with load and displacement test results, excess pore pressure ratios, and settlement contours resulting from blasting.

4.1 4-Pile Group and 0.6-m CISS Pile Site Characterization

A detailed subsurface investigation was performed at the 4-pile group and 0.6-m CISS pile site. The location of each in-situ test performed prior to foundation testing is presented in Figure 4.2. Soil boring log BH-2 (Figure 4.3) shows a loose poorly graded sand extending from the original ground surface to a depth of 5.6 m. This is underlain by a soft gray clay layer with occasional sand seams extending to a depth of 10.2 m. A loose silty sand layer was found between 10.2 m and 13.7 m. A soft gray clay extended from 13.7 m to the bottom of the boring at 19 m. SPT blow counts in the saturated sand varied between 5 and 13. The low SPT blow counts are an indication that the sand is susceptible to liquefaction.

Cone penetration tests were performed at various stages in the test program. Seven CPT's were performed prior to lateral load testing. Six of the seven cone logs shown in Figures 4.4 through 4.9 were performed prior to foundation installation. The seventh CPT log (Figure 4.10) presents data between piles within the 4-pile group after foundation installation. There was not a significant change in the measured tip resistance after installation of the pile group. The CPT results generally show a tip resistance varying between 4 and 10 MPa in the saturated sand layer.

Shear and compression wave velocity testing was also performed at the site. Down-hole, suspension logger and seismic CPT's were used to obtain shear wave velocities. The suspension logger and seismic CPT tests were performed prior to foundation installation. The down-hole test utilized a steel pipe pile within the 4-pile group to house the geophone and obtain shear wave velocities. The suspension logger was used to obtain compression wave velocities. Results of the shear and compression wave velocity tests are shown in Figure 4.11. Additional shear wave velocity tests were performed after the first lateral load test and again after stone columns were installed. A comparison of down-hole shear wave velocity profiles at various stages of the testing program is shown in Figure 4.12.

4.2 4-Pile Group and 0.6-m CISS Pile Test Results

The first lateral load test at the 4-pile group/0.6-m CISS pile site was conducted on February 4, 1999. A plan view of this test set-up is shown in Figure 4.13. Similar to the single pile test, a static test was performed prior to blasting. The actuator loading was controlled using displacement limits. When either the CISS pile or the 4-pile group reached the target displacement, loading began to decrease until the zero displacement target was achieved.

4.2.1 Load-Displacement Results for Test 1

The static test consisted of pulling the piles together up to a displacement of 38 mm. The piles were then cyclically loaded through a series of displacements after blasting where the piles were pushed apart. The first of ten post-blast series consisted of the piles being displaced through one 76 mm cycle, one 152 mm cycle, and eleven 228 mm cycles. Post-blast Series 2 through 6 consisted of one 76 mm cycle, one 152 mm cycle and four 228 mm cycles. Post-blast Series 7 through 10 consisted of one 75 mm cycle and four 150 mm cycles. The magnitude of the displacement was reduced for the later series to prevent individual load cells within the 4-pile group from being loaded beyond the calibrated region. Load versus displacement for the 4-pile group is presented in Figures 4.14 through 4.16. Load versus displacement for the 0.6-m CISS pile is presented in Figures 4.17 through 4.19. Both the 4-pile group and 0.6-m CISS pile showed an initial decrease in stiffness with each cycle of loading and then an increase in stiffness as pore water pressures decreased.

4.2.2 Excess Pore Pressure Response for Test 1

Transducers used to measure pore water pressure were placed throughout the test site. The depth and location of each PPT is shown in Figure 4.20. Measured pore water pressures were used to calculate the excess pore pressure ratio at each transducer location. Excess pore pressure ratios and the variation of load with time are shown in Figures 4.21 through 4.31. PPT data to the east (near the 4-pile group) is presented first, and transducer data to the west (around the CISS pile) is presented last. Excess pore pressure ratios varied according to depth and location. Excess pore pressure ratios at a distance of 4.2 m from the CISS pile or pile group generally tend to be lower than ratios near the pile. Initial excess pore pressure ratios ranged from as low as 40 percent to just over 100 percent. The change in excess pore pressure ratio close to the CISS pile and pile group was affected dramatically by loading and unloading of the pile as seen in the figures.

4.3 Site Characterization after Installation of Stone Columns

Within days of the first blast, the excavation was backfilled, and twenty-four 0.9-m diameter stone columns were installed in a 4 by 6 grid centered around the piles. The stone columns extended through the sand layer and were spaced at approximately 2.4 meters on center. Installation of the stone columns is shown in Figure 4.32. A series of CPT's were performed after the first blast prior to stone column installation and after the stone columns were installed. The location of the stone columns and in-situ tests after the first blast is shown in Figure 4.33. CPT results prior to stone column installation are presented in Figures 4.34 through 4.37. CPT results after stone column installation are presented in Figures 4.38 through 4.47. Measured tip resistance from cone penetration tests prior to stone column installation rarely exceed 10 MPa. Tip resistance values after stone column installation reach as high as 35 MPa. The difference between tip resistance before and after stone column installation provided evidence that the loose sand deposit had been significantly densified as a result of the stone columns.

4.4 Stone Column Test Results

The lateral load tests provided further evidence of improved ground behavior as a result of installing stone columns. In addition to the standard 16 charges used to liquefy the site, an additional 14 charges were placed around the perimeter of the improved area in an attempt to liquefy the area surrounding the stone columns.

4.4.1 Load-Displacement Results with Stone Columns

Load versus displacement for the 4-pile group and 0.6-m CISS pile is presented in Figures 4.48 through 4.50 and Figures 4.51 through 4.53, respectively. The static tests were run using displacement control. The target displacements for the static test were 3 mm and 38 mm. The piles were unloaded between the two target displacements. The post blast testing was controlled using load limits as the piles were pushed apart to prevent individual load cells within the 4-pile group from being loaded beyond the calibrated region. As the piles were unloaded, a zero displacement target was set to control the actuator. The first post blast series consisted of one cycle with a maximum load of 450 kN and twelve cycles at a maximum load of 600 kN. The remaining post blast cycles consisted of one 200 kN loading cycle and four 600 kN loading cycles. Stiffness of the pile group and the CISS pile reduced during the first post blast series. Displacements continued to increase slightly during each loading cycle. The effect of pore water pressure buildup on the load-displacement response was minimal.

4.4.2 Excess Pore Pressure Response with Stone Columns

The pore water pressure transducers were again placed around the pile group and CISS pile. In addition to the transducers around the piles, an array of transducers in the north-south direction was placed just to the east of the CISS pile. The depth and location of each transducer is shown in Figure 4.54. The excess pore pressure ratios are shown in Figures 4.55 through 4.63 with data to the west (near the CISS pile) presented first, and transducer data to the east (around the 4-pile group) presented last. Immediately after blasting, the excess pore pressure ratios ranged between 30% and 85% adjacent to the CISS pile and between 50% and 95% adjacent to the pile group. Within one minute after blasting, the pore pressure ratios had dropped dramatically, and approximately 10 minutes after blasting, the excess pore pressure ratios were generally 10% ore less. The stone columns sufficiently densified the loose sand to prevent liquefaction from occurring during this test.

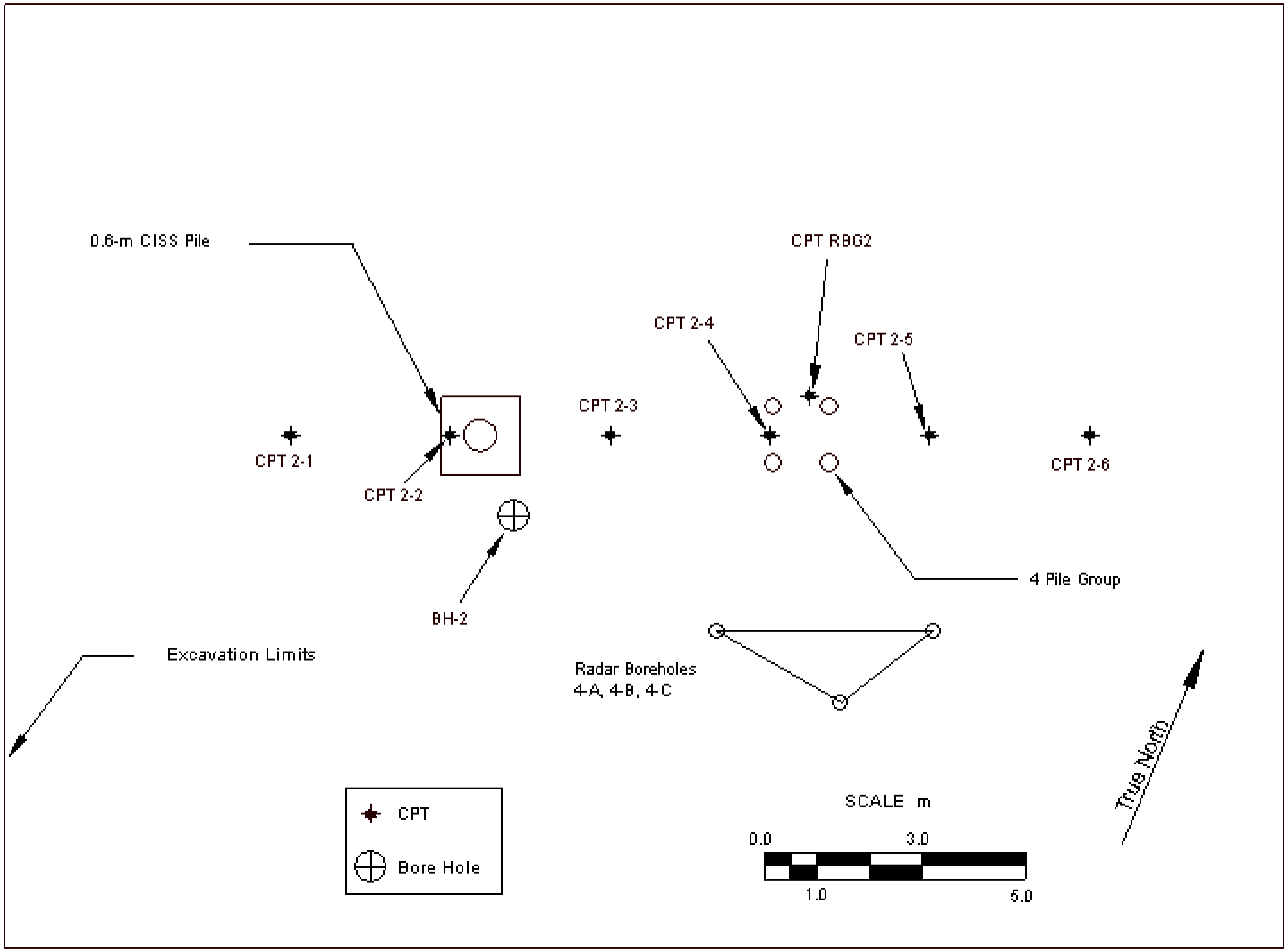
4.5 Blast Induced Settlement

Upon completion of each test, a settlement survey was conducted. Settlement contours are presented in Figure 4.64 due to the first blast and Figure 4.65 due to the first and second blast combined. The contours of settlement are in millimeters, while the coordinates from the edge of the excavation are in meters. The amount of settlement following the first blast was very similar to the single pile test, though essentially no settlement was measured due to the second blast.

4.6 Summary

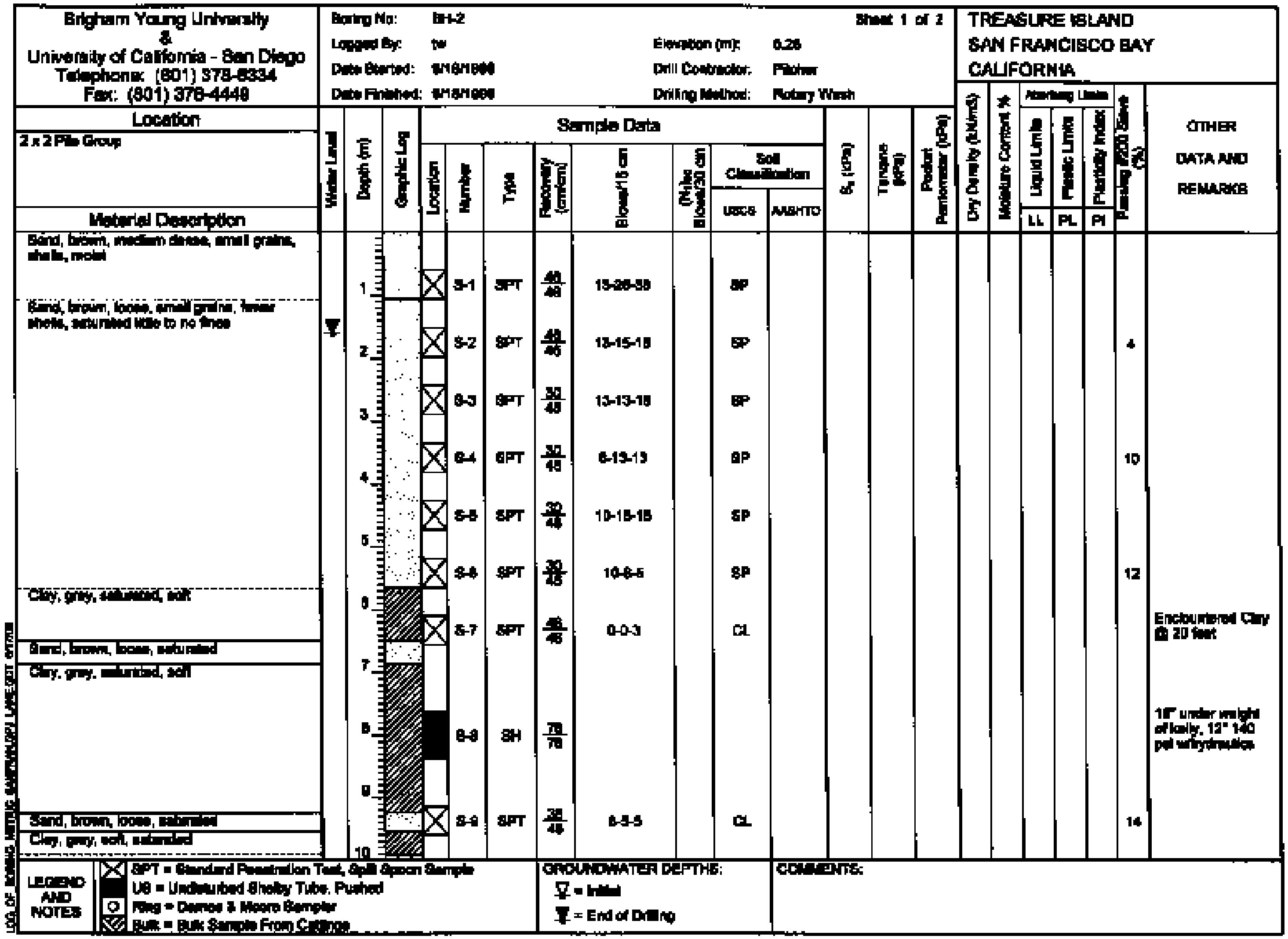
In summary, two series of lateral load tests were performed on the 4-pile group and a 0.6-m CISS pile. Both static and post-liquefaction tests were performed. Foundation stiffness was observed to decrease after detonation of the explosives due to an increase in the pore water pressure. As the pore water pressure dissipated, foundation stiffness increased. The effect of pore-pressure on foundation stiffness is discussed in more detail in Chapters 7 and 8. It was observed that the 0.6-m CISS pile was stiffer than the 4-pile group during the static test, but was less stiff during the liquefaction testing. The installation of the stone columns dramatically increased the soil density, preventing liquefaction from occurring during the second blast. As a result, the foundation stiffness increased substantially as compared to the liquefied case for the first blast. This is some of the first full-scale quantitative evidence of the effectiveness of stone columns in increasing the lateral capacity of pile foundations in liquefiable soil.

Figure 4.1 4-Pile Group/0.6-m CISS Pile Test Set-Up



Note:
CPT No's 1-6 Completed 10/19/98
CPT No's RBG-1 Completed 1/30/99

Figure 4.2 Location Map of Insitu Tests Prior to Blasting



Number	Type	Recovery (cm/cm)	Blows/15 cm	USCS	Passing #200 Sieve (%)	Other Data and Remarks
S-1	SPT	46/46	15-26-35	SP		
S-2	SPT	46/46	13-15-15	SP	4	
S-3	SPT	30/45	13-13-15	SP		
S-4	SPT	30/45	6-13-13	SP	10	
S-5	SPT	30/45	10-15-15	SP		
S-6	SPT	30/45	10-6-5	SP	12	
S-7	SPT	46/46	0-0-3	CL		Encountered Clay @ 20 feet
S-8	SH	76/76				18" under weight of kelly, 12" 140 psi w/hydraulics
S-9	SPT	38/45	5-5-5	CL	14	

Figure 4.3a Soil Boring Log for 4-Pile Group/0.6m CISS Pile Area

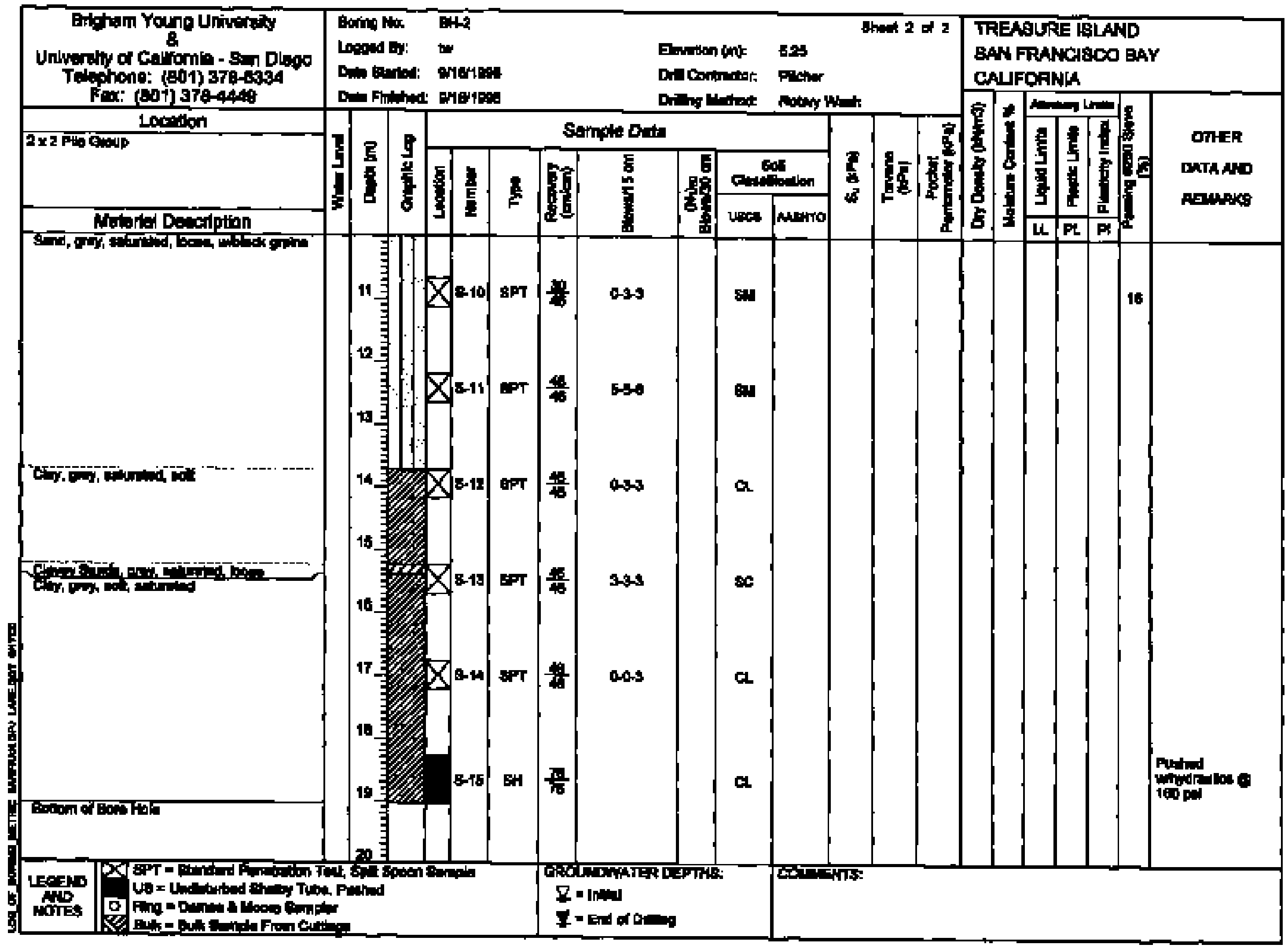


Figure 4.3b Soil Boring Log for 4-Pile Group/0.6m CISS Pile Area

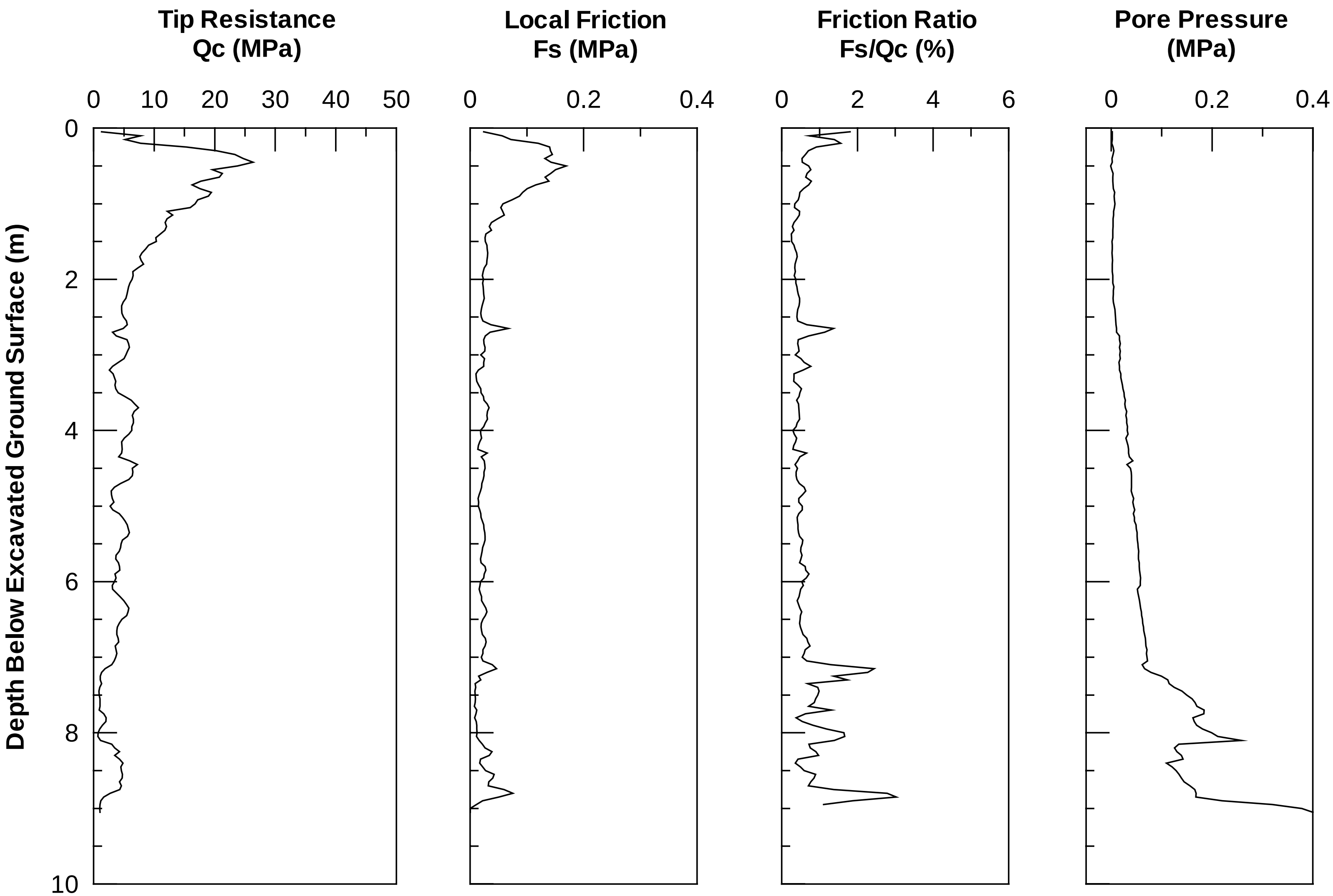


Figure 4.4 CPT2-1 Logs for 4 Pile Group/0.6m CISS Pile Test Area (10-30-98)

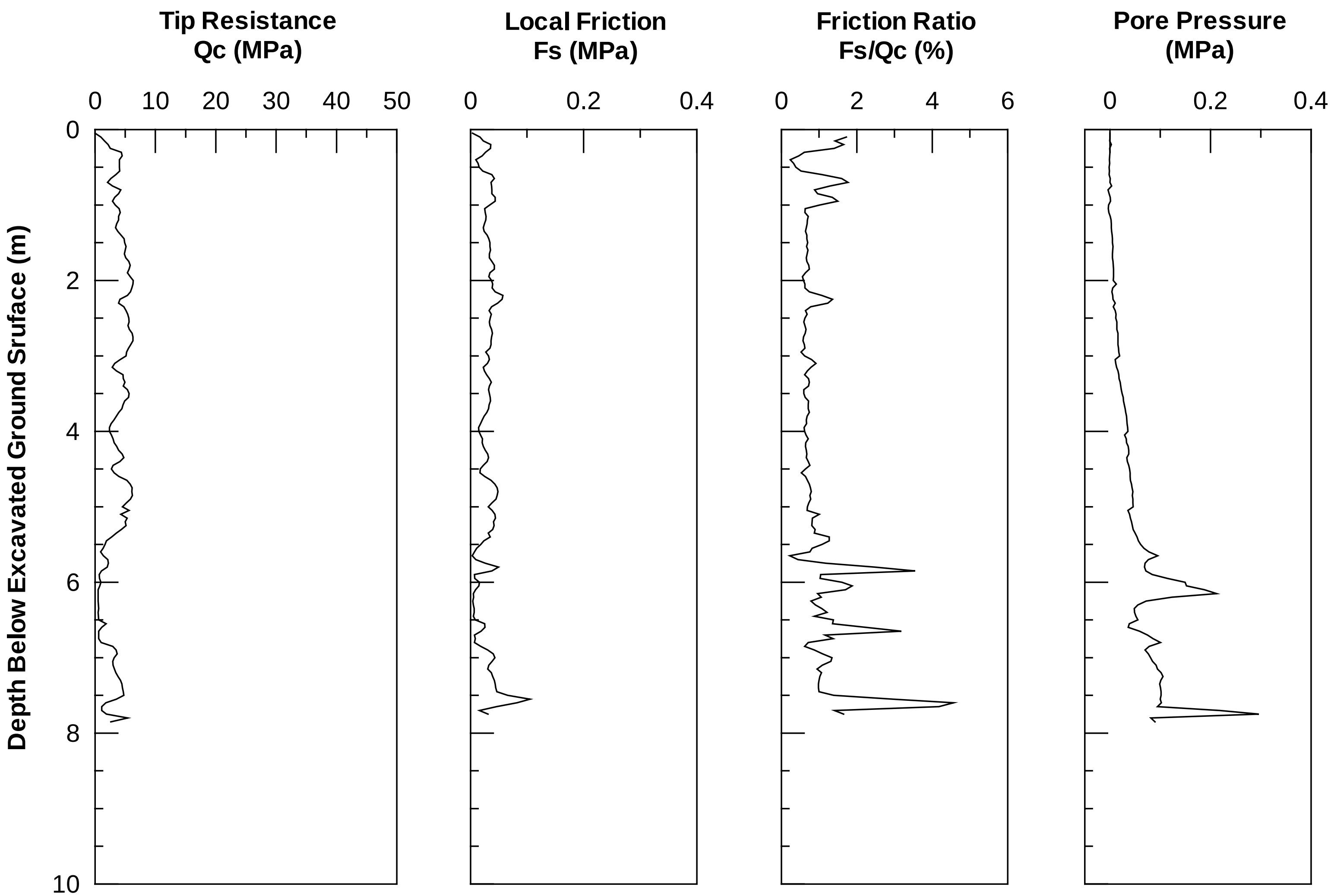


Figure 4.5 CPT2-2 Logs for 4-Pile Group/0.6m CISS Pile Test Area (11-06-98)

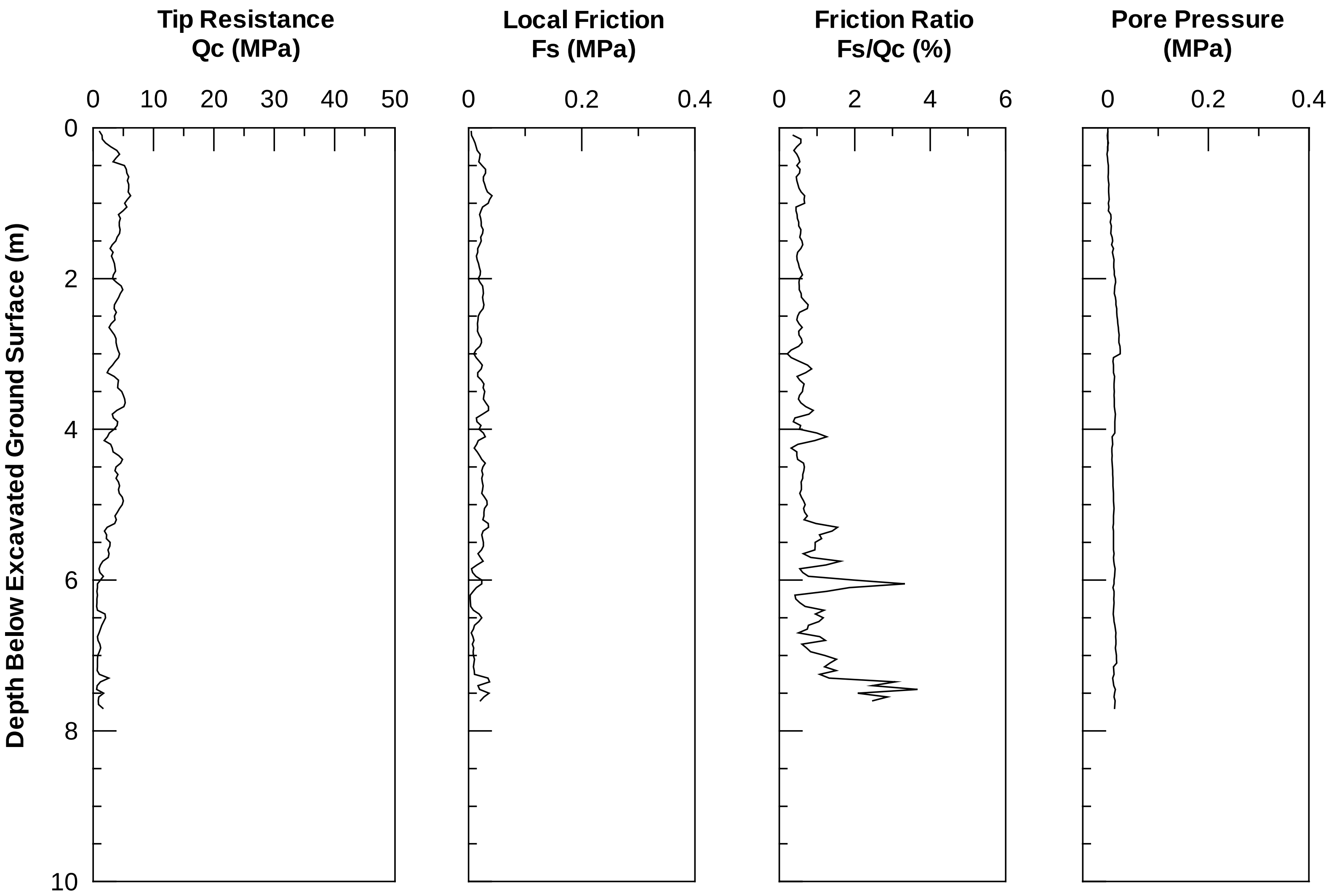


Figure 4.6 CPT2-3A Logs for 4-Pile Group/0.6m CISS Pile Test Area (11-06-98)

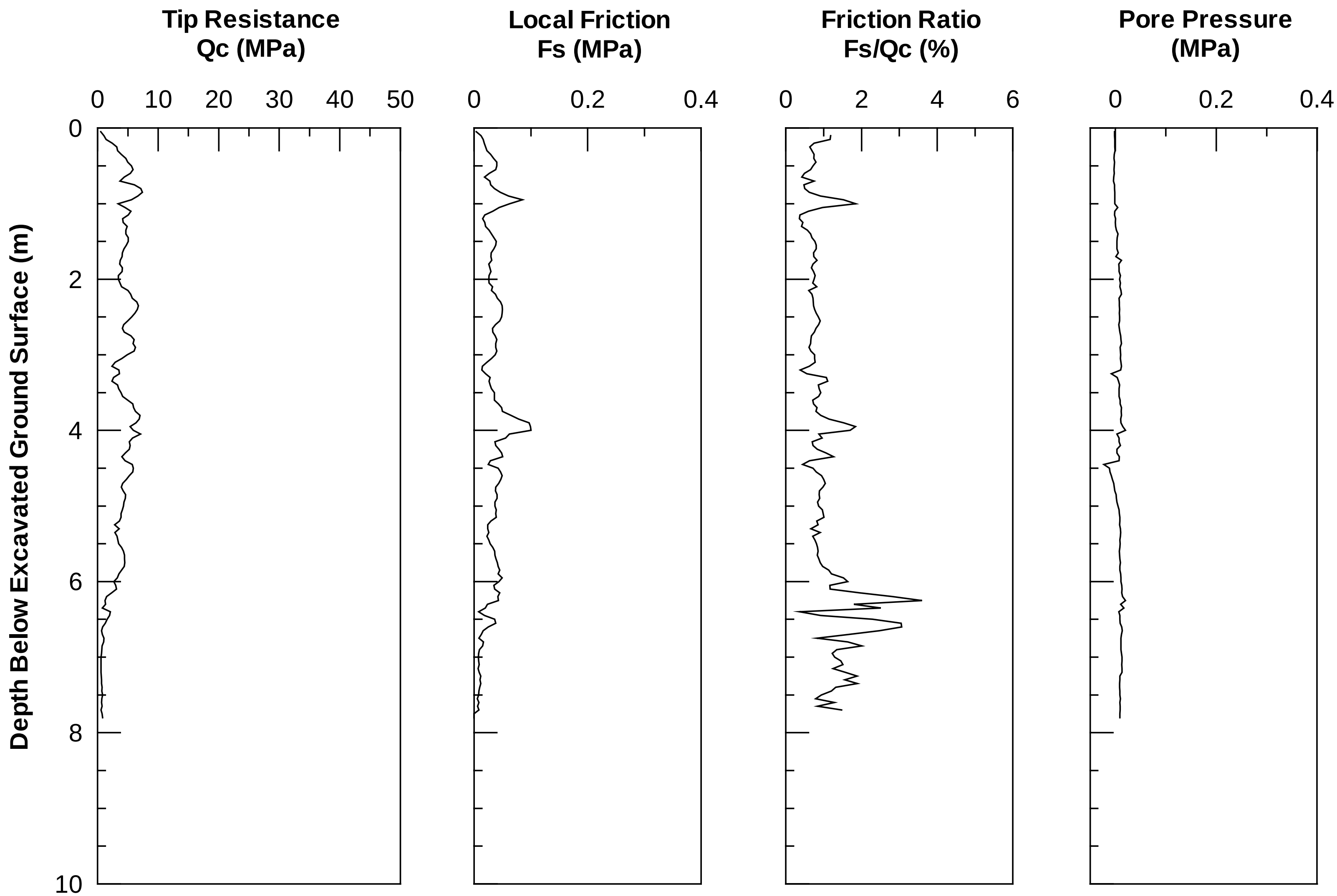


Figure 4.7 CPT2-4 Logs for 4-Pile Group/0.6m CISS Pile Test Area (10-30-98)

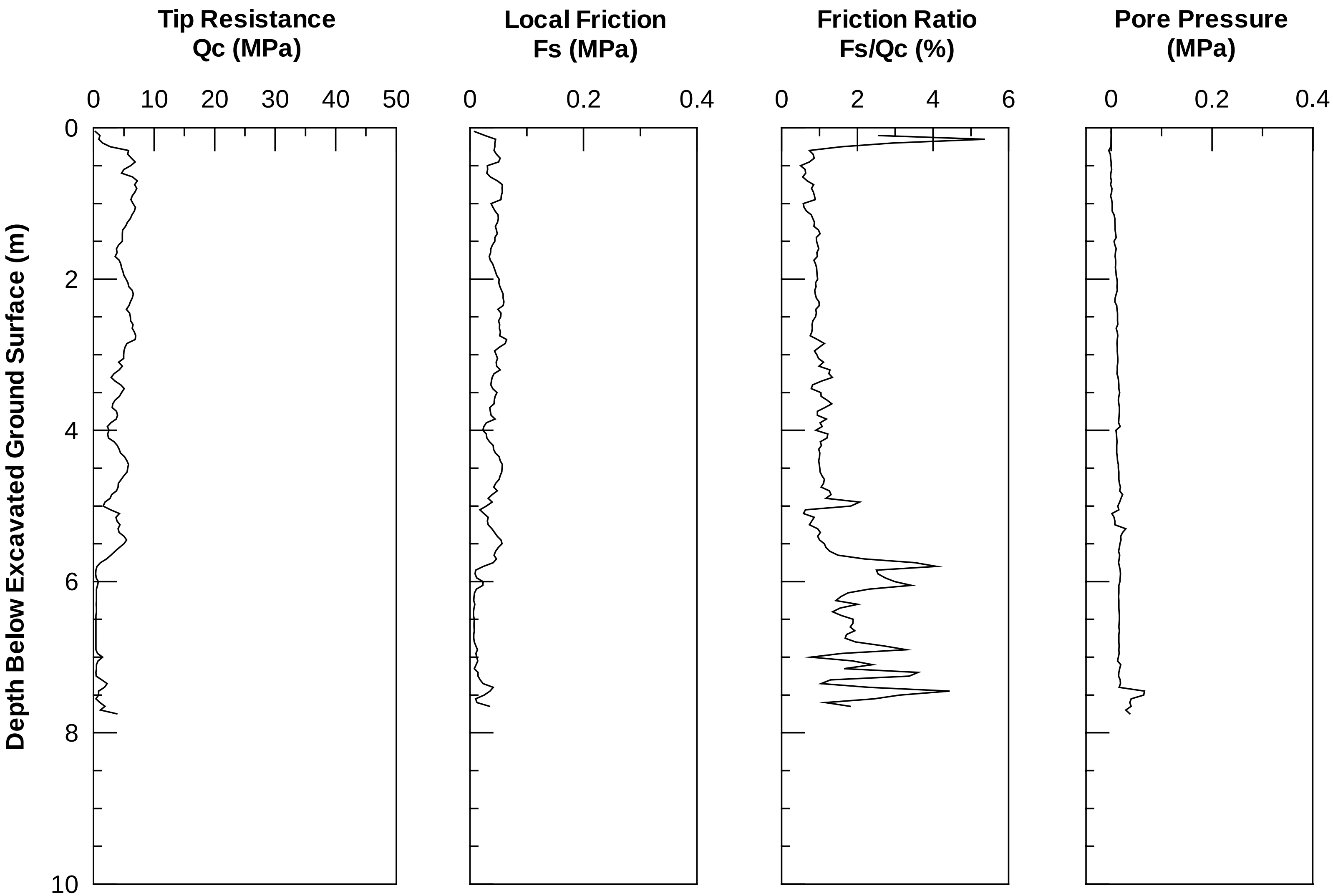


Figure 4.8 CPT2-5 Logs for 4-Pile Group/0.6m CISS Pile Test Area (10-30-98)

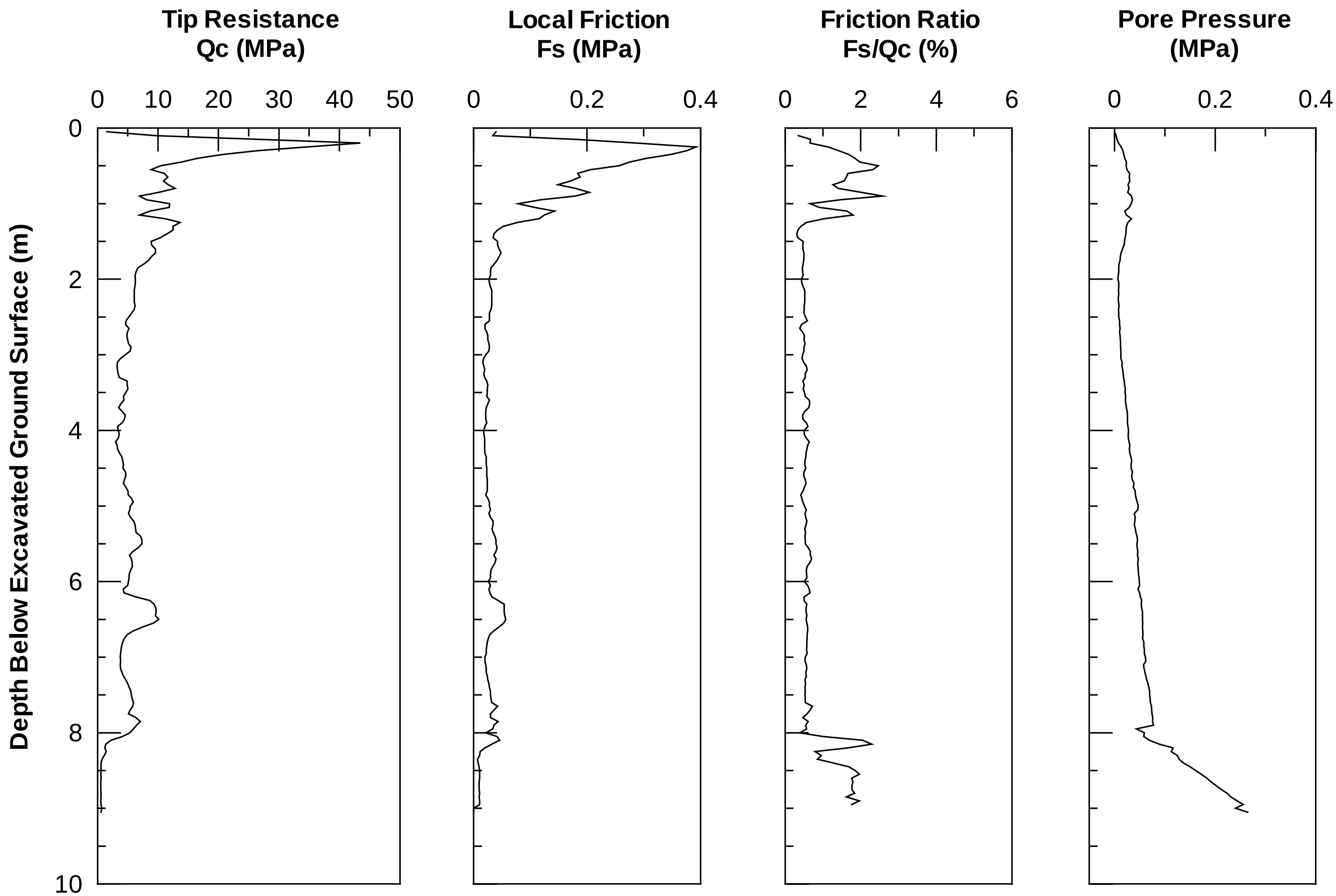


Figure 4.9 CPT2-6 Logs for 4-Pile Group/0.6m CISS Pile Test Area (10-30-98)

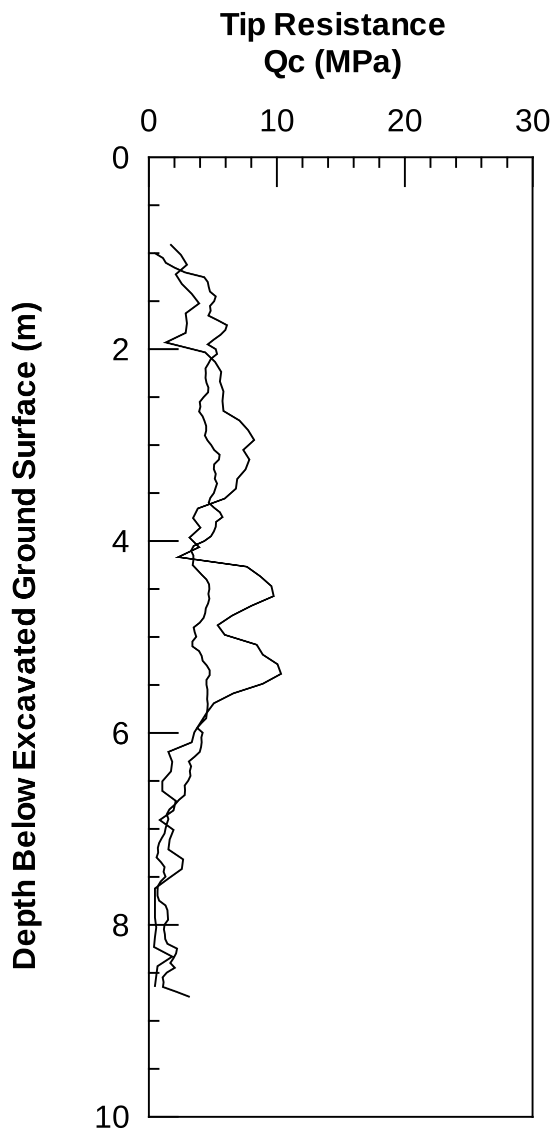


Figure 4.10 CPT RBG2 Logs for 4-Pile Group/0.6m CISS Pile Test Area After Foundation Installation (1-30-99)

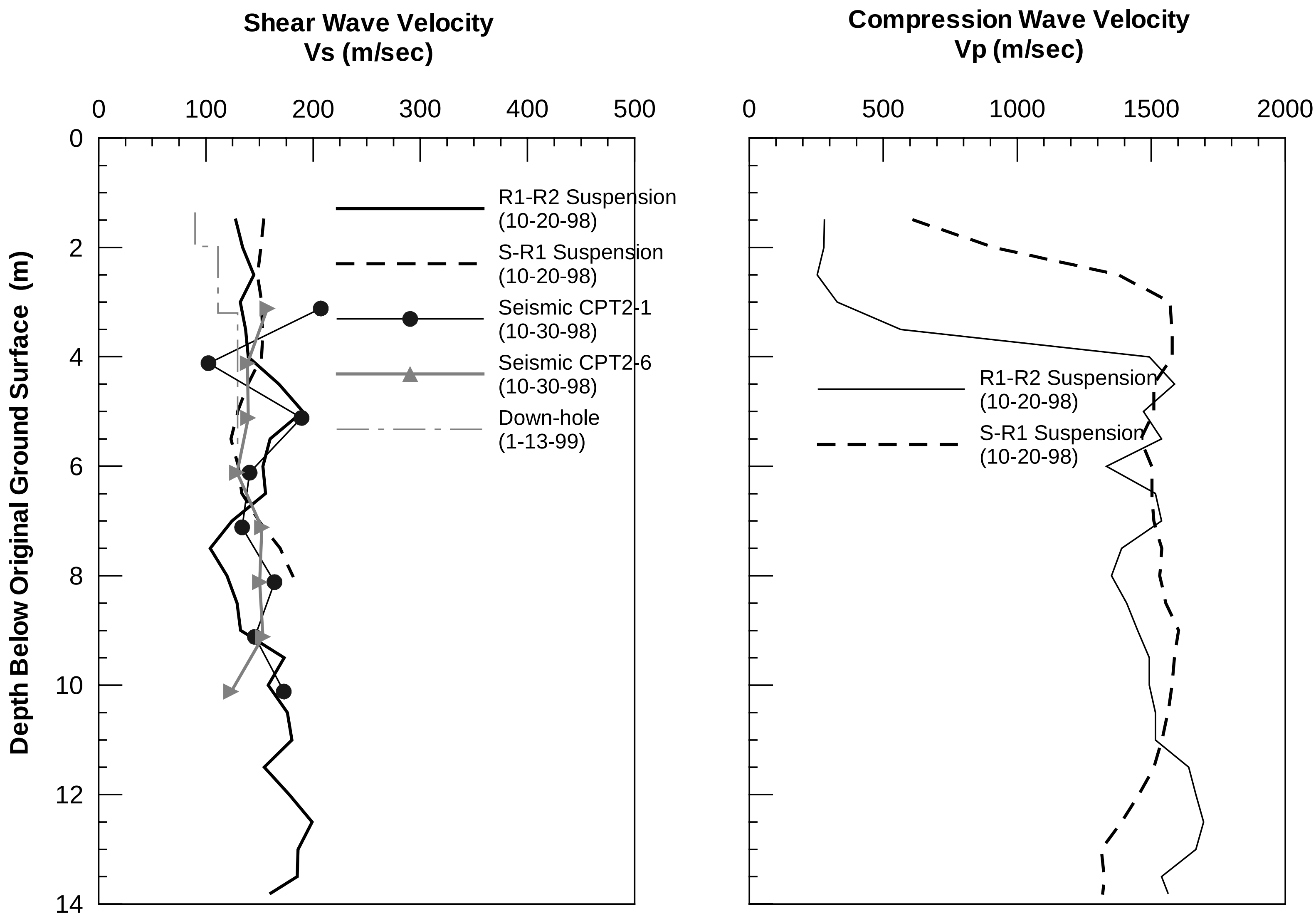


Figure 4.11 Pre-blast Velocity Profile for 4-Pile Group/0.6m CISS Pile a) Shear Wave b) Compression Wave

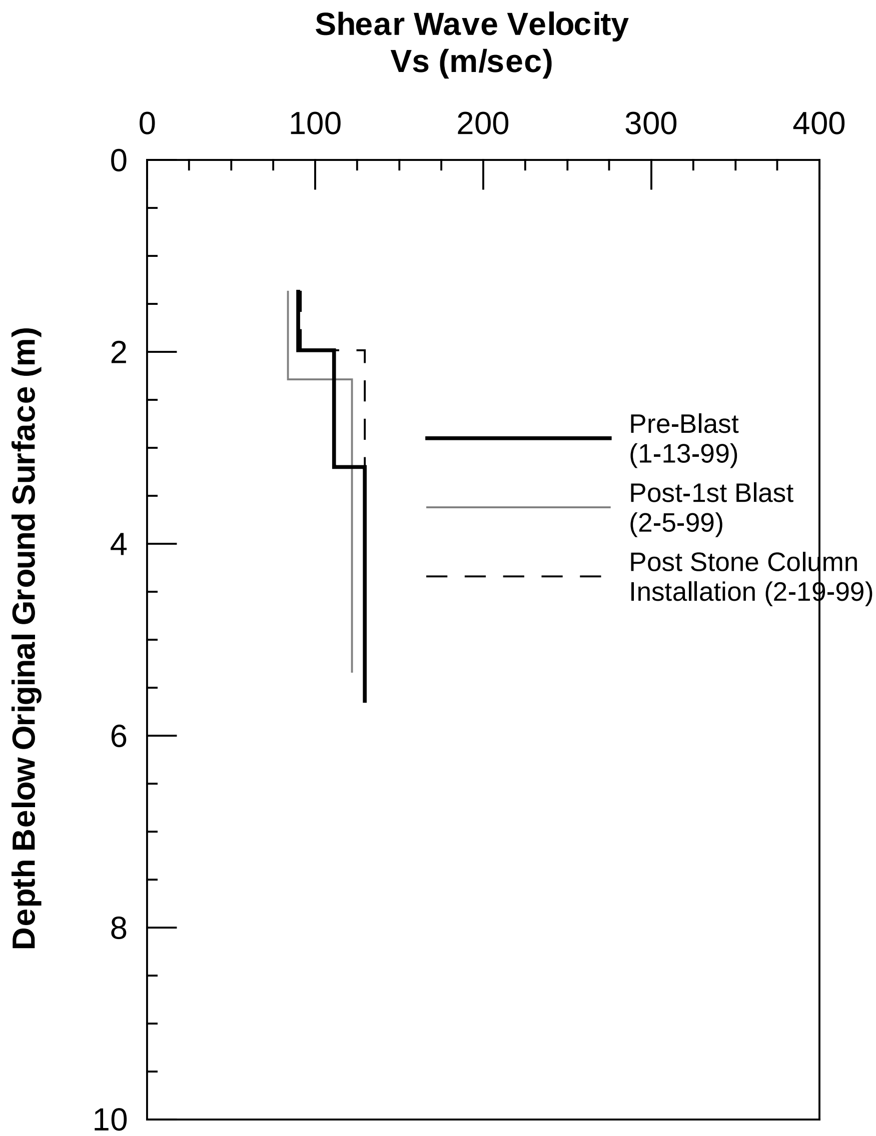


Figure 4.12 Comparison of Down-hole Shear Wave Velocity Profile before 1st Blast, after 1st Blast and after Stone Column Installation for the 4-Pile Group/0.6m CISS Pile Test Area

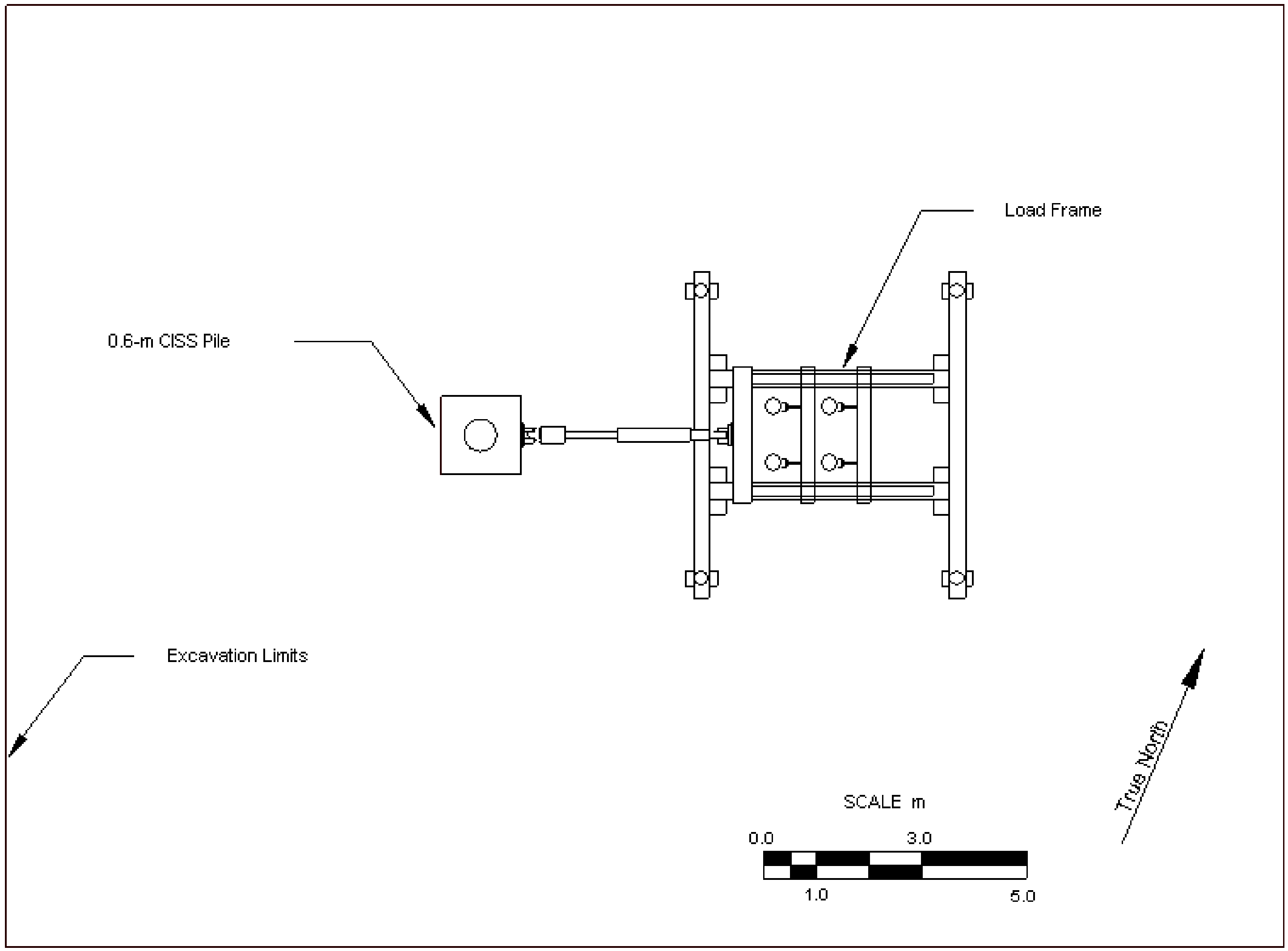


Figure 4.13 Plan View of 4-Pile Group/0.6m CISS Pile Test Set-Up

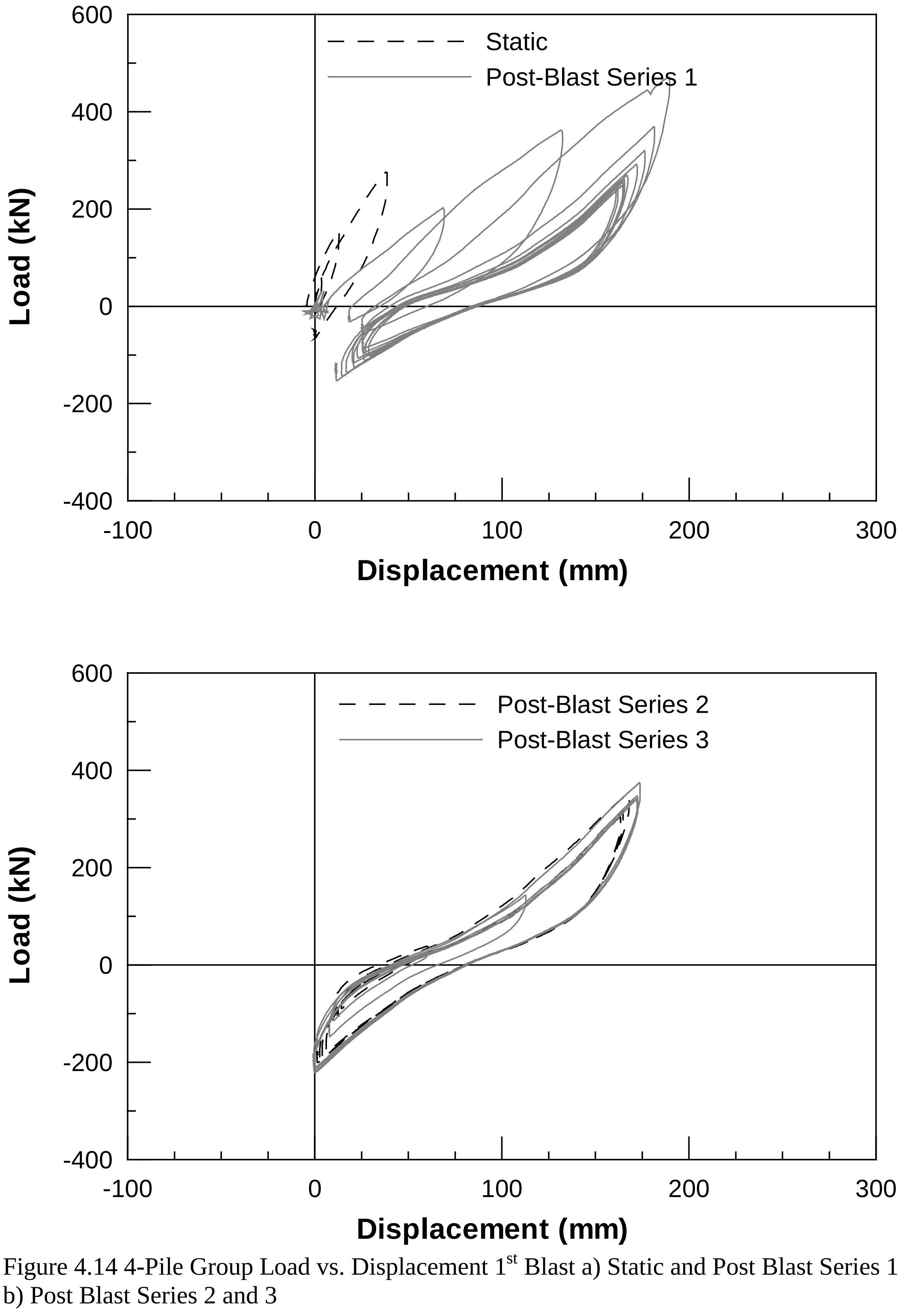


Figure 4.14 4-Pile Group Load vs. Displacement 1st Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3

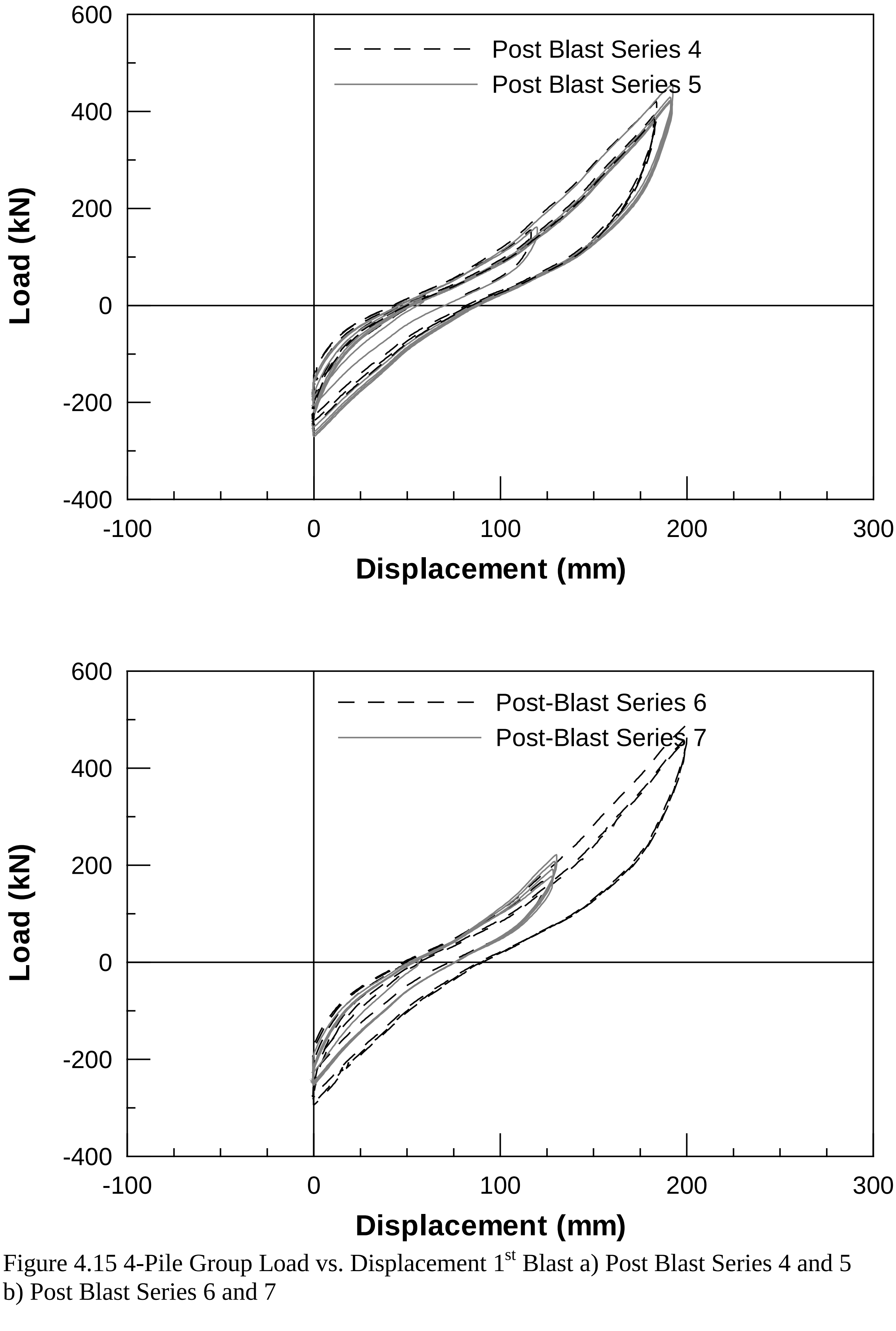


Figure 4.15 4-Pile Group Load vs. Displacement 1st Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

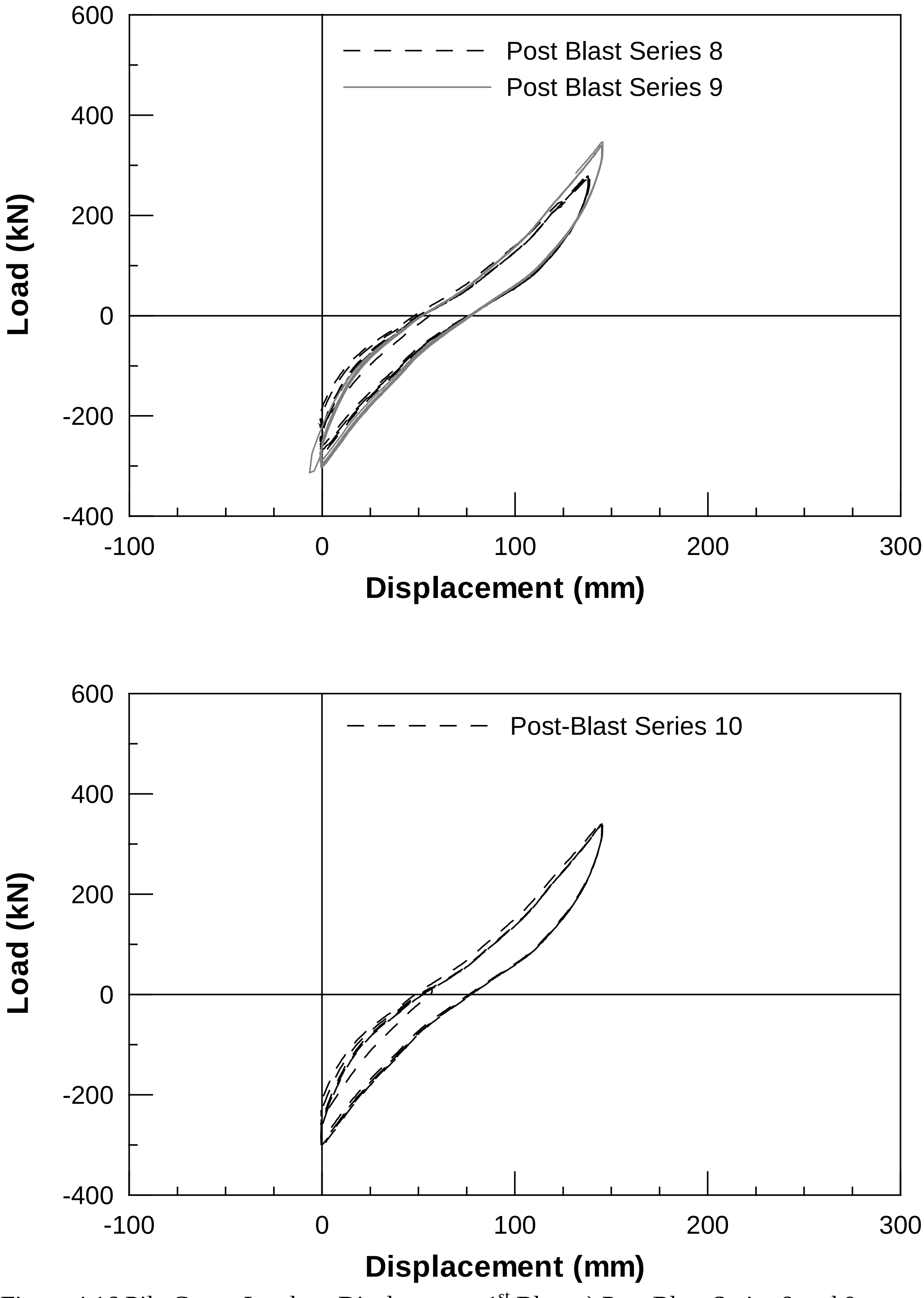


Figure 4.16 Pile Group Load vs. Displacement 1st Blast a) Post Blast Series 8 and 9 b) Post Blast Series 10

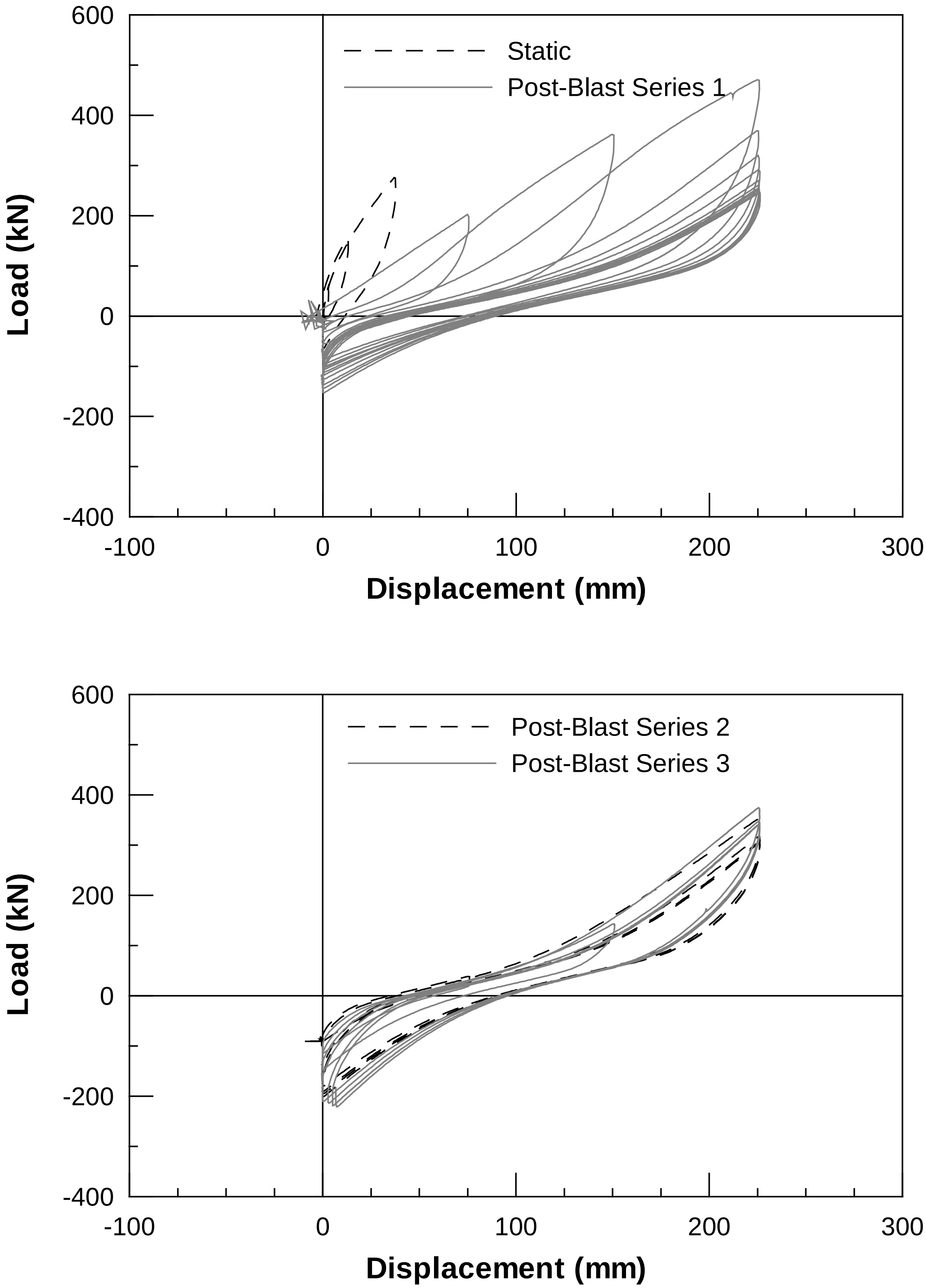
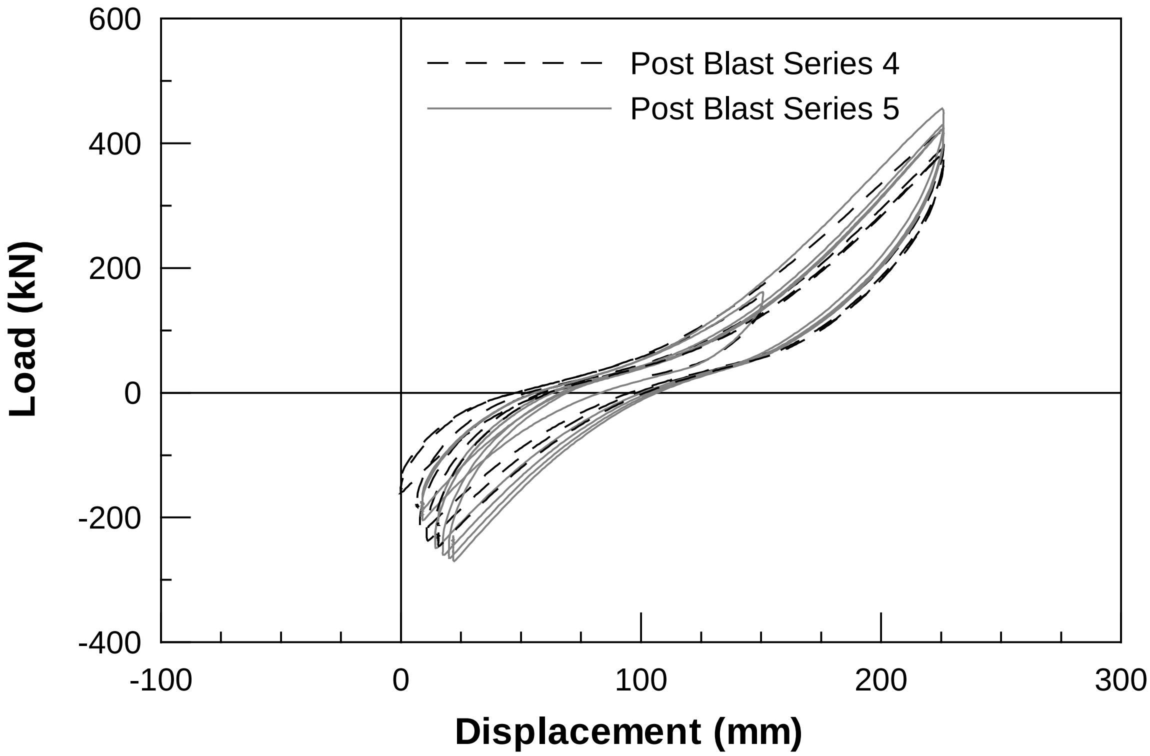


Figure 4.17 0.6m CISS Load vs. Displacement 1st Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3



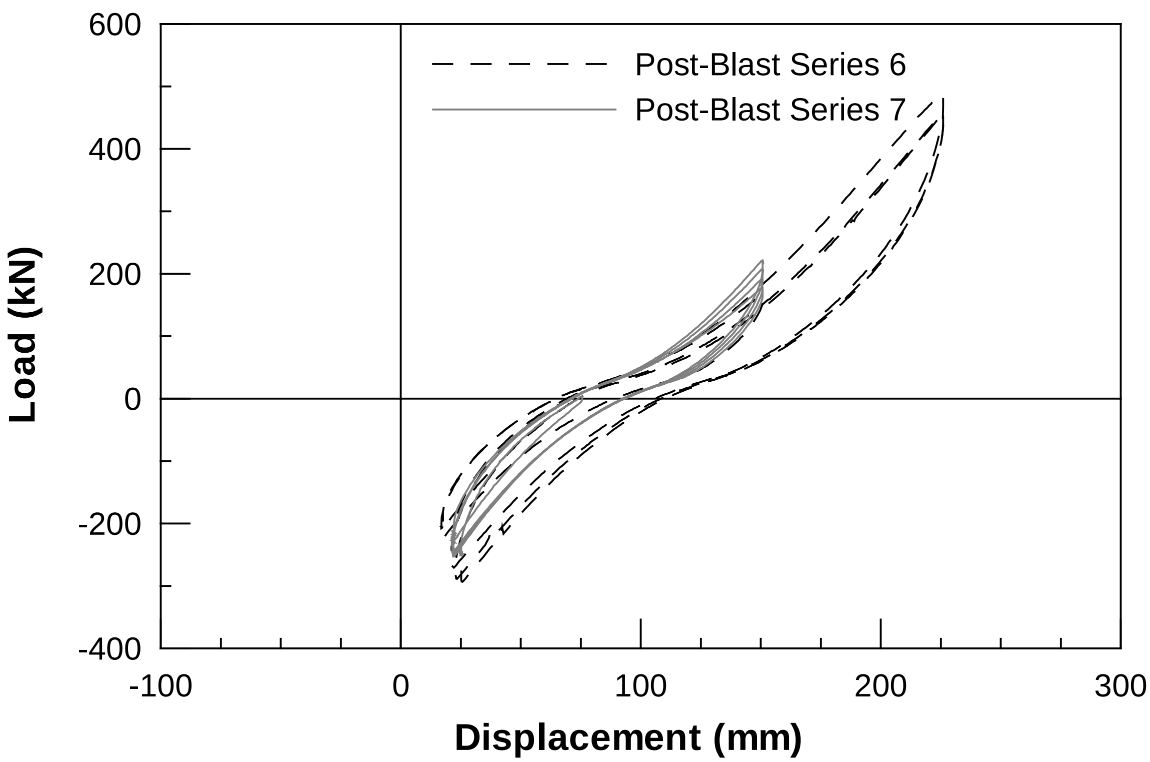


Figure 4.18 0.6m CISS Load vs. Displacement 1st Blast a) Post Blast Series 4 and 5
b) Post Blast Series 6 and 7

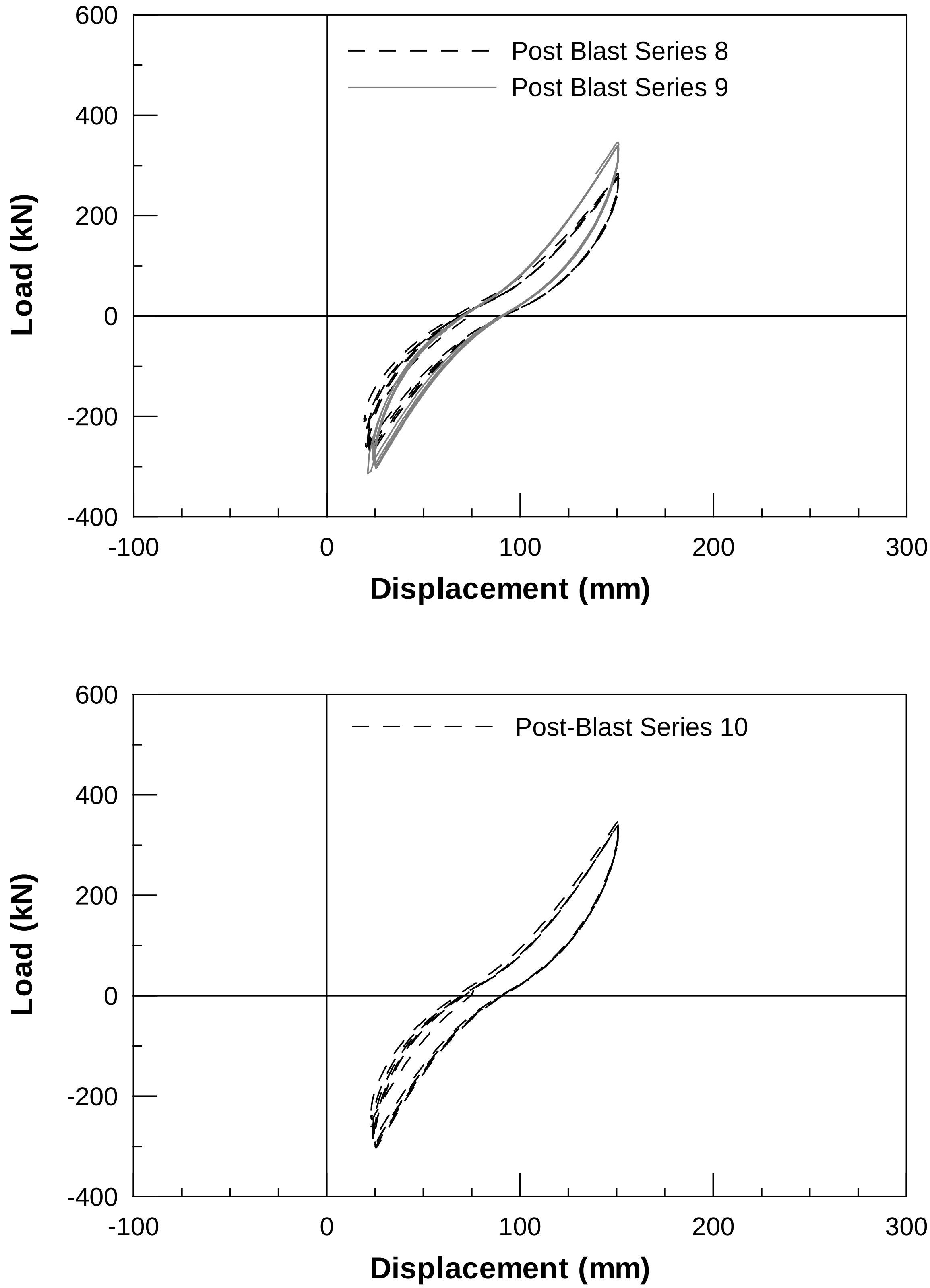


Figure 4.19 0.6m CISS Load vs. Displacement 1st Blast a) Post Blast Series 8 and 9
b) Post Blast Series 10

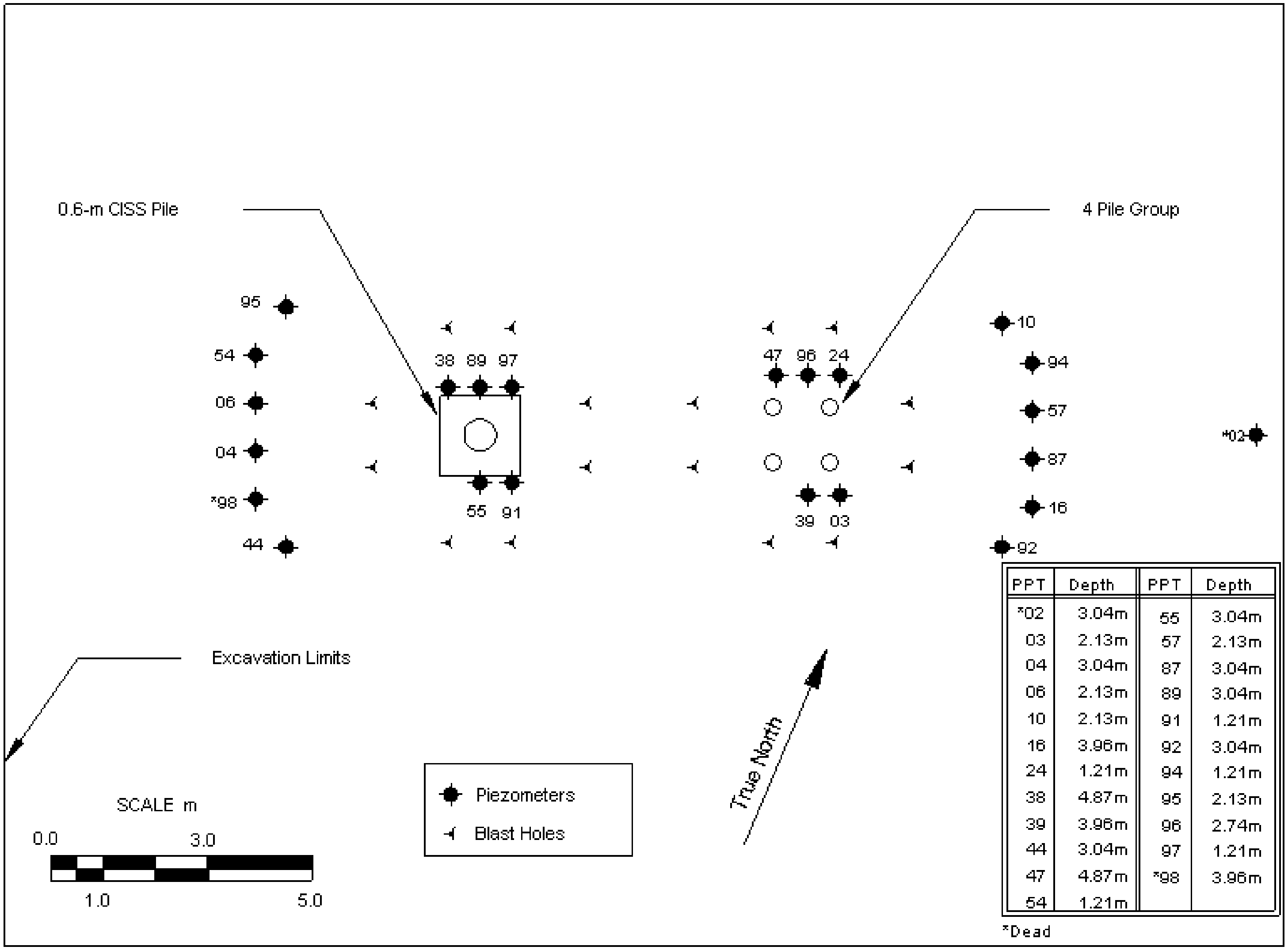


Figure 4.20 Location Map of Pore Pressure Transducers for 4-Pile Group/0.6m CISS Pile Site

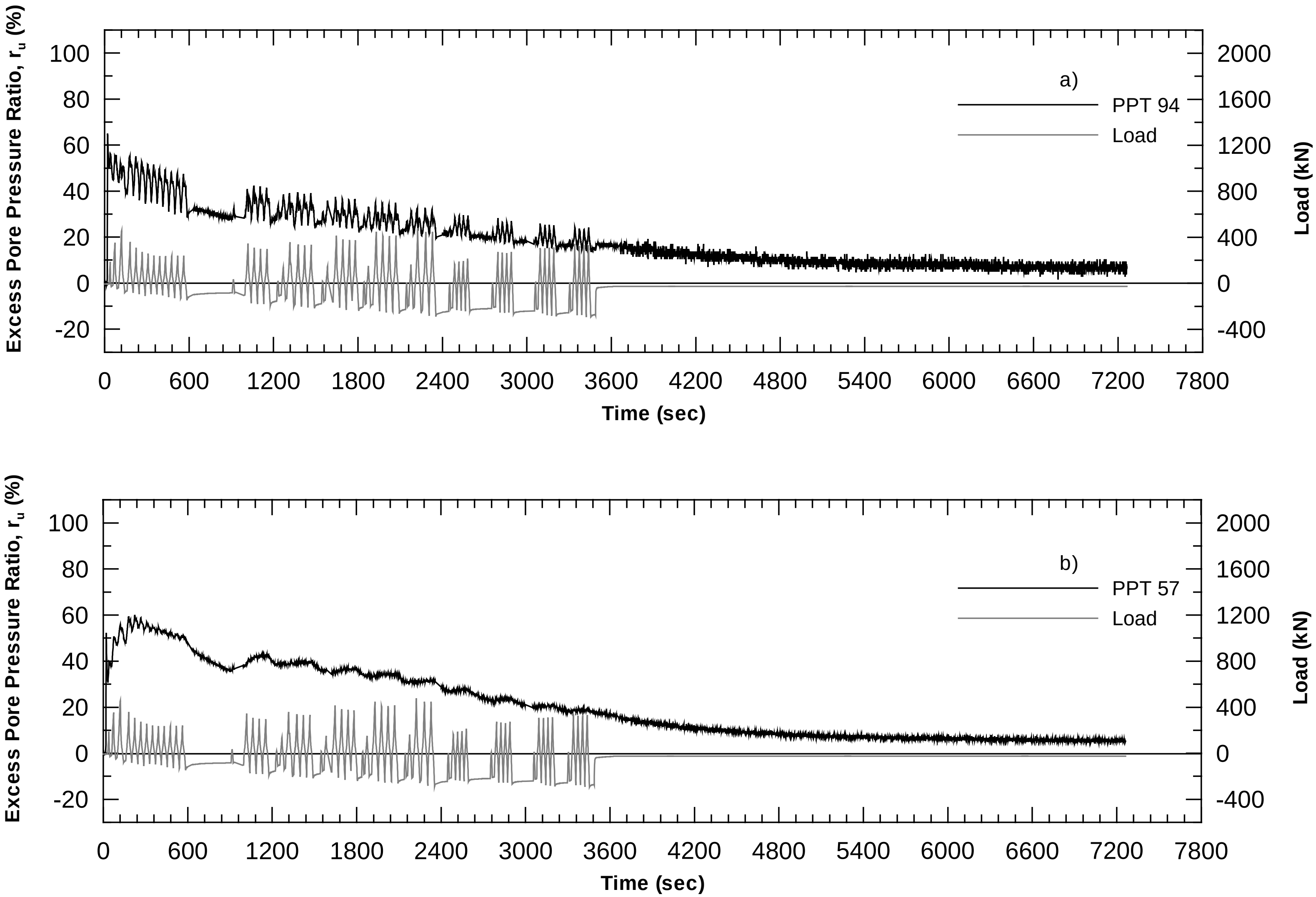


Figure 4.21 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT94   b)PPT57

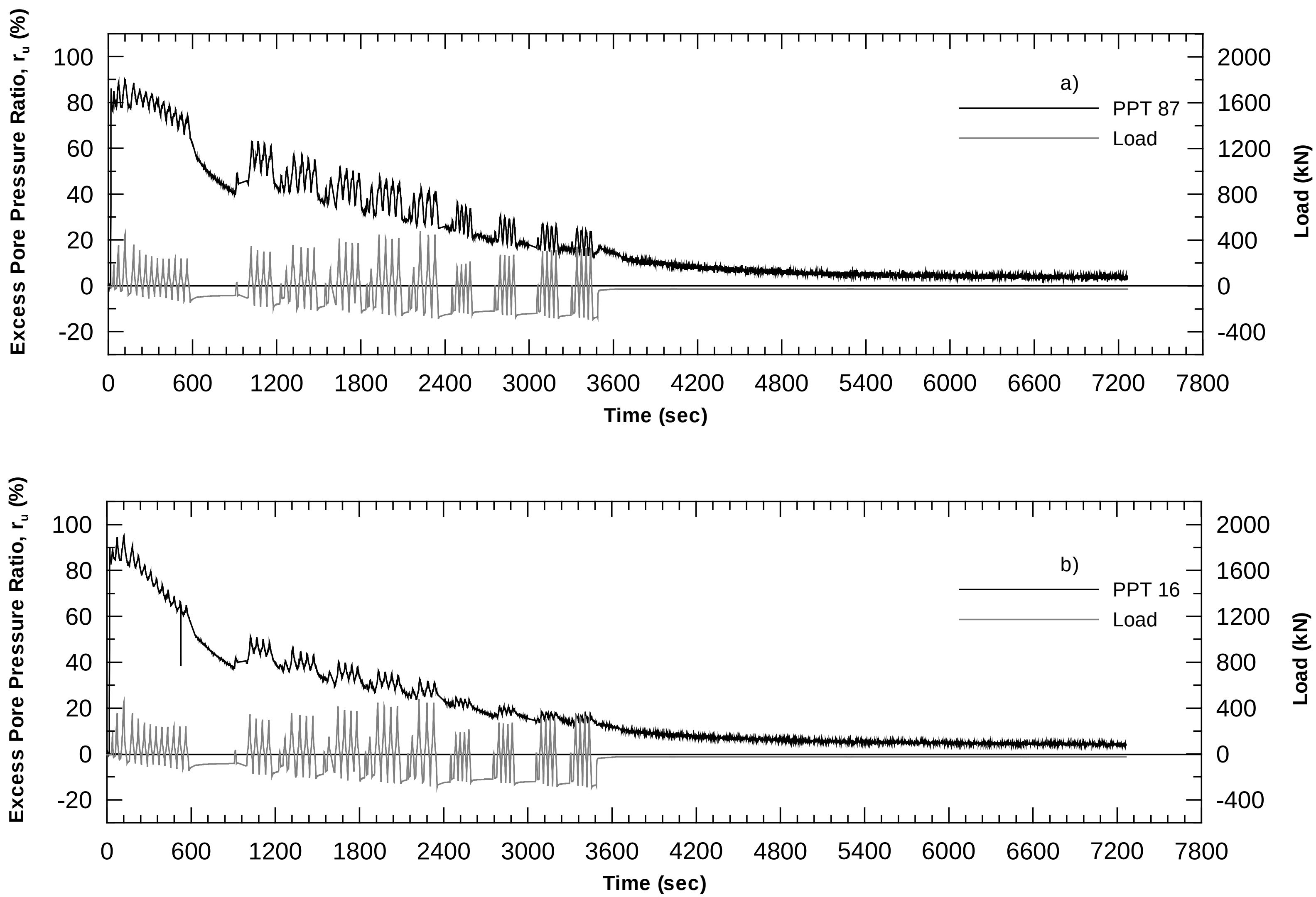


Figure 4.22 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT87 b)PPT16

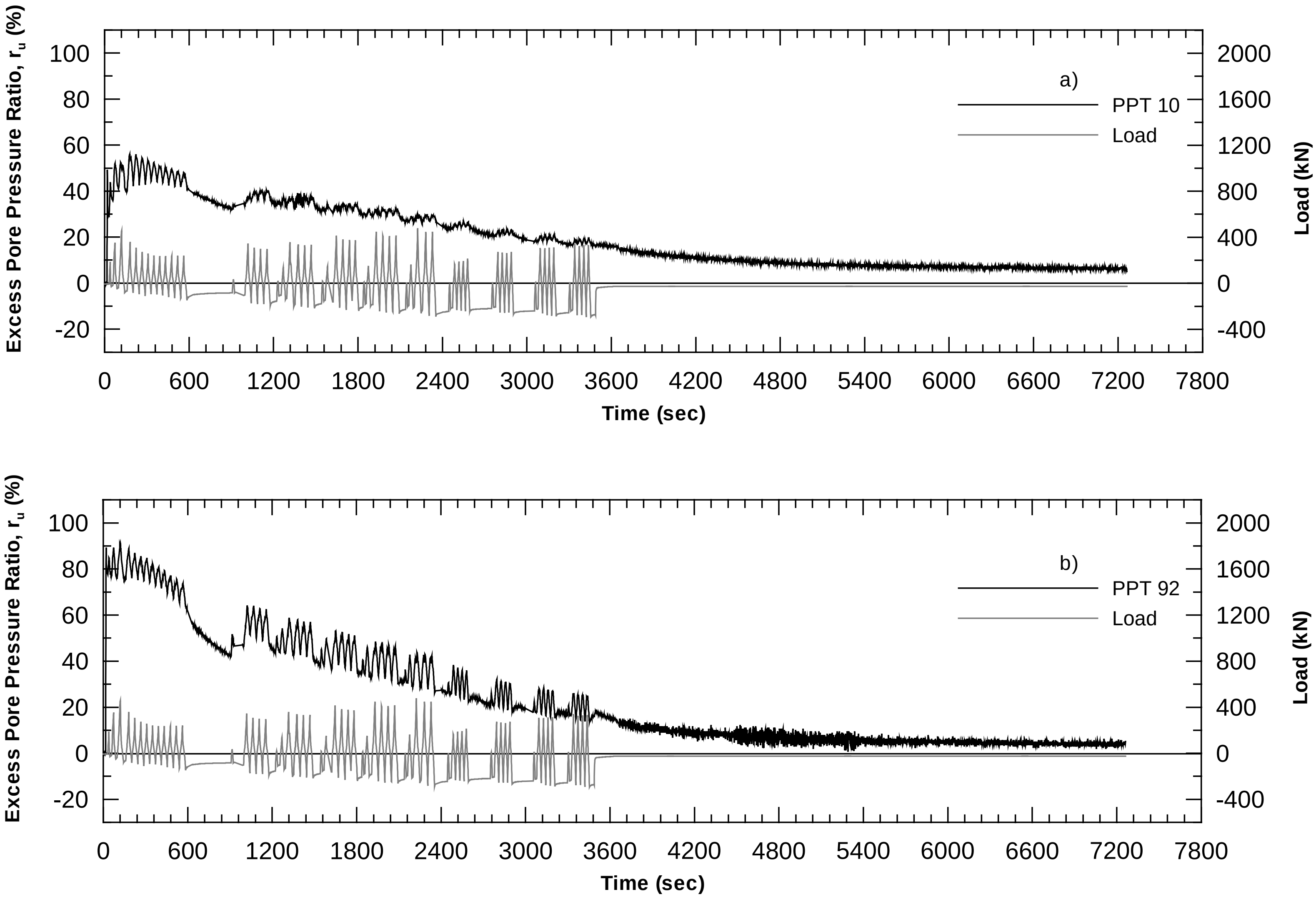


Figure 4.23 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT10 b)PPT92

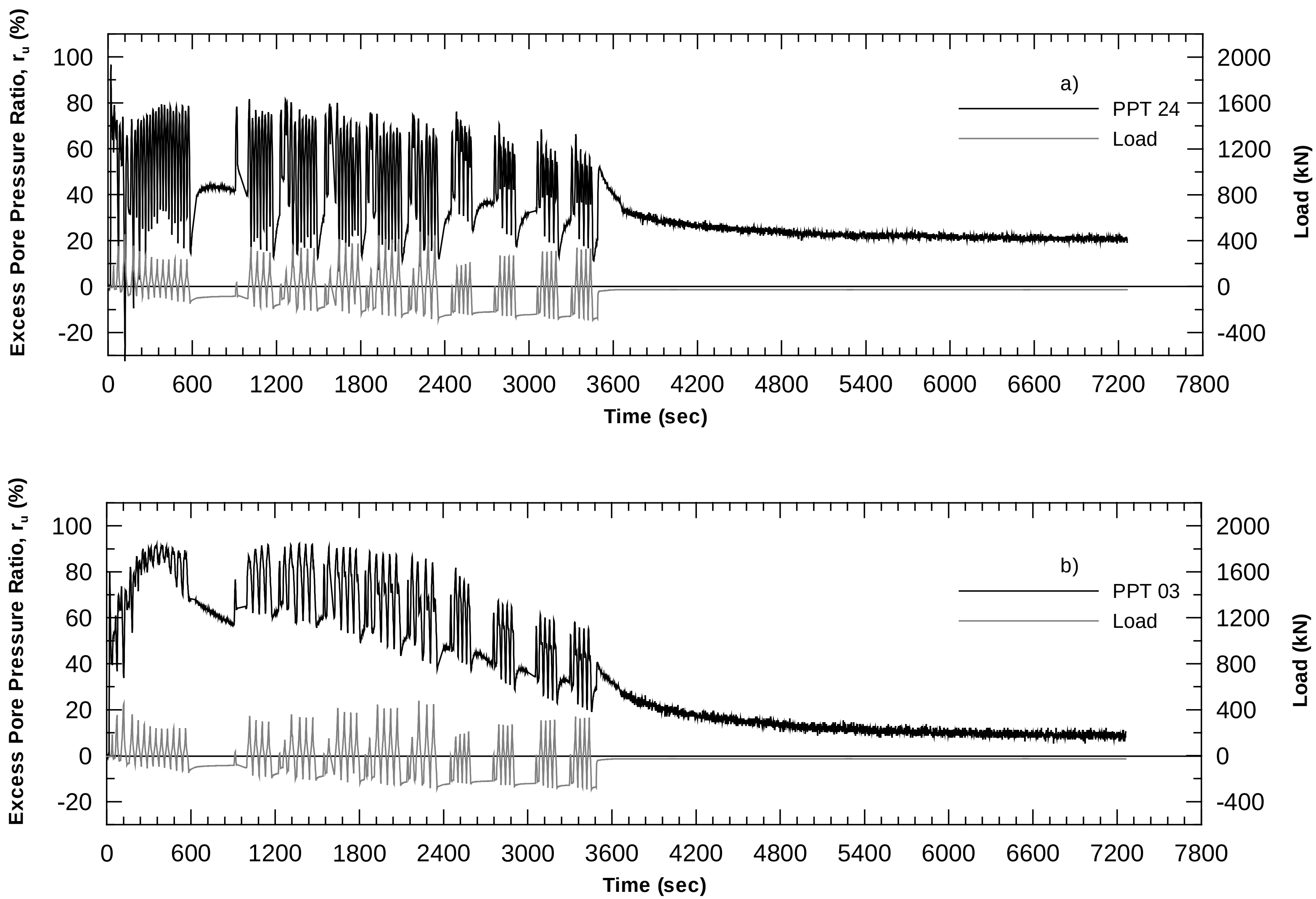


Figure 4.24 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT24 b)PPT3

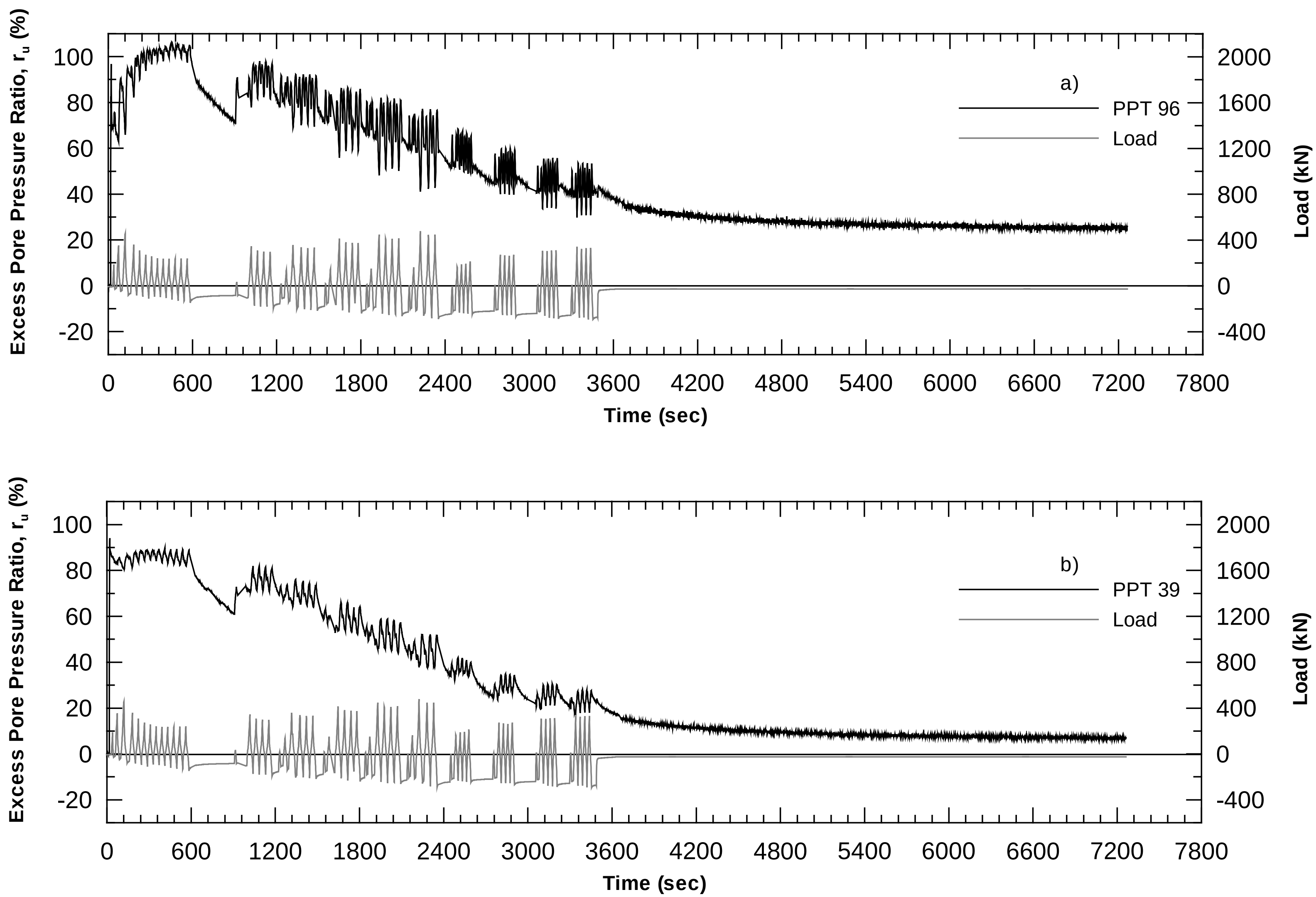


Figure 4.25 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT96 b)PPT39

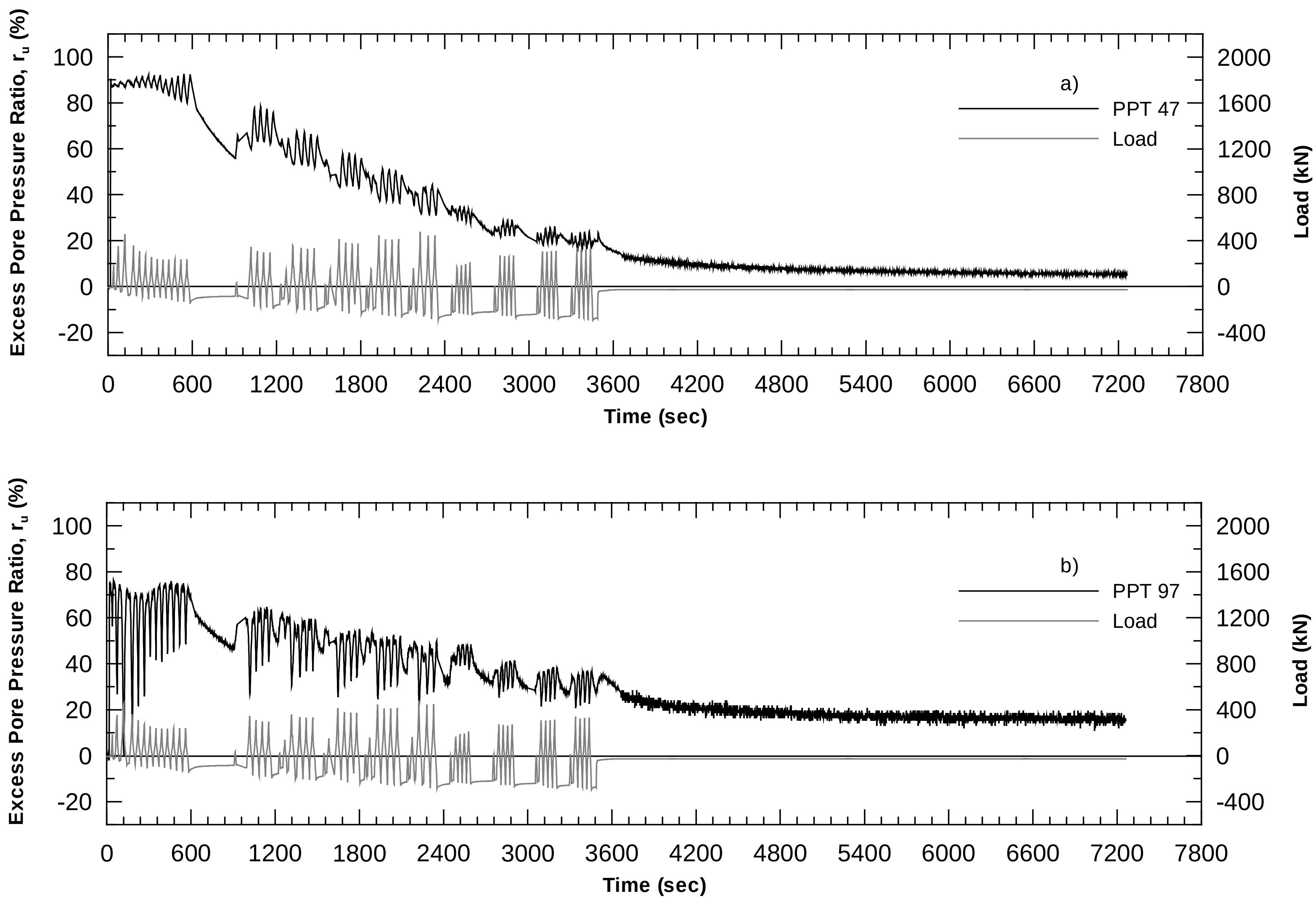


Figure 4.26 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT47 b)PPT97

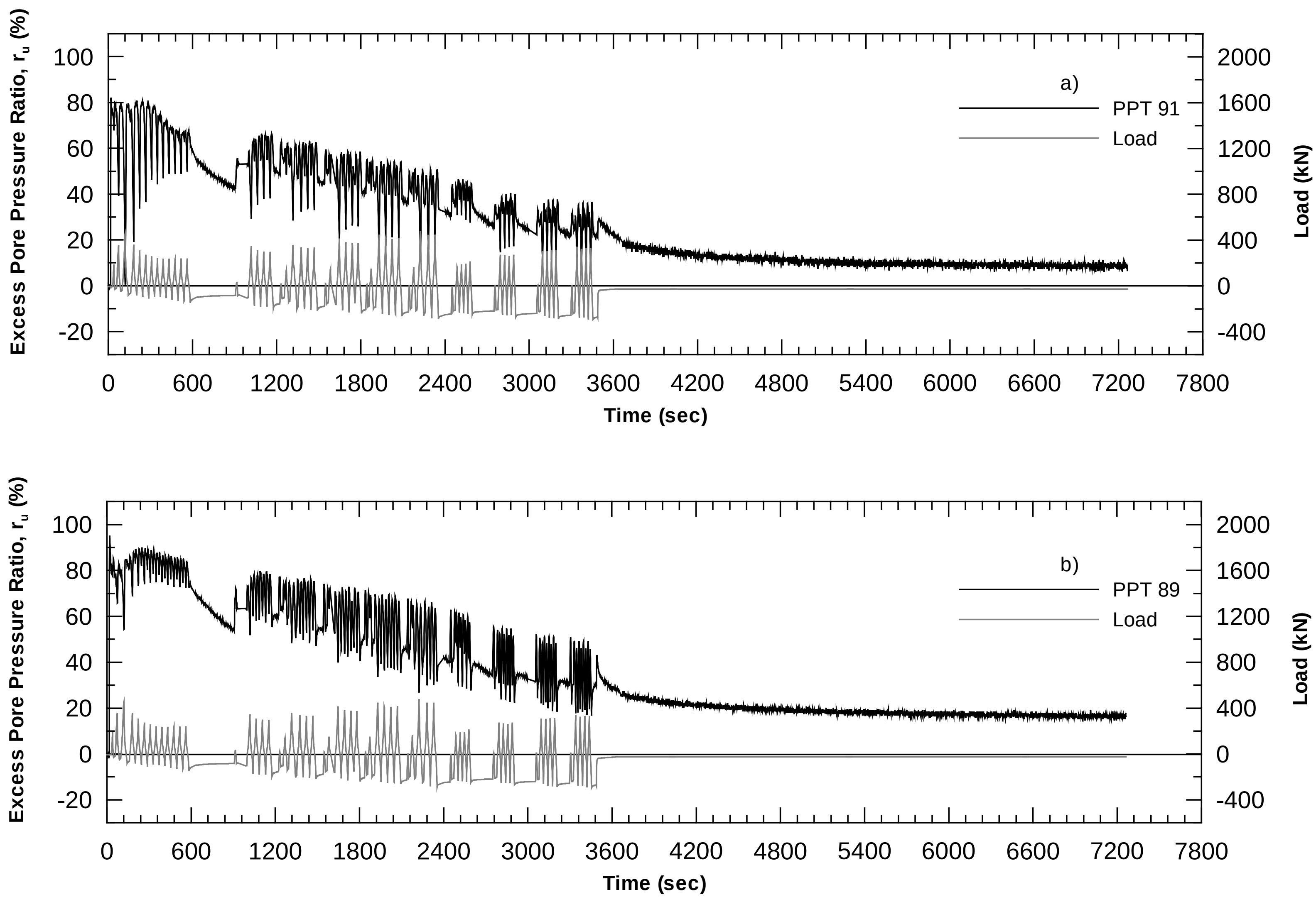


Figure 4.27 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT91 b)PPT89

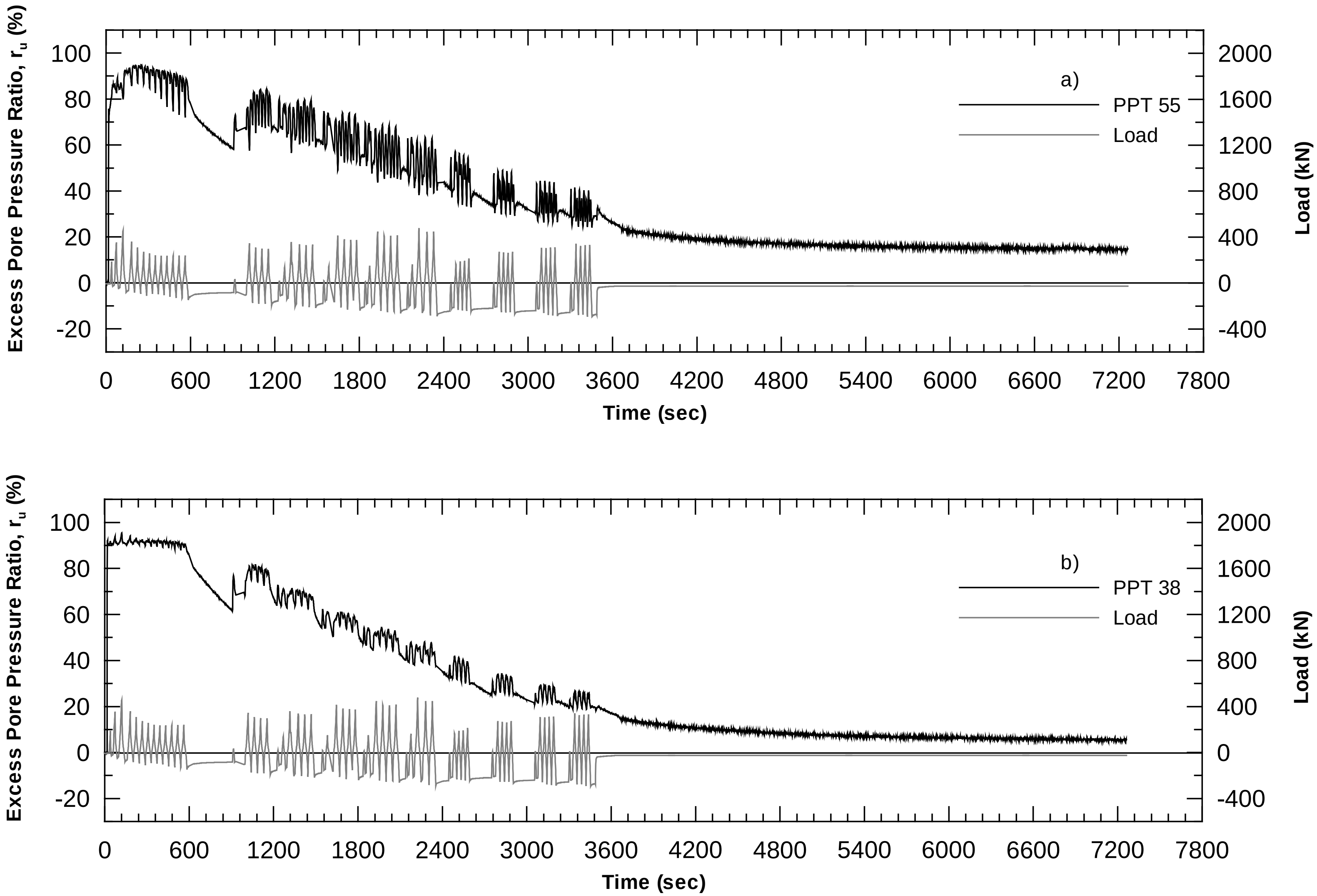


Figure 4.28 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT55 b)PPT38

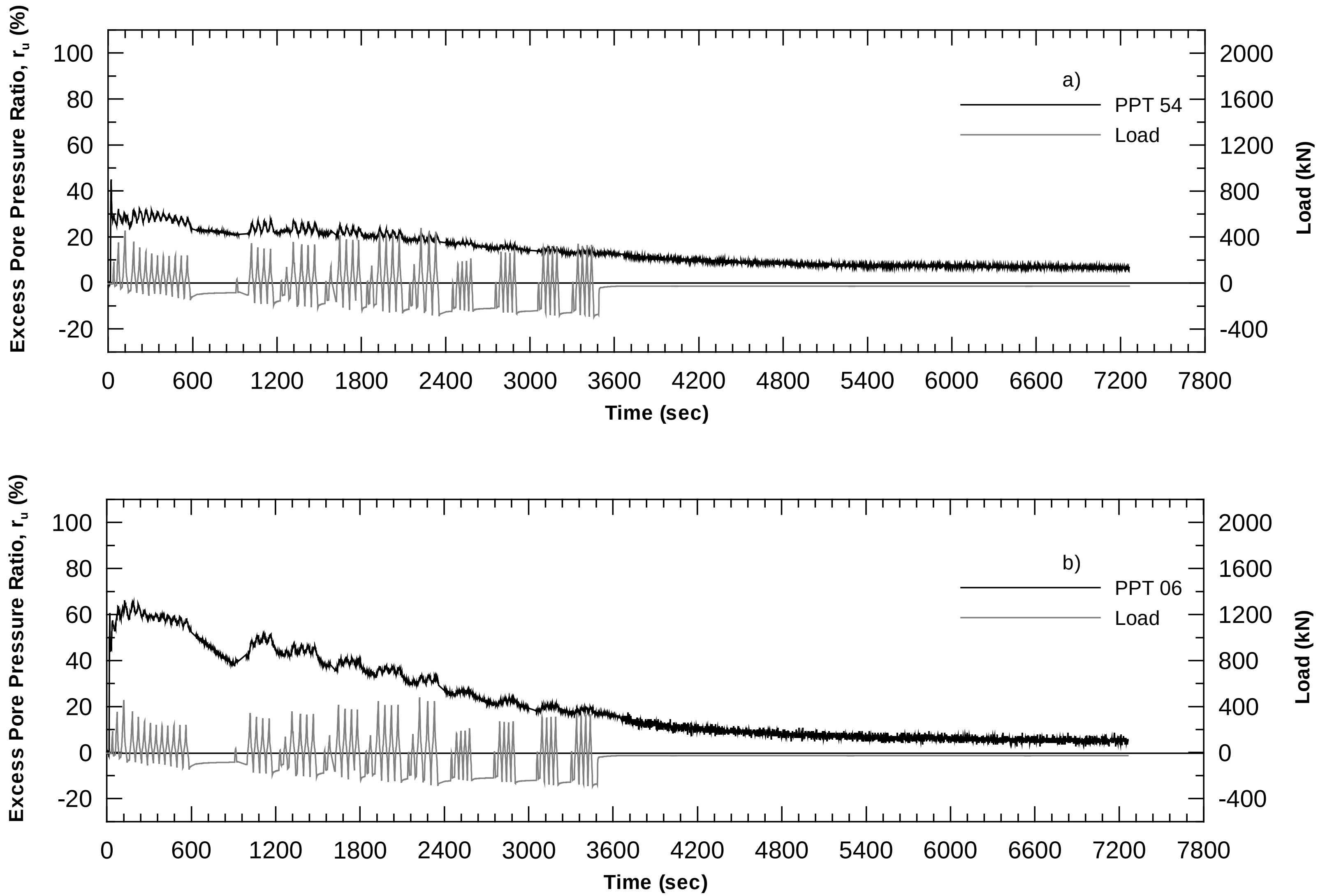


Figure 4.29 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT54 b)PPT06

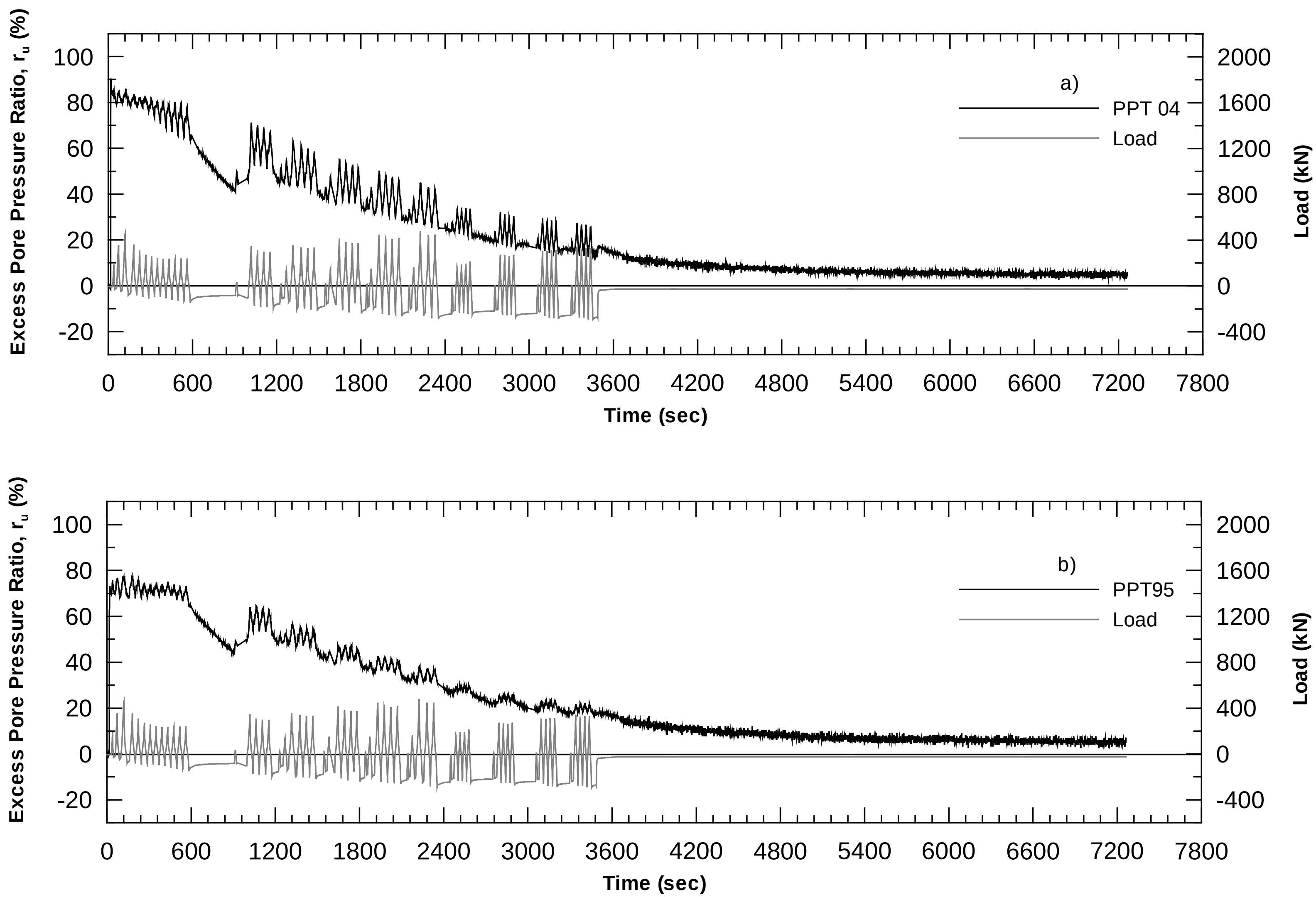


Figure 4.30 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast a)PPT04   b)PPT95

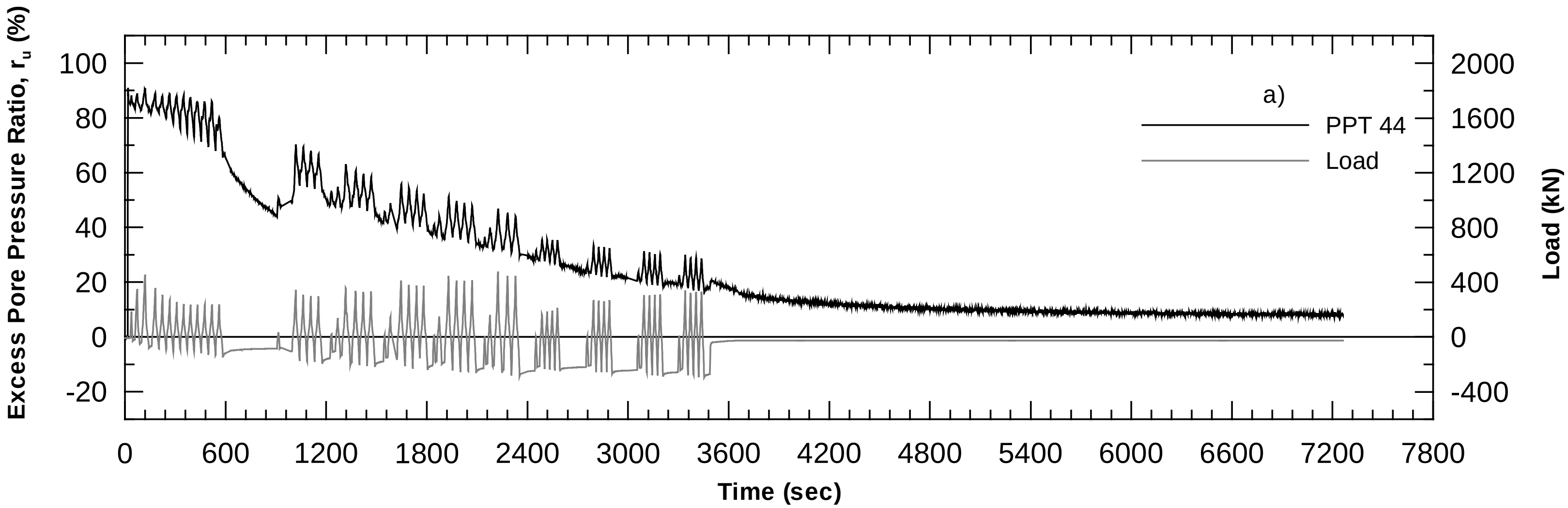


Figure 4.31 Excess Pore Pressure Ratio for 4-Pile Group/0.6m CISS Pile 1st Blast PPT44

Figure 4.32 Installation of Stone Columns

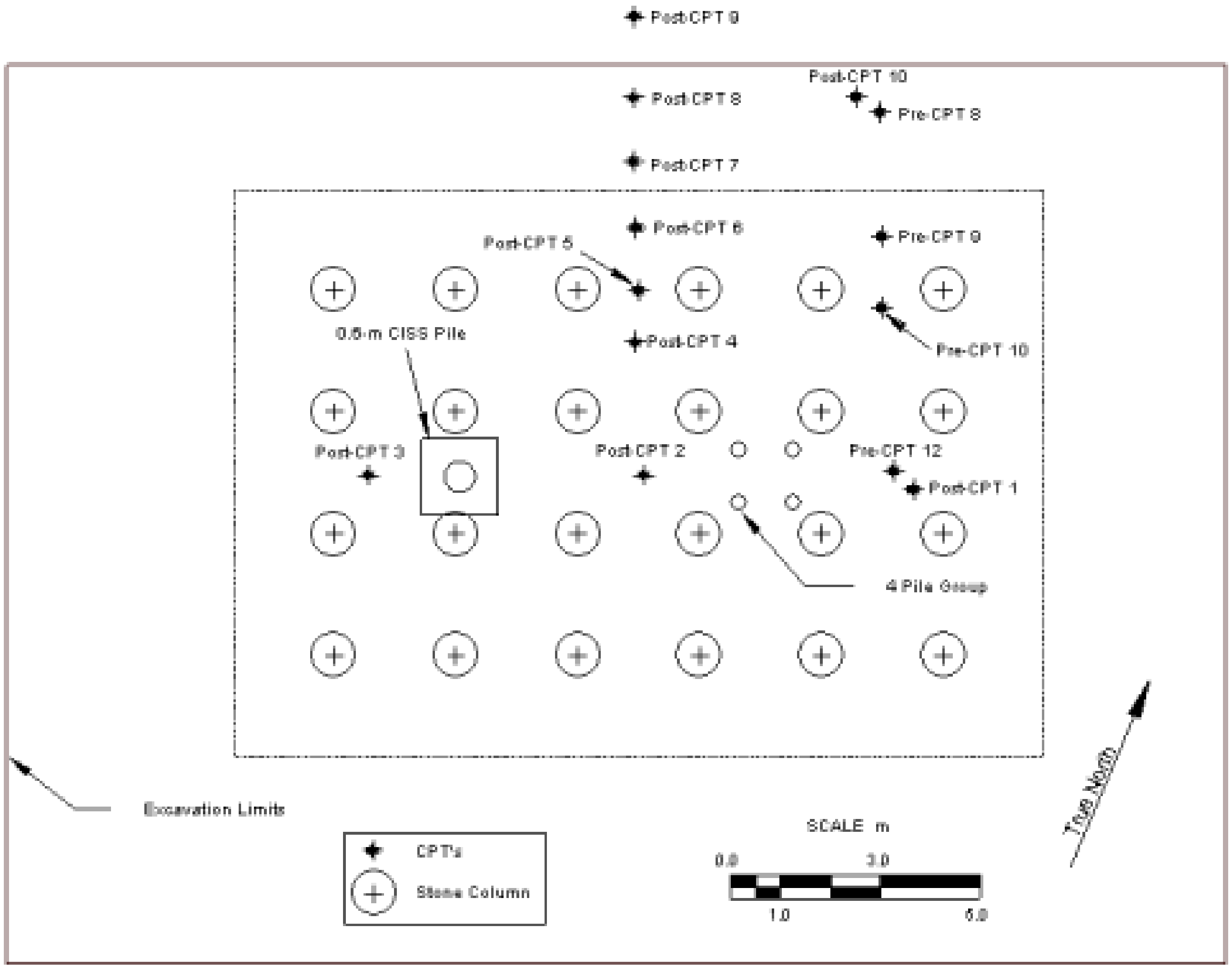


Figure 4.33 Location Map of Secondary In-Situ Tests and Stone Columns for 2nd Blast

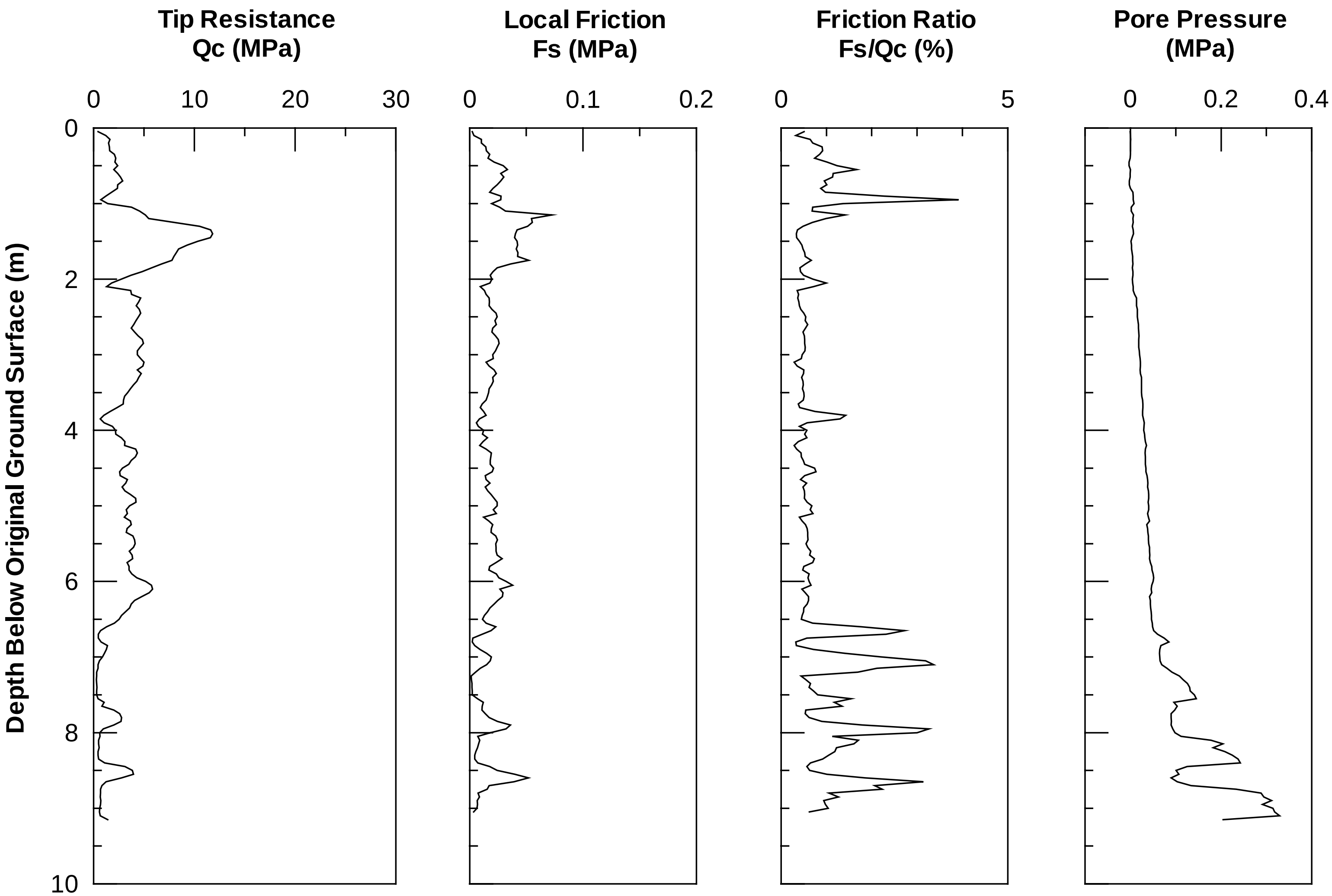


Figure 4.34 Pre-treatment CPT08 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-08-99)

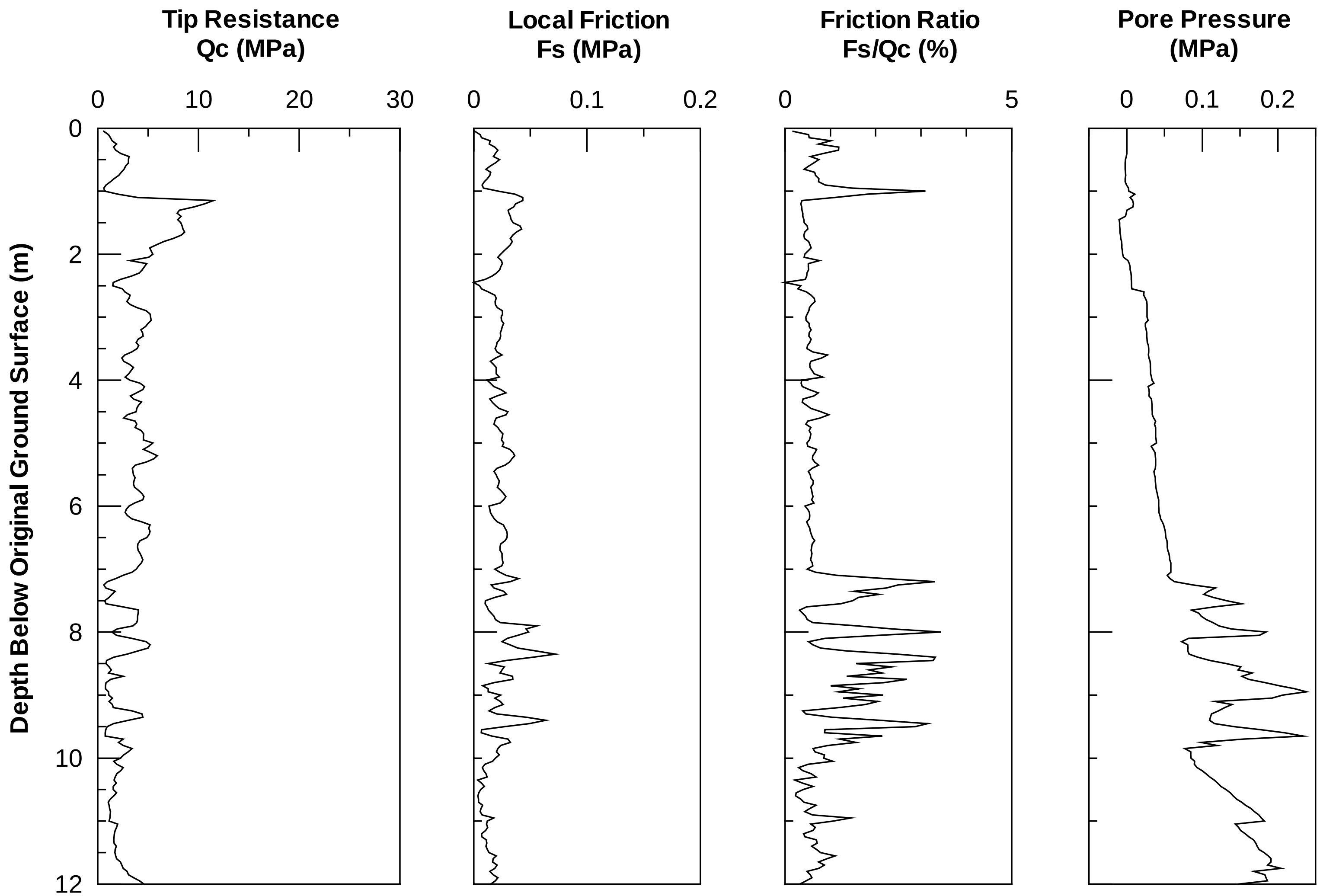


Figure 4.35 Pre-treatment CPT09 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-08-99)

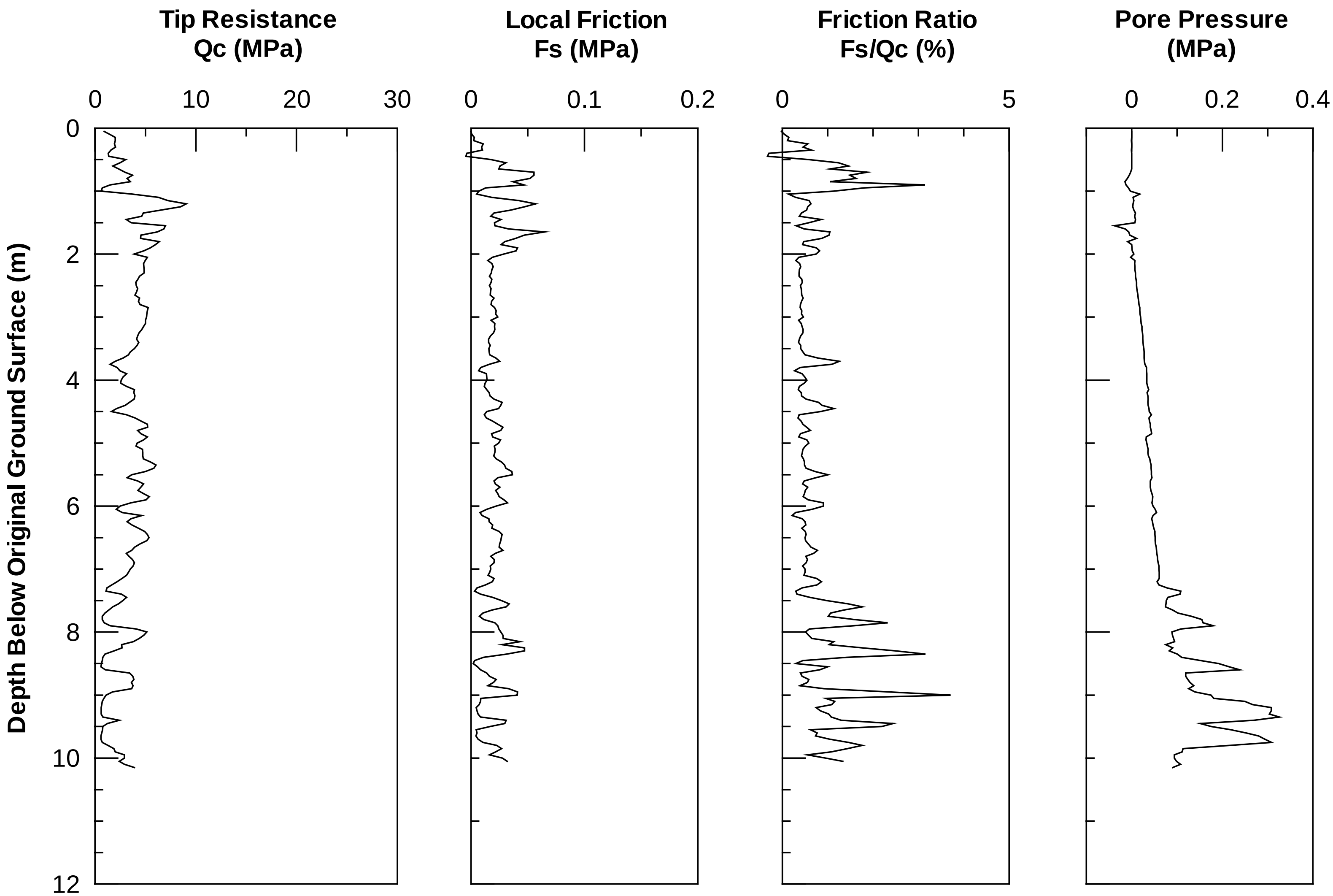


Figure 4.36 Pre-treatment CPT10 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-08-99)

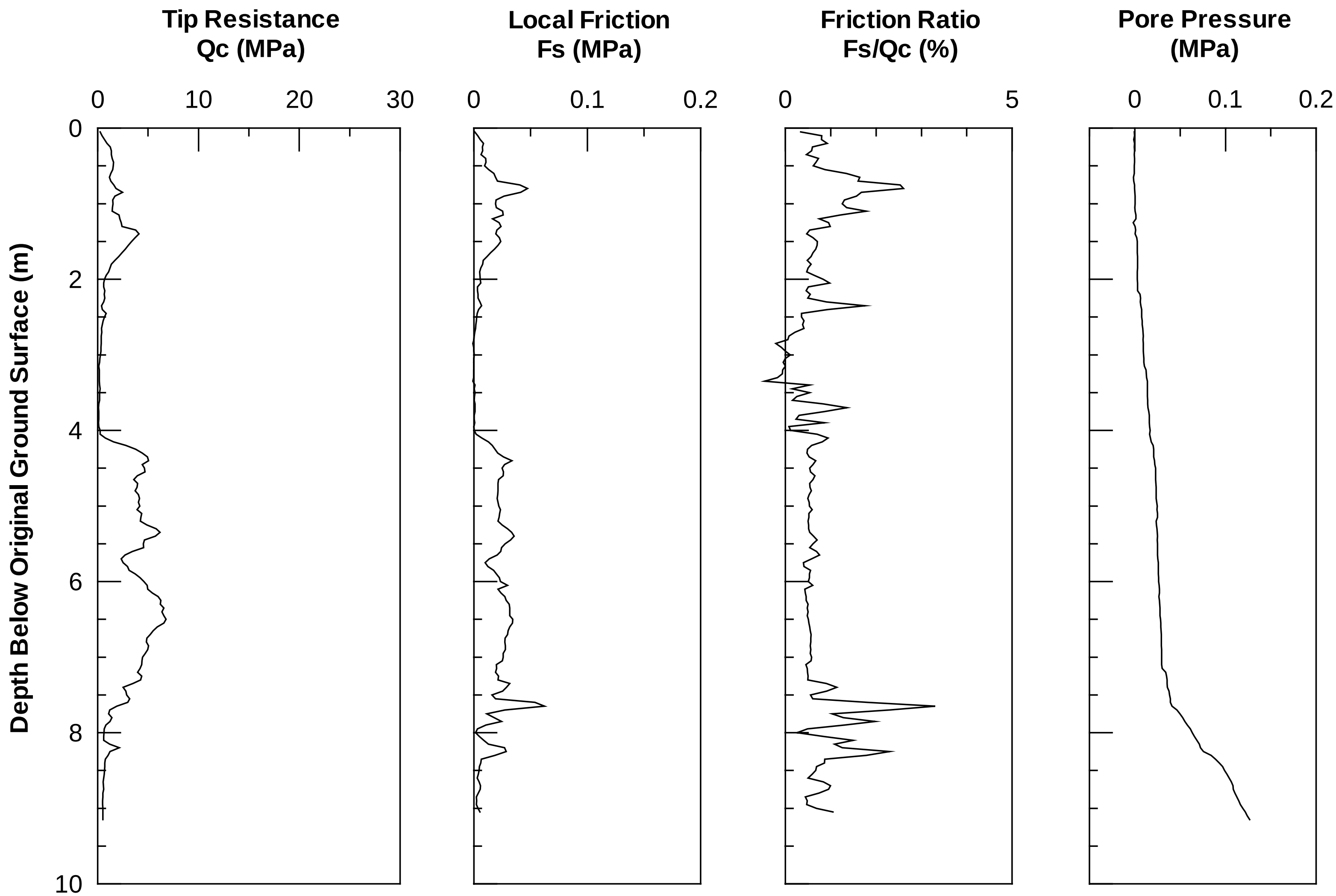


Figure 4.37 Pre-treatment CPT12 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-08-99)

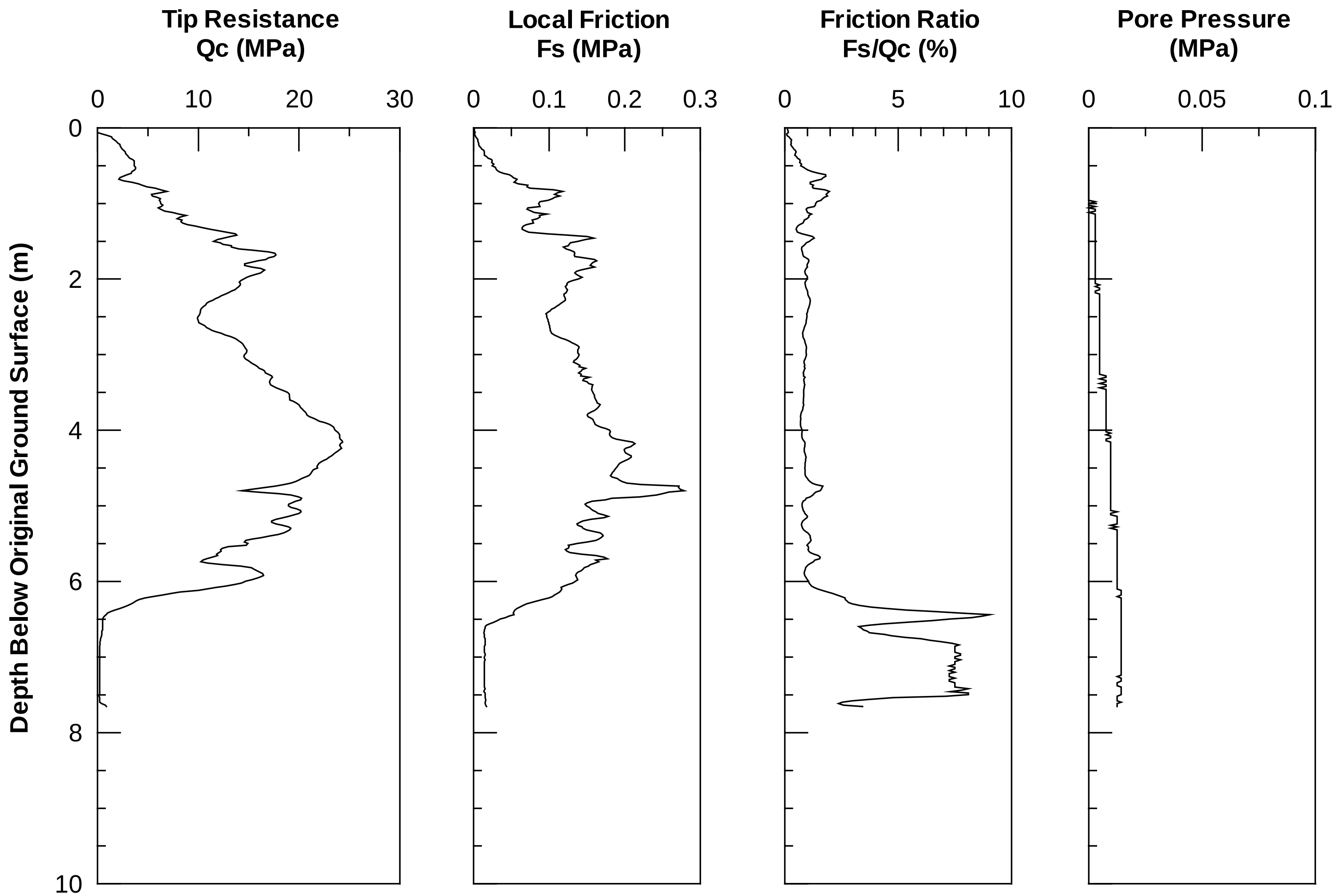


Figure 4.38 Post-treatment CPT1 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

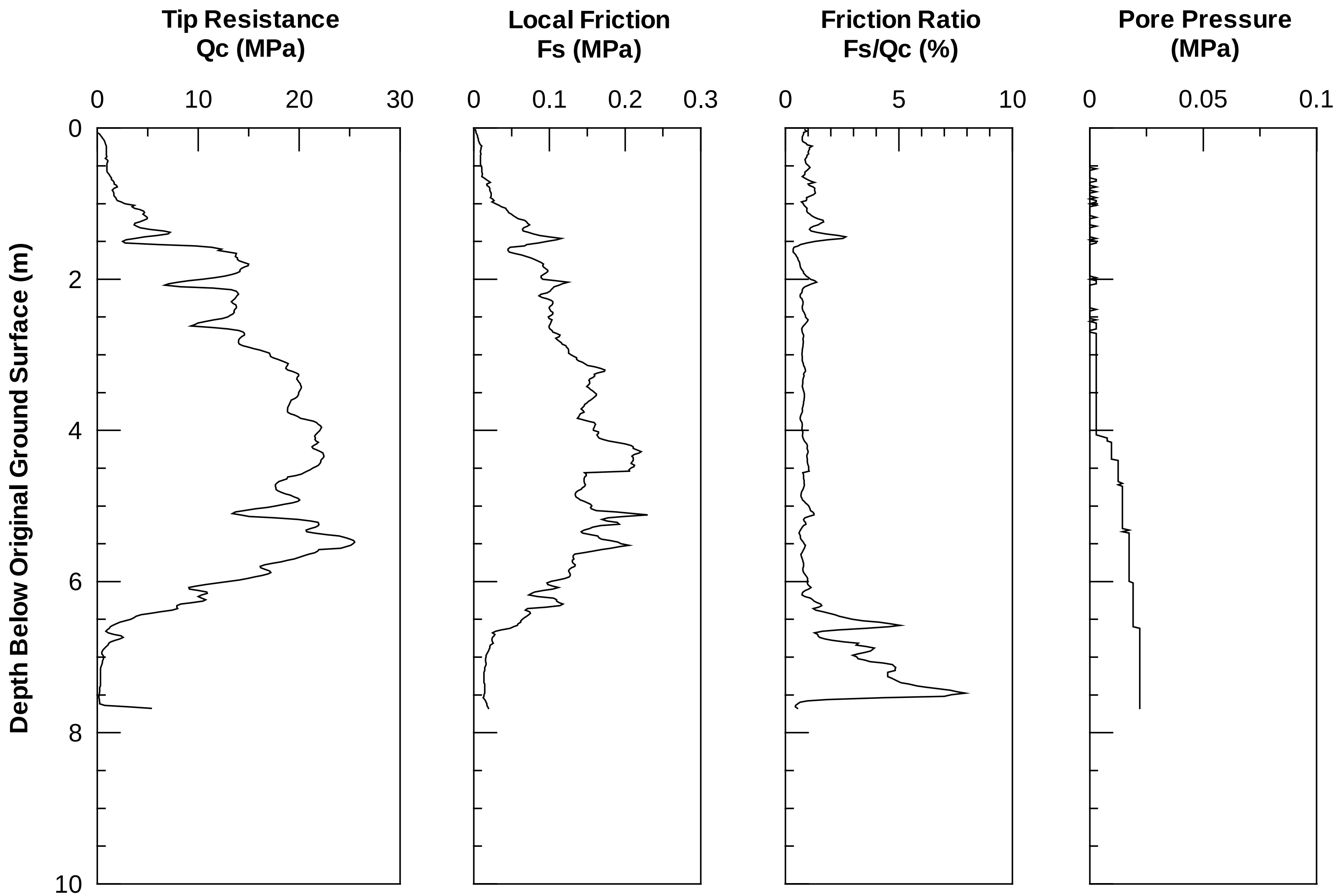


Figure 4.39 Post-treatment CPT2 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

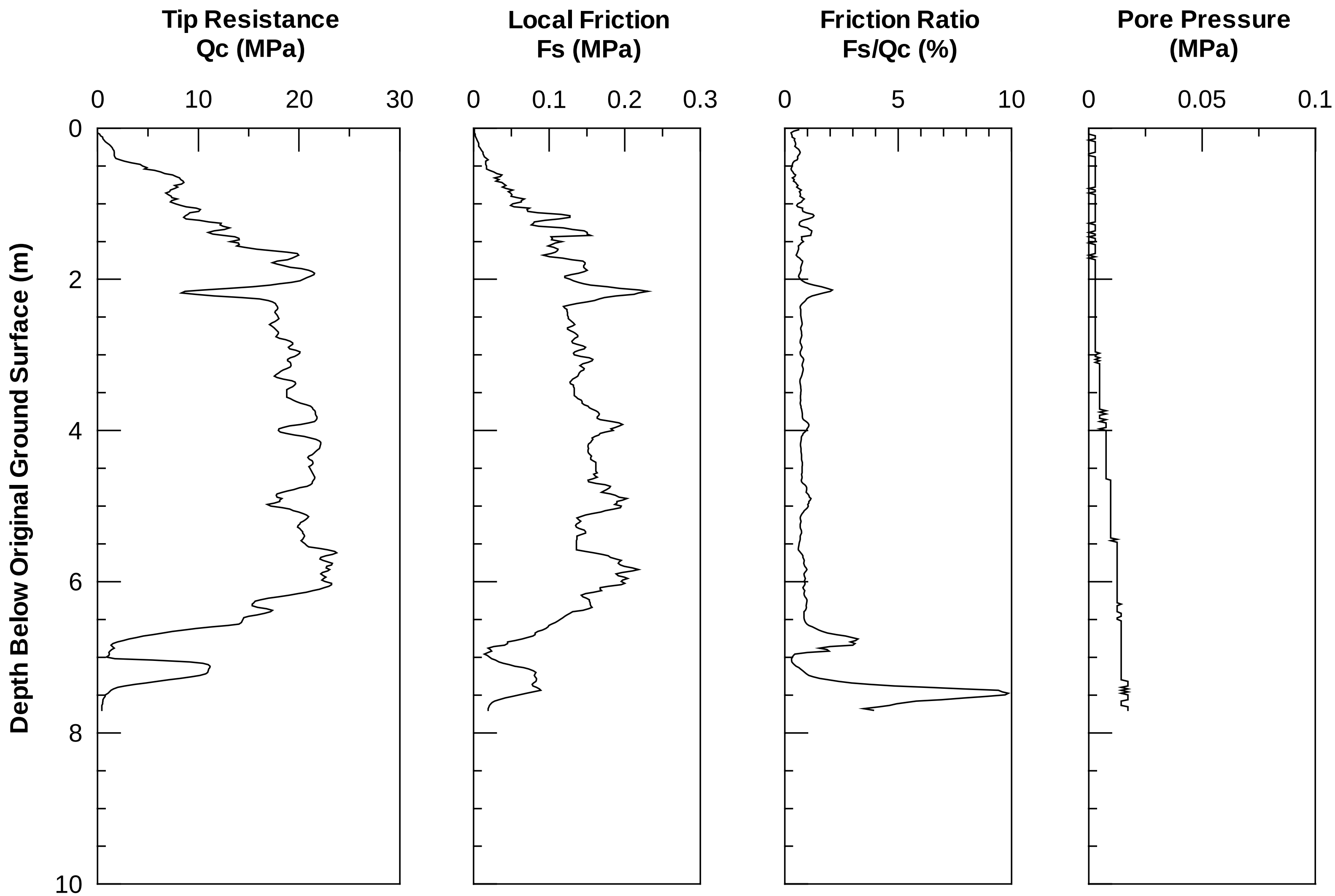


Figure 4.40 Post-treatment CPT3 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

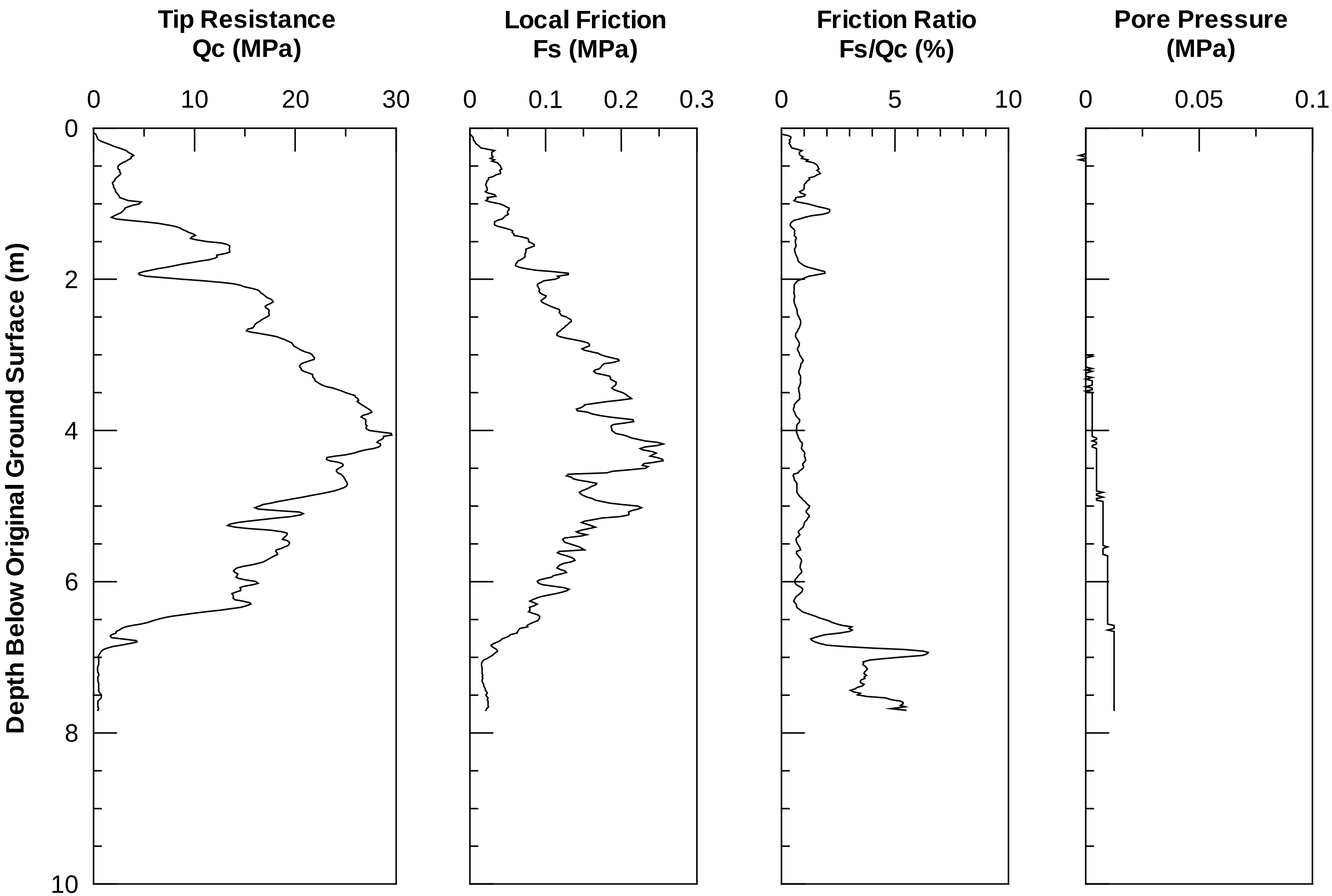


Figure 4.41 Post-treatment CPT4 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

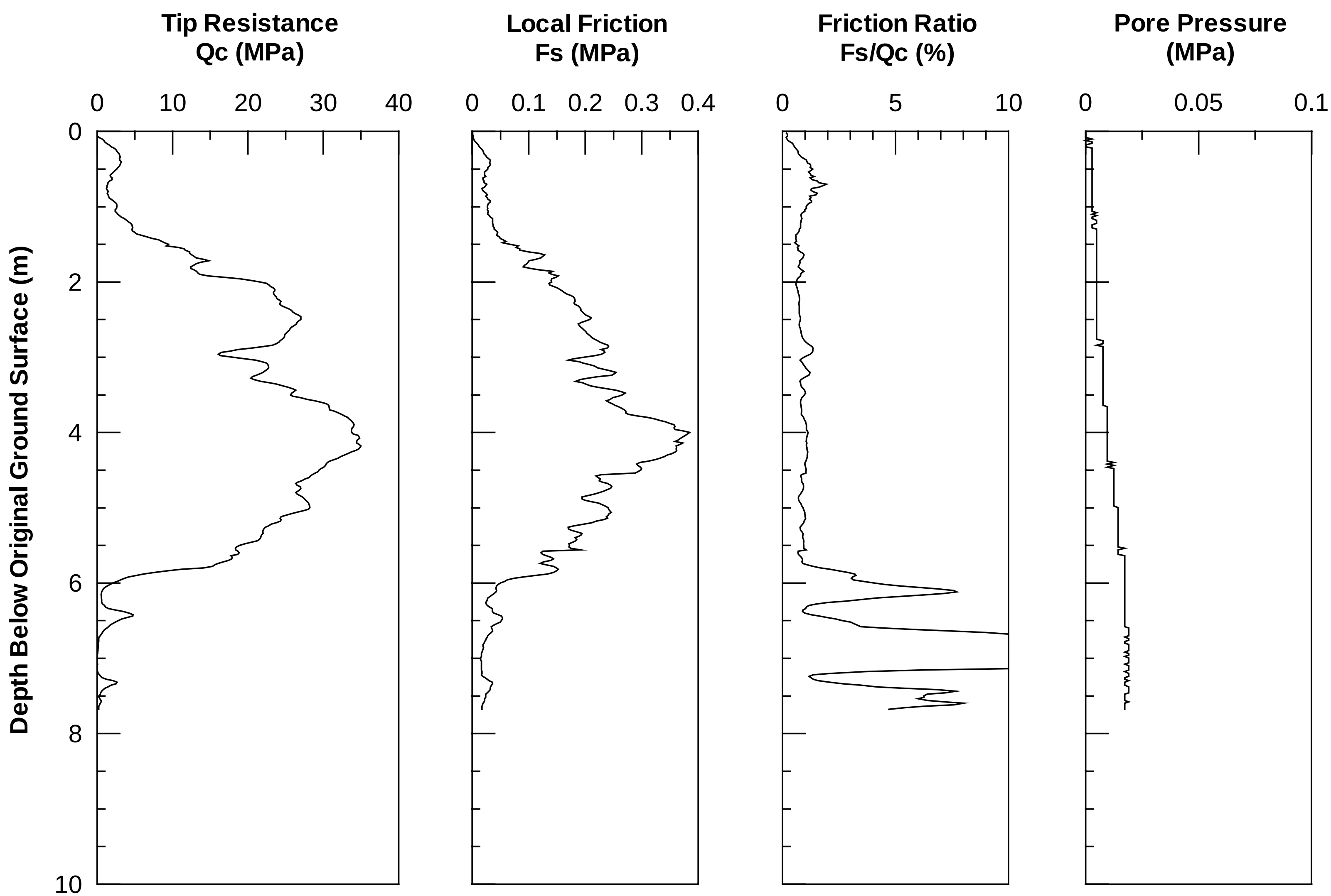


Figure 4.42 Post-treatment CPT5 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

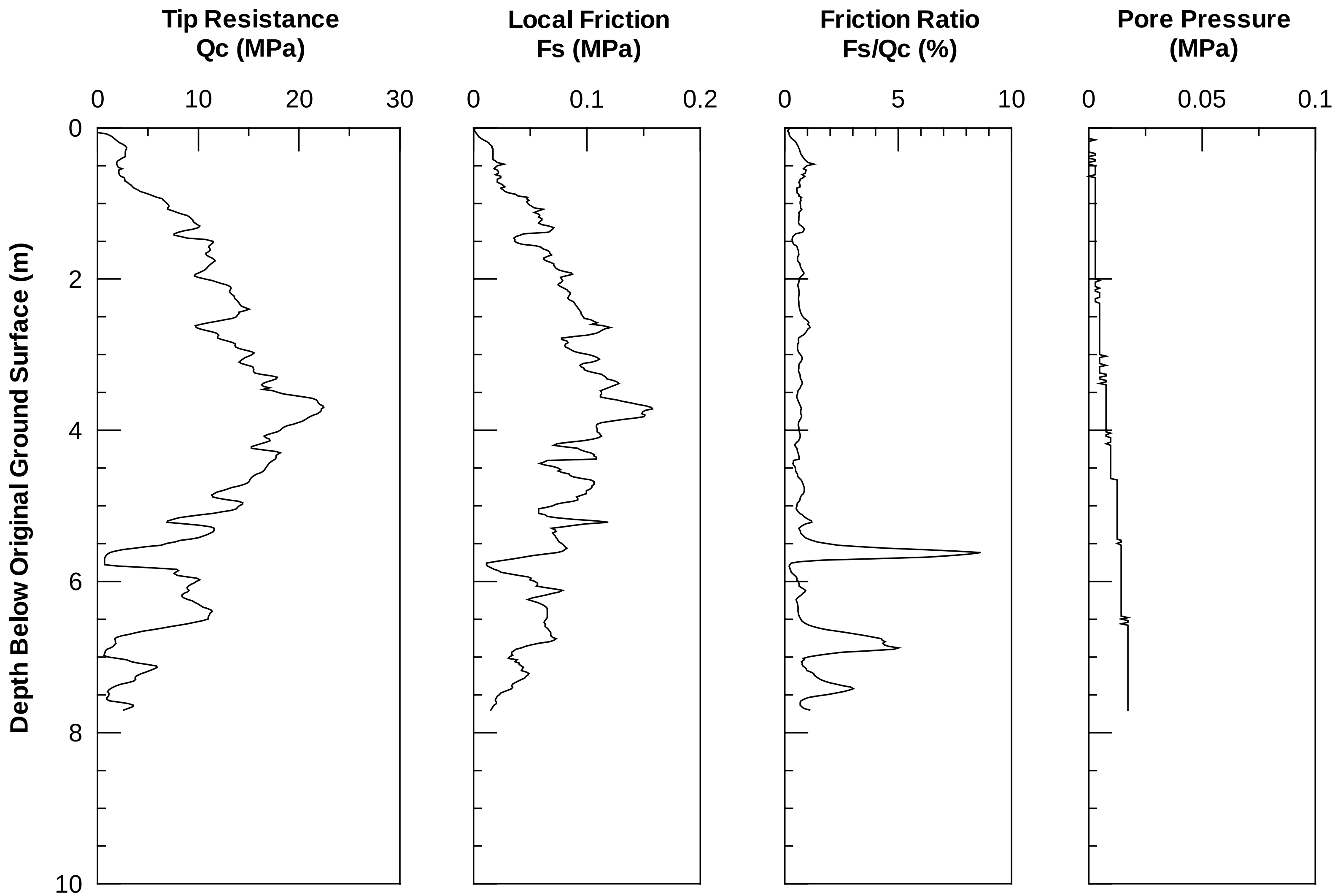


Figure 4.43 Post-treatment CPT6 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

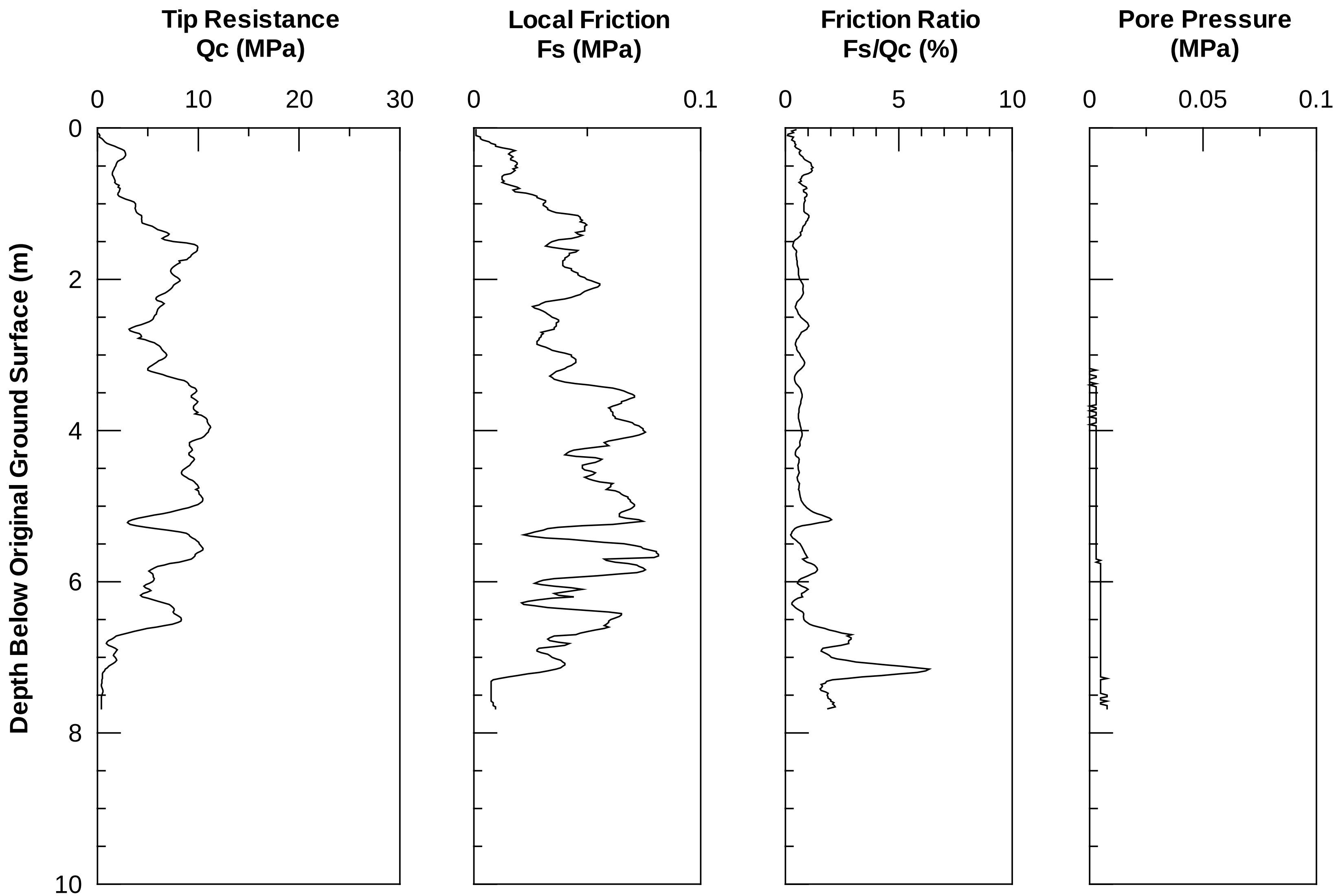


Figure 4.44 Post-treatment CPT7 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

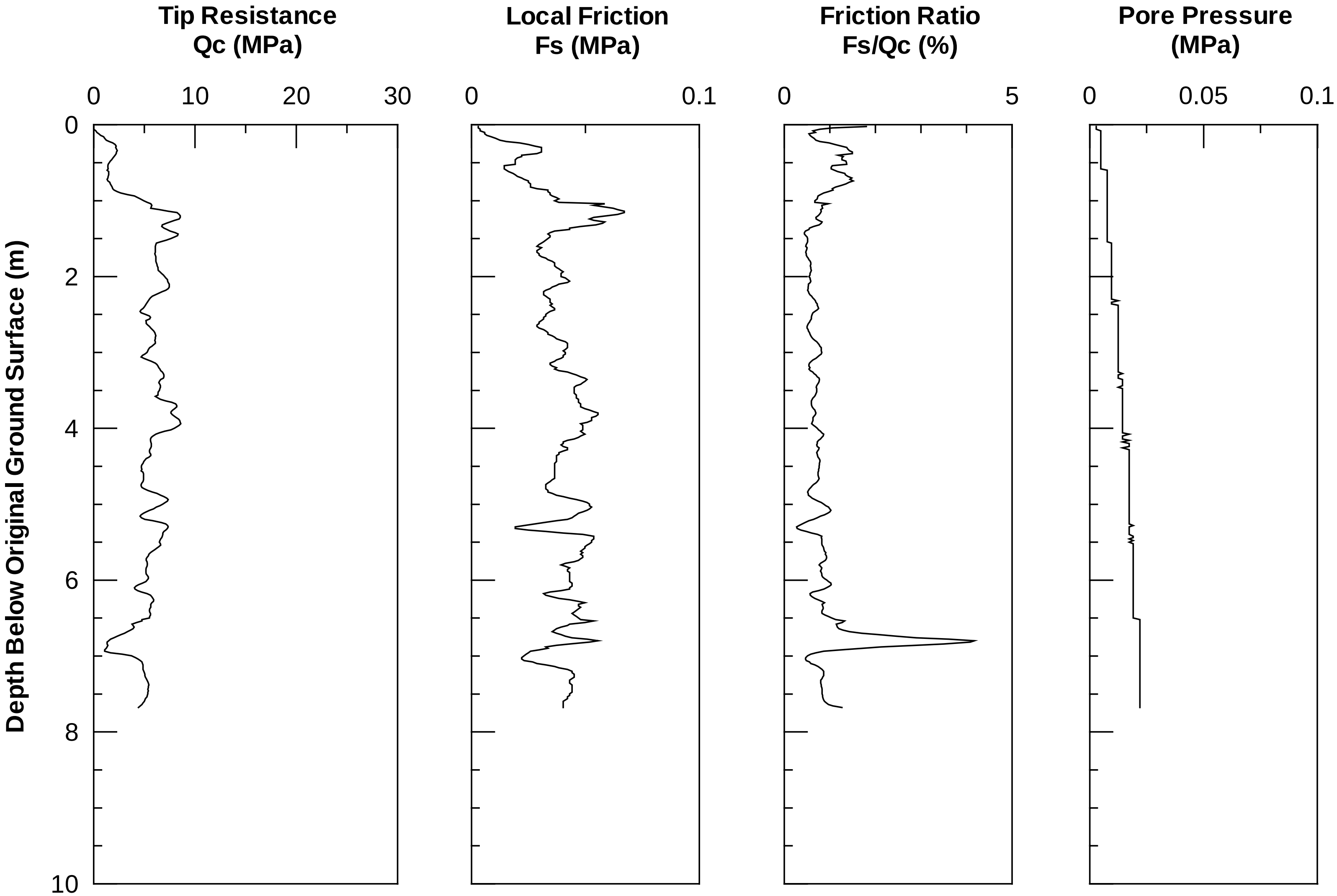


Figure 4.45 Post-treatment CPT8 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

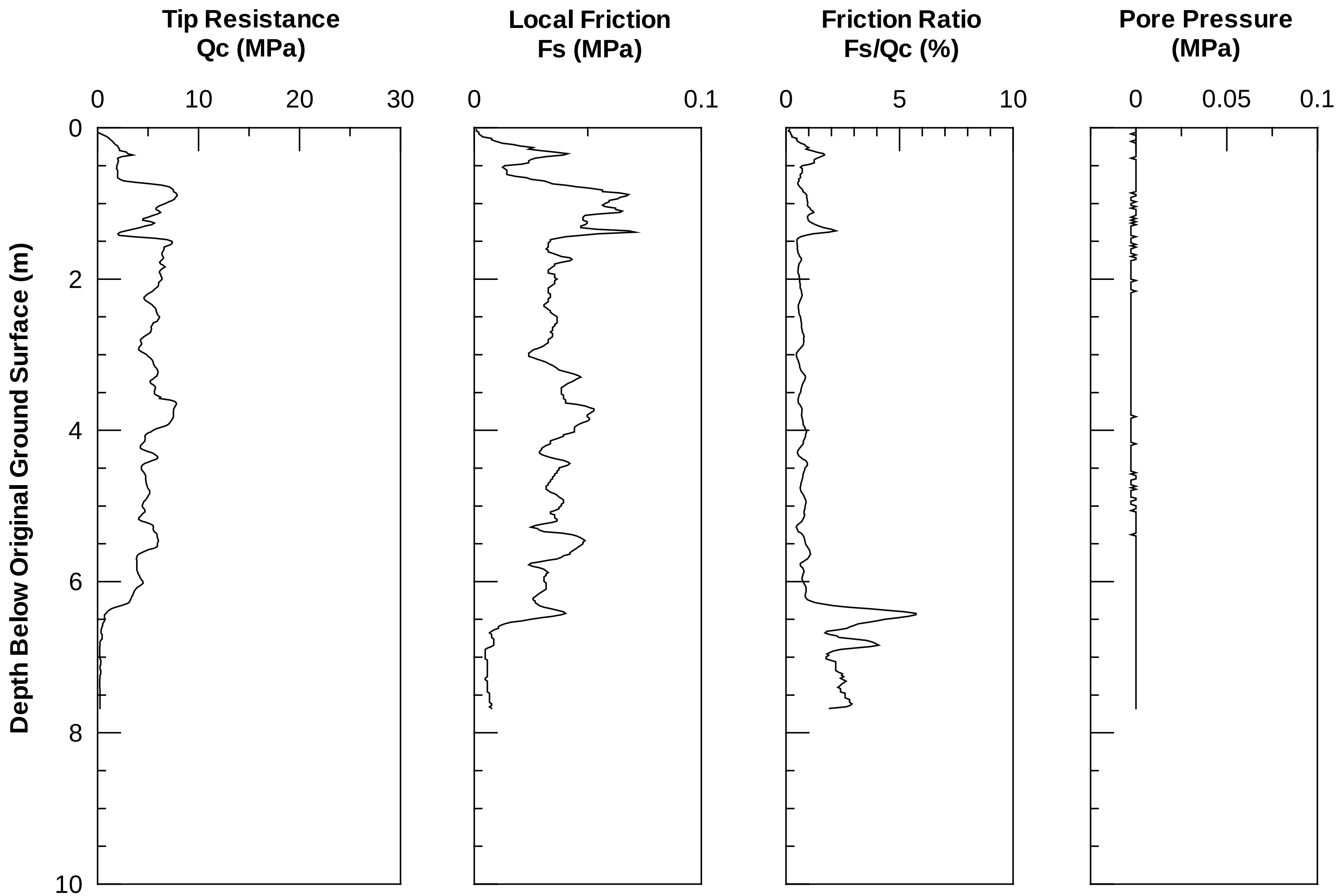


Figure 4.46 Post-treatment CPT9 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

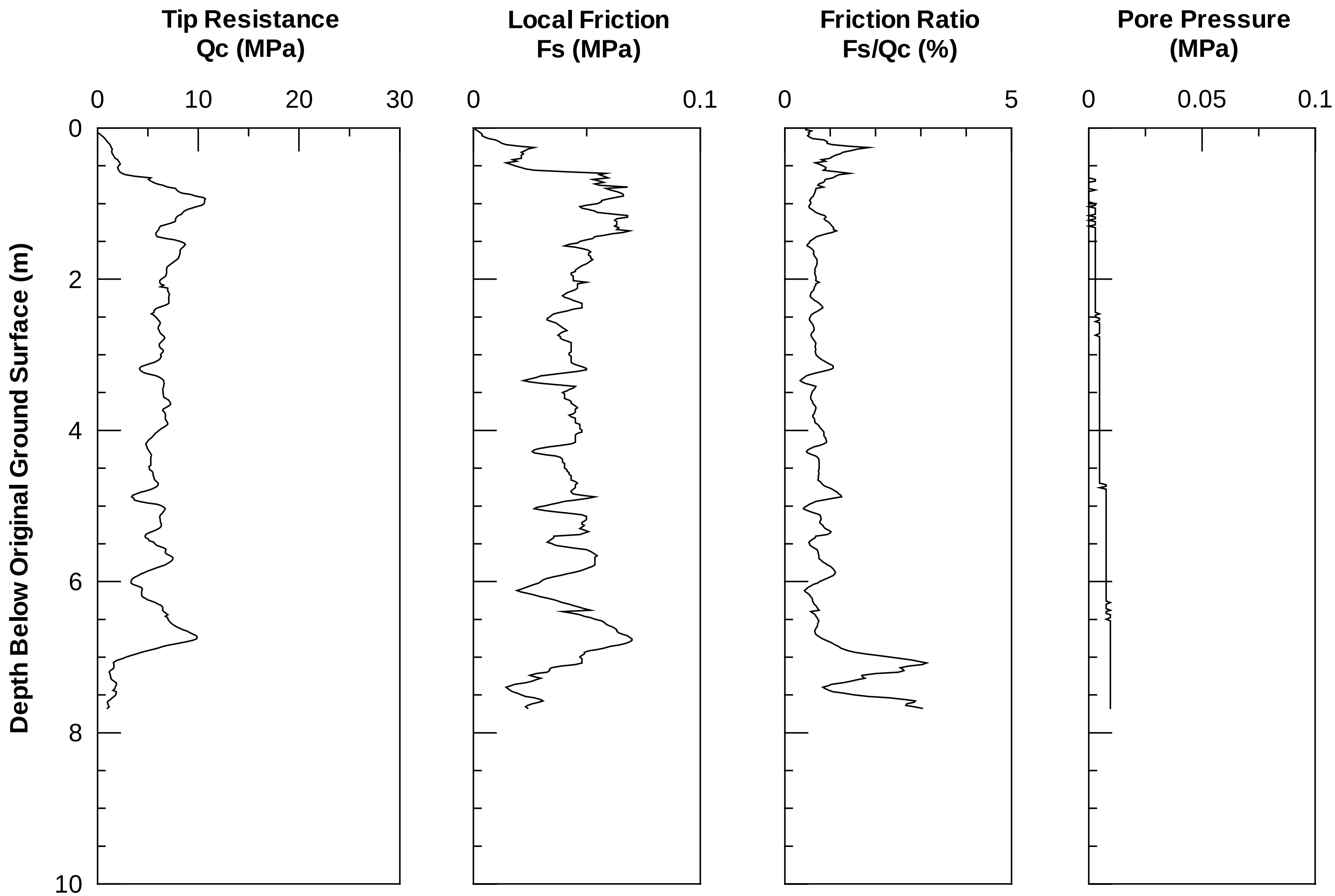


Figure 4.47 Post-treatment CPT10 Logs for 4-Pile Group/0.6m CISS Pile Test Area (2-16-99)

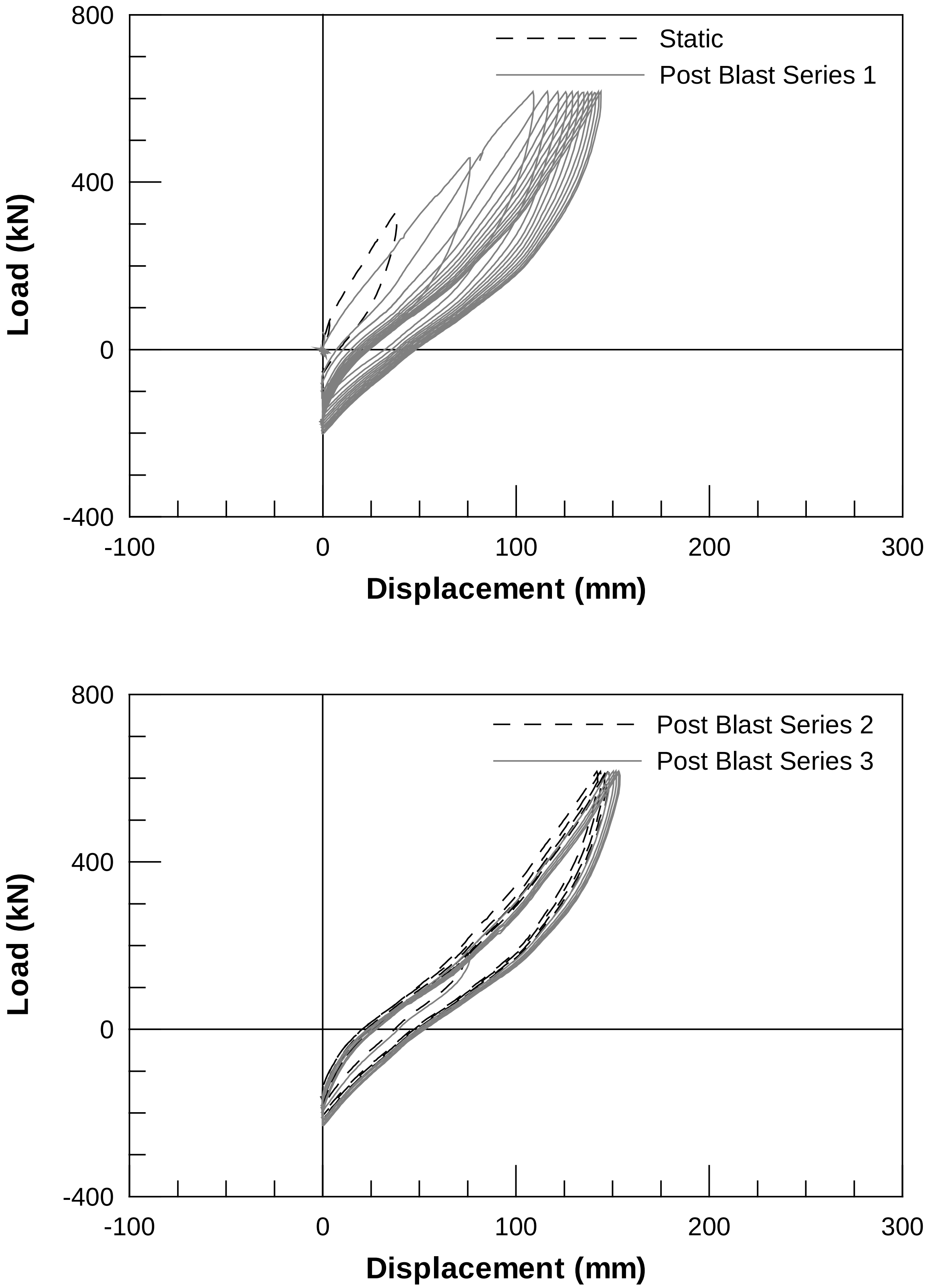
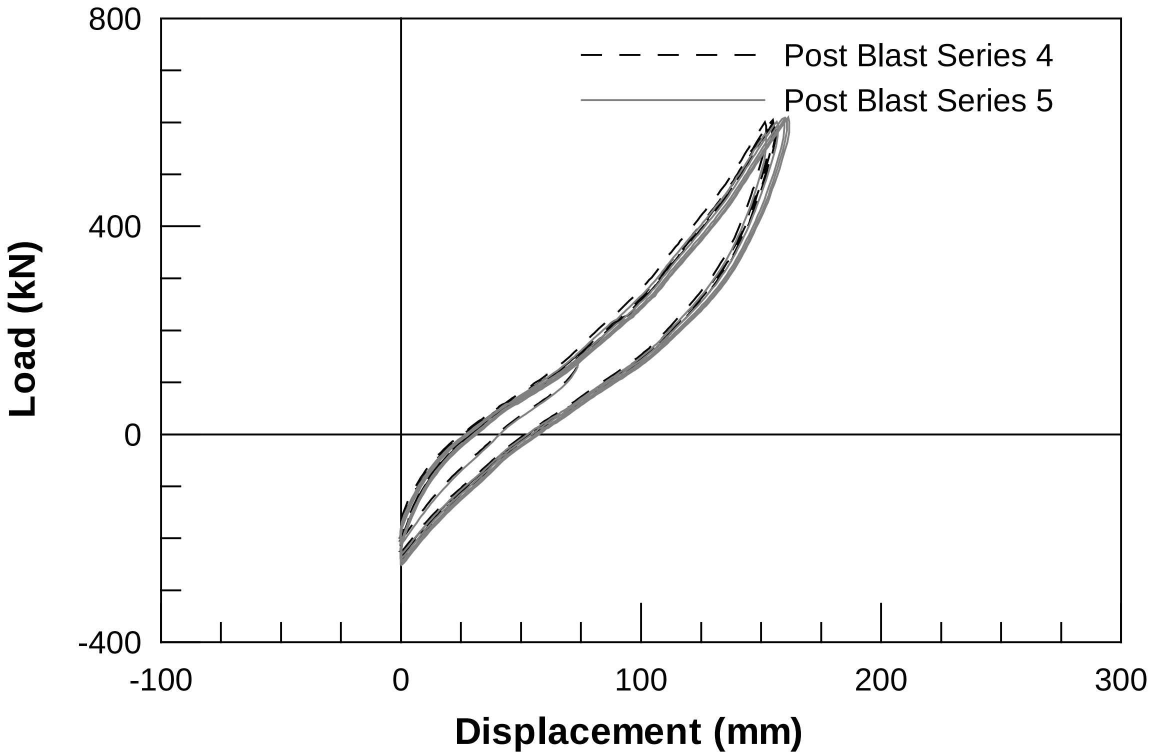


Figure 4.48 4-Pile Group Load vs. Displacement 2nd Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3



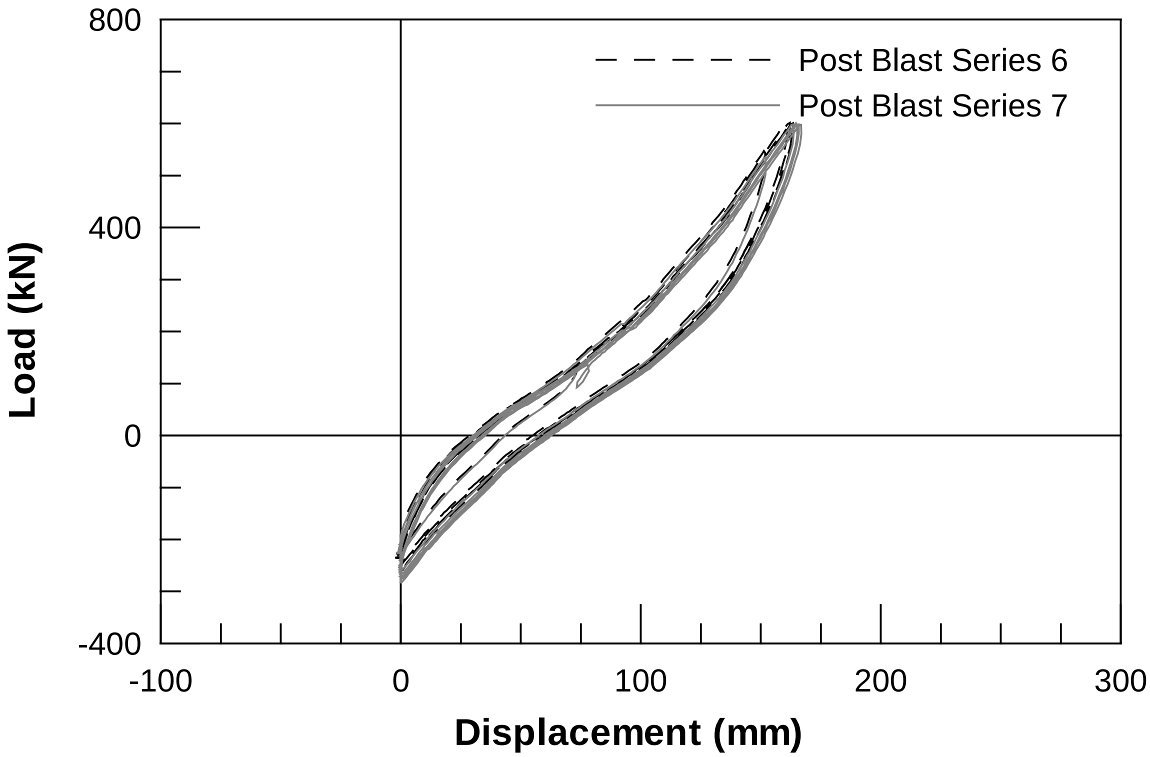


Figure 4.49 4-Pile Group Load vs. Displacement 2nd Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

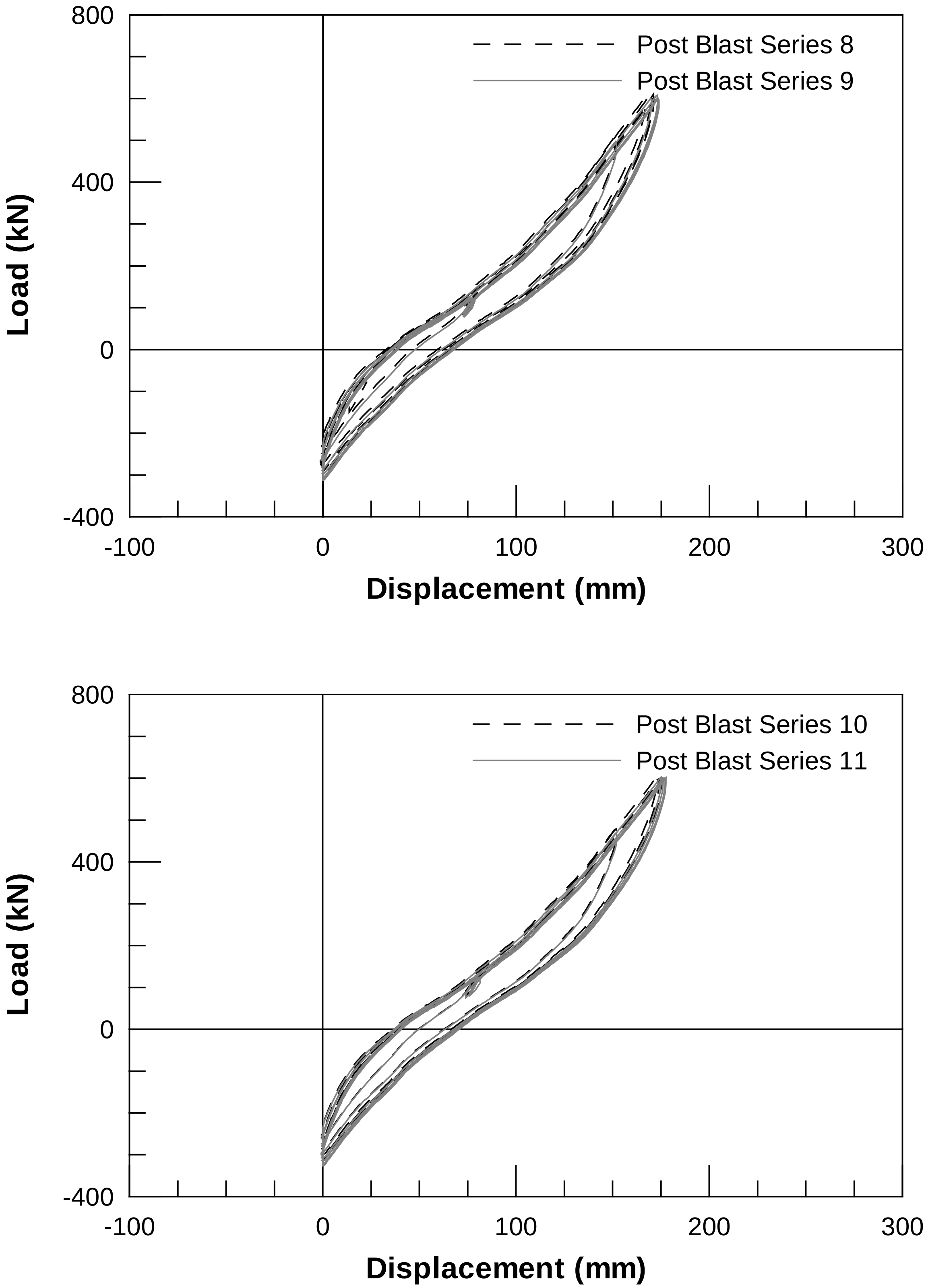
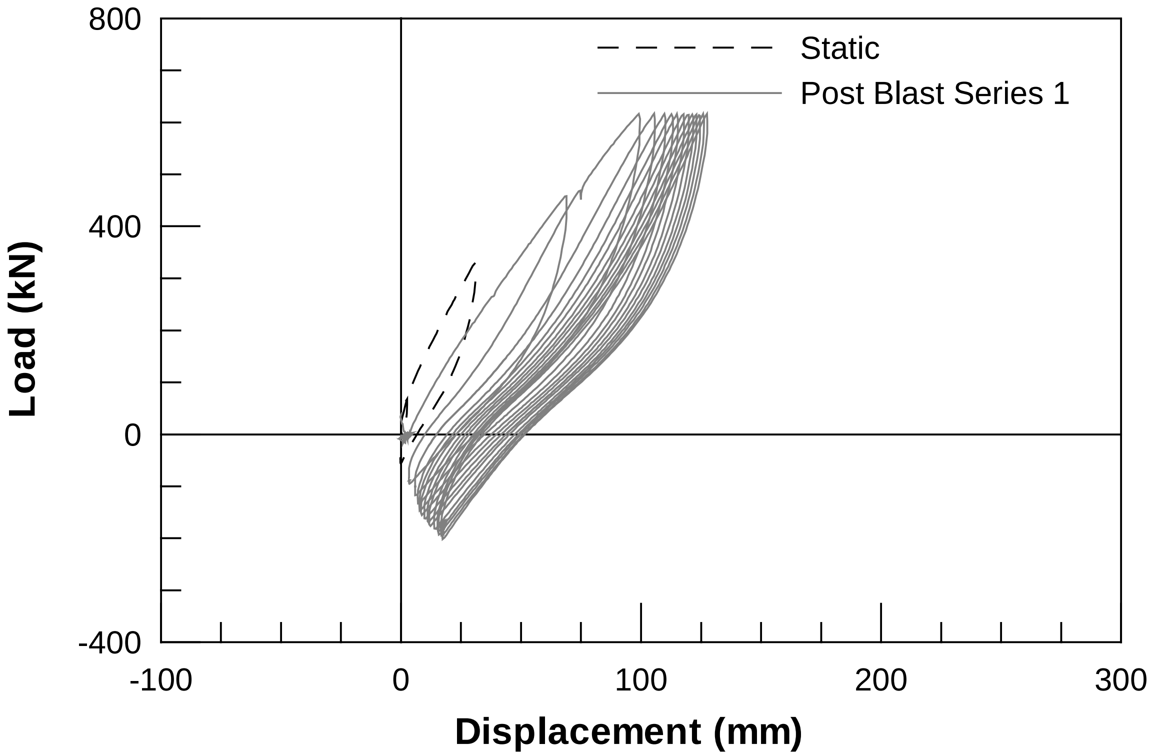


Figure 4.50 4-Pile Group Load vs. Displacement 2nd Blast a) Post Blast Series 8 and 9 b) Post Blast Series 10 and 11



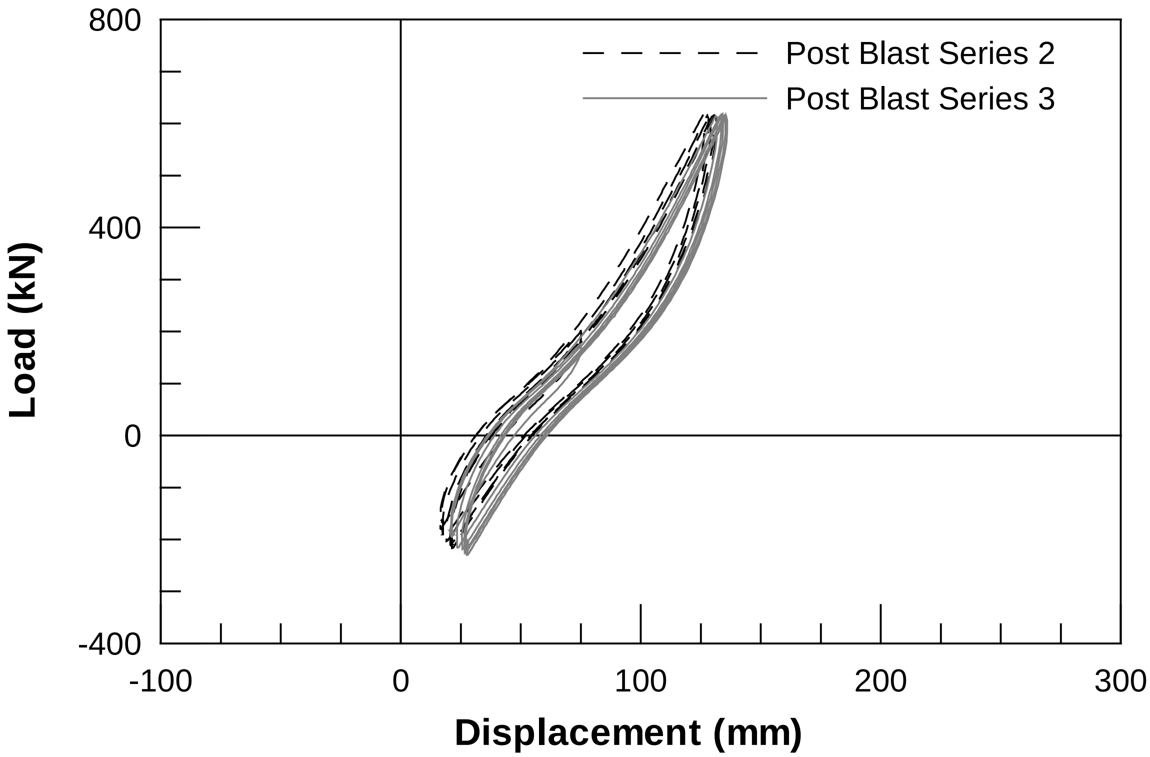


Figure 4.51 0.6m CISS Load vs. Displacement 2nd Blast a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3

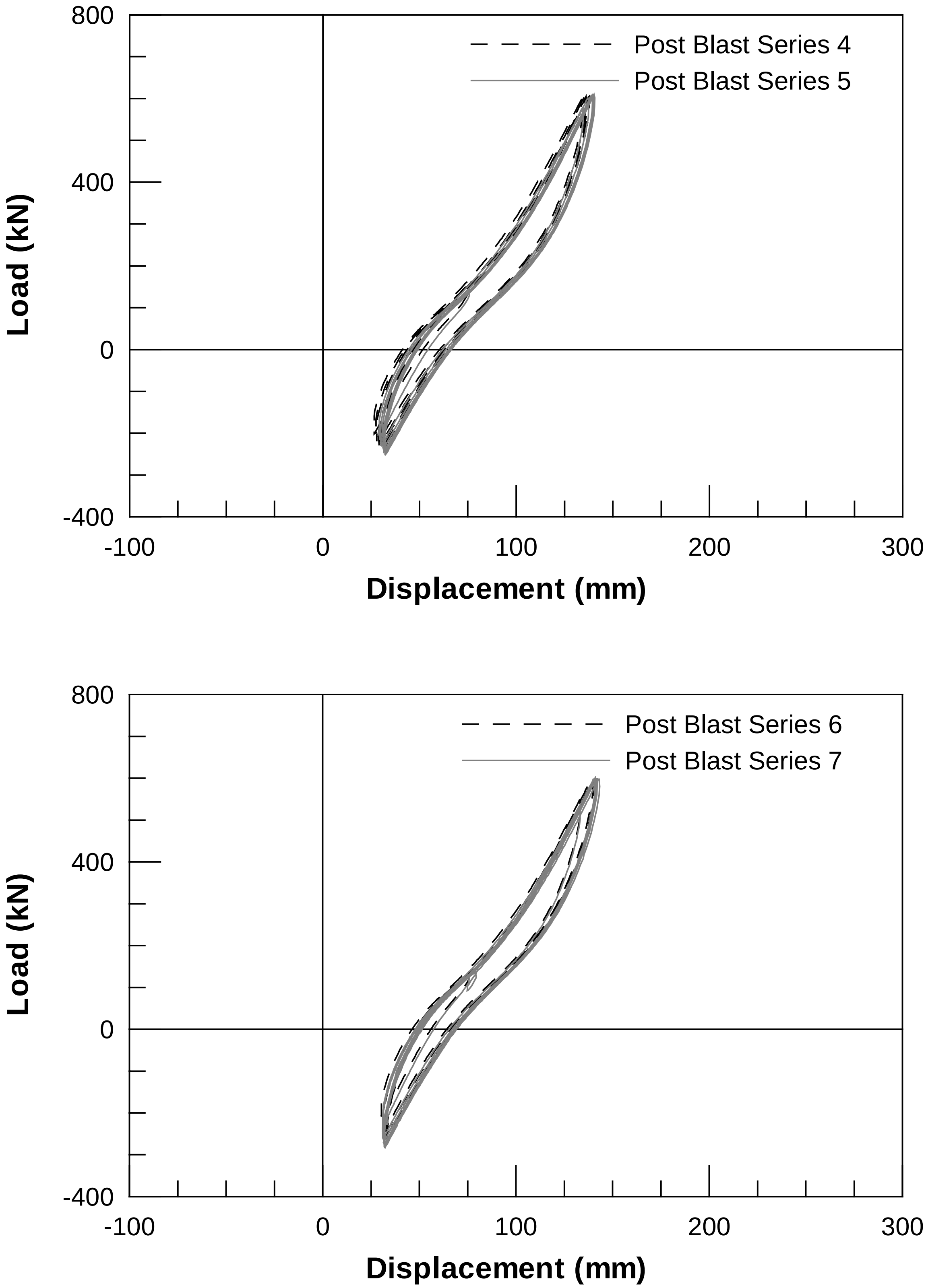


Figure 4.52 0.6m CISS Load vs. Displacement 2nd Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

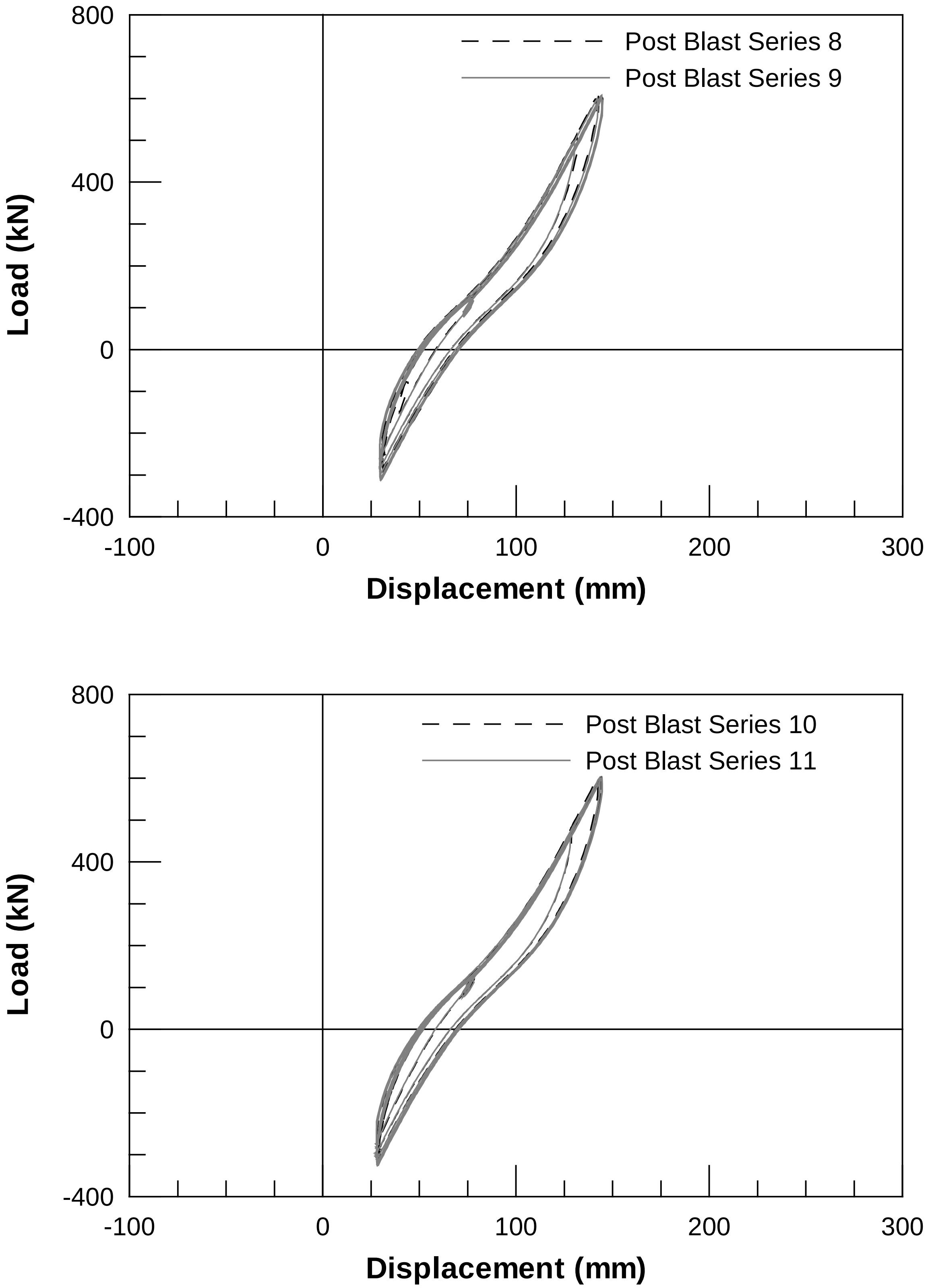


Figure 4.53 0.6m CISS Load vs. Displacement 2nd Blast a) Post Blast Series 8 and 9
b) Post Blast Series 10 and 11

PPT	Depth
10	3.96m
38	3.04m
39	3.04m
44	2.13m
47	1.21m
57	2.74m
*91	3.96m
92	3.04m
94	3.04m
95	2.13m
504	3.04m
505	4.87m
855	3.04m
4290	4.27m
4301	1.98m
4507	1.37m
4857	3.04m
5090	4.87m
5698	3.04m
5699	3.04m

*Dead

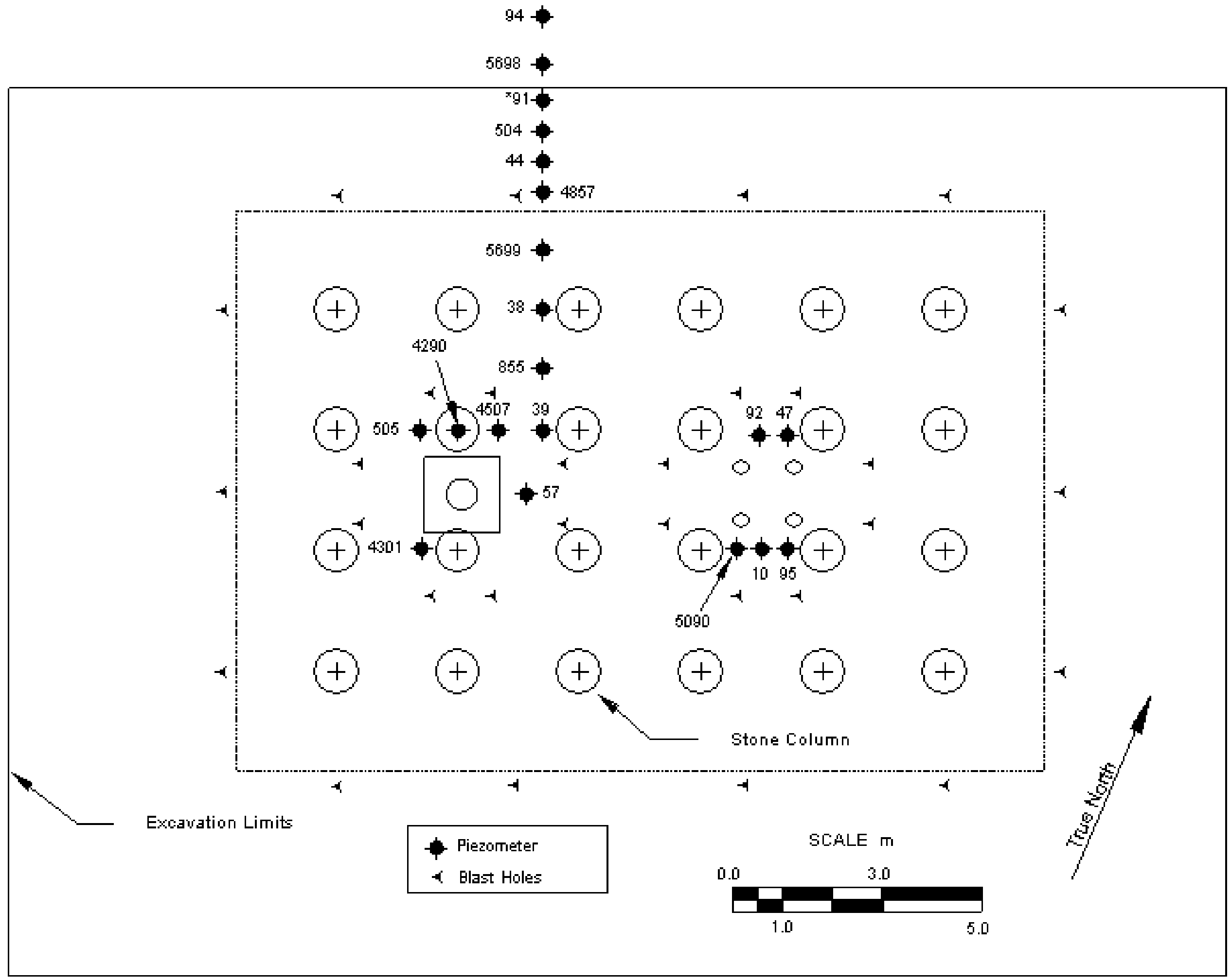


Figure 4.54 Location Map of Pore Pressure Transducers and Explosives for Stone Columns Test

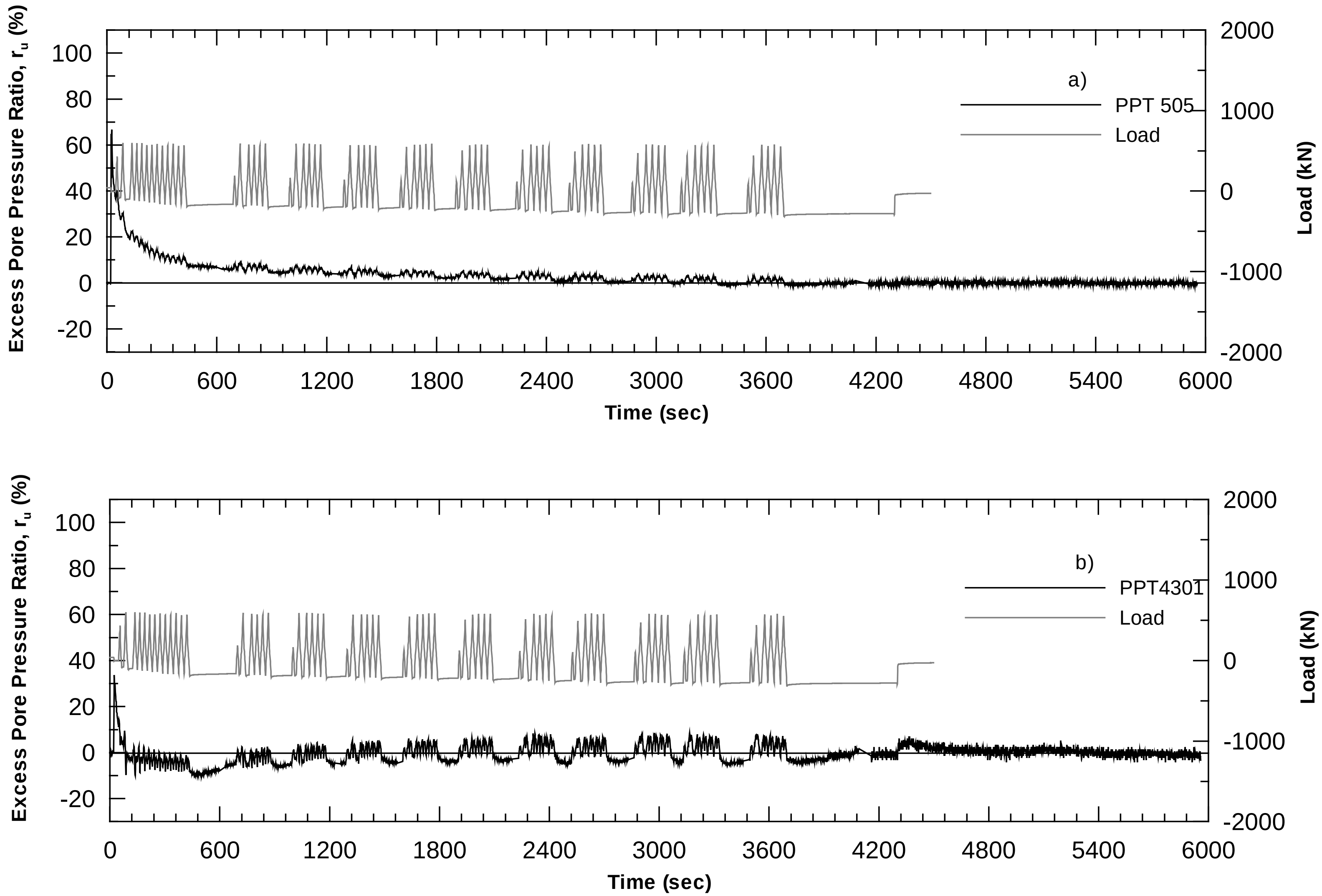


Figure 4.55 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT505 b)PPT4301

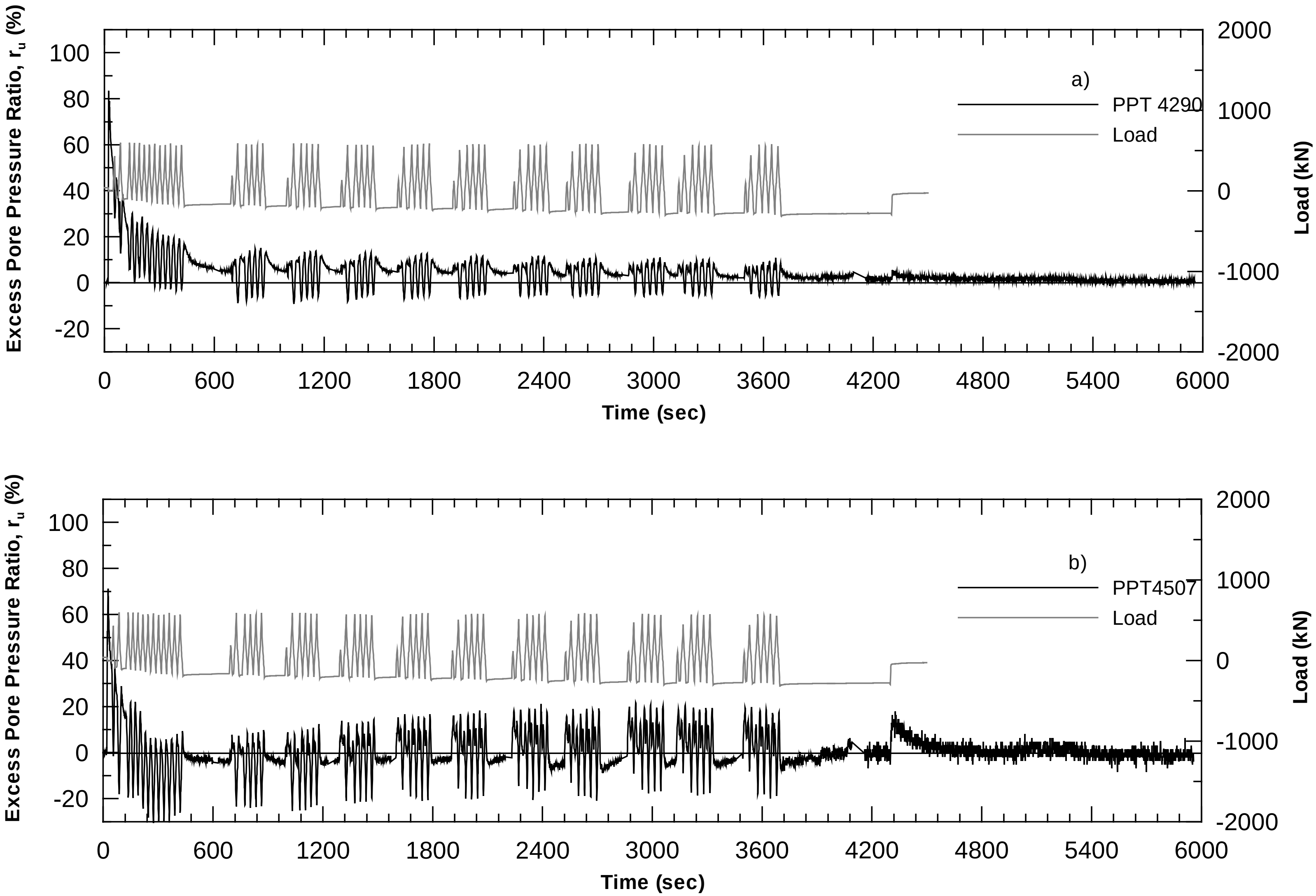


Figure 4.56 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT4290 b)PPT4507

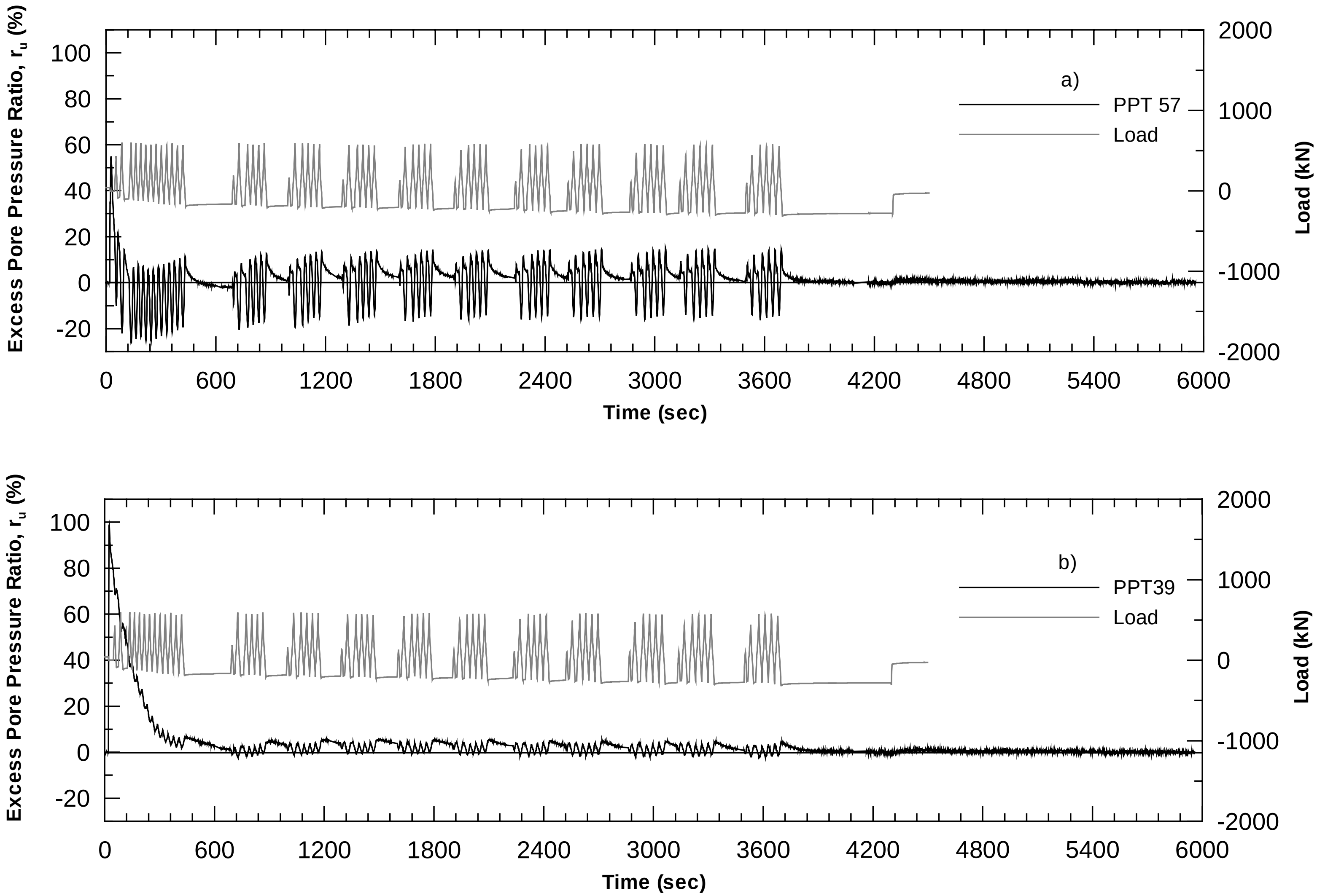


Figure 4.57 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT57 b)PPT39

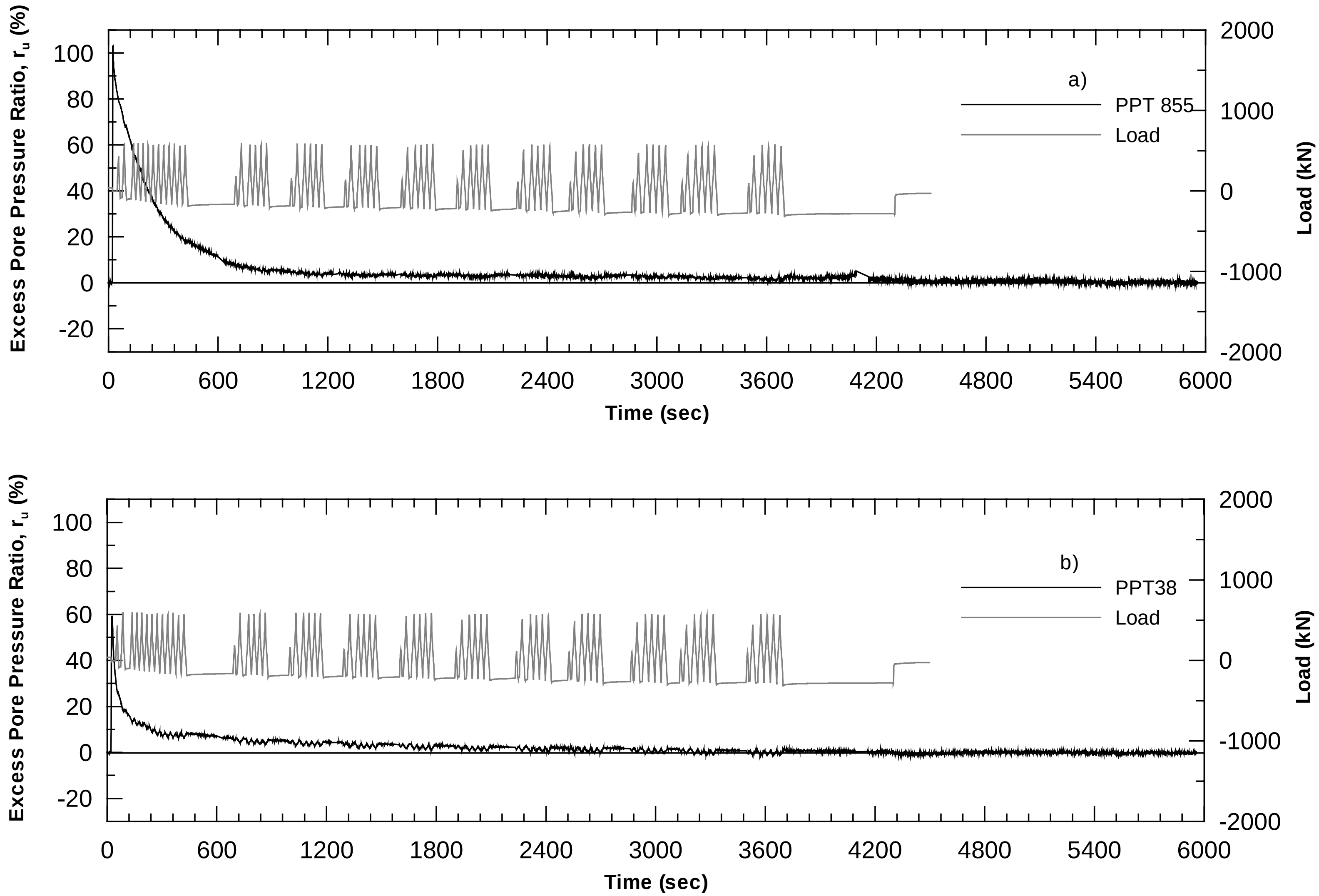


Figure 4.58 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT855 b)PPT38

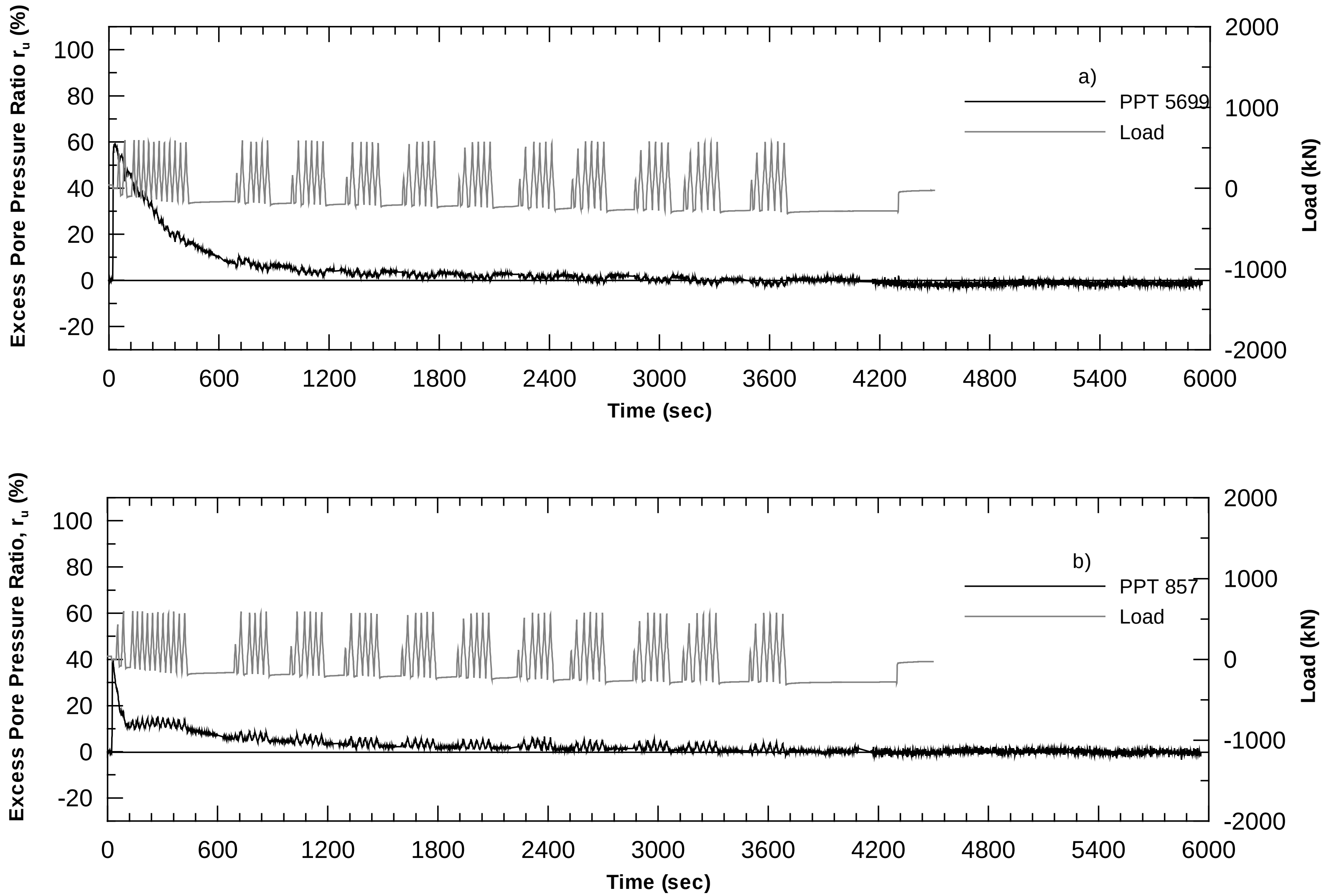


Figure 4.59 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT5699 b)PPT857

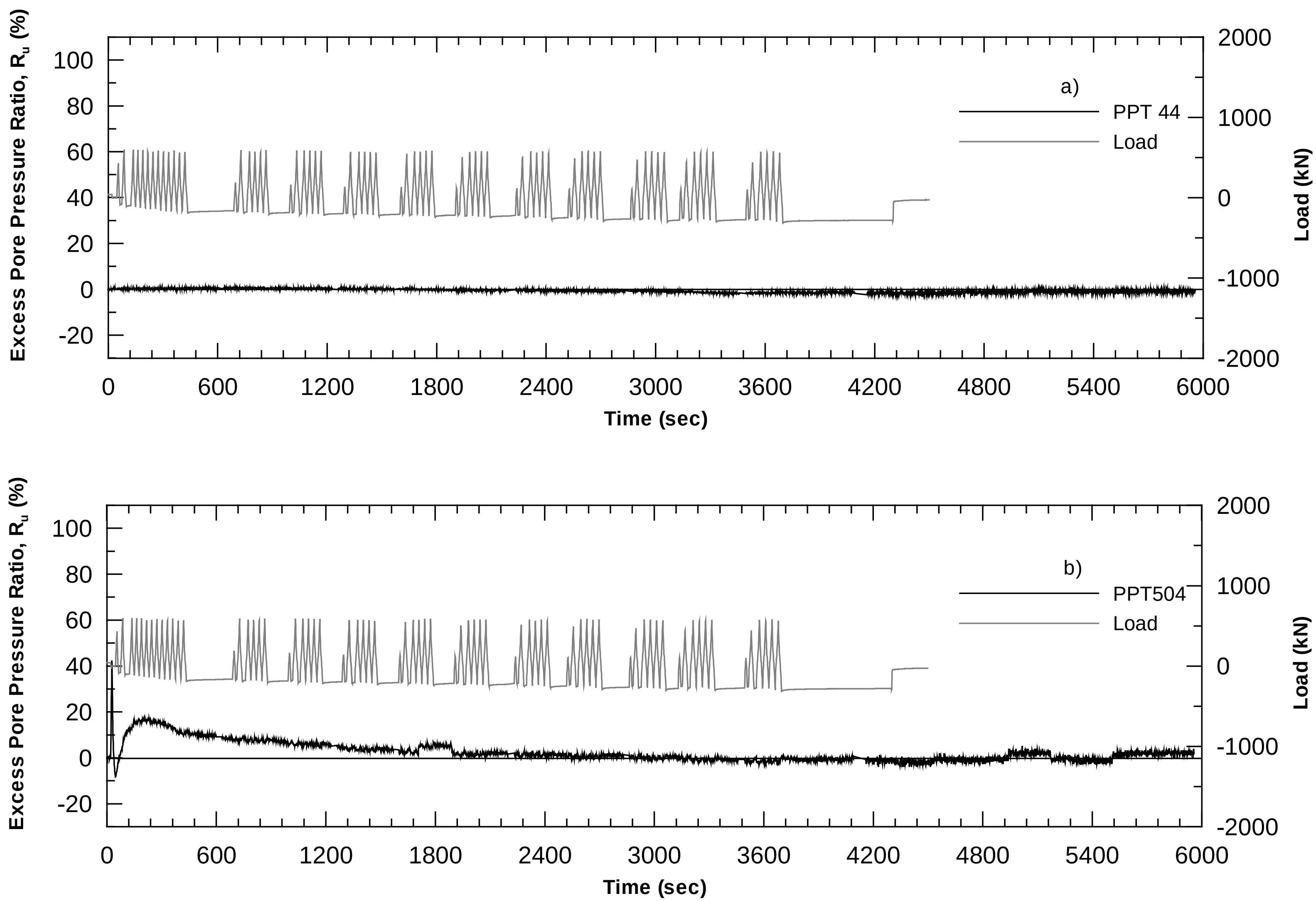


Figure 4.60 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT44 b)PPT504

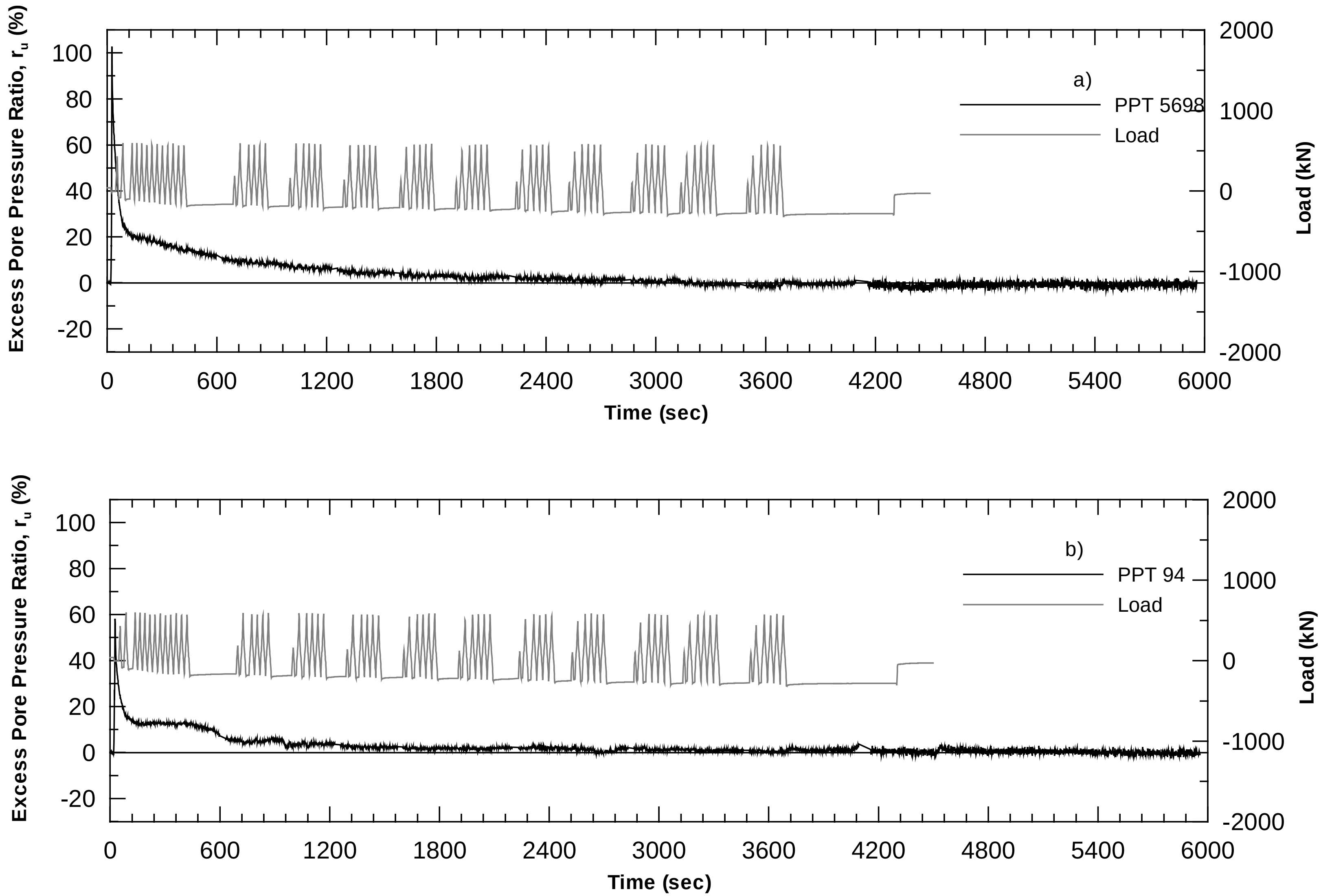


Figure 4.61 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT5698 b)PPT94

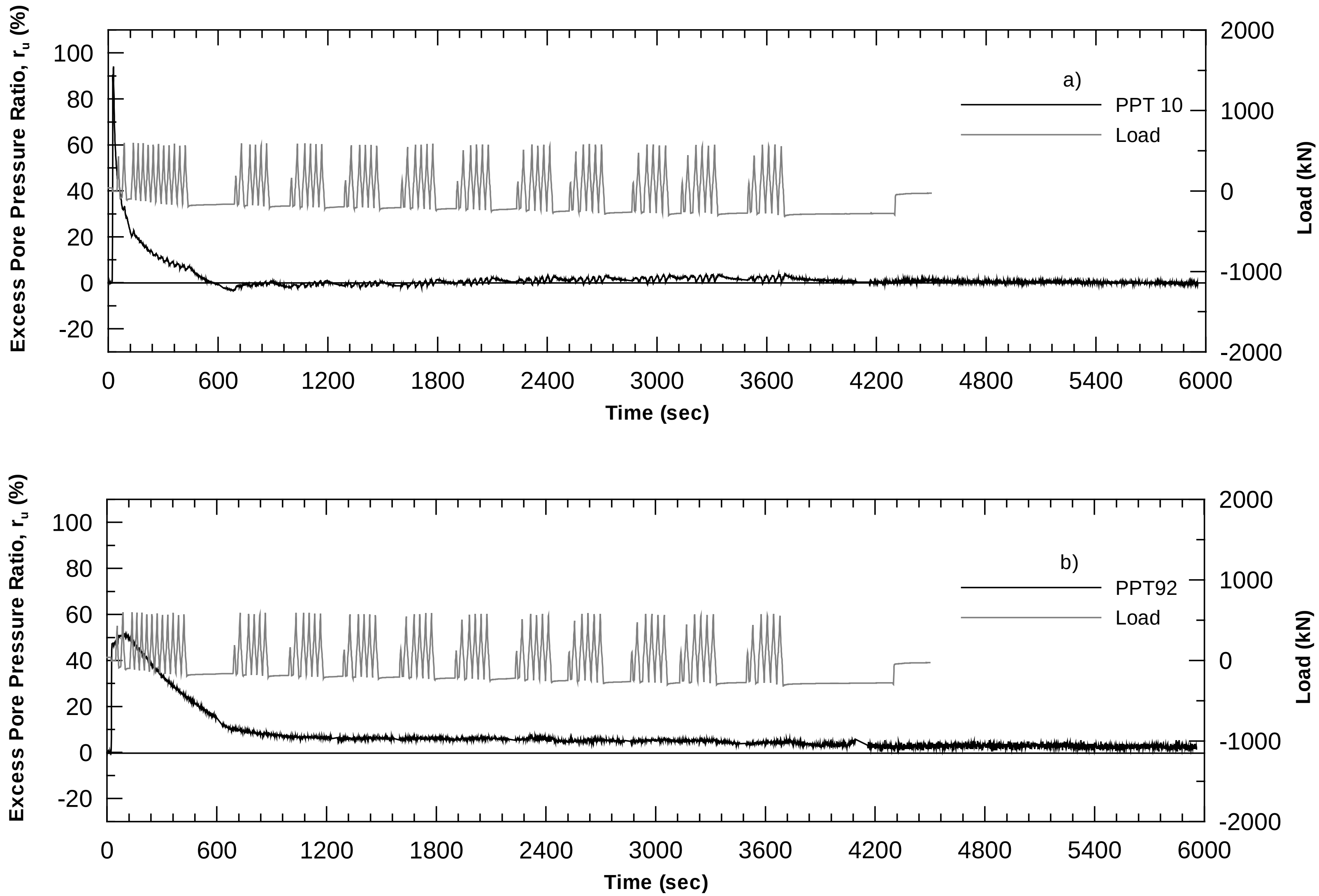


Figure 4.62 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT10 b)PPT92

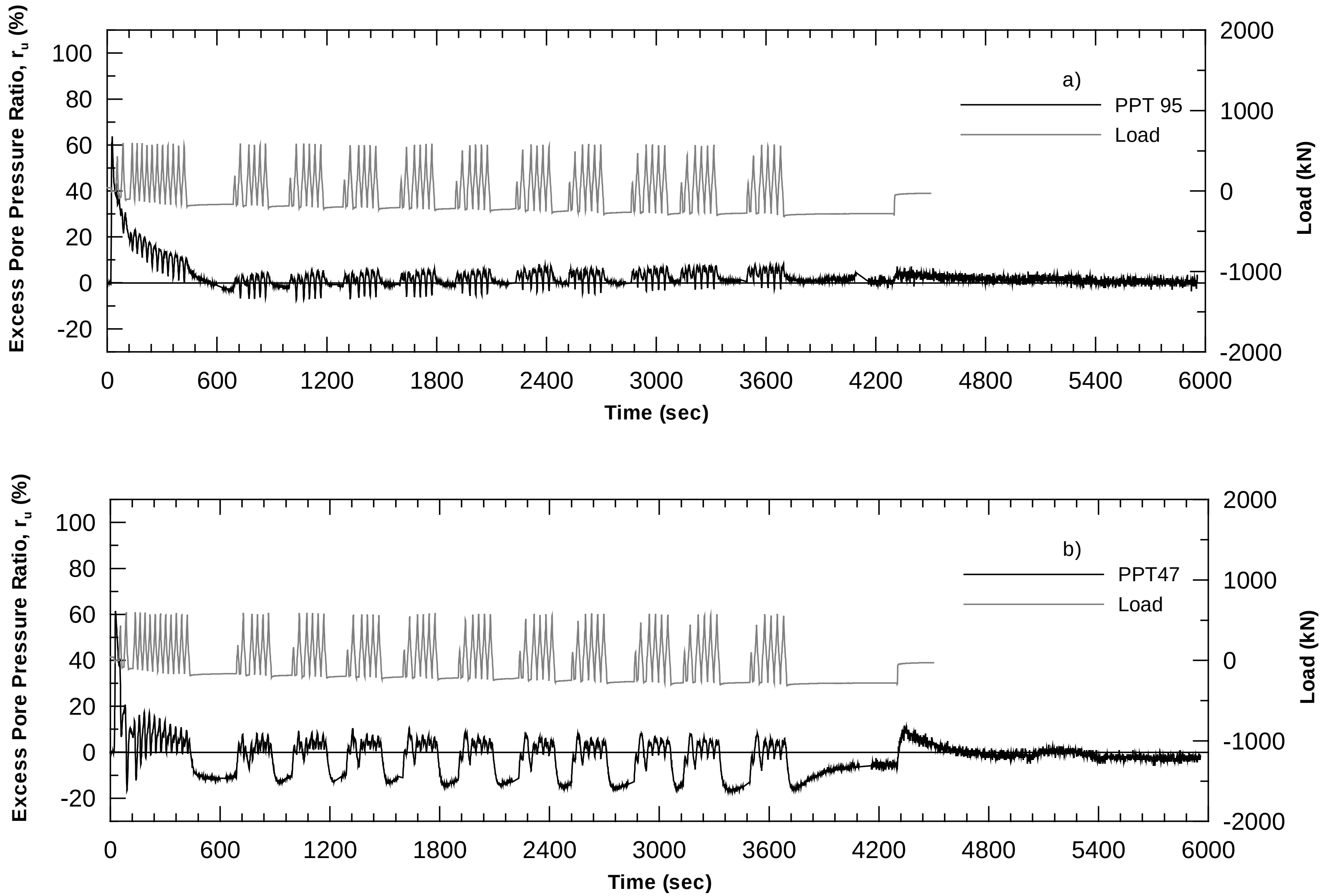


Figure 4.63 Excess Pore Pressure Ratio 4-Pile Group/0.6m CISS Pile 2nd Blast a) PPT95 b)PPT47

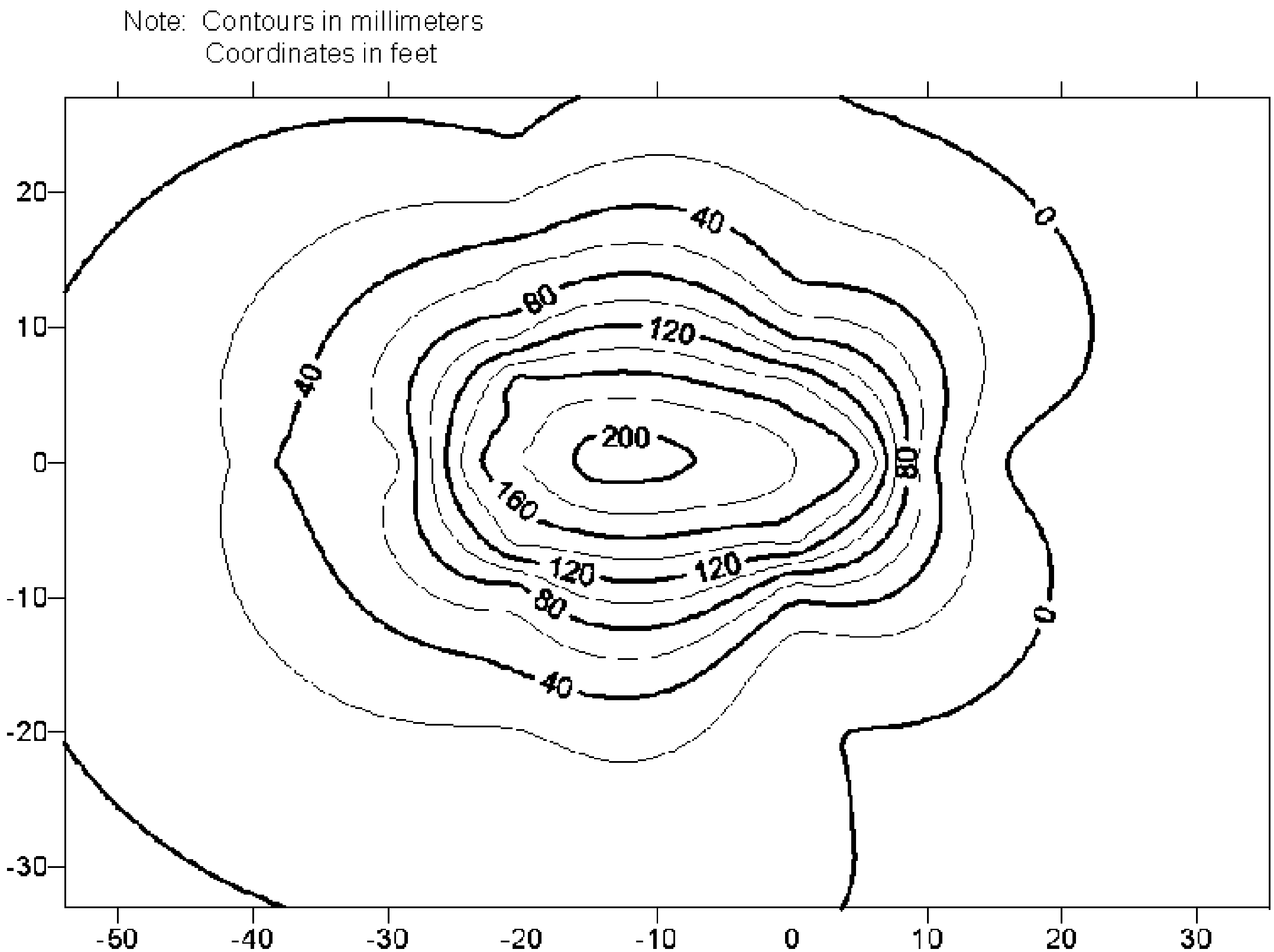


Figure 4.64 Contour of Settlement for 4-Pile Group/0.6m CISS Pile 1st Blast

5 9-PILE GROUP AND 0.9-m CISS PILE TESTS

The 9-pile group and 0.9-m CISS pile tests were performed to further investigate effects of group behavior of closely spaced piles and large diameter piles in liquefied soils. The 9-pile group consisted of 0.32 m outside diameter pipe piles spaced at 3.5 pile diameters driven to a depth of 12 m. The 0.9-m CISS pile consisted of a 0.9-m outside diameter steel shell with an 11-mm wall thickness driven to a depth of 14.8 m, after which soil inside the 0.9-m steel shell was augered out. In all cases, driving was very easy. A rebar cage consisting of thirteen 35.8 mm (#11) bars spaced at 169 mm on center and 15.9 mm (#5) spiral with a 165 mm spacing was then placed inside the steel shell which was then filled with concrete. A 2.2 MN hydraulic actuator placed between the pile group and CISS pile applied lateral loads during testing. Pile instrumentation consisted of strain gages along the depth of the pile and displacement transducers at the pile head. Pore pressure transducers were placed along the sides of the piles and at various distances in front of the piles. Data from the subsurface investigation and in-situ testing are presented along with load and displacement test results, excess pore pressure ratios, and settlement contours resulting from blasting.

5.1 9-Pile Group and 0.9-m CISS Pile Site Characterization

A subsurface investigation was performed at the 9-pile group/0.9-m CISS pile site before foundation testing was conducted. The location of the soil boring BH-3 along with other in-situ tests is shown in Figure 5.1. The soil log from BH-3 is presented in Figure 5.2. The water table at the time of boring was 1.5 m below the original ground surface. The soil profile consists of a loose sand layer extending from the ground surface to a depth of 8.1 m, which is underlain by a gray clay layer extending from 8.1 m to 9.9 m. Another loose sand layer was found between 9.9 m and 13.1 m. A gray clay extended from 13.1 m to 24.4 m. The soil sample recovered at the bottom of the hole at 24.8 m consisted of silt and silty sand. Low SPT blow counts were observed at this site indicating a susceptibility to liquefaction.

In addition to SPT's, five CPT's were performed at this site before foundation installation (Figures 5.3 through 5.7), one after foundation installation but before lateral load testing (Figure 5.8), and two additional CPT's were performed approximately four months after foundation testing (Figure 5.9 through 5.10). The cone tip resistance from the CPT's performed before foundation installation show values typically between 4 and 7 Mpa in the upper sand layer with little difference in the measured tip resistance after pile installation. The low tip resistance confirms the liquefaction susceptibility of the sand layer.

Shear and compression wave velocity tests were also conducted at the 9-pile group/0.9-m CISS pile site. A seismic CPT test, two down-hole tests, and two suspension logger tests were performed to obtain the shear wave velocity profile. Results of the seismic CPT differ greatly compared to the other shear wave velocity tests. This may be due to the location of CPT3-6 being near the edge of the excavation where the soil density or in-situ stress was significantly greater than toward the middle of the excavation. There is reasonably good agreement between the down-hole and suspension logger tests. The suspension logger was also used to measure compression wave velocity. Velocity profiles for both shear and compression waves are presented in Figure 5.11.

5.2 9-Pile Group and 0.9-m CISS Pile Test Results

A plan view of the test set-up is shown in Figure 5.12, and a profile of the test set-up profile is shown in Figure 5.13. The lateral load for the 9-pile group/0.9-m CISS pile test was applied approximately 1 m above the excavated ground surface. The test procedure and results are described below.

5.2.1 Load-Displacement Results for Test 1

Lateral load testing of the 9-pile group/0.9-m CISS pile commenced on February 18, 1999. The lateral loading consisted of a static test and ten post blast loading series. The lateral load tests were run under displacement control. When either the 9-pile group or 0.9-m CISS pile reached the target displacement, the piles were unloaded until one reached the zero displacement target. The two target displacements for the static test were 3 mm and 38 mm. The 9-pile group reached the target displacement during the static test resulting in the 0.9-m CISS pile displacing less than the 9-pile group. The first post blast series consisted of ten cycles with a target displacement of 37 mm, one cycle at 76 mm, one cycle at 152 mm, and eleven cycles at 228 mm. Approximately 12 minutes elapsed during the first post blast load series. Post blast Series 2 through 10 consist of one 76 mm, one 152 mm, and four 228 mm cycles, and lasted for approximately 4 minutes each. Load versus displacement for the 9-pile group is presented in Figures 5.14 through 5.16. The load-displacement plots for the 0.9-m CISS pile are shown in Figures 5.17 through 5.19. The soil pile system decreased in stiffness during the first five cycles of post blast Series 1. The decrease in stiffness for the last six 228 mm cycles was negligible. After approximately four cycles the soil-pile stiffness of the 9-pile group and

0.9-m CISS pile had decreased to 37% and 25% of the static stiffness respectively. The soil-pile stiffness incrementally increased during post blast Series 2 through 10. Approximately 45% of the static soil pile stiffness was regained by the last load series for the 9-pile group and 40% for the 0.9-m CISS pile. The increase in pile stiffness during the loading series is due to a reduction in pore water pressure with time.

5.2.2 Excess Pore Pressure Response for Test 1

Pore pressure transducers were used to obtain excess pore pressure ratios during the lateral load testing and for approximately one hour after loading stopped. The location and depth below grade of each transducer is shown in Figure 5.20. Pore pressures were measured for approximately two hours after detonation of the down-hole explosives and are shown in Figures 5.21 through 5.33. Transducer data is presented according to location starting with transducers north of the 9-pile group and ending with transducers south of the 0.9-m CISS pile.

A review of excess pore pressure ratios show initial ratios ranging from 70% to over 100% immediately following the blast indicating liquefaction had occurred. Sand boils began forming minutes after detonation of the explosives, providing additional evidence of liquefaction. Large fluctuations in the excess pore pressure ratio near the pile indicate the influence the piles have on the nearby soil, especially at shallow depths where pile displacements are greatest.

Approximately one hour after the blast, excess pore pressure ratios generally ranged from 20% to 50% which correlates well with excess pore pressure ratios at the same time for the 0.6-m CISS and 4-pile group test. However, excess pore pressure ratios adjacent to the 0.9-m CISS pile at depths of 1.1 and 2.0 meters were as high as 70%. The higher pore pressure ratios near the 0.9-m CISS pile may be a result of pile diameter effects. The excess pore pressure ratios had were close to pre-blast levels after approximately two hours.

5.2.3 Load-Displacement Results for Test 2

The second 9-pile group/0.9-m CISS pile lateral load test occurred on February 19, 1999, one day after the first test. The static test consisted of displacing the piles to 7 mm for five minutes, increasing and to a displacement of 13 mm and holding for eight minutes, increasing and holding a displacement of 19 mm for five minutes, and increasing to and maintaining a final displacement of 26 mm for five minutes. After the 26 mm displacement, the piles returned to a zero displacement and then loaded to a displacement of 38 mm. Approximately 25 minutes elapsed during the static test. The first post blast load series took 14 minutes to execute. The remaining series occurred at 5-minute intervals. The first nine post blast series occurred during the first hour after blasting. The tenth post blast series took place approximately two hours after blasting and the eleventh post blast series occurred 2.5 hours after blasting. Post blast Series 1 consisted of one displacement cycle at 56 mm, on cycle at 128 mm, one cycle at 192 mm, three cycles at 228 mm, one cycle at 278 mm, and four cycles at 300 mm. Post blast Series 2 through 4 onsisted of one 72 mm cycle, one 150 mm cycle, one 228 mm cycle, and three 300 mm cycles. After post blast Series 4 residual displacement of the 0.9-m CISS pile exceed the 72 mm target displacement, therefore post blast Series 5 through 10 did not contain the 72 mm cycle. The eleventh post blast series consisted of one 174 mm displacement, one 205 mm displacement, one 236 mm displacement, and three 270 mm displacements. Load-displacement data is presented in Figures 5.34 through 5.36 and Figures 5.37 through 5.39 for the 9-pile group/0.9-m CISS pile respectively. The 9-pile group reached the target displacements for the static test while the 0.9-m CISS pile reached the target displacement for the post blast testing. Upon detonation of the explosives, the soil-pile systems began to decrease in stiffness. The soil-pile stiffness at the end of post blast Series 1 was 50% and 30% of the static stiffness for the 9-pile group and 0.9-m CISS pile respectively. Approximately one hour after the blast, the 9-pile group and 0.9-m CISS pile had regained 60% and 50% of the static stiffness. The soil-pile system stiffness increased to approximately 70% and 60% of the static stiffness for the 9-pile group and 0.9-m CISS pile respectively 2.5 hours after the blast. The increase in stiffness between each load series is attributed to the decrease in pore water pressure with time.

5.2.4 Excess Pore Pressure Response for Test 2

The location and depth of pore pressure transducers used to obtain excess pore pressure ratios is presented in Figure 5.20. Figures 5.40 through 5.52 show the variation of excess pore pressure ratio and load with time. Data from the PPT's is presented according to transducer location from north to south. Excess pore pressure ratios generally range between 50% and 100% immediately following the blast and 0% and 20% approximately 70 minutes after blasting. Sand boils began to form approximately three minutes after detonation of the explosives. Generally, pore pressure dissipation between one and two hours after the blast was negligible.

5.3 Blast Induced Settlement

An elevation survey was performed prior to and after blasting in an effort to measure settlement due to blasting. Contours of settlement due to the first and second blast are presented in Figures 5.53 and 5.54. The contours of settlement are in centimeters, while the coordinates from the edge of the excavation are in meters. Settlement of nearly 320 mm was observed after the first test, and an additional 170 mm at the conclusion of the second test.

5.4 Summary

Two series of lateral load tests were performed on a 9-pile group and a 0.9-m CISS pile. Both static and post liquefaction tests were performed. Pile stiffness was observed to decrease after detonation of the explosives due to an increase in pore water pressure. As the pore pressures decreased, pile stiffness increased. The effect of pore-pressure on foundation stiffness is discussed in more detail in Chapters 7 and 8. It was observed that the 0.9-m CISS pile was more stiff during the static test while the 9-pile group was more stiff during the liquefaction testing.

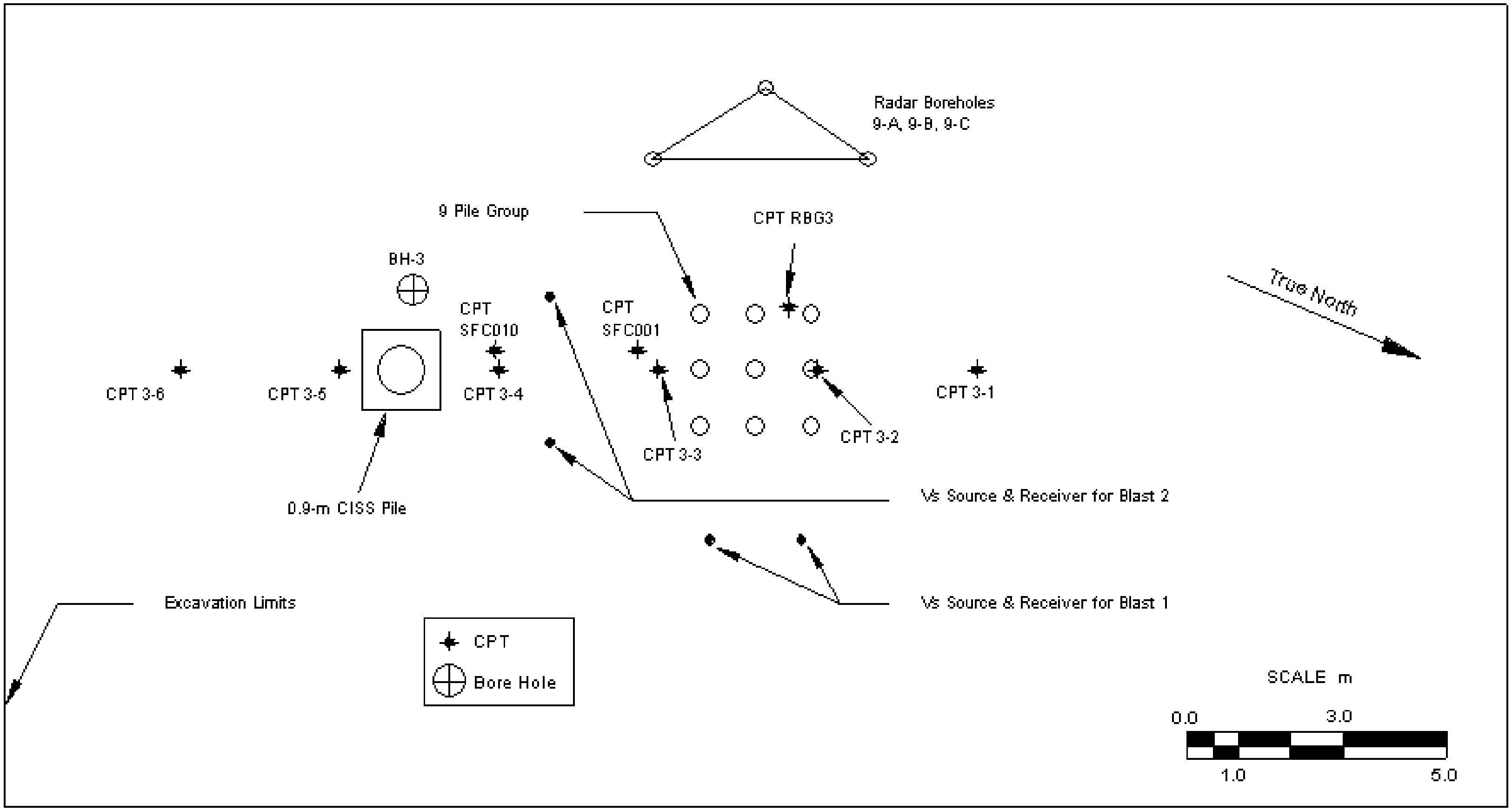


Figure 5.1 Location Map of In-Situ Tests for 9-Pile Group/0.9m CISS Pile Test Area

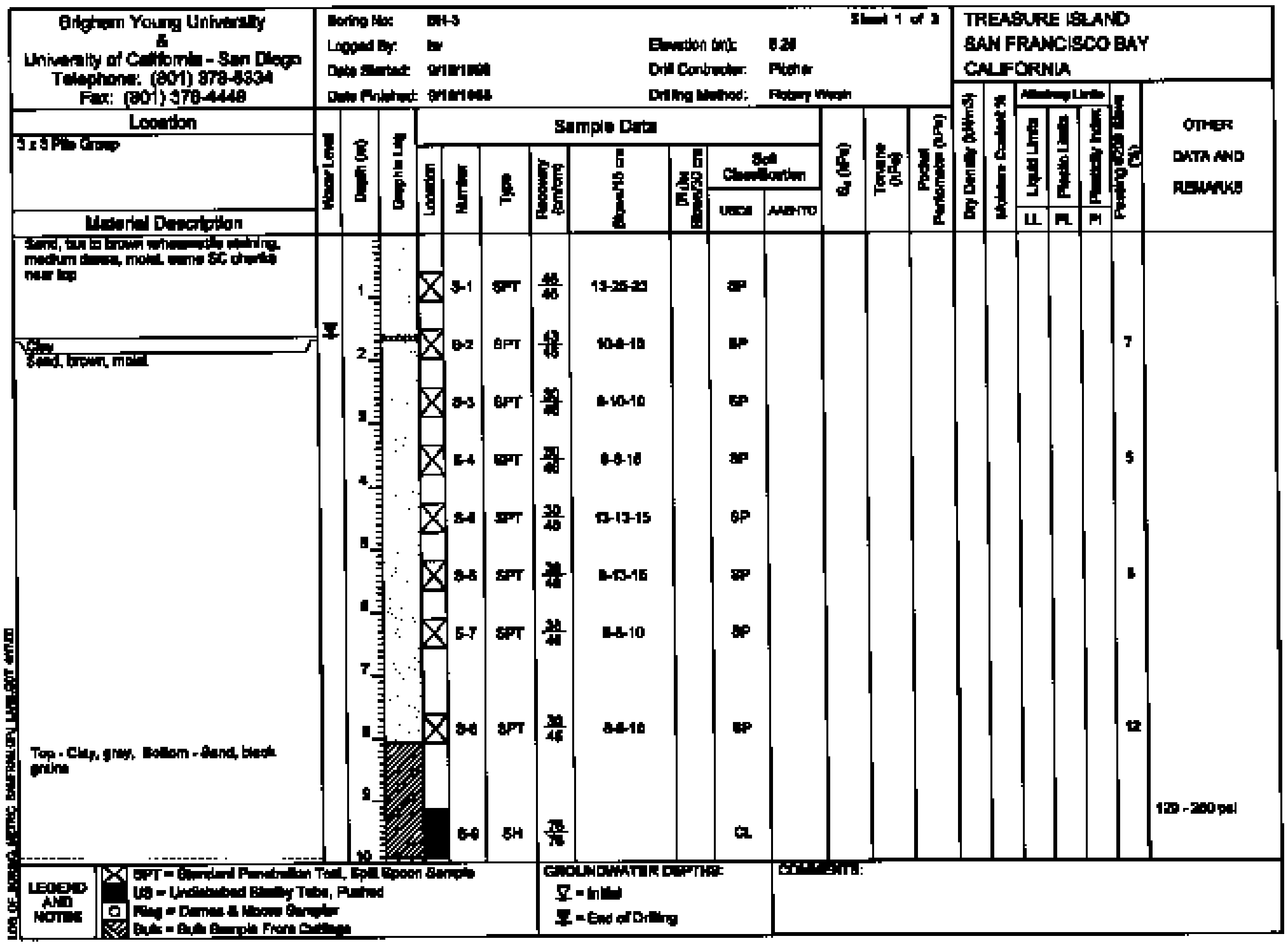


Figure 5.2a Soil Boring Log for the 9-Pile Group/0.9m CISS Pile Test Area

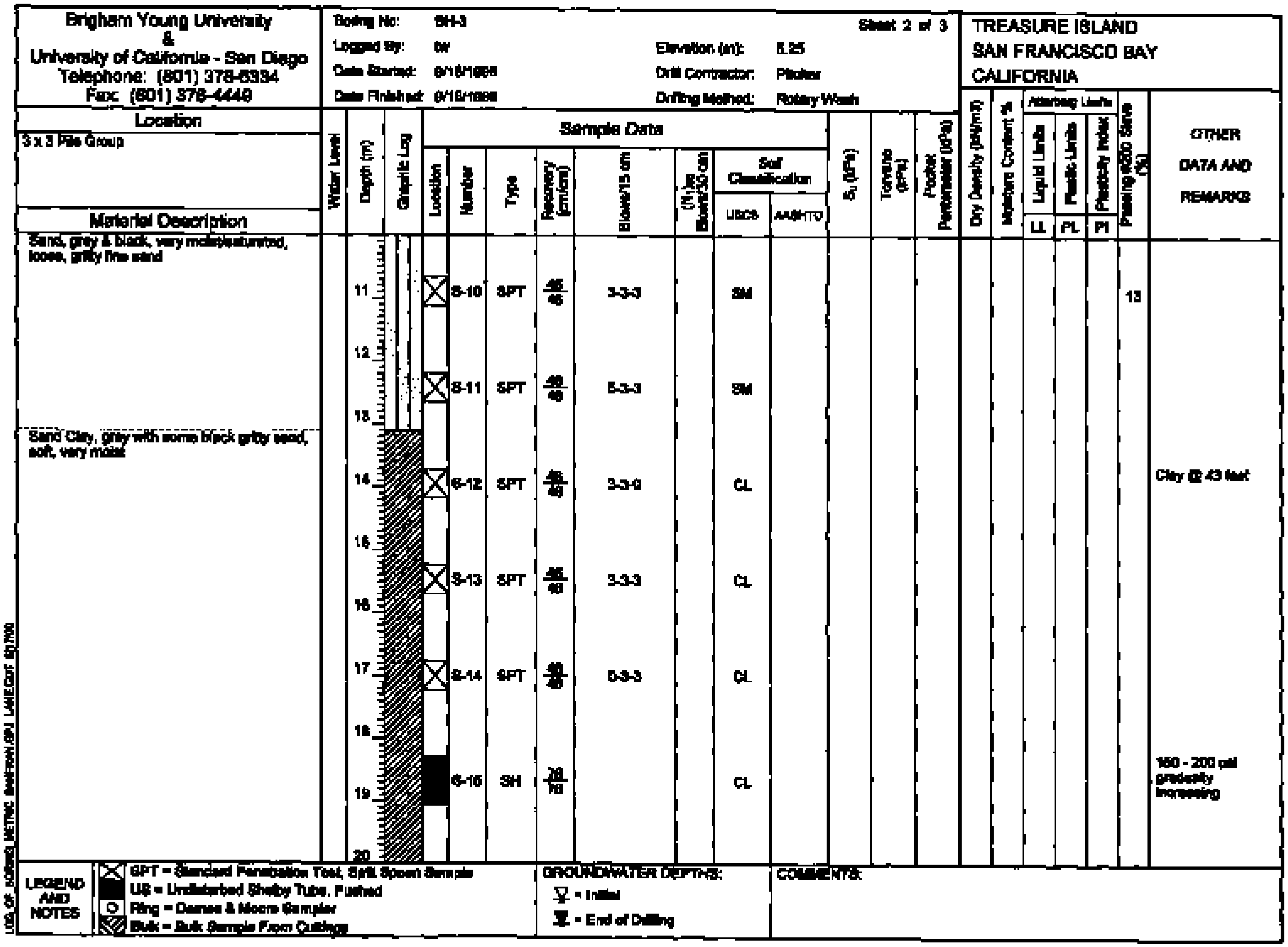


Figure 5.2b Soil Boring Log for the 9-Pile Group/0.9m CISS Pile Test Area

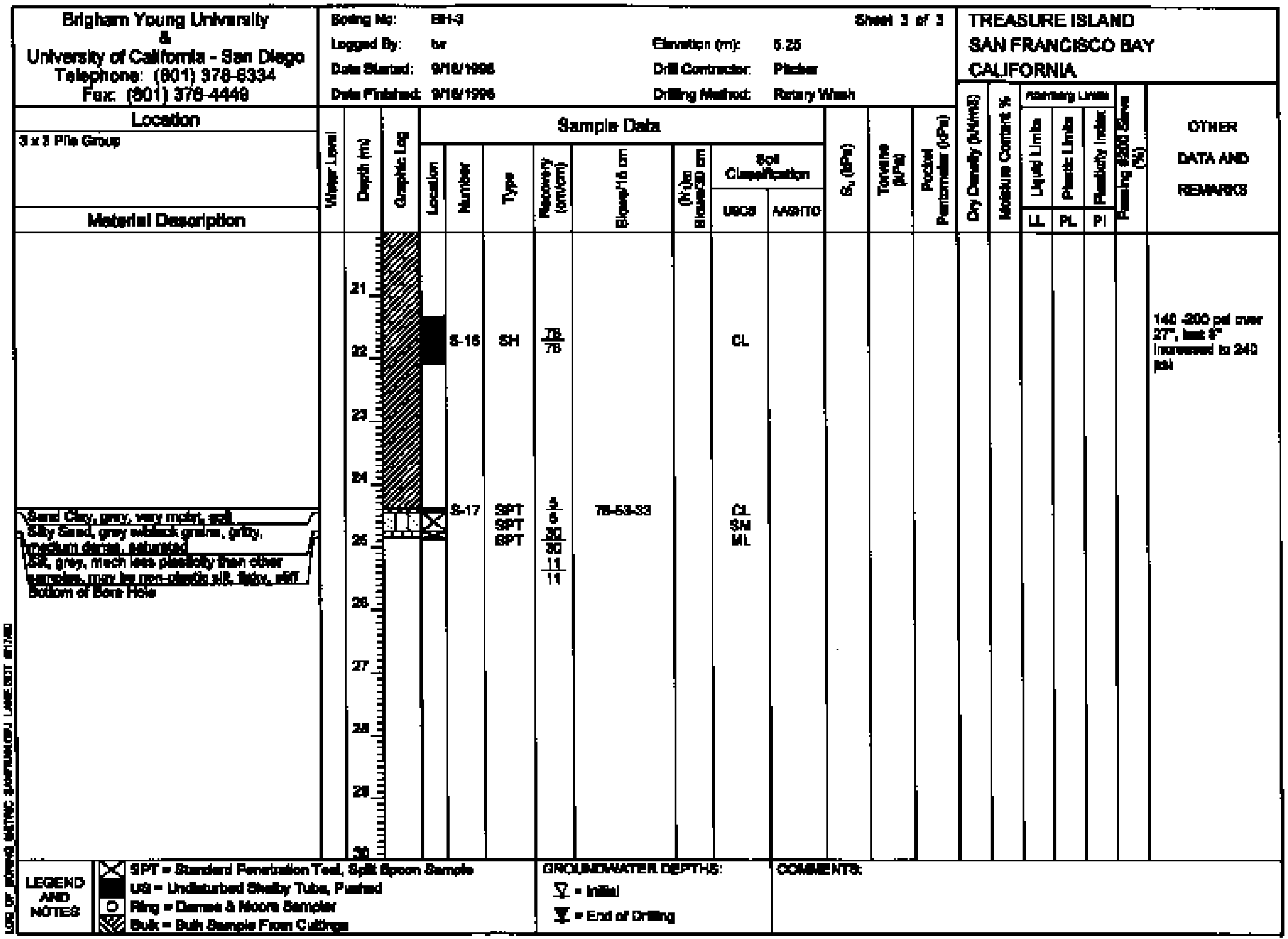


Figure 5.2c Soil Boring Log for the 9-Pile Group/0.9m CISS Pile Test Area

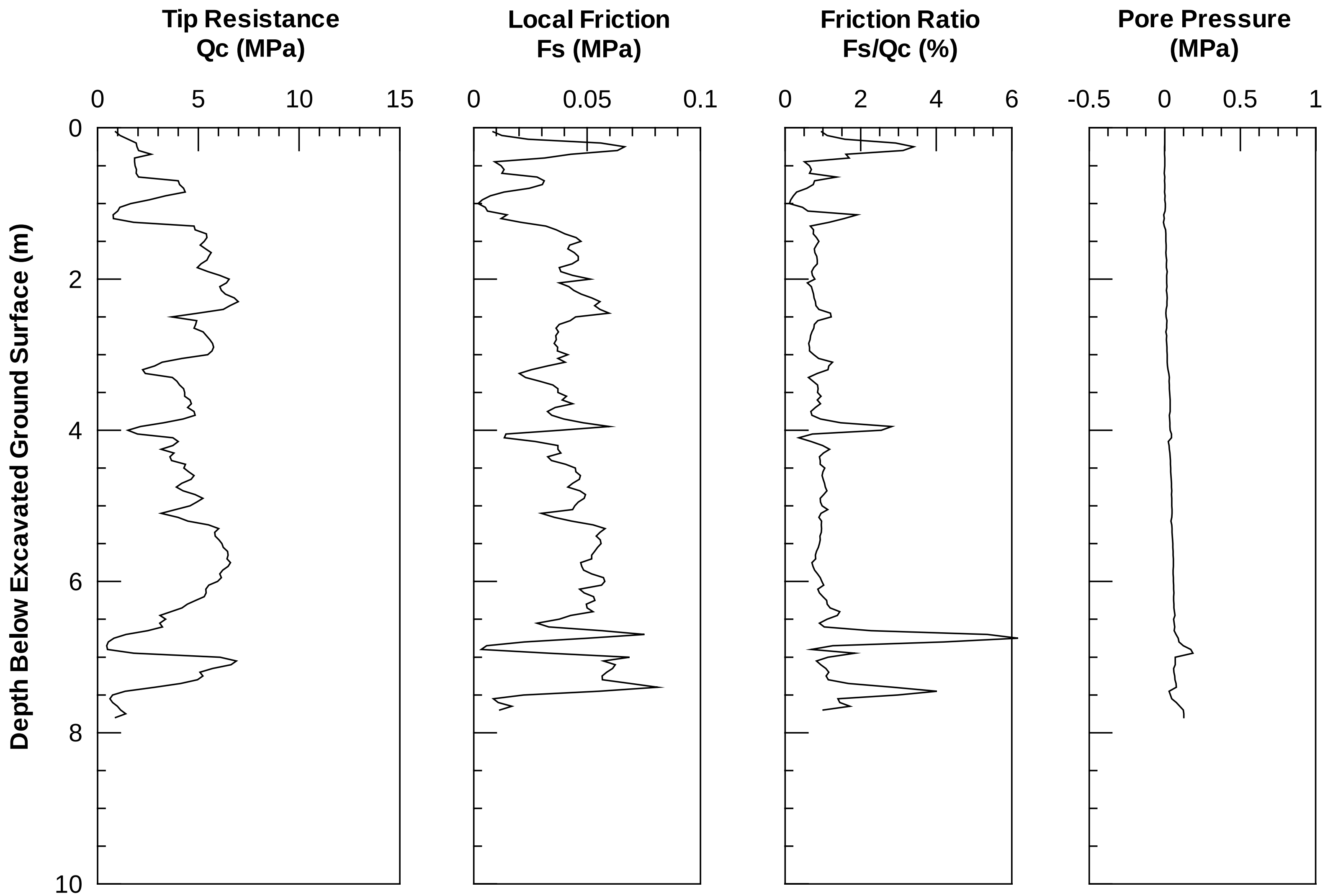


Figure 5.3 CPT3-2 Logs for 9-Pile Group/0.9m CISS Pile Test Area (10-30-98)

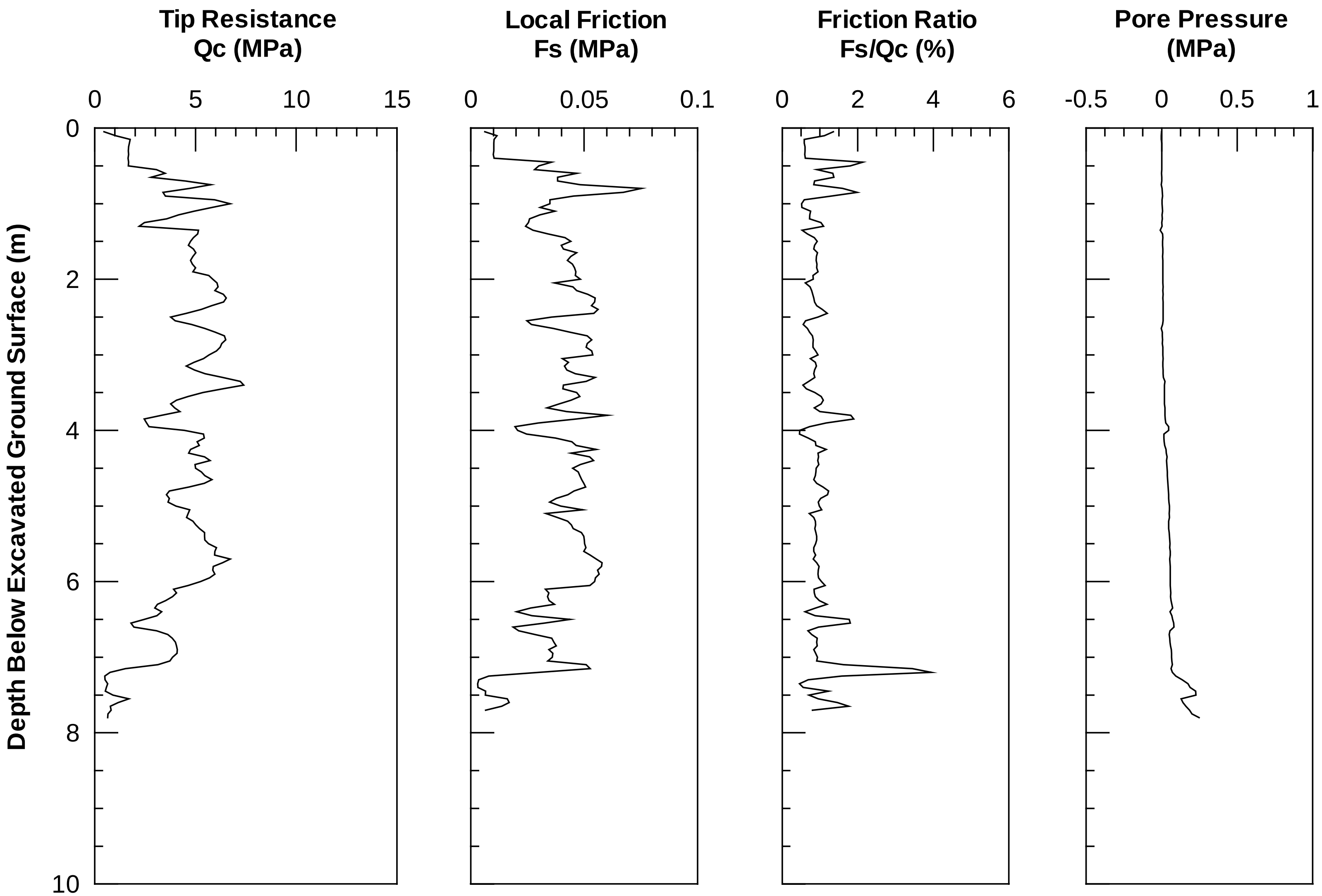


Figure 5.4 CPT3-3 Logs for 9-Pile Group/0.9m CISS Pile Test Area (10-30-98)

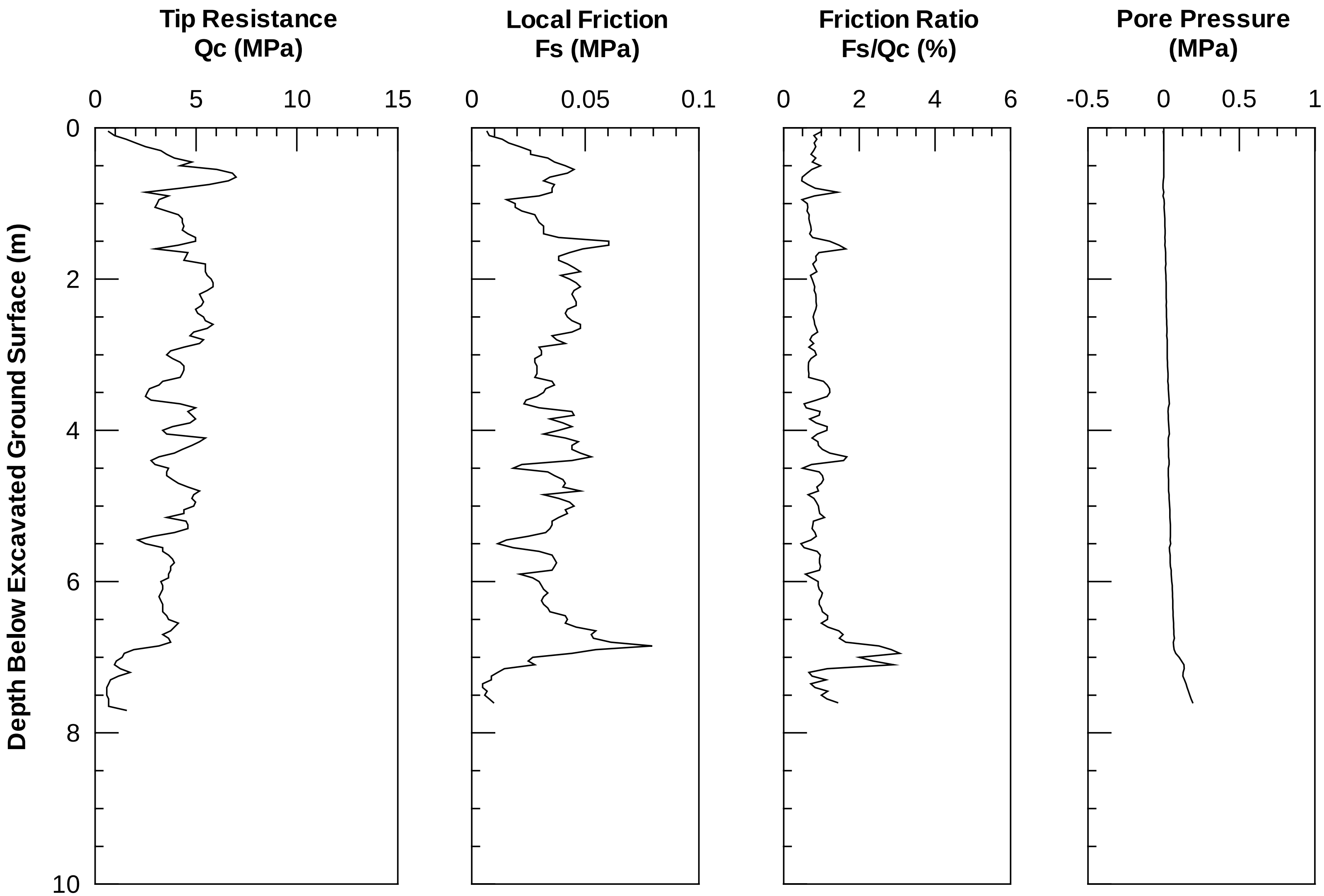


Figure 5.5 CPT3-4 Logs for 9-Pile Group/0.9m CISS Pile Test Area (10-30-98)

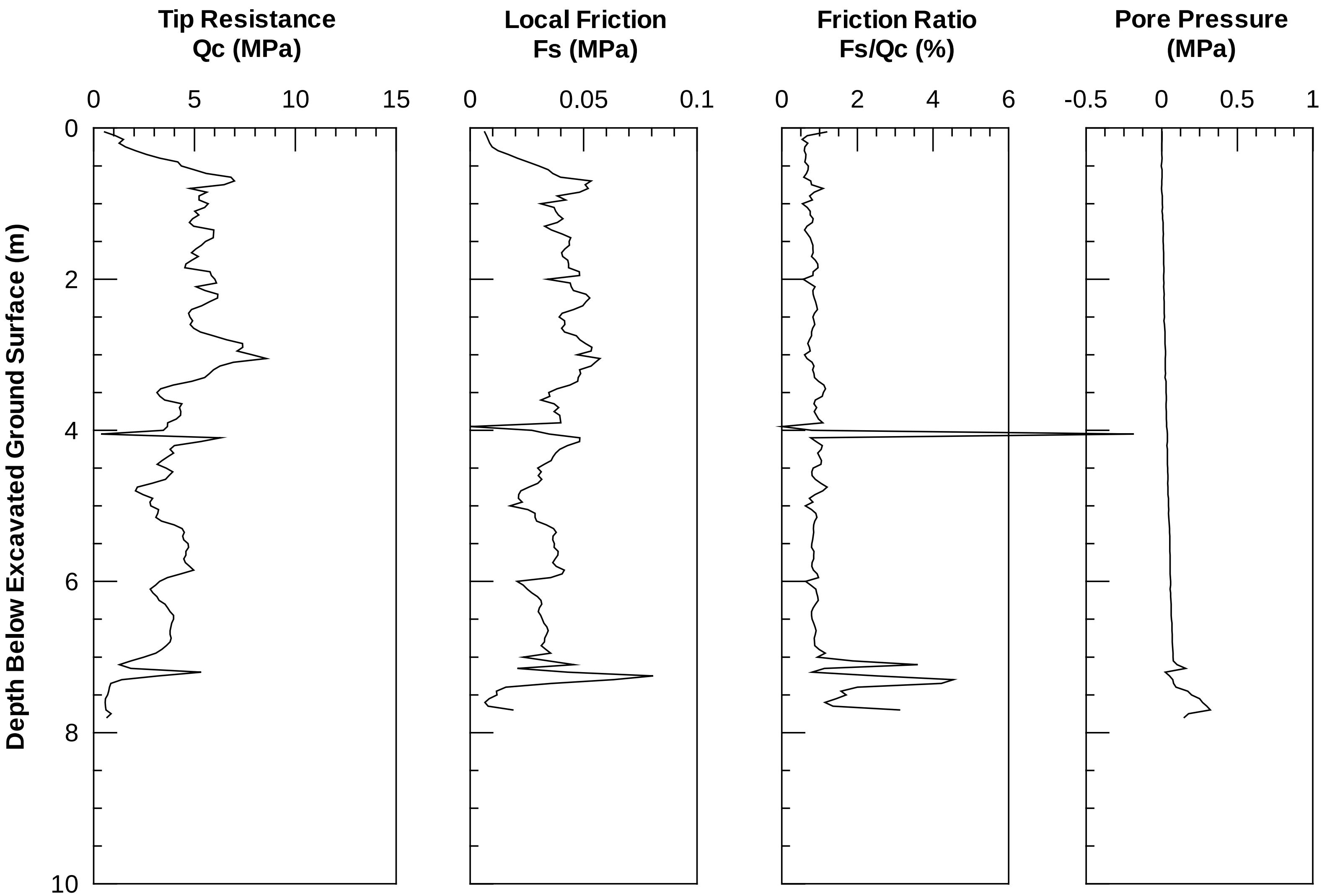


Figure 5.6 CPT3-5 Logs for 9-Pile Group/0.9m CISS Pile Test Area (10-30-98)

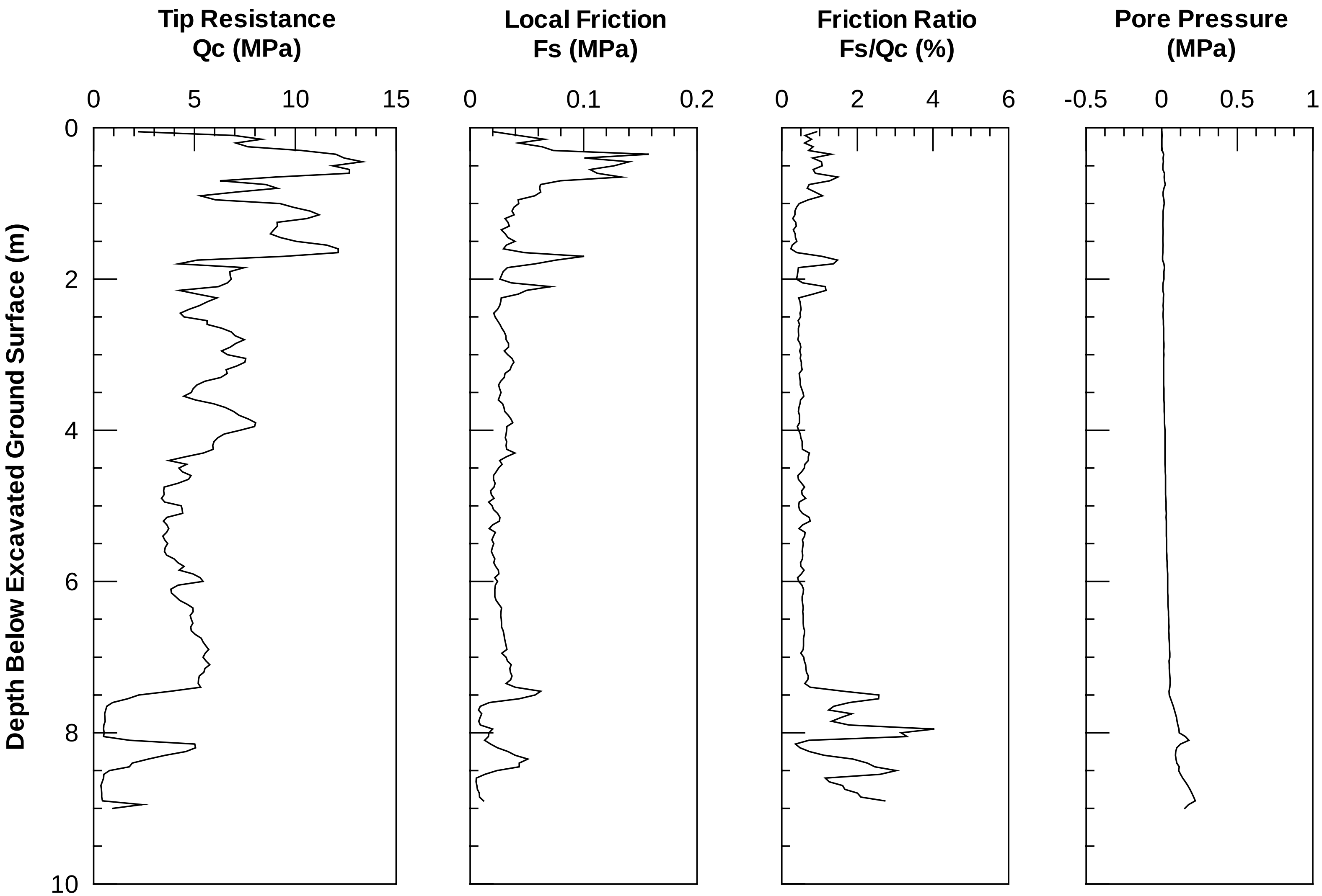


Figure 5.7 CPT3-6 Logs for 9-Pile Group/0.9m CISS Pile Test Area (10-30-98)

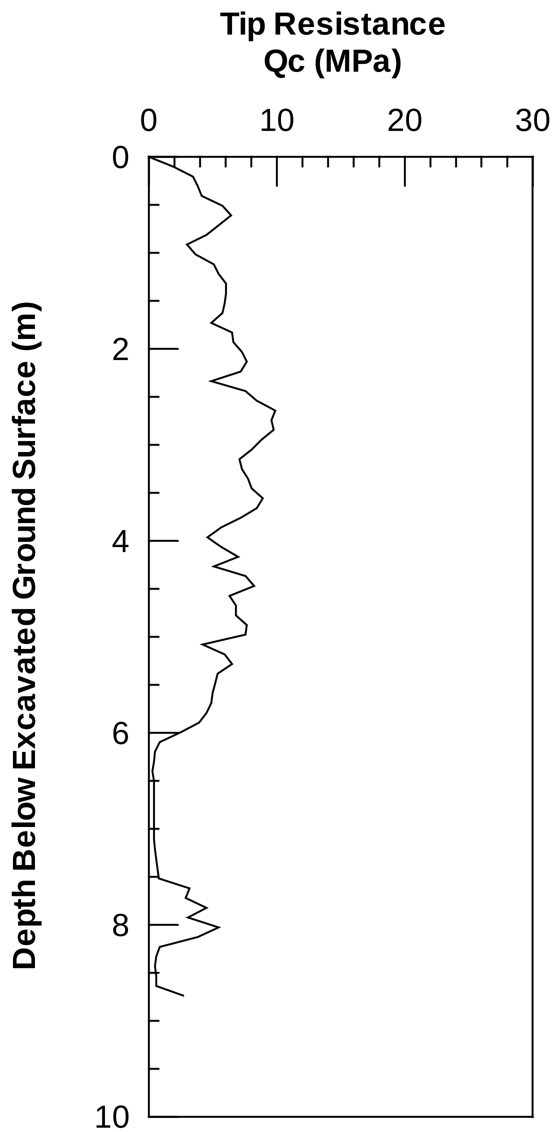


Figure 5.8 CPT RBG3 Logs for 9-Pile Group/0.9m CISS Pile Test Area (1-30-99)

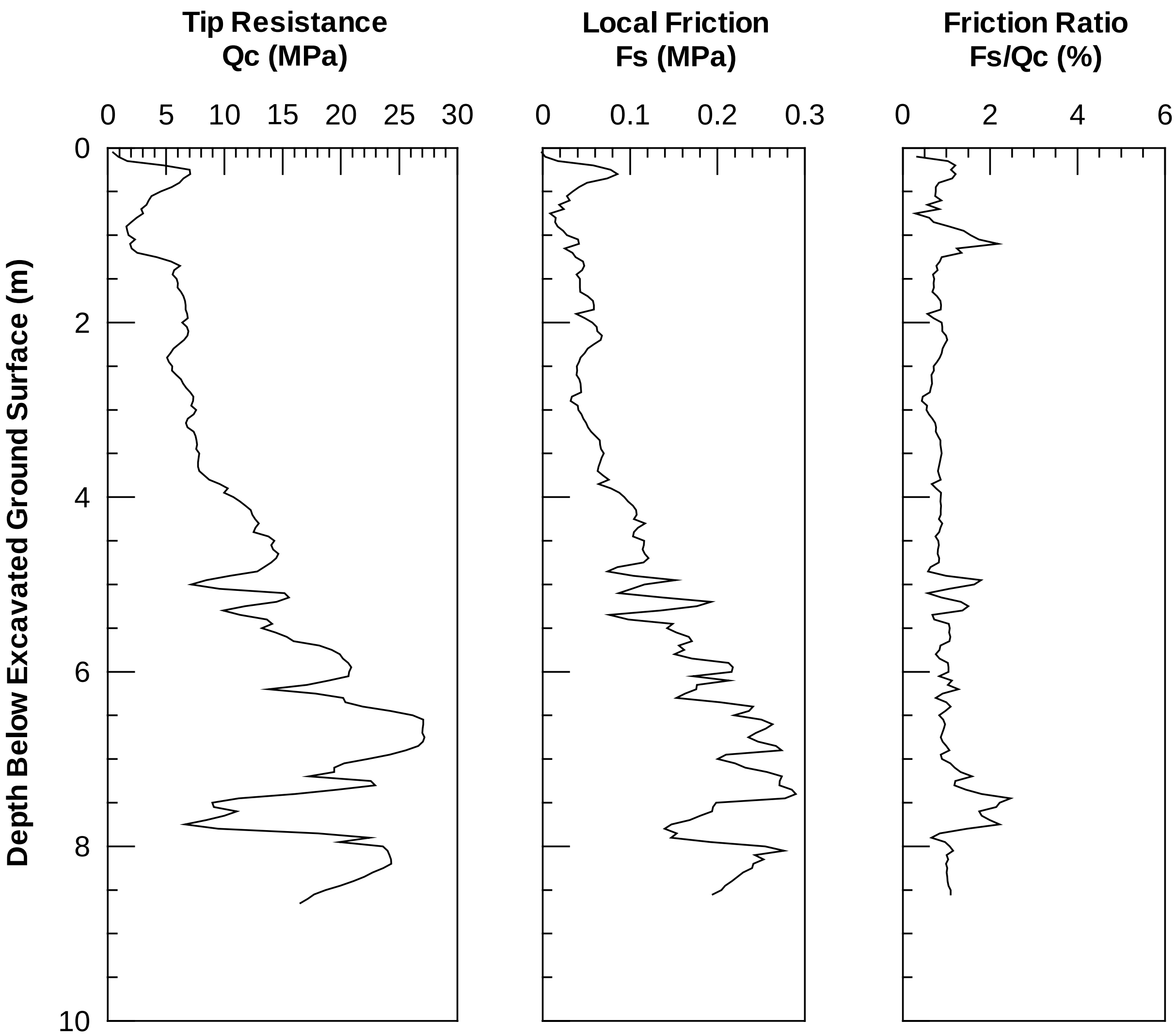


Figure 5.9 CPT SFC001 Logs for 9-Pile Group/0.9m CISS Pile Test Area (6-24-99)

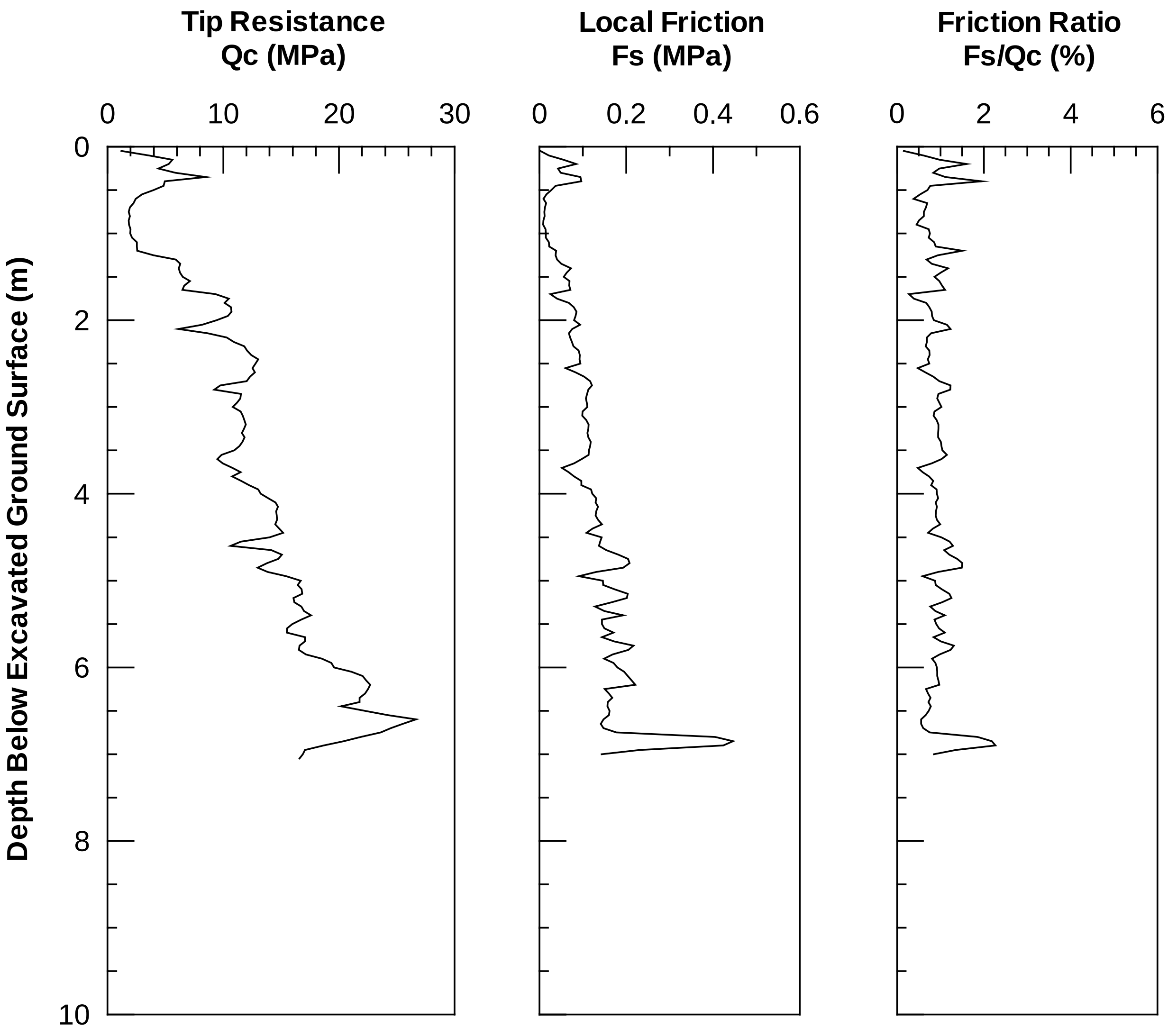


Figure 5.10 CPT SFC010 Logs for 9-Pile Group/0.9m CISS Pile Test Area (6-24-99)

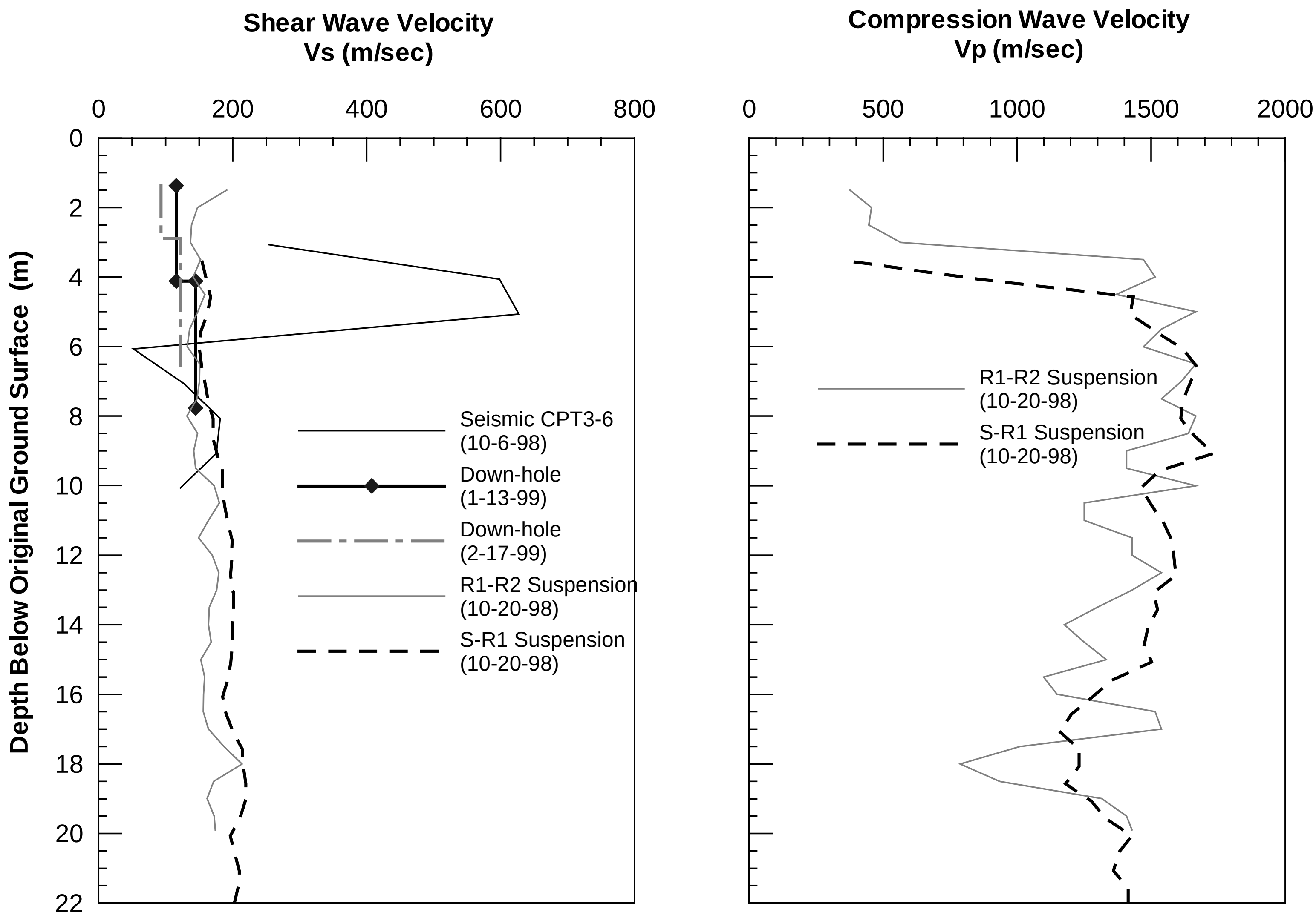


Figure 5.11 9-Pile Group/0.9m CISS Pile Site Pre-blast Velocities a) Shear Wave b) Compression Wave

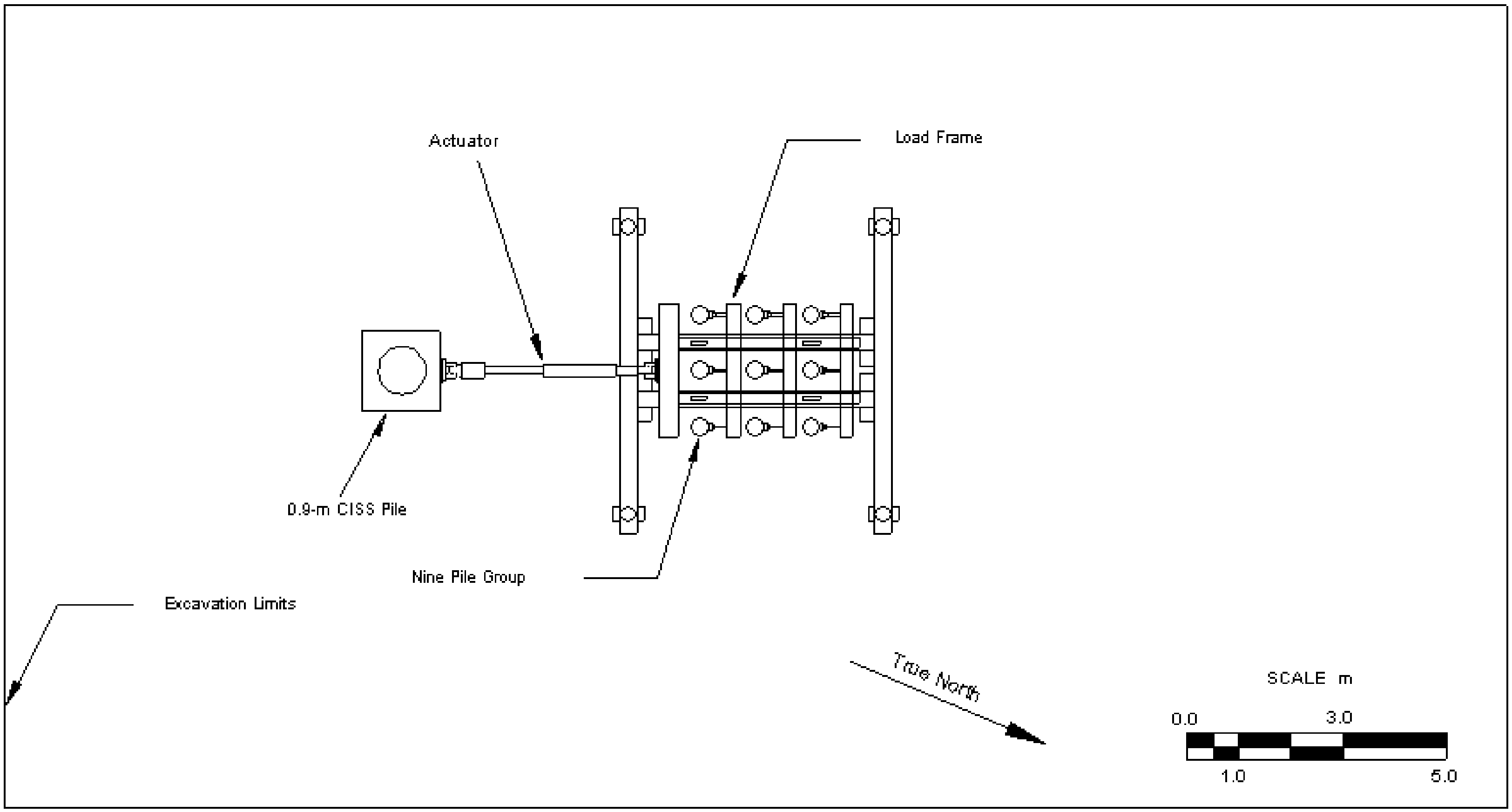


Figure 5.12 Plan View of 9-Pile Group/0.9m CISS Pile Test Set-Up

Figure 5.13 Profile of 9-Pile/0.9-m CISS Pile Test Set-Up

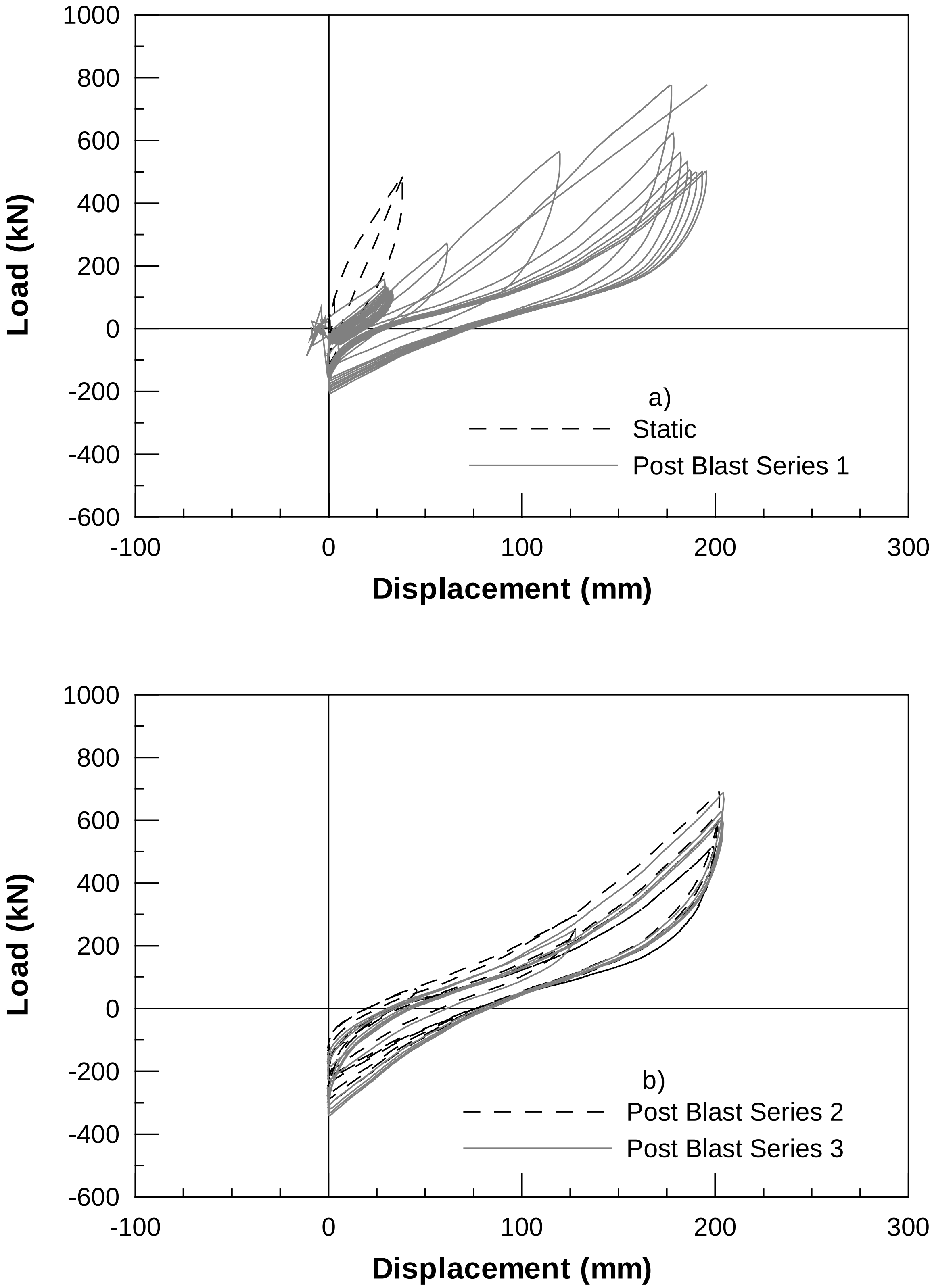


Figure 5.14 Load vs. Displacement for 9-Pile Group a) Static and Post Blast Series 1
b) Post Blast Series 2 and 3

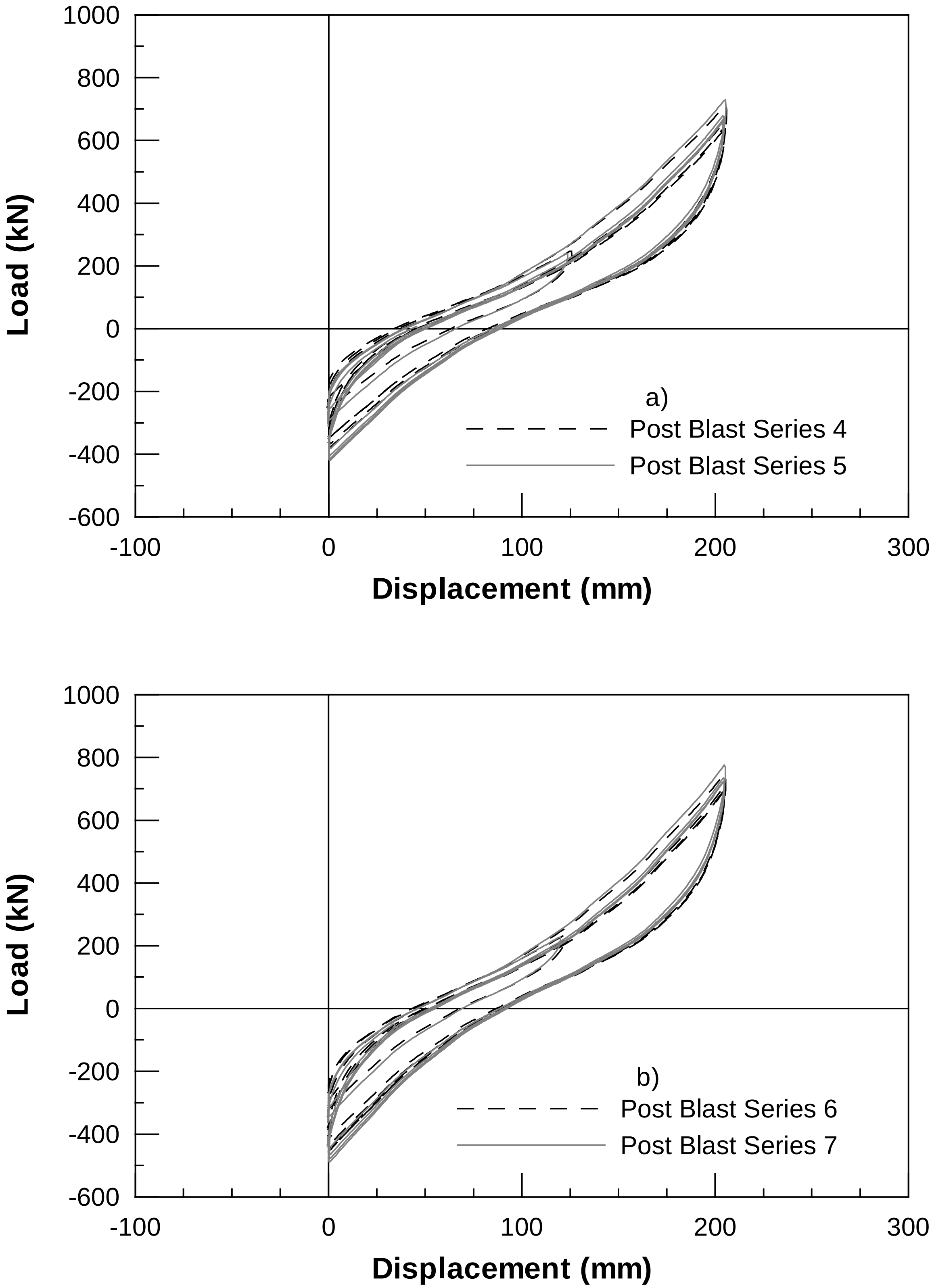


Figure 5.15 Load vs. Displacement for 9-Pile Group a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

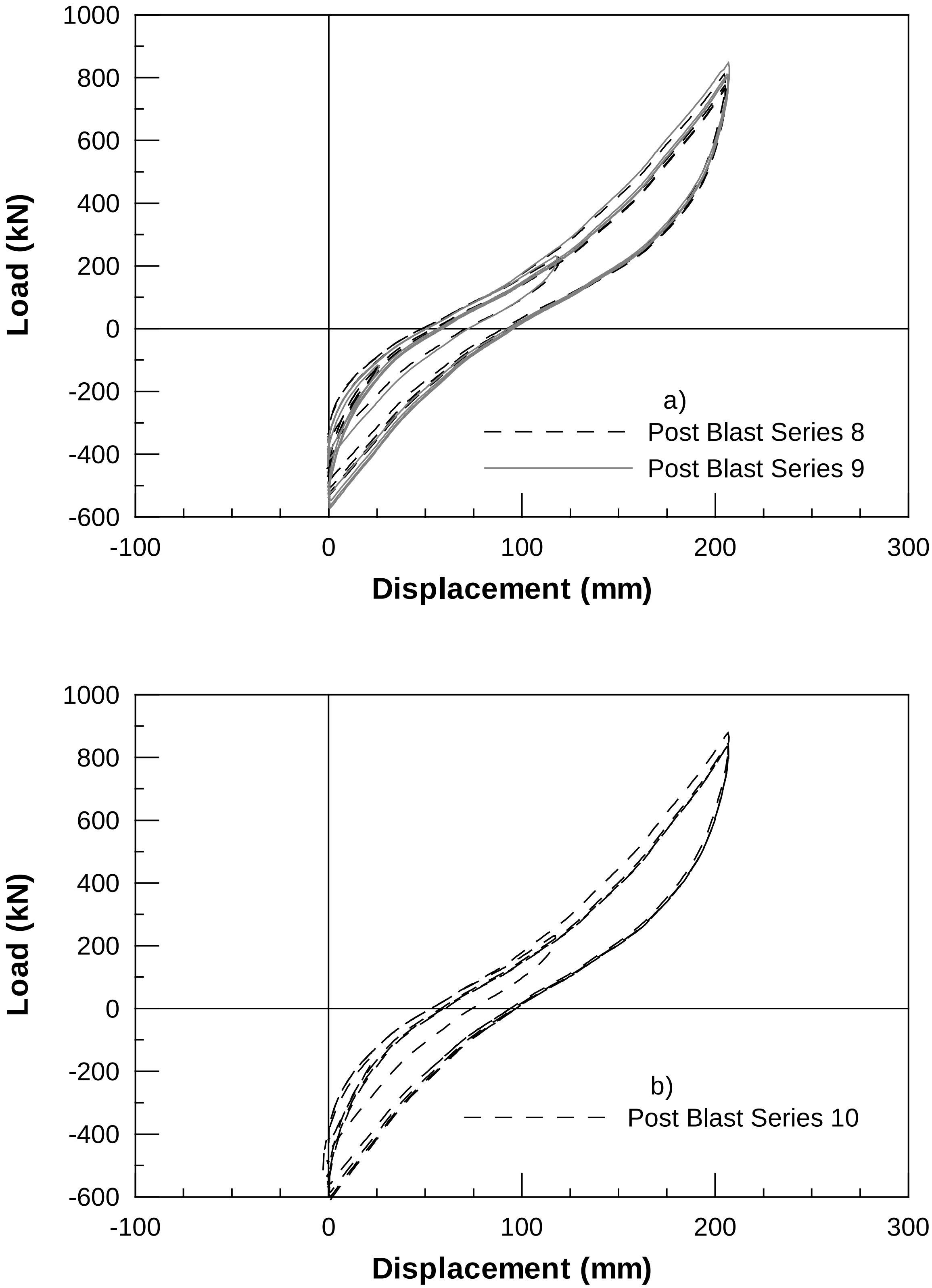
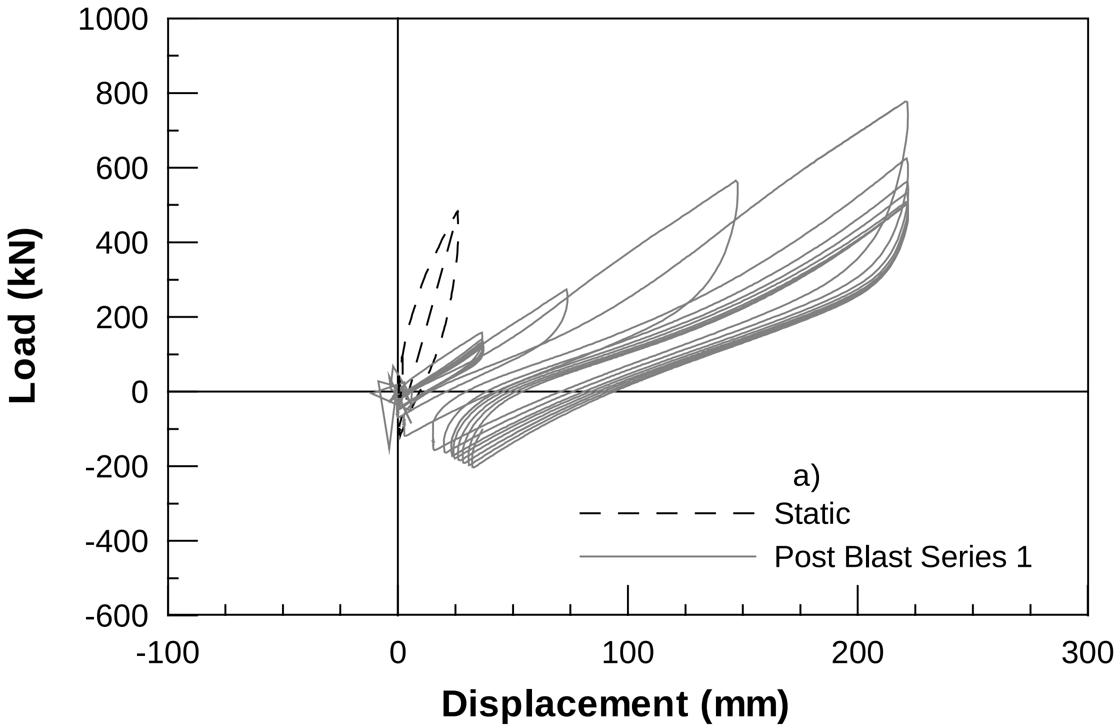


Figure 5.16 Load vs. Displacement for 9-Pile Group a) Post Blast Series 8 and 9 b) Post Blast Series 10



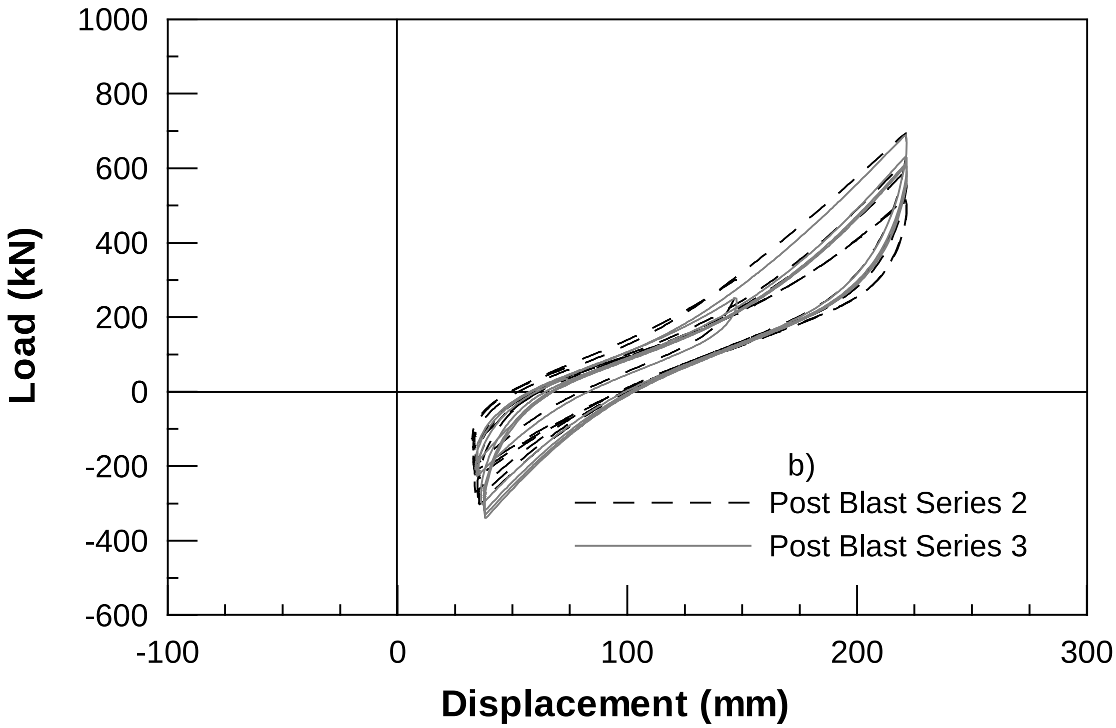


Figure 5.17 Load vs. Displacement for 0.9m CISS Pile a) Static and Post Blast Series 1 b) Post Blast Series 2 and 3

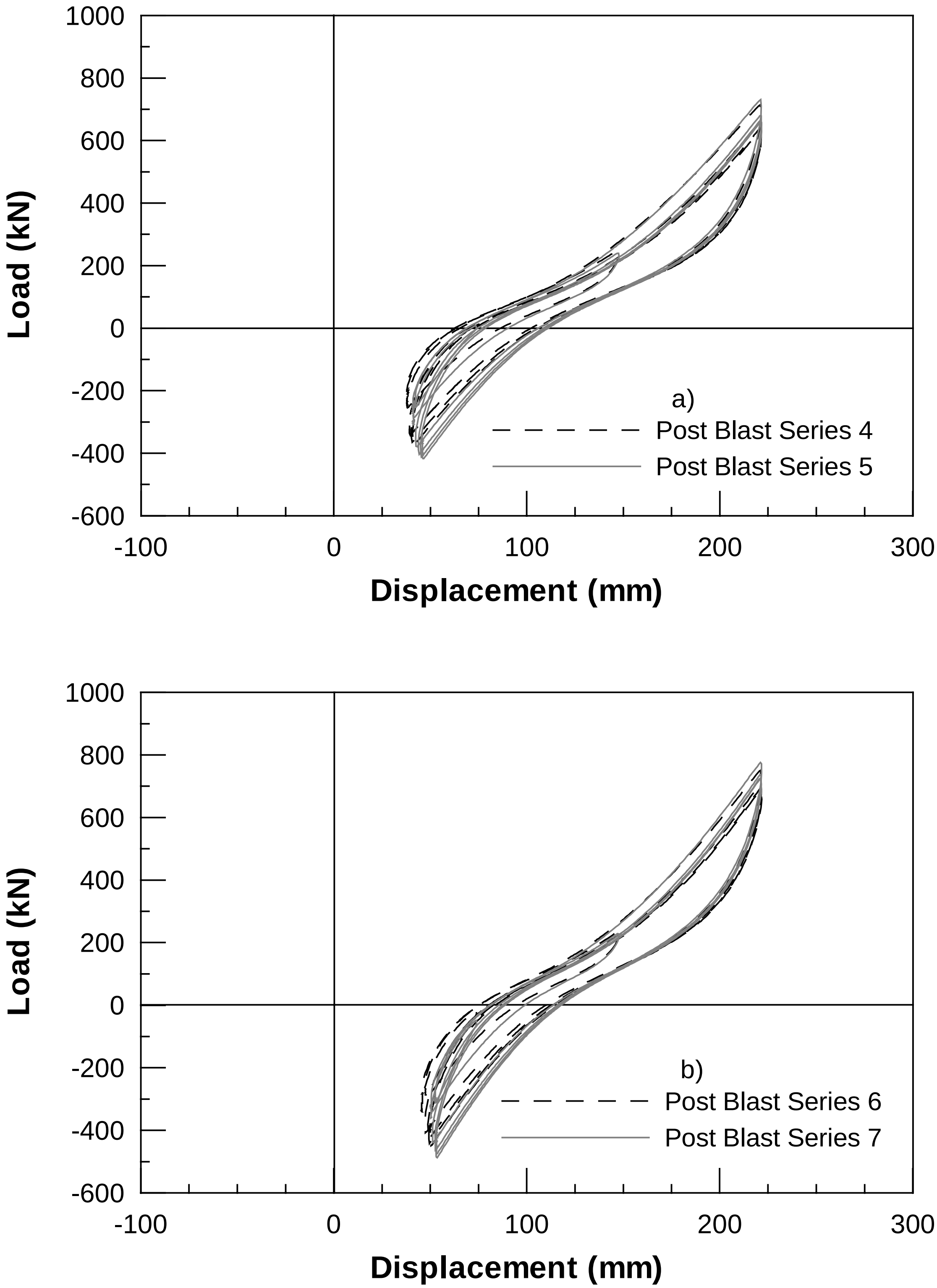


Figure 5.18 Load vs. Displacement for 0.9m CISS Pile a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

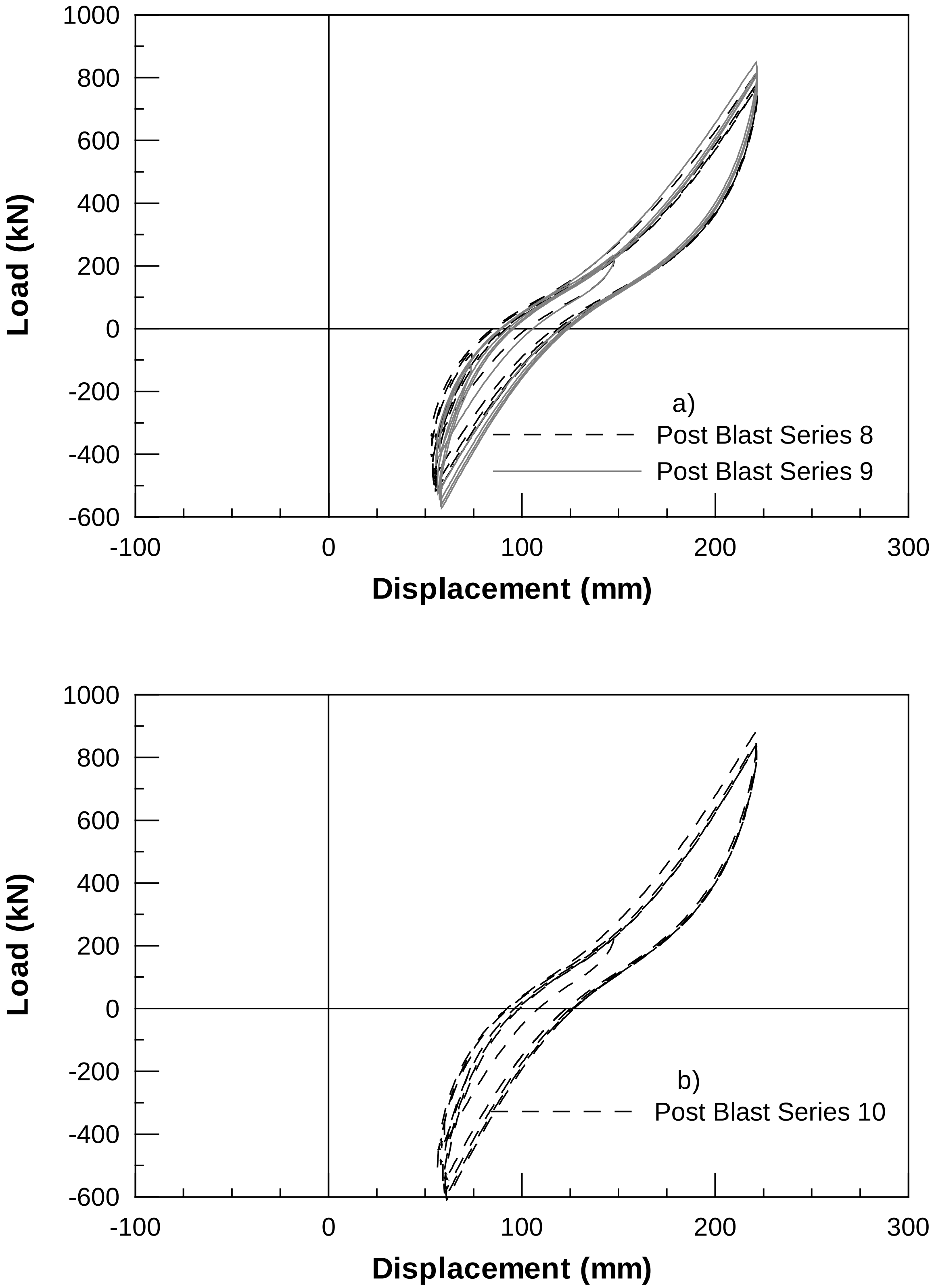


Figure 5.19 Load vs. Displacement for 0.9m CISS Pile a) Post Blast Series 8 and 9 b) Post Blast Series 10

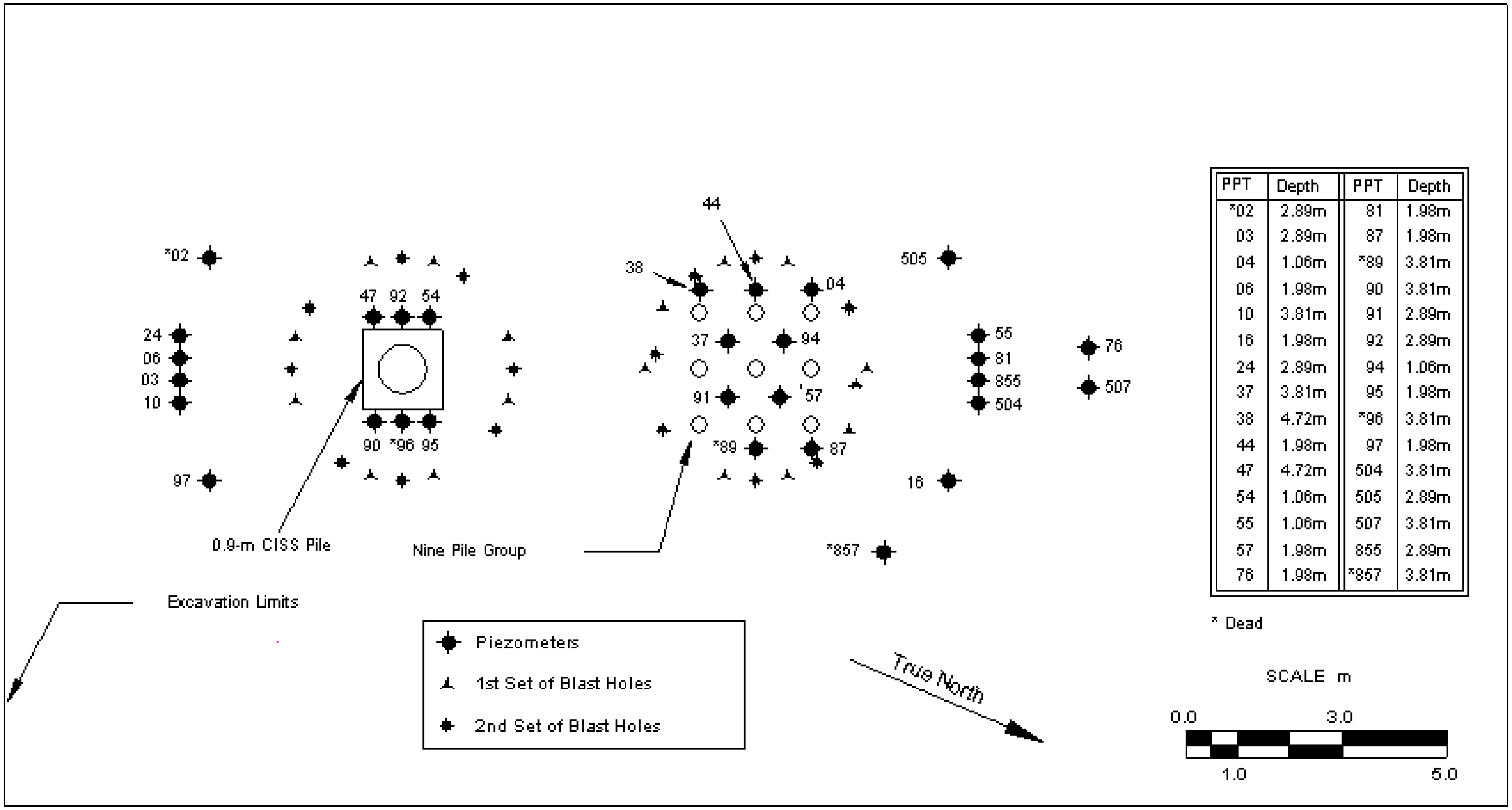


Figure 5.20 Location Map of Pore Pressure Transducers and Explosives for 9-Pile Group/0.9m CISS Pile Test

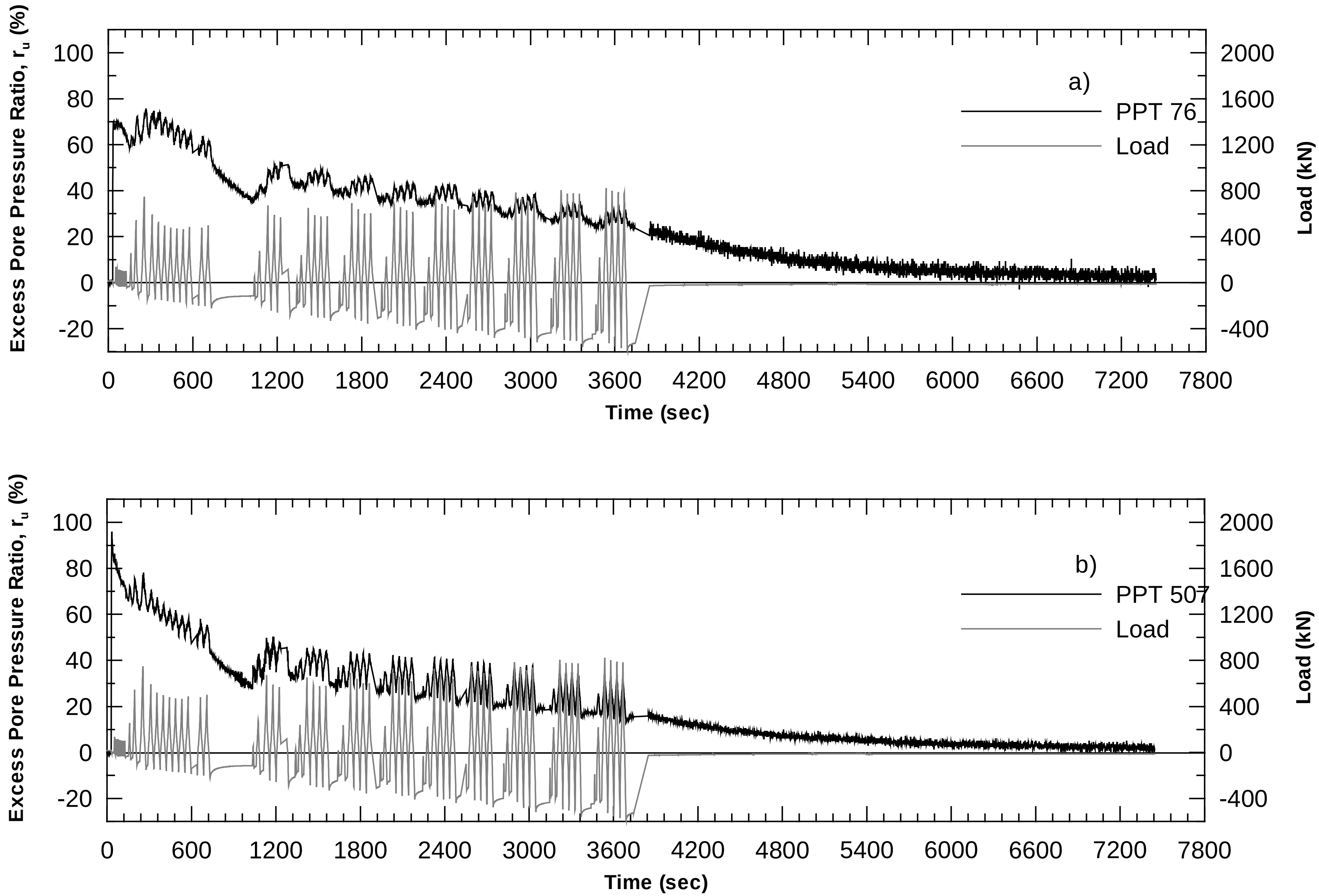


Figure 5.21 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT76 b)PPT507

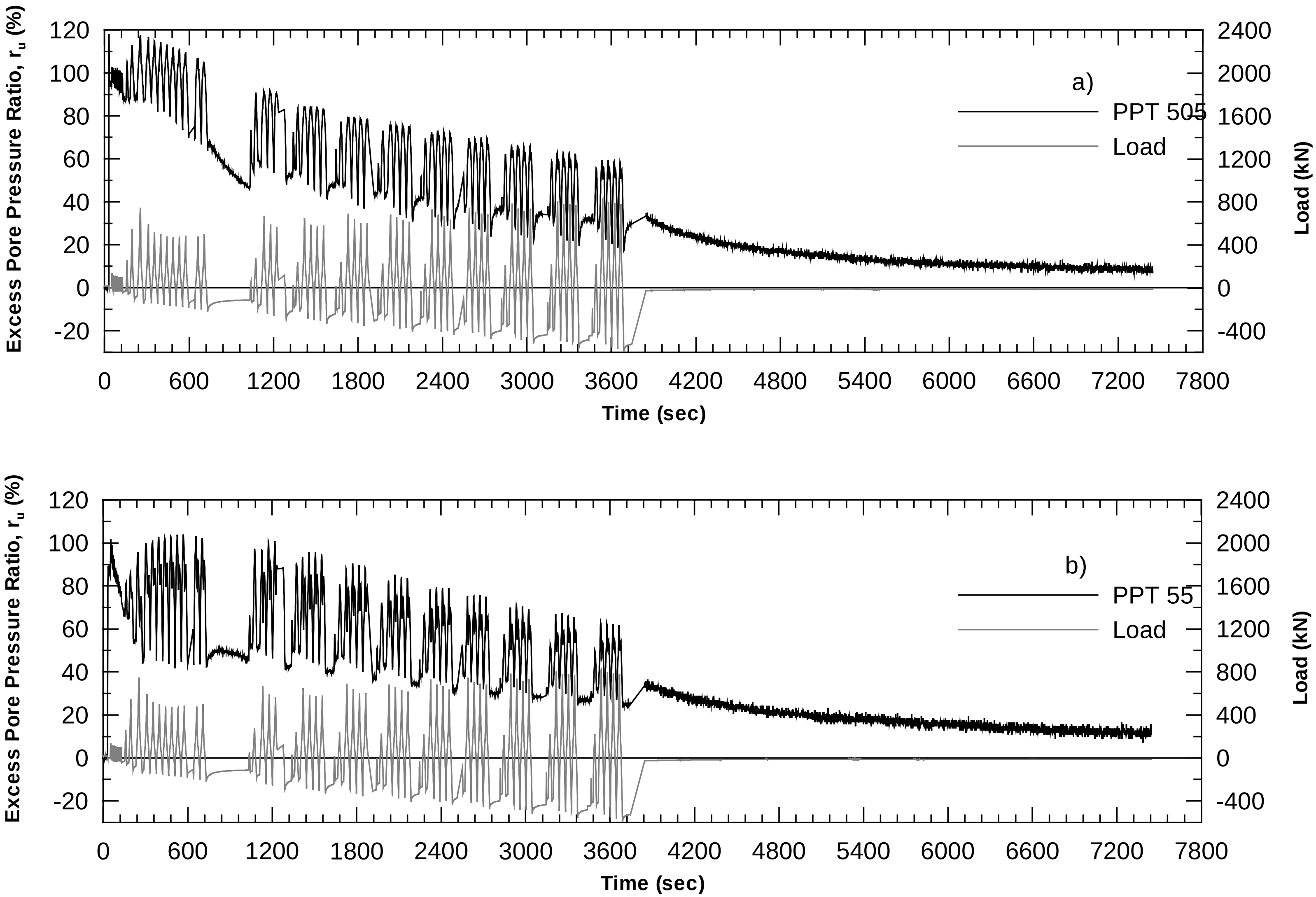


Figure 5.22 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT505 b)PPT55

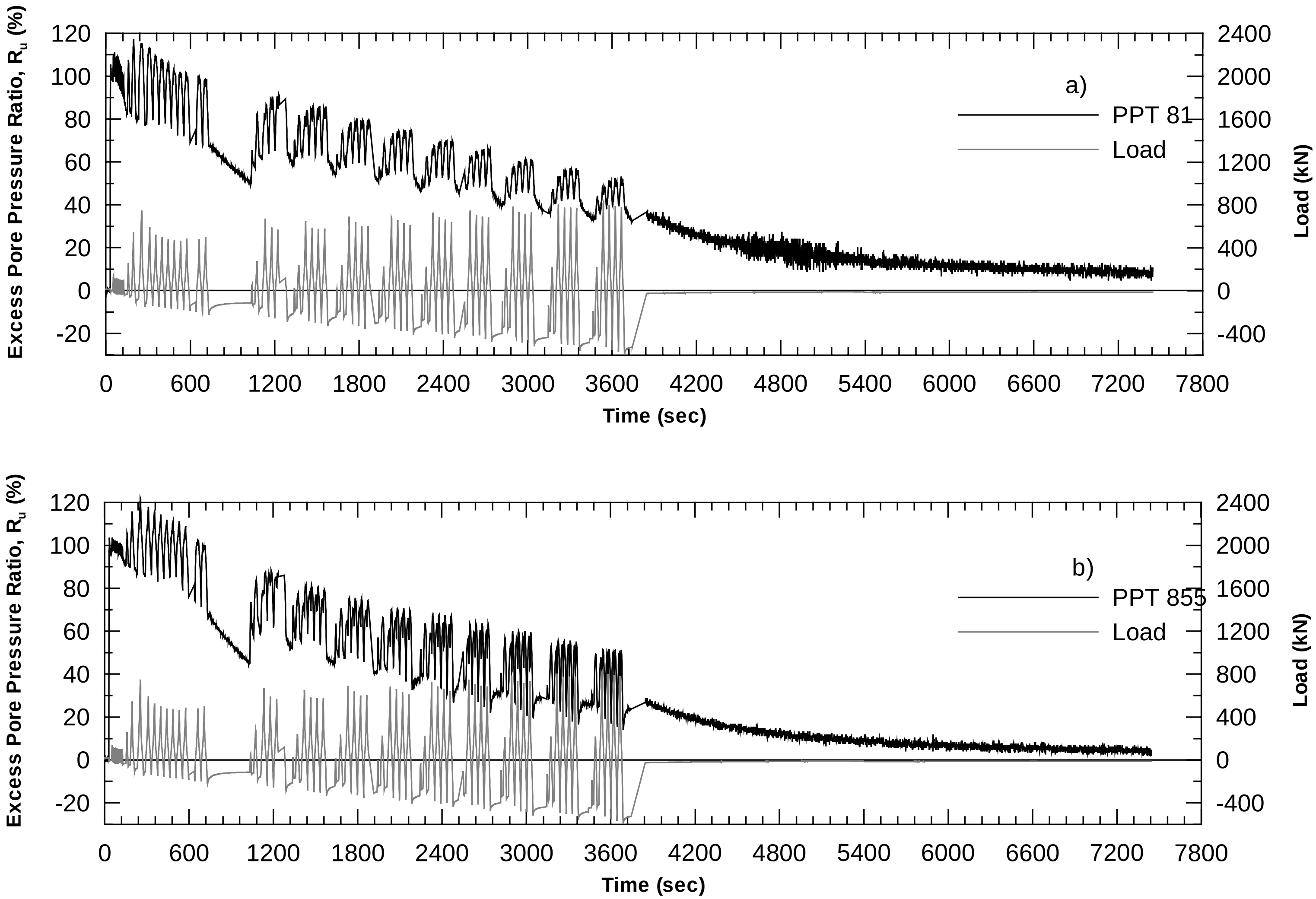


Figure 5.23 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT81 b)PPT855

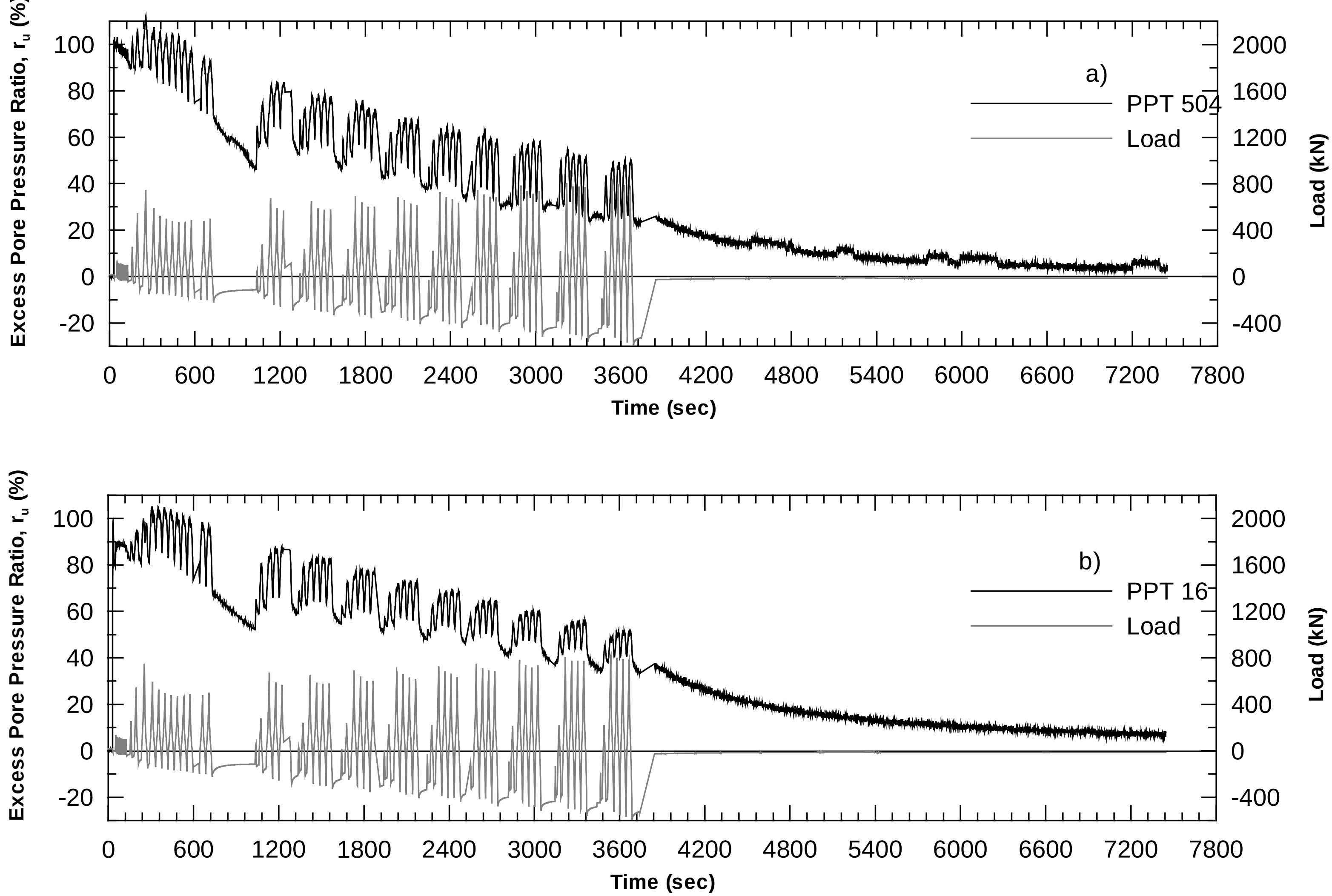


Figure 5.24 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT504 b)PPT16

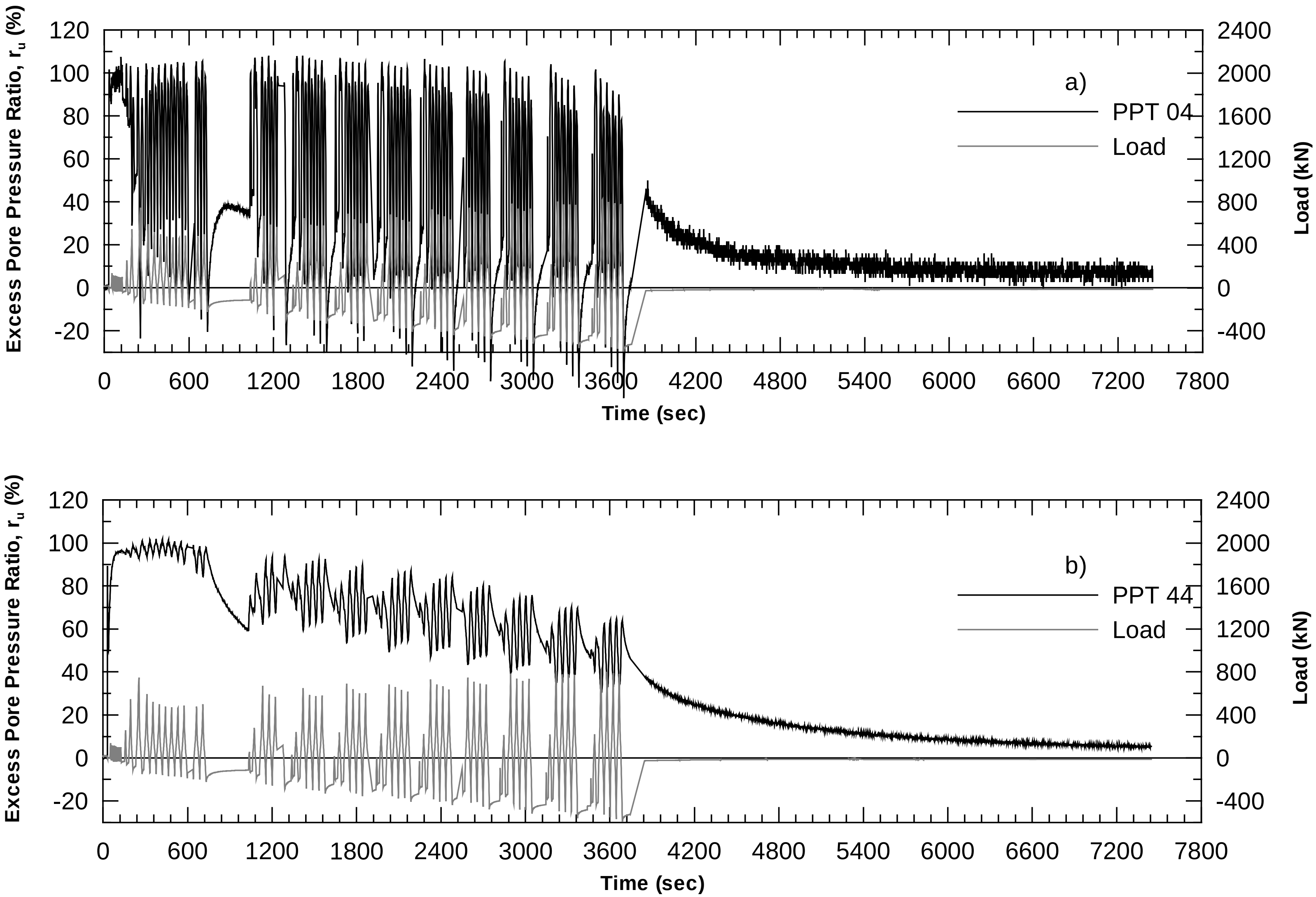


Figure 5.25 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT04 b)PPT44

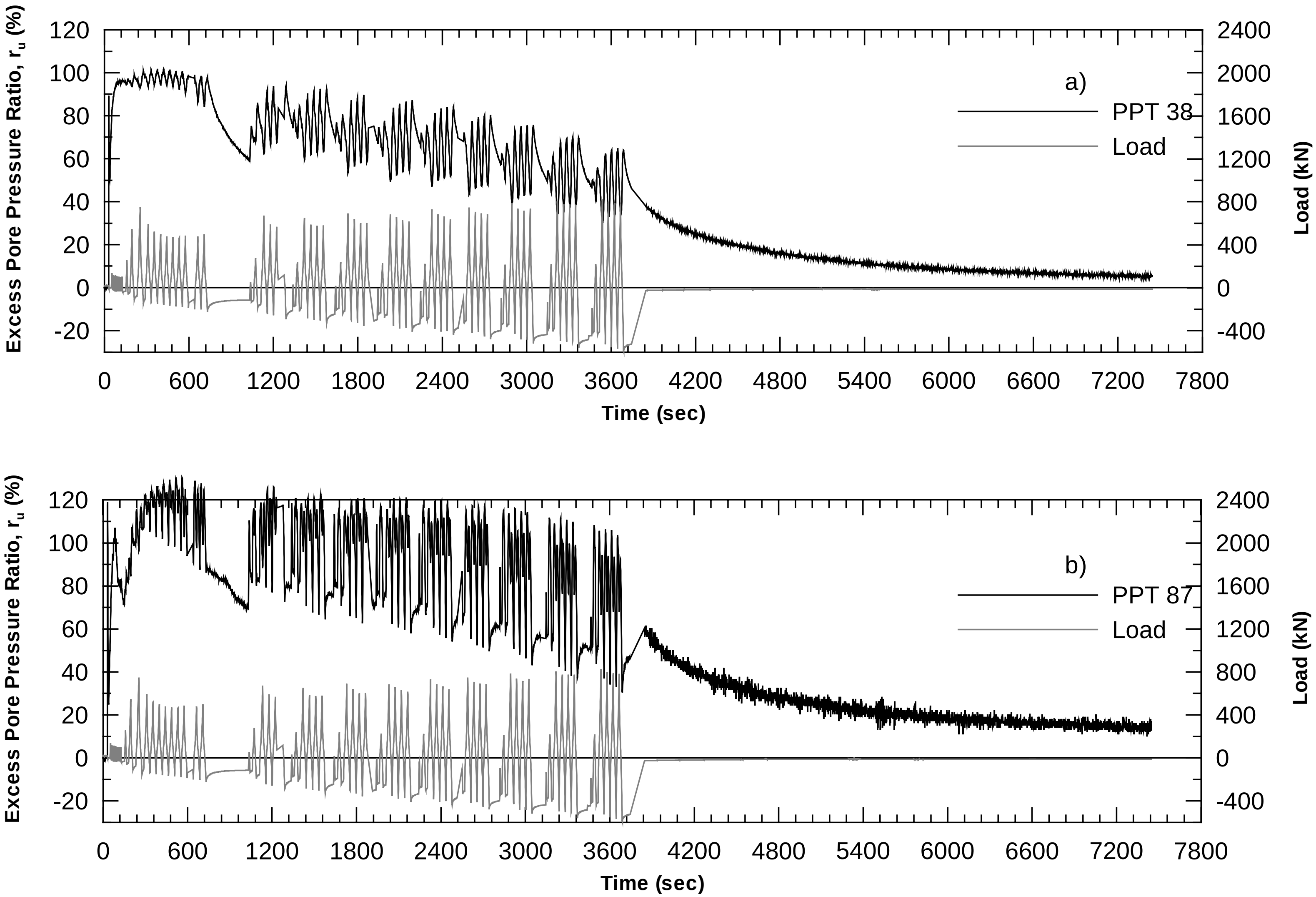


Figure 5.26 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT38 b)PPT87

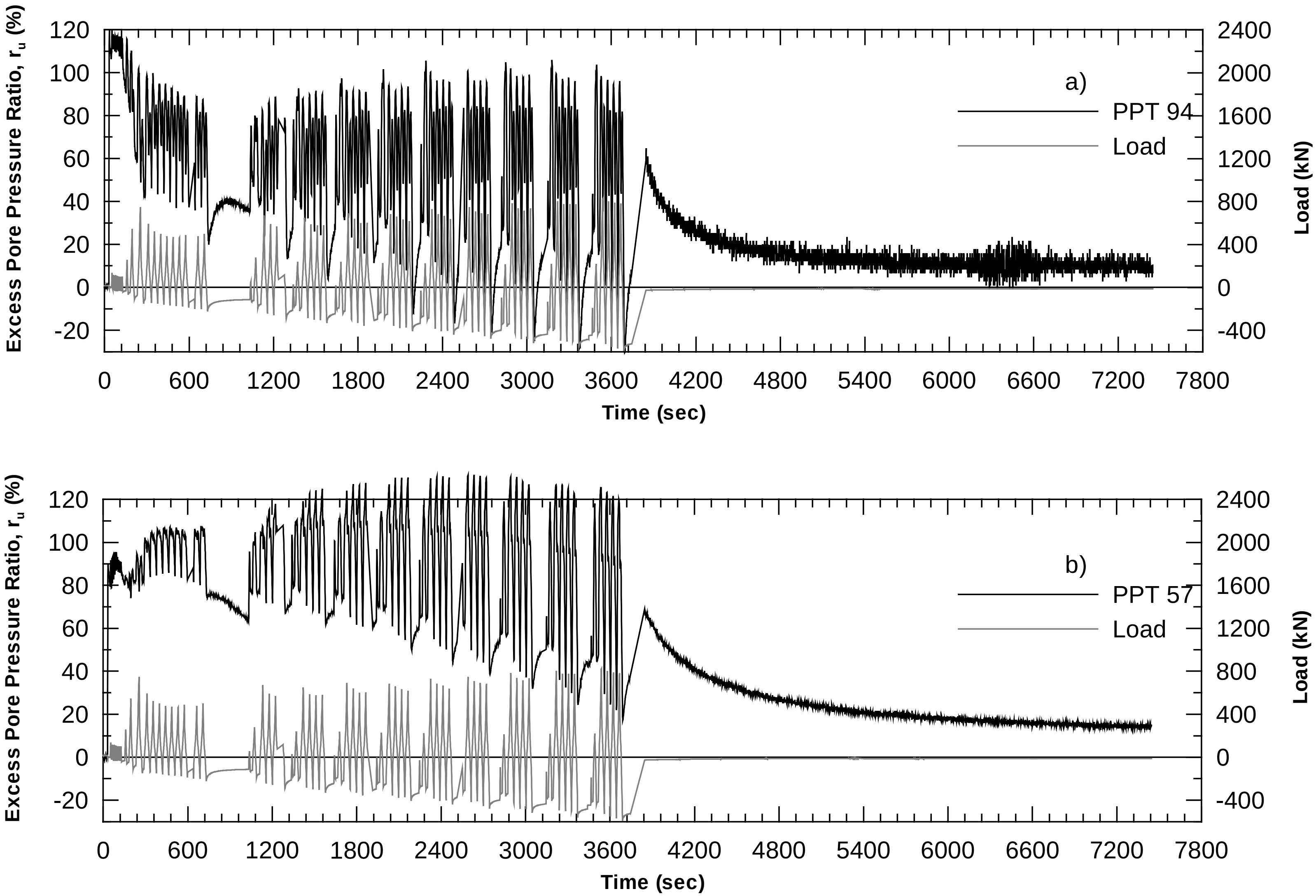


Figure 5.27 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT94 b)PPT57

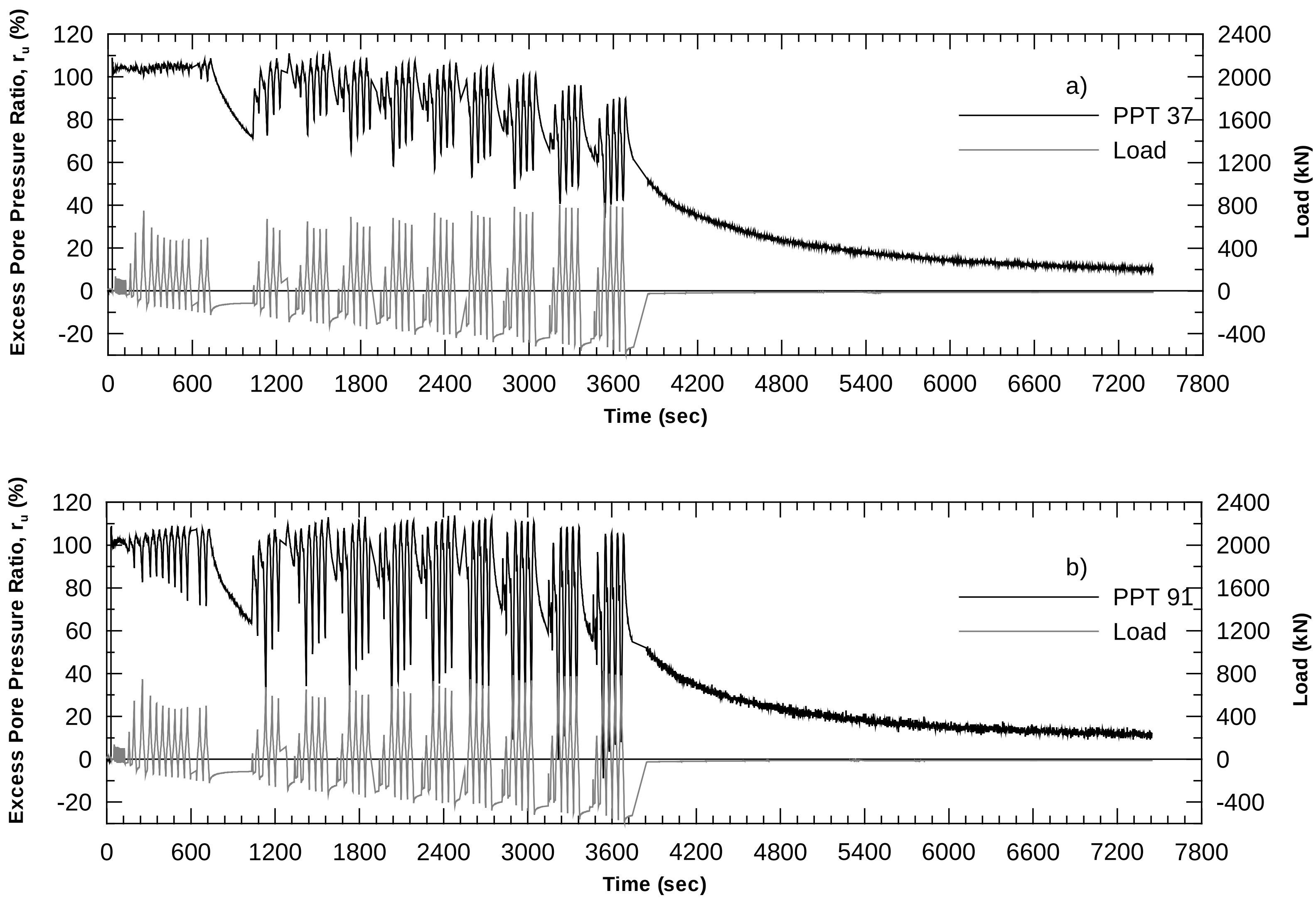


Figure 5.28 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT37 b)PPT91

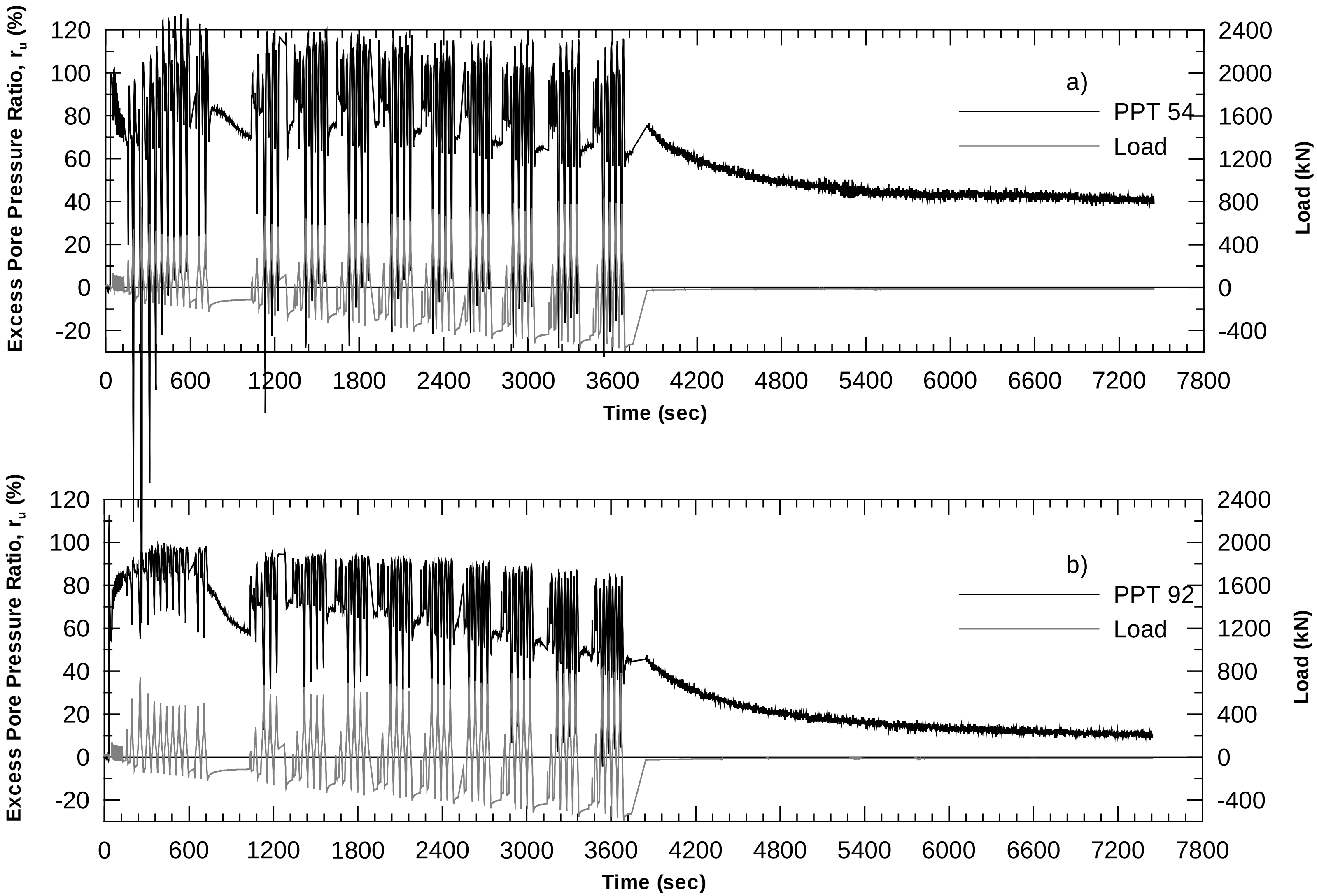


Figure 5.29 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT54 b)PPT92

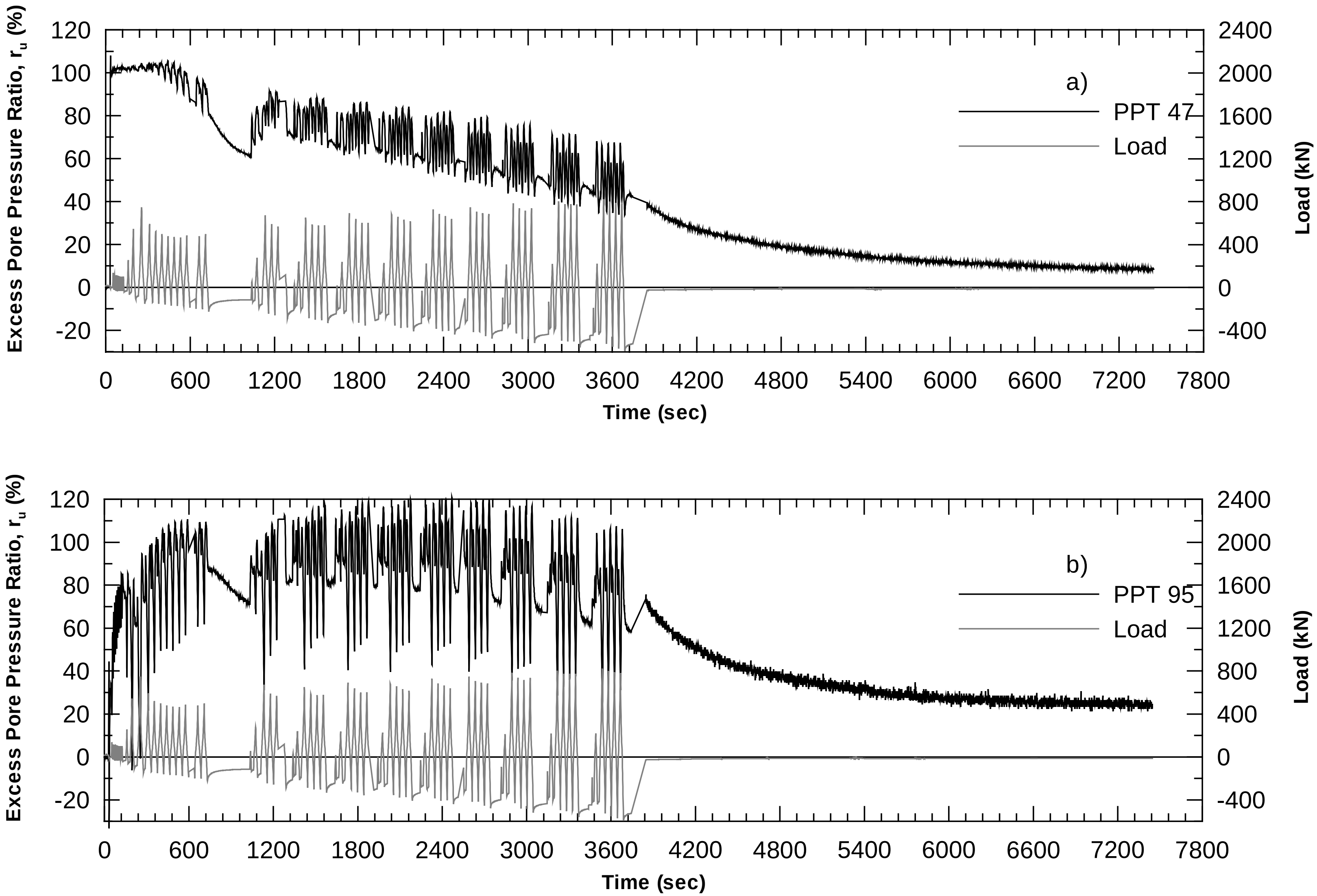


Figure 5.30 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT47 b)PPT95

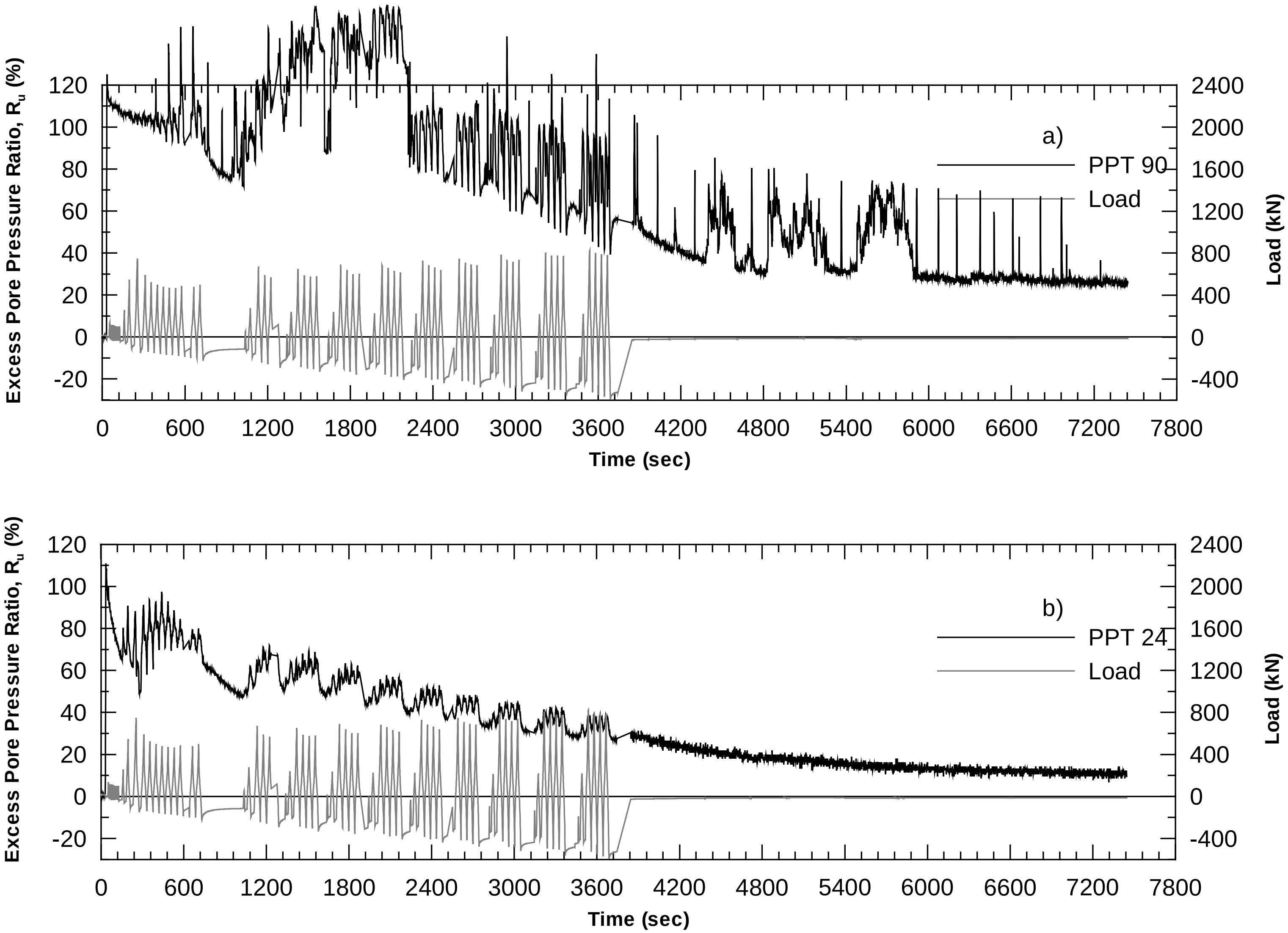


Figure 5.31 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT90 b)PPT24

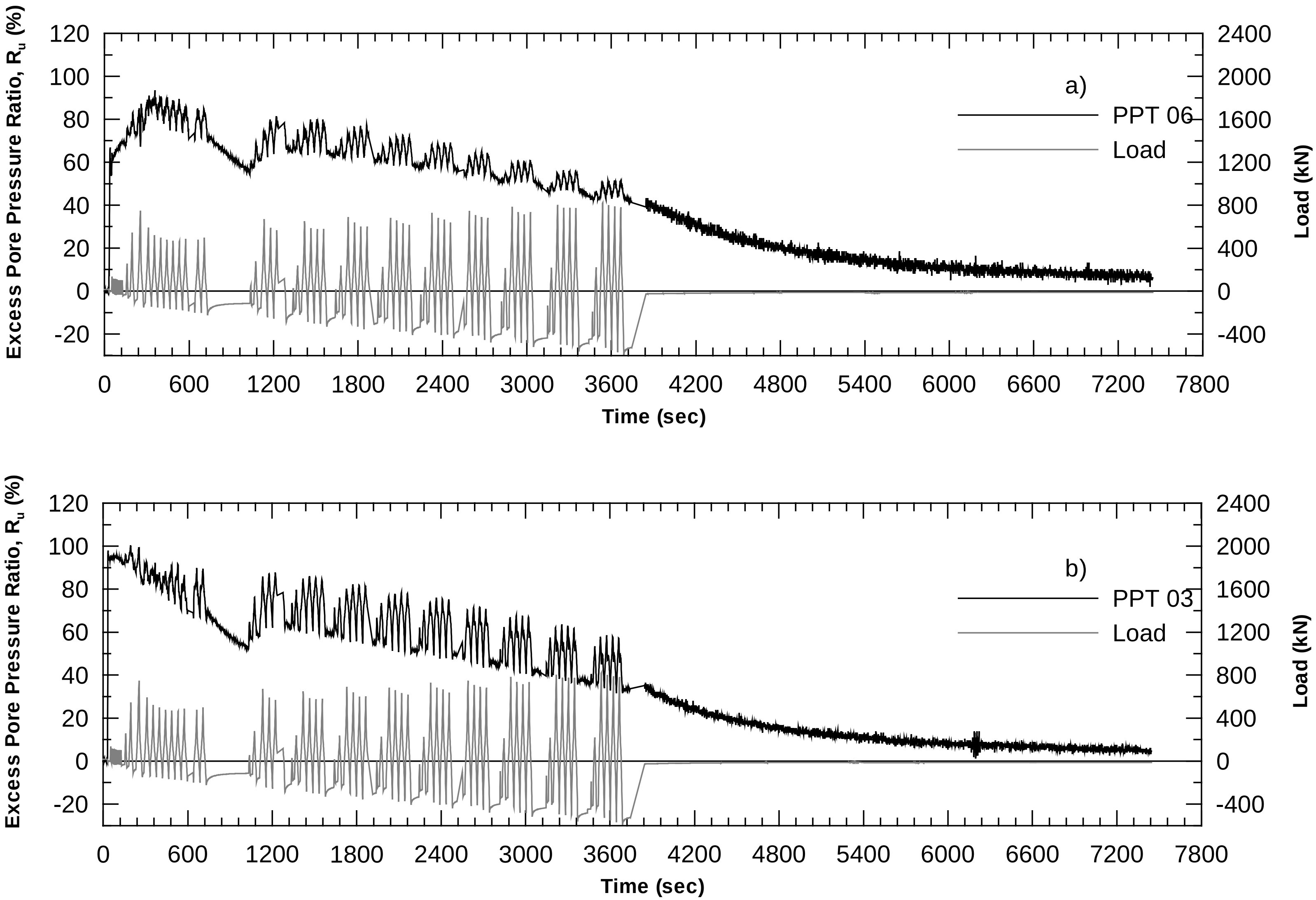


Figure 5.32 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT06 b)PPT03

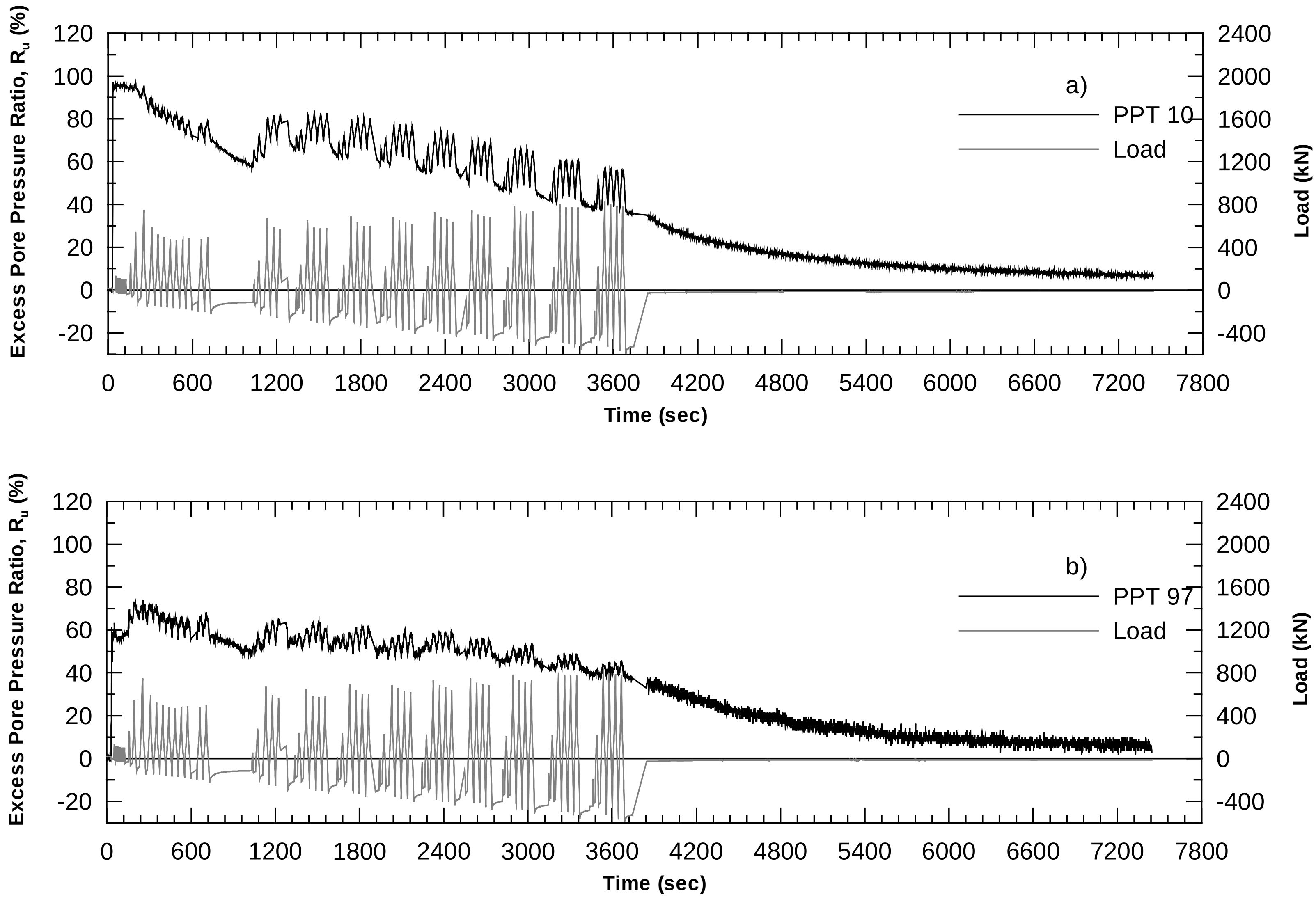
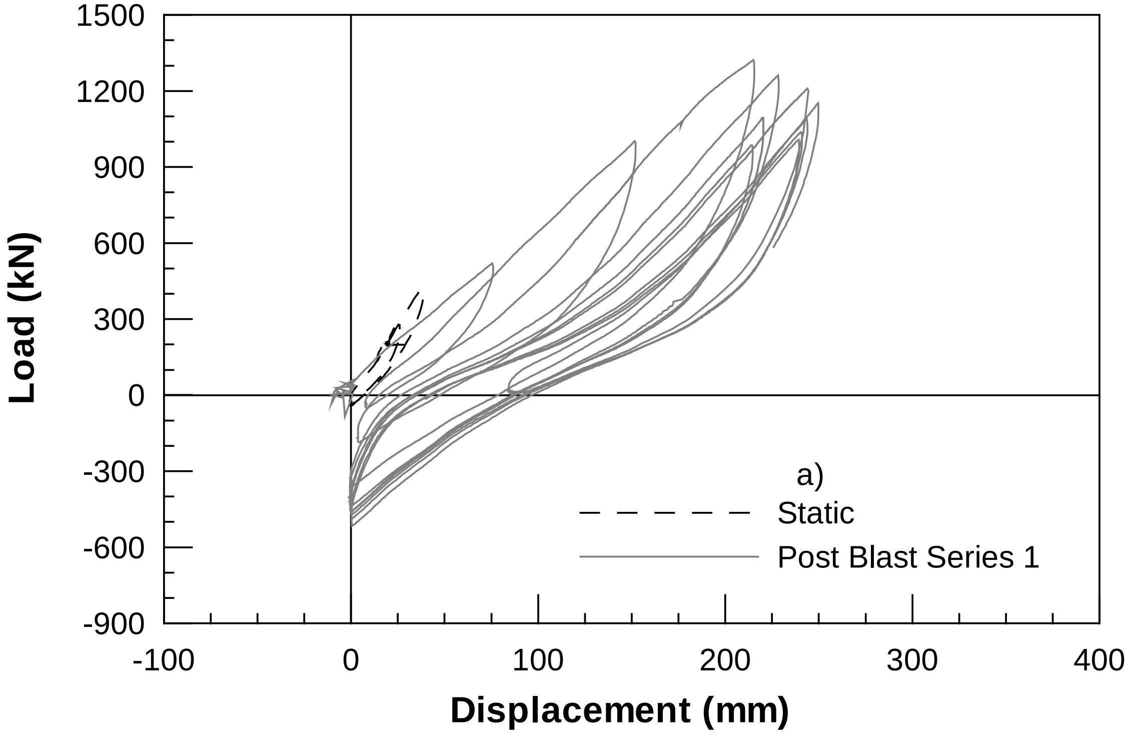


Figure 5.33 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 1st Blast a)PPT10 b)PPT97



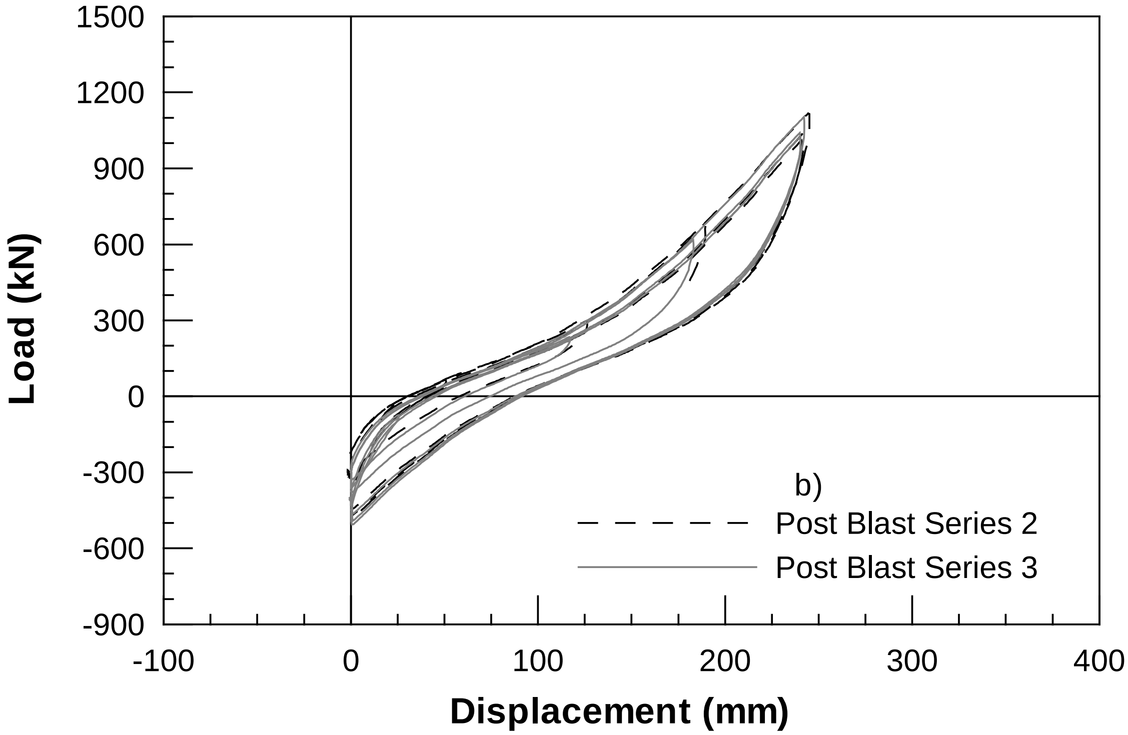


Figure 5.34 Load vs. Displacement 9-Pile Group 2nd Blast a)Static and Post Blast Series 1 b) Post Blast Series 2 and 3

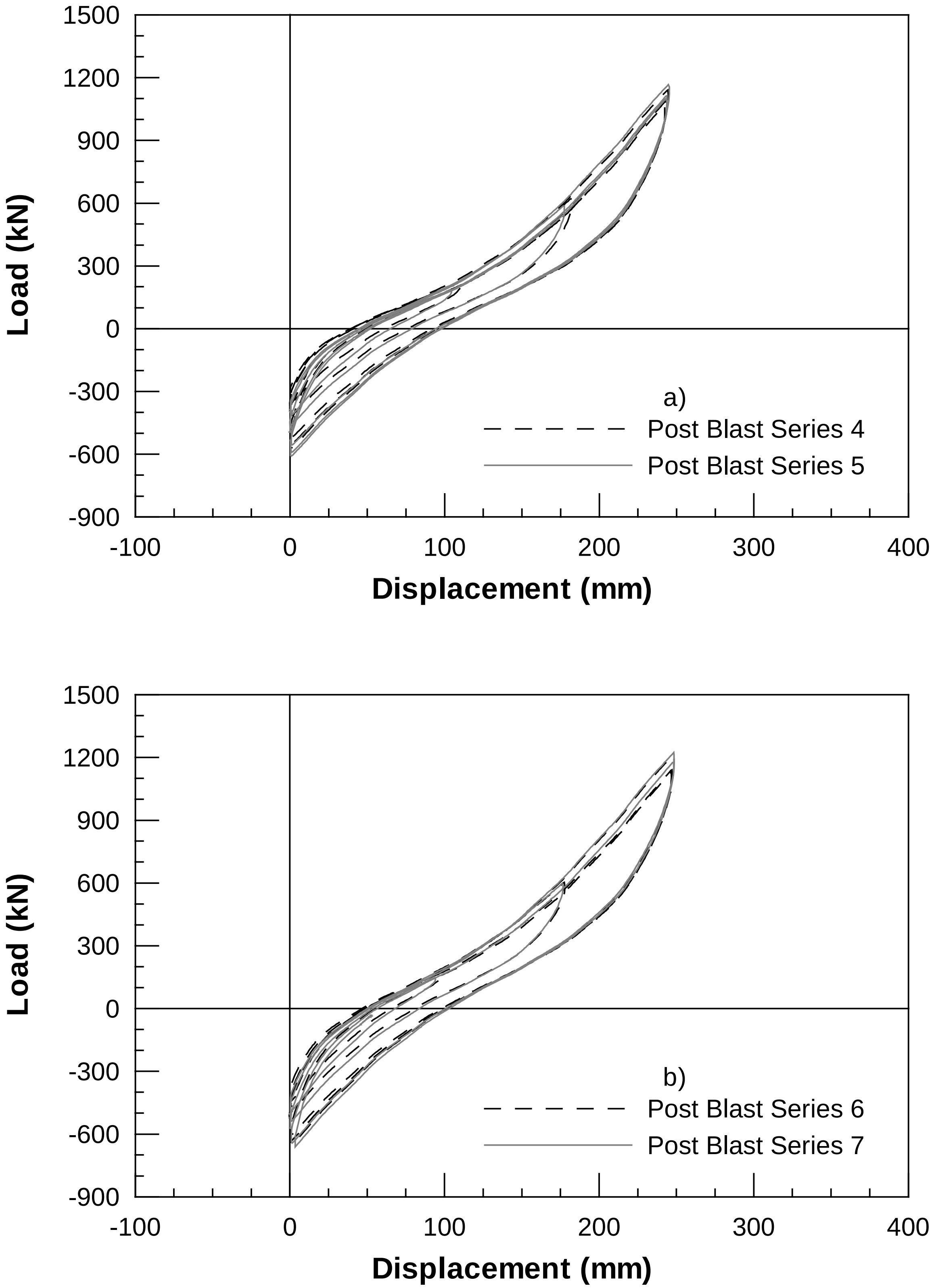


Figure 5.35 Load vs. Displacement 9-Pile Group 2nd Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

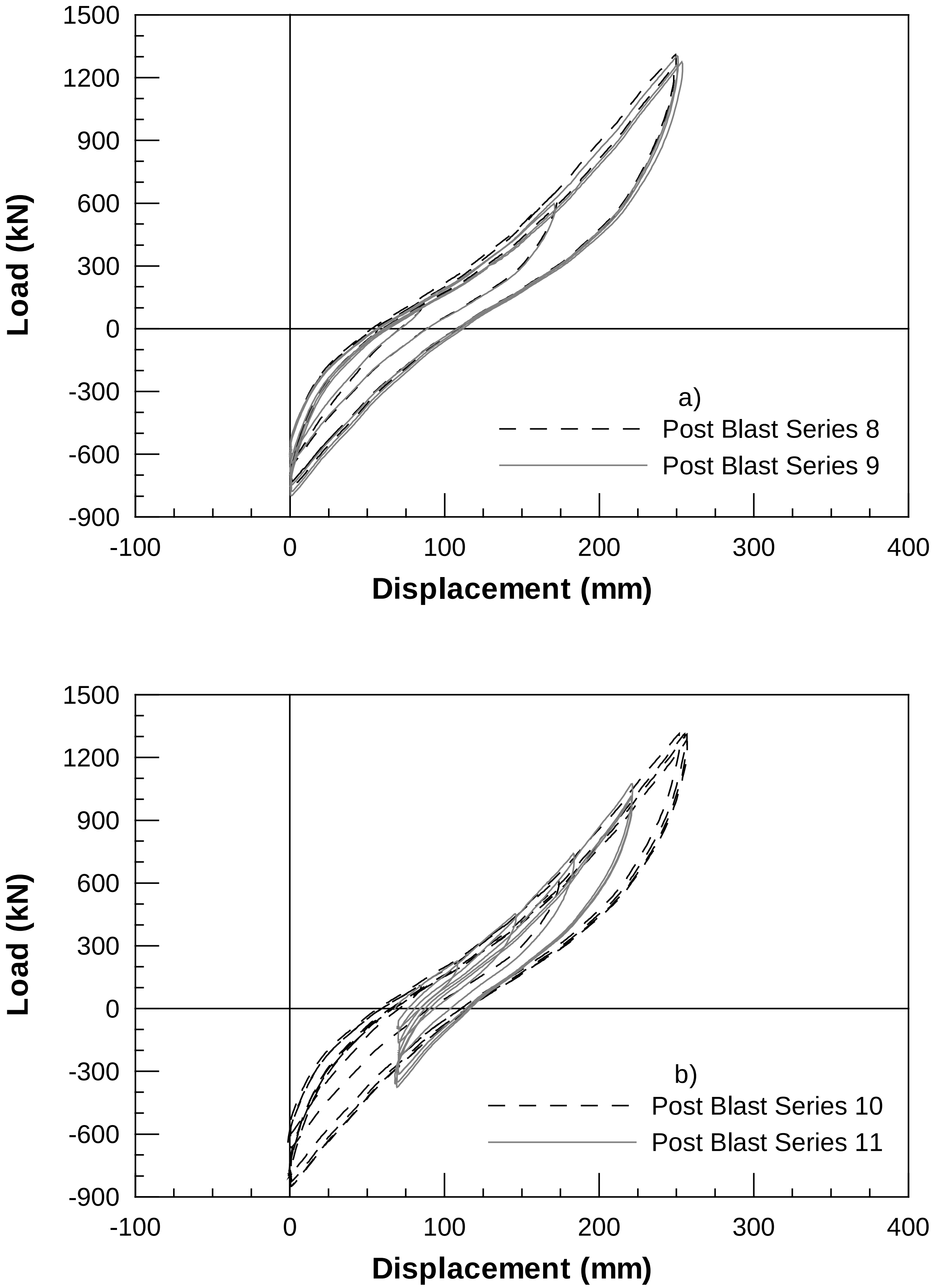


Figure 5.36 Load vs. Displacement 9-Pile Group 2nd Blast a)Post Blast Series 8 and 9 b) Post Blast Series 10 and 11

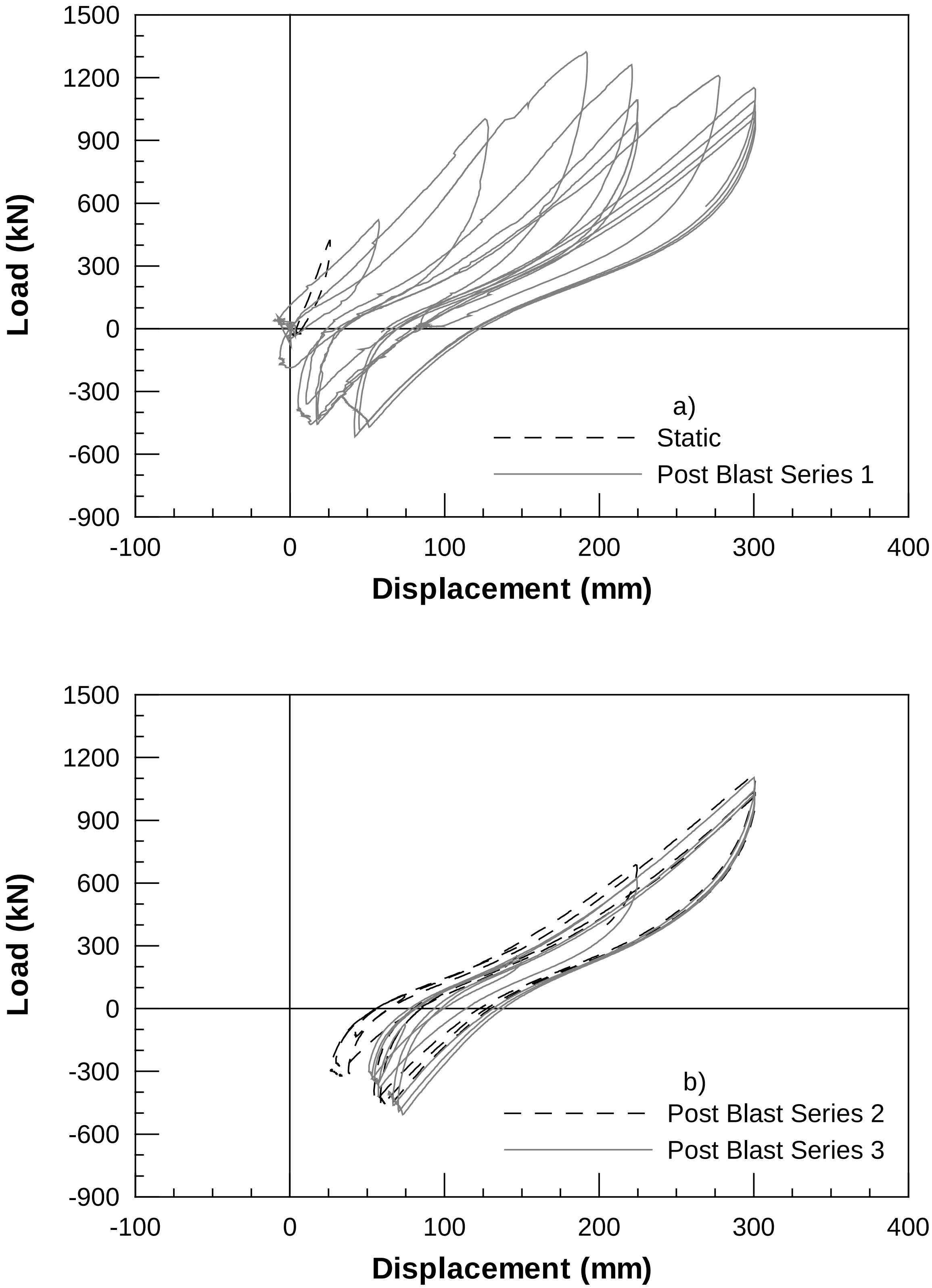


Figure 5.37 Load vs. Displacement for 0.9m CISS Pile 2nd Blast a)Static and Post Blast Series 1 b) Post Blast Series 2 and 3

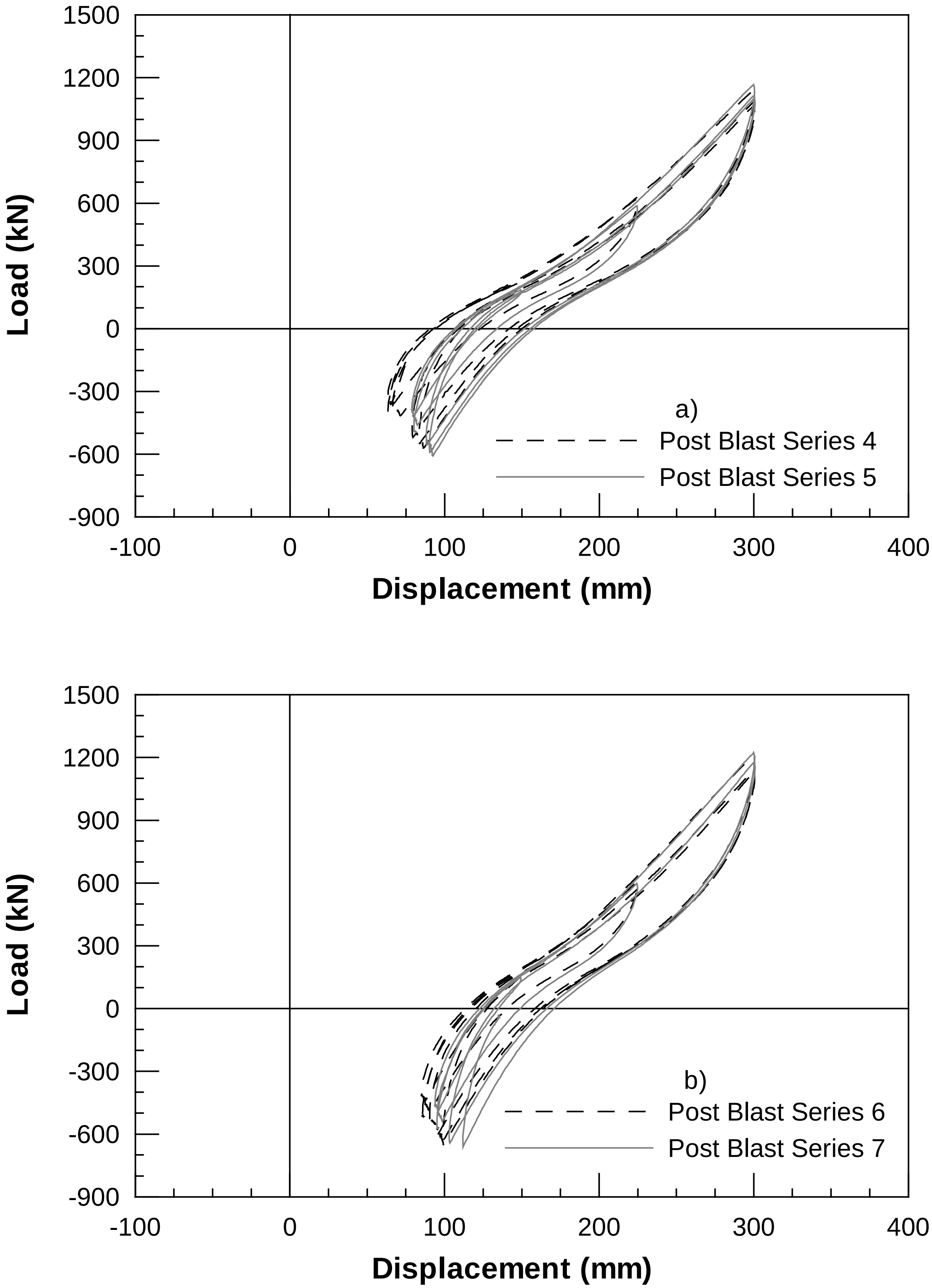


Figure 5.38 Load vs. Displacement for 0.9m CISS pile 2nd Blast a) Post Blast Series 4 and 5 b) Post Blast Series 6 and 7

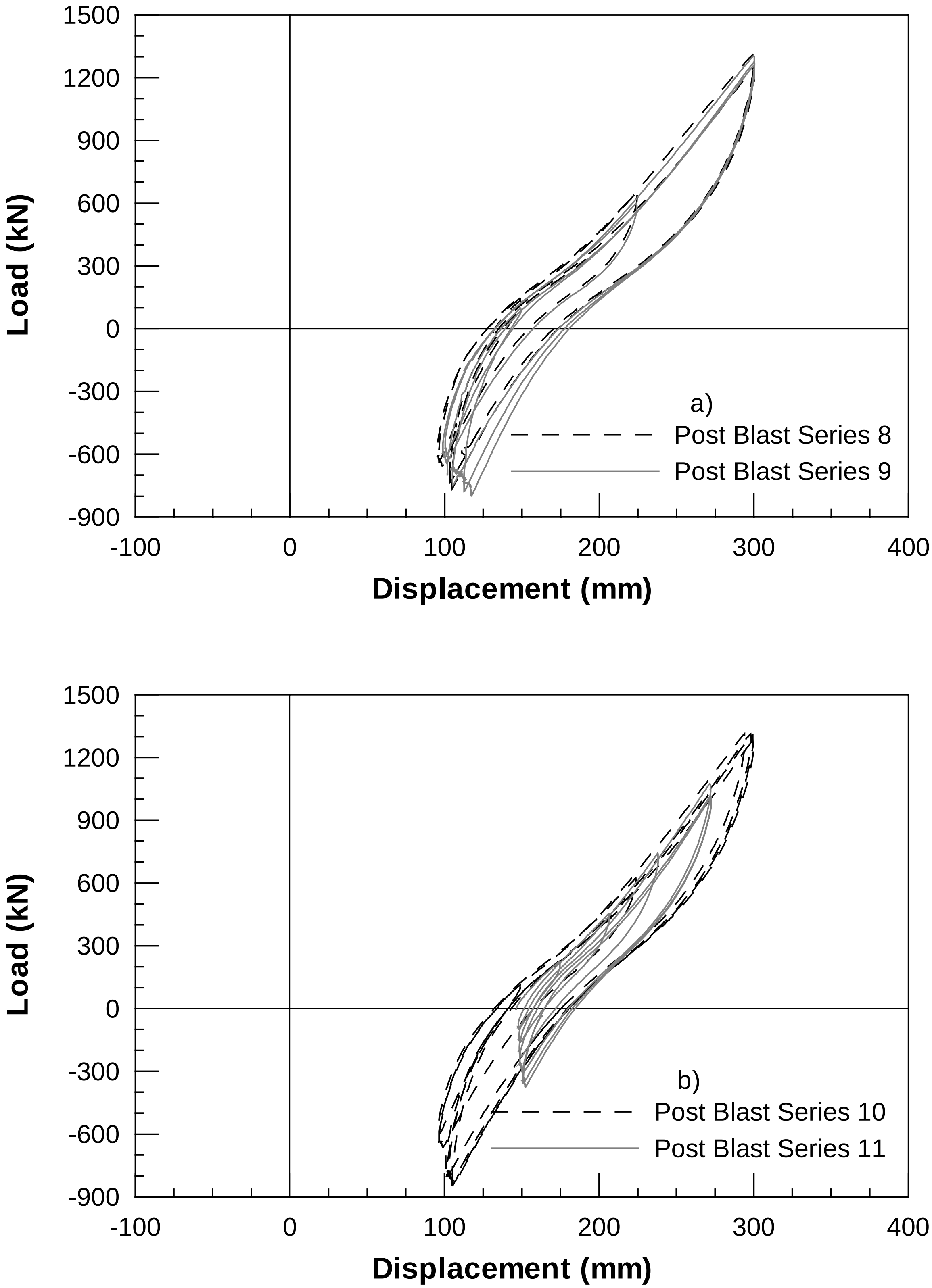


Figure 5.39 Load vs. Displacement for 0.9m CISS pile 2nd Blast a) Post Blast Series 8 and 9 b) Post Blast Series 10 and 11

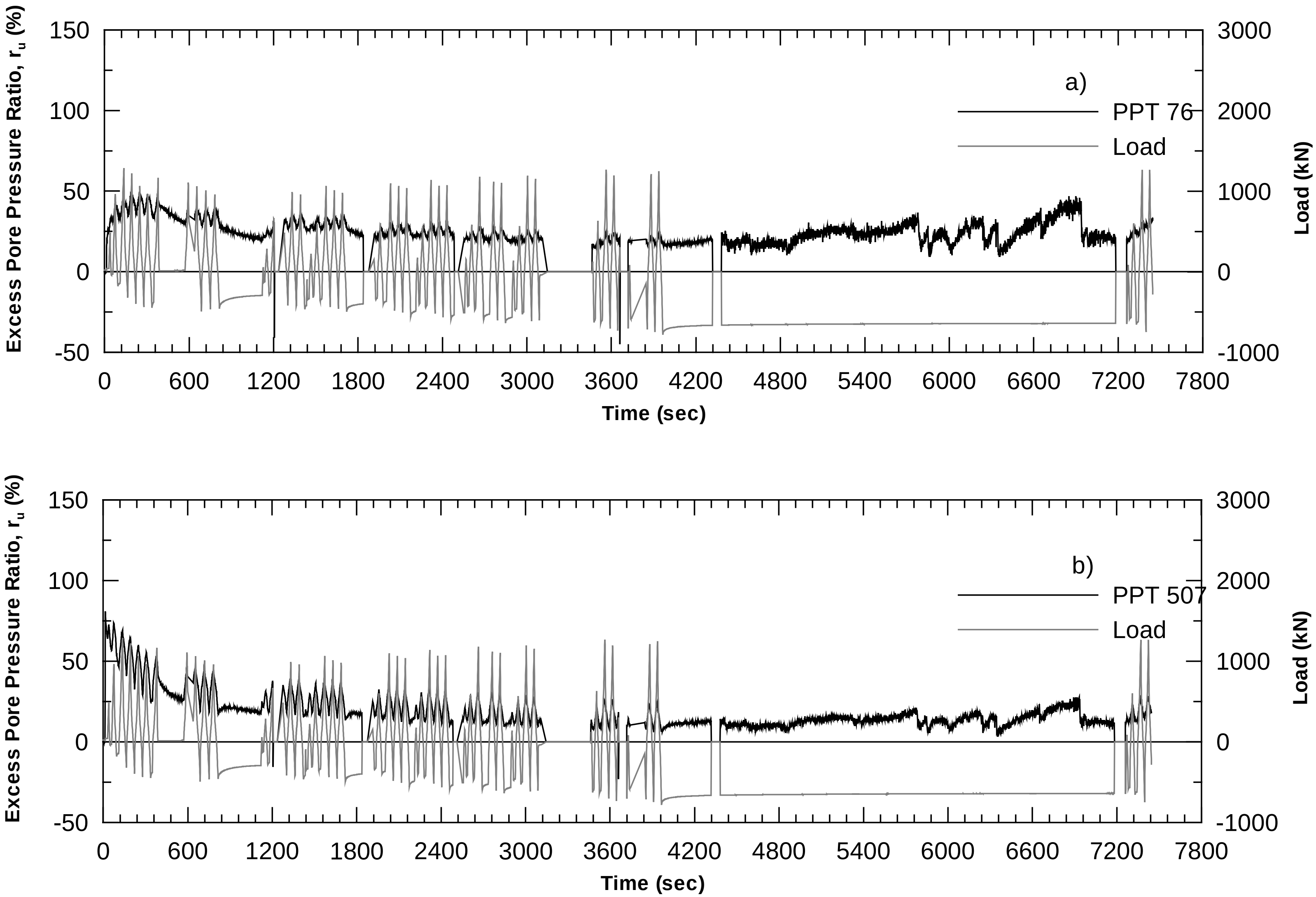


Figure 5.40 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT76 b)PPT507

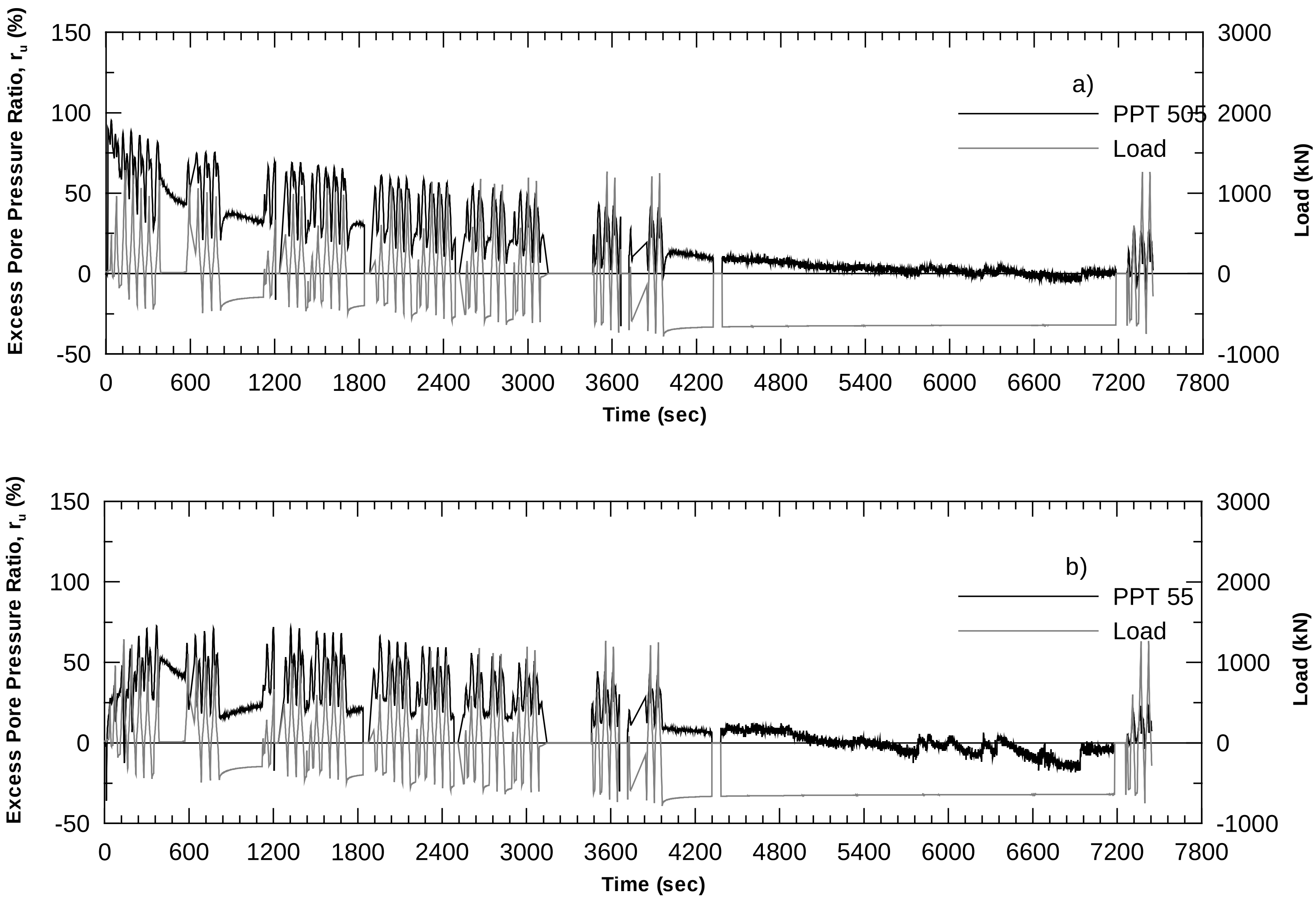


Figure 5.41 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT505 b)PPT55

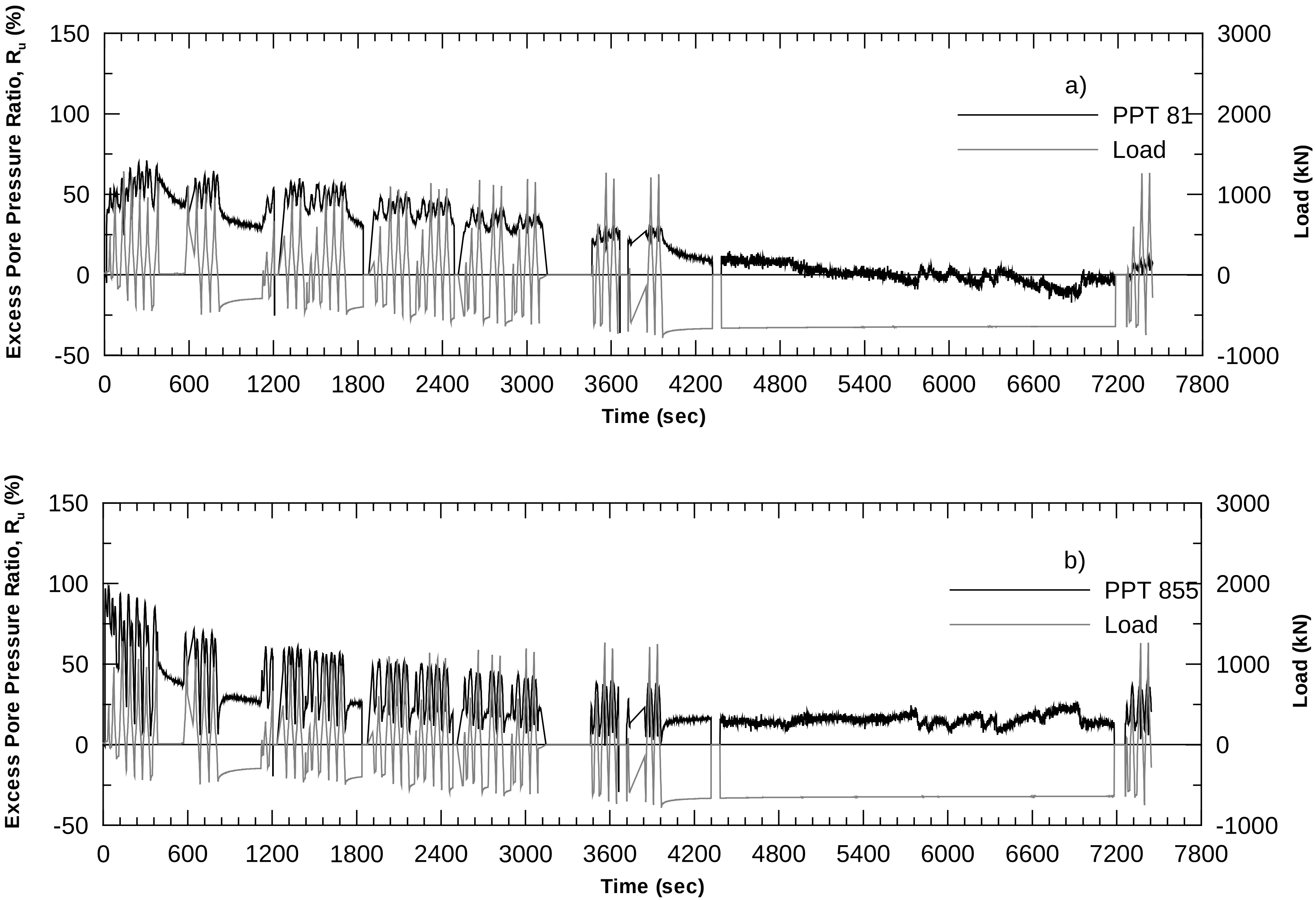


Figure 5.42 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT81 b)PPT855

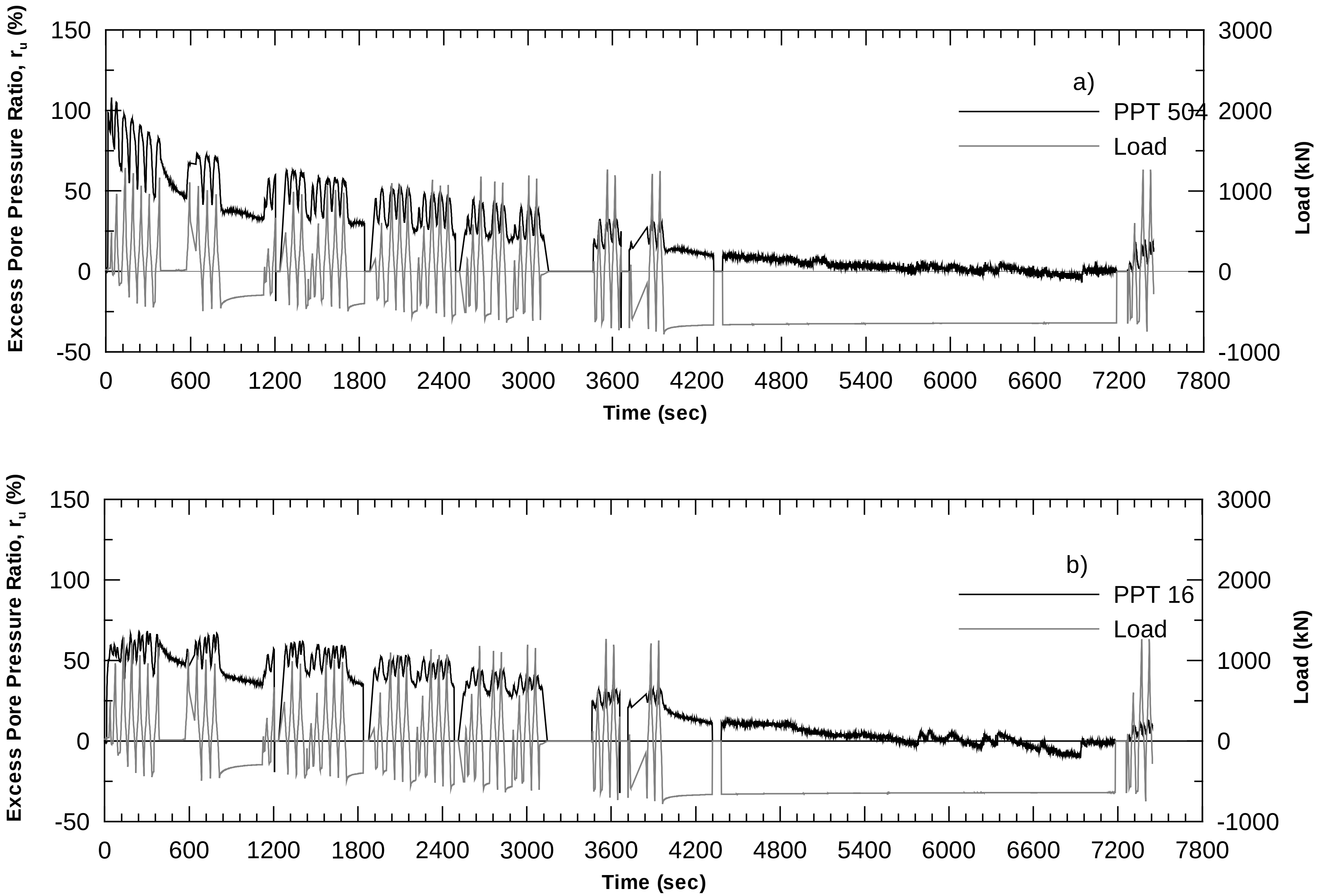


Figure 5.43 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT504 b)PPT16

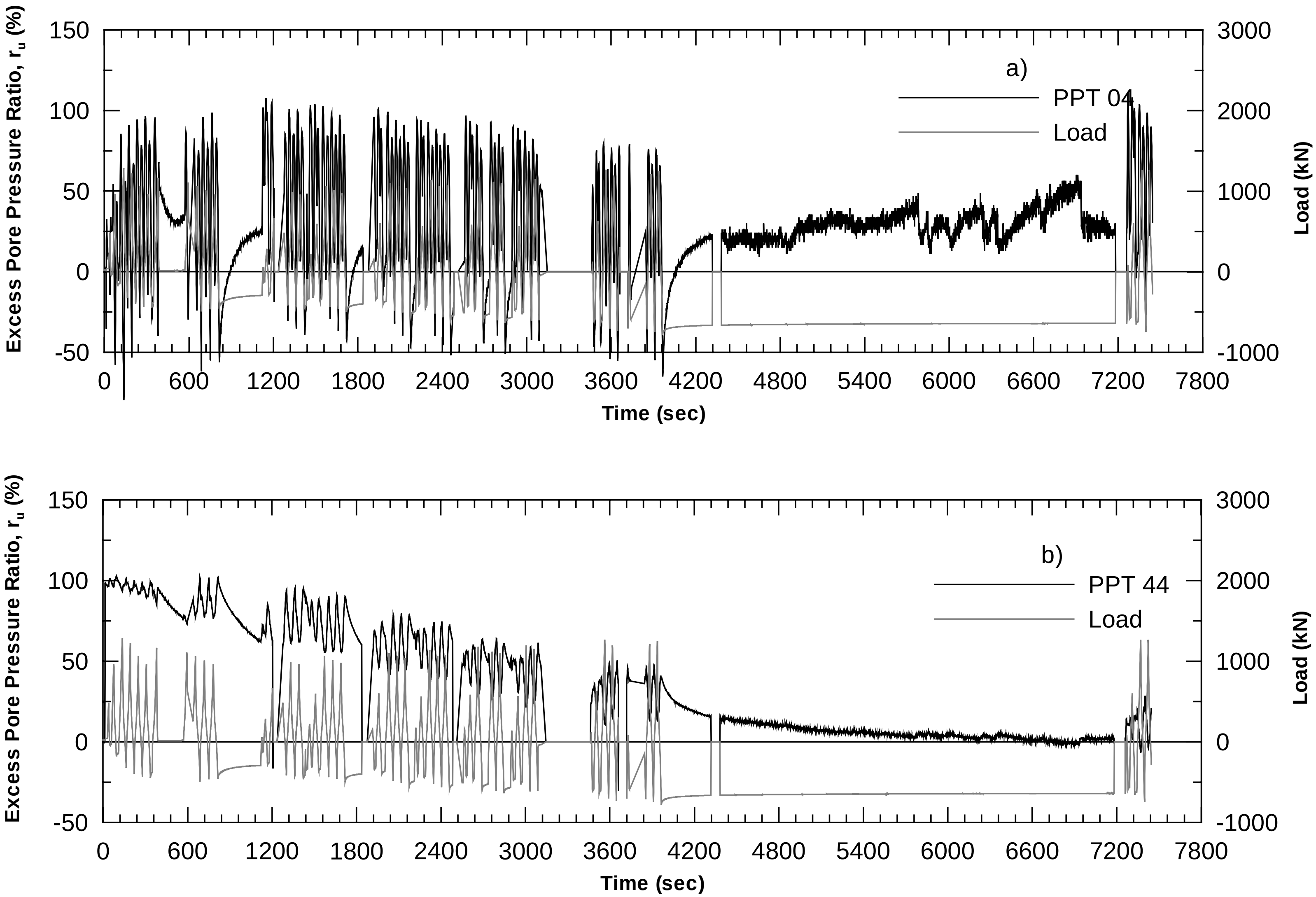


Figure 5.44 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT04 b)PPT44

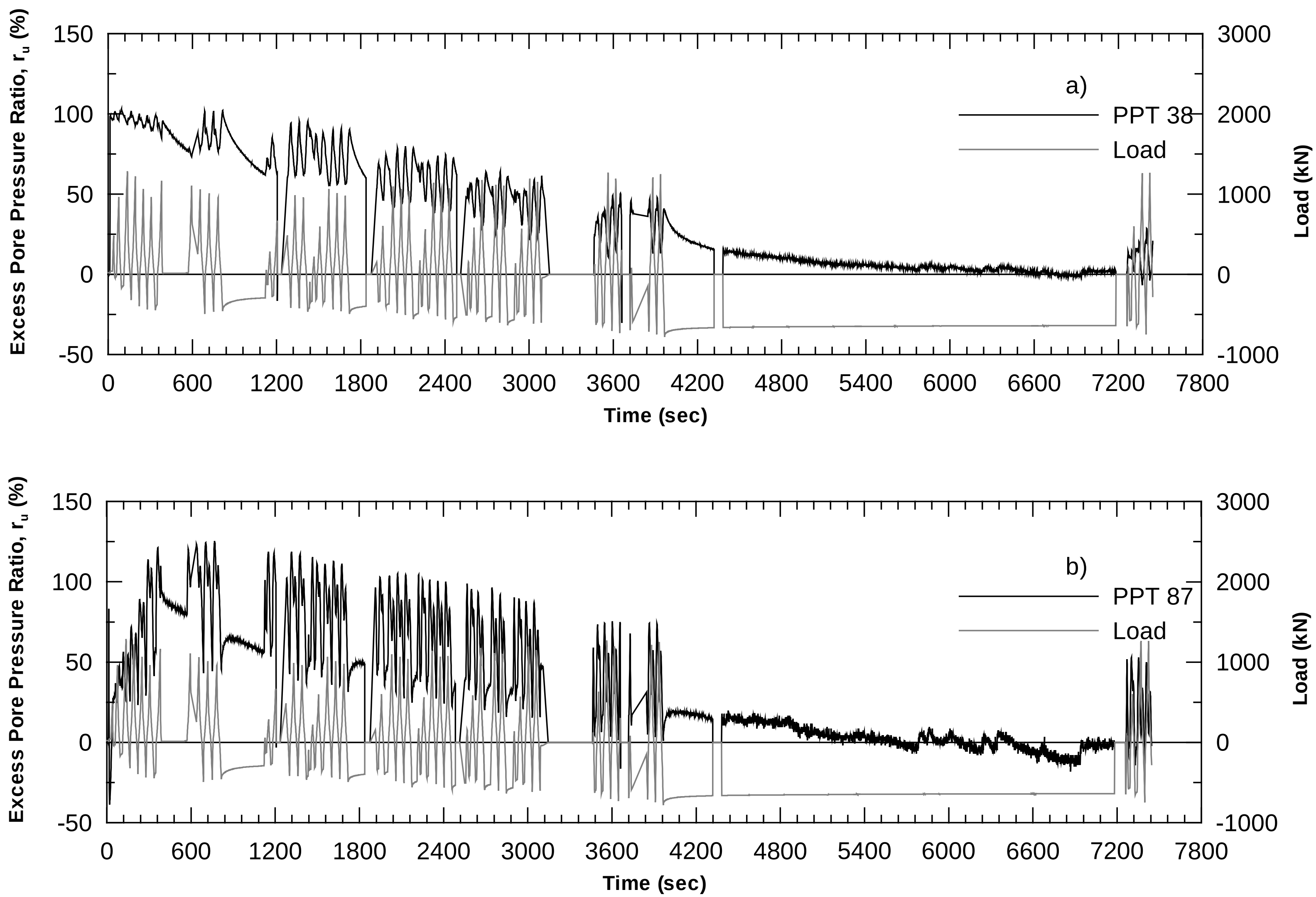


Figure 5.45 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT38 b)PPT87

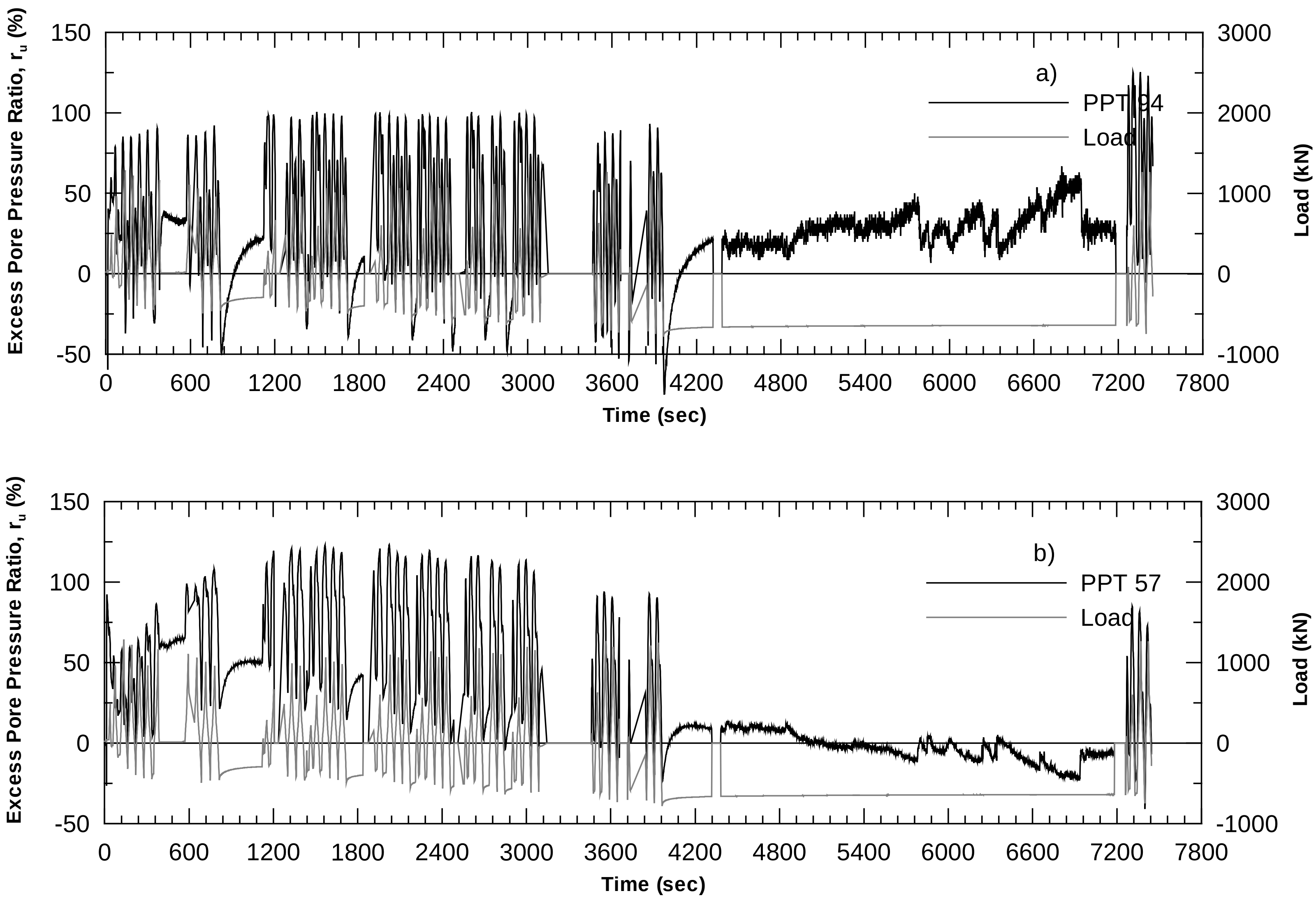


Figure 5.46 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT94 b)PPT57

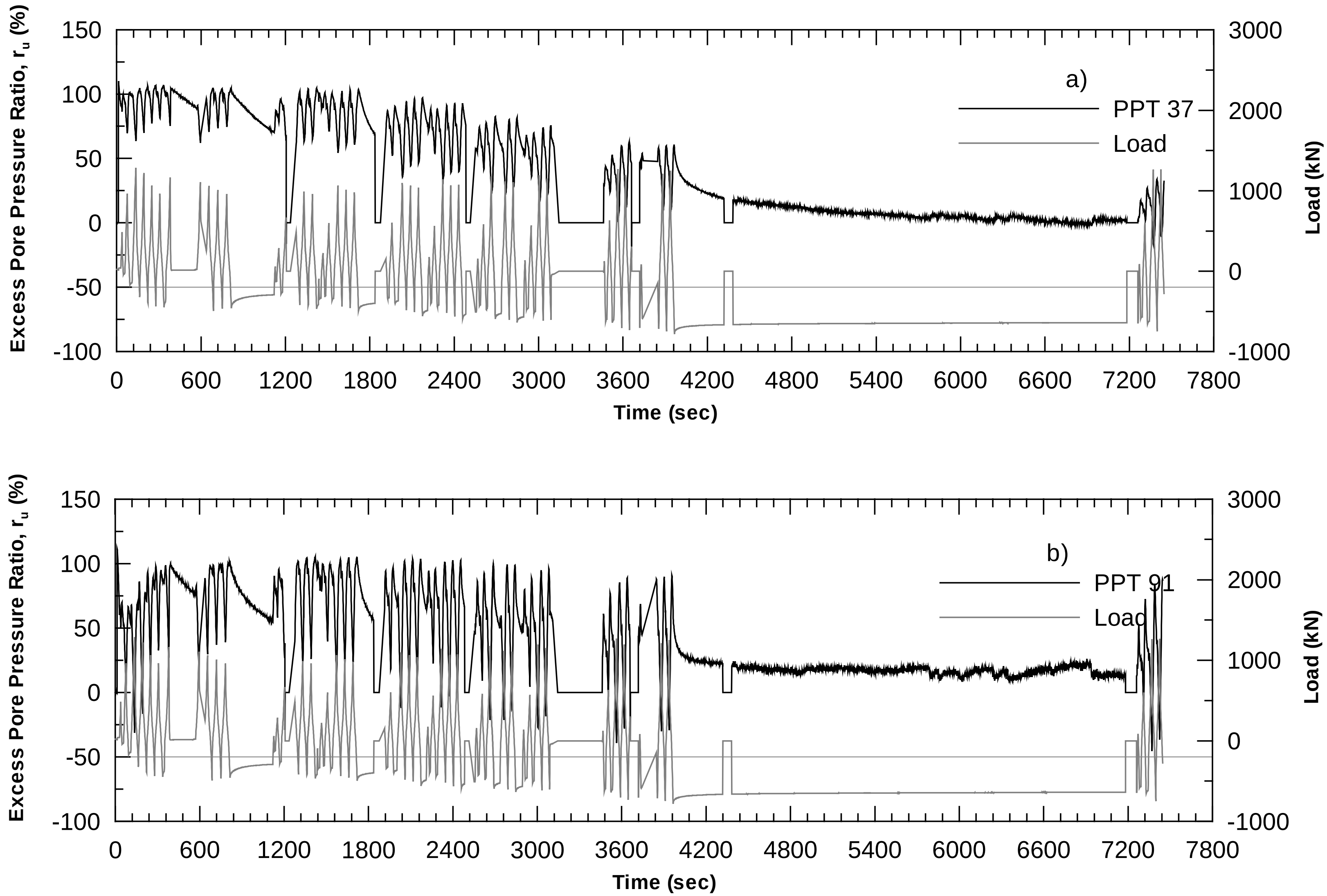


Figure 5.47 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT37 b)PPT91

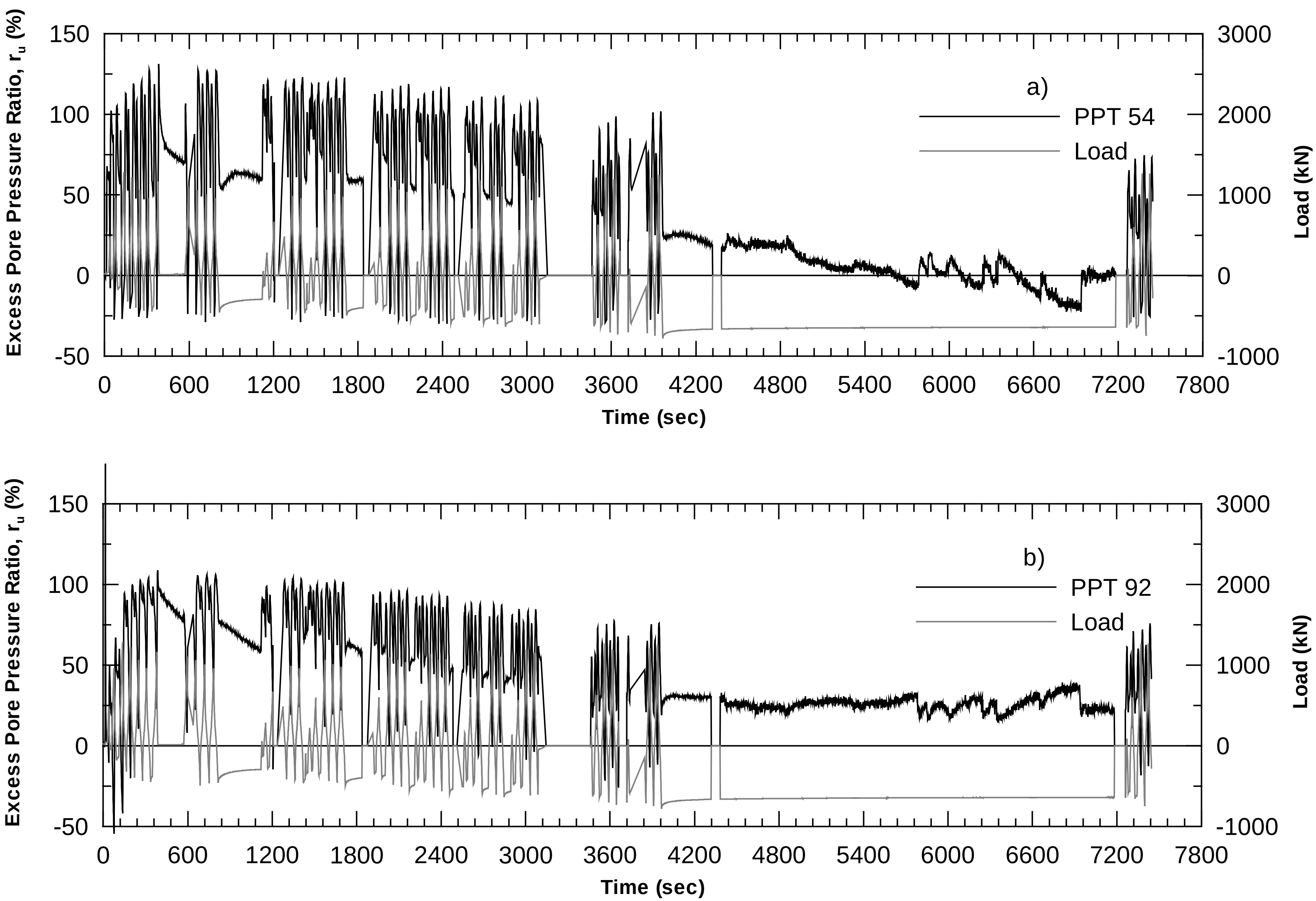


Figure 5.48 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT54 b)PPT92

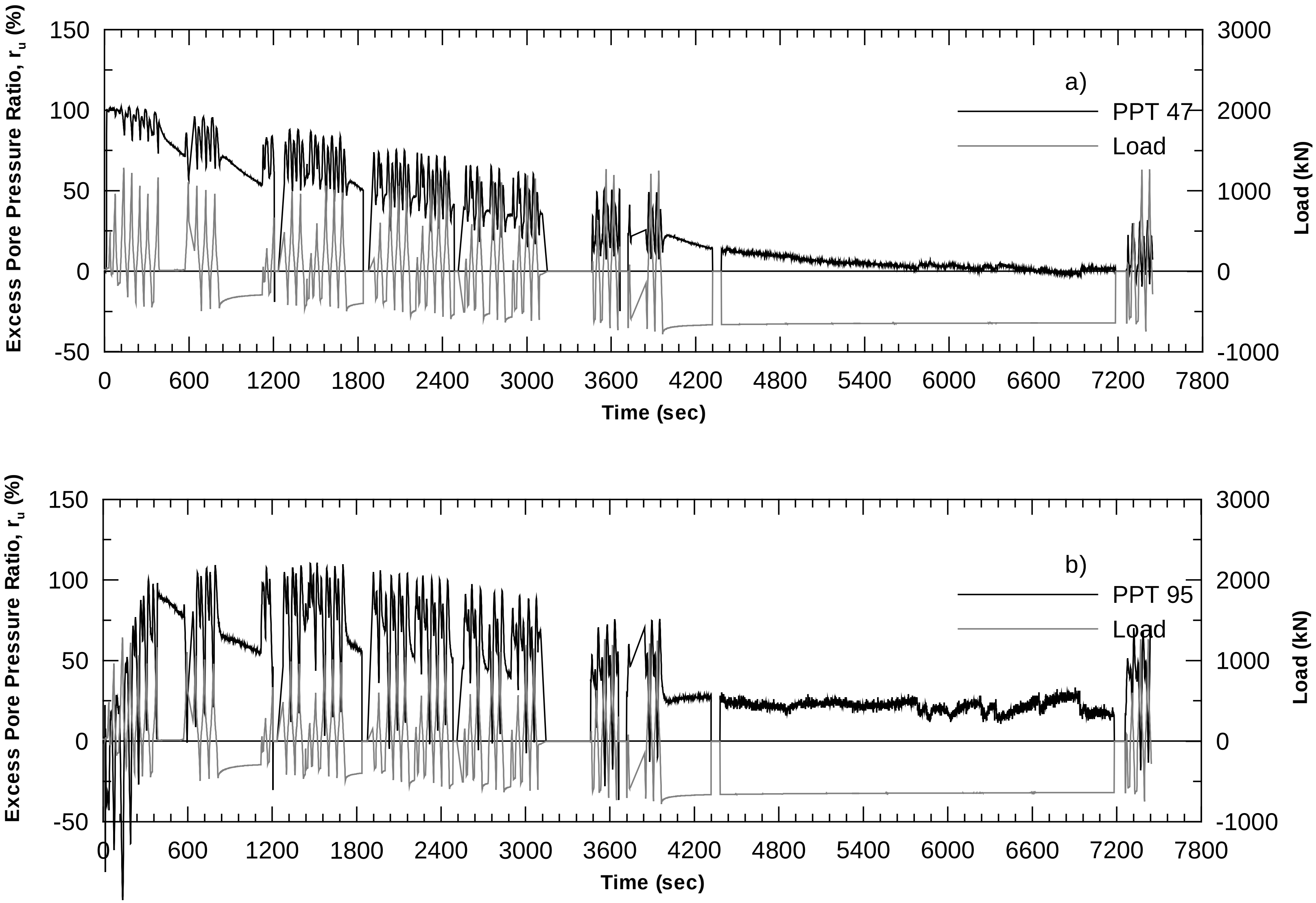


Figure 5.49 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT47 b)PPT95

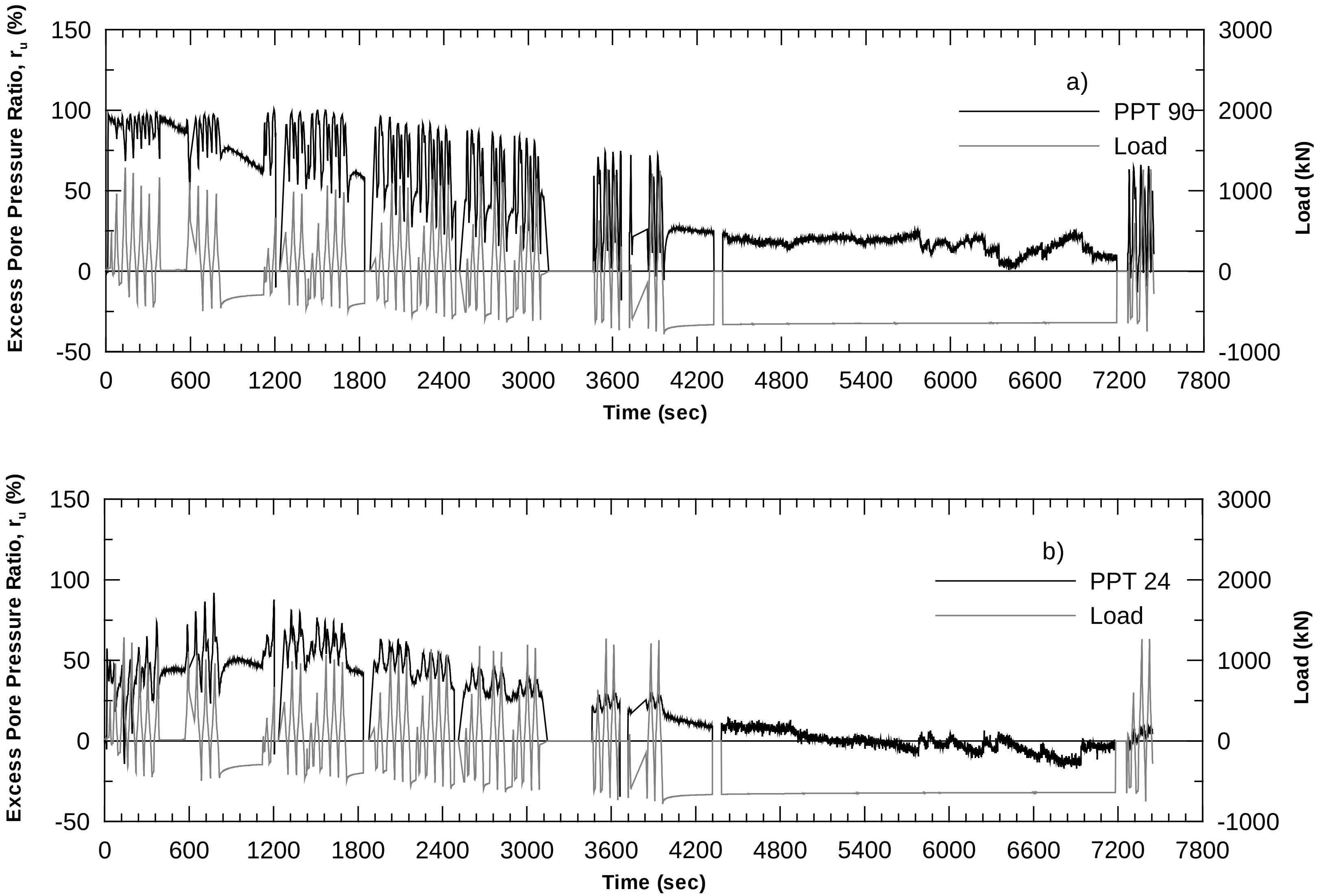


Figure 5.50 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT90 b)PPT24

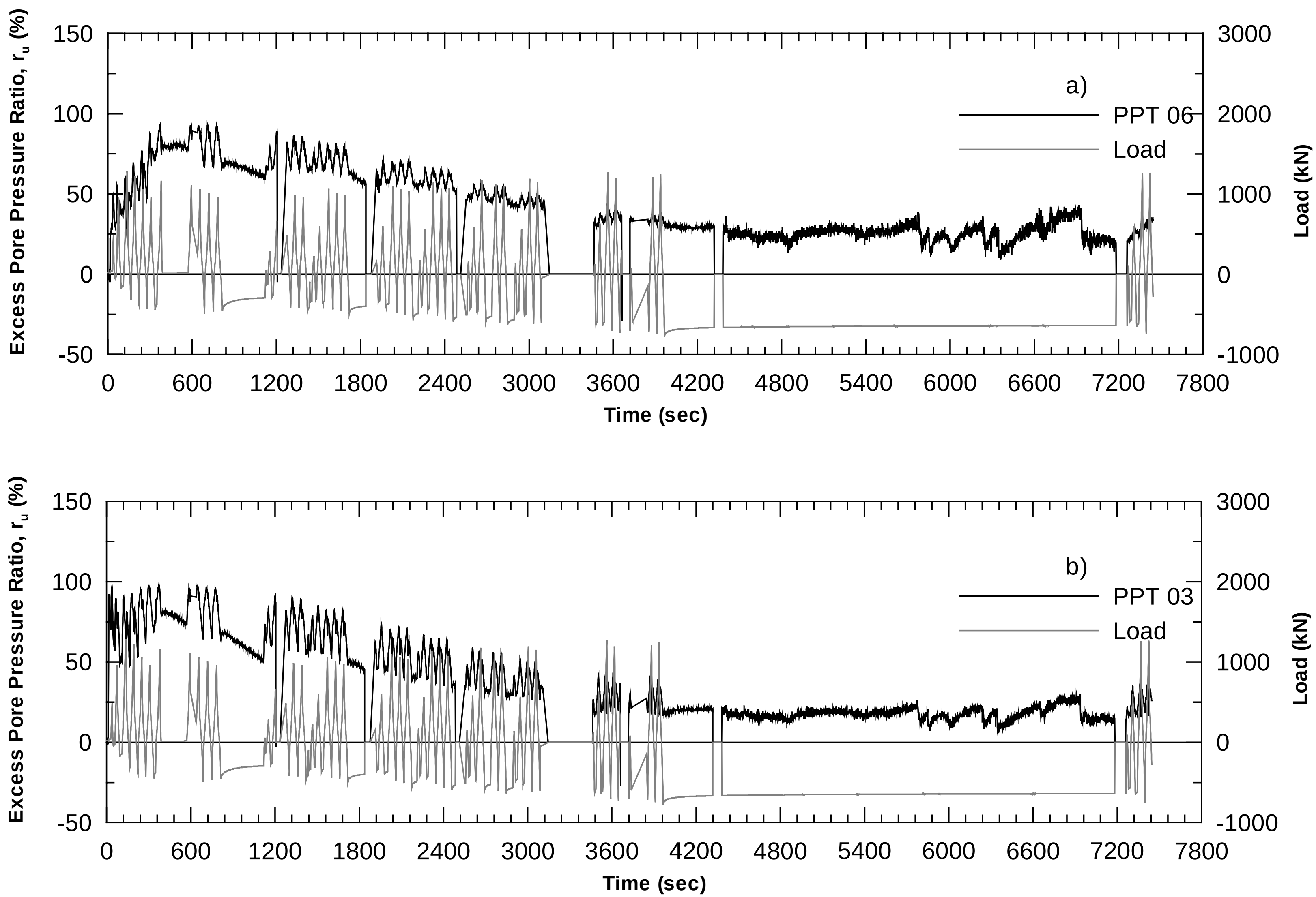


Figure 5.51 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT06 b)PPT03

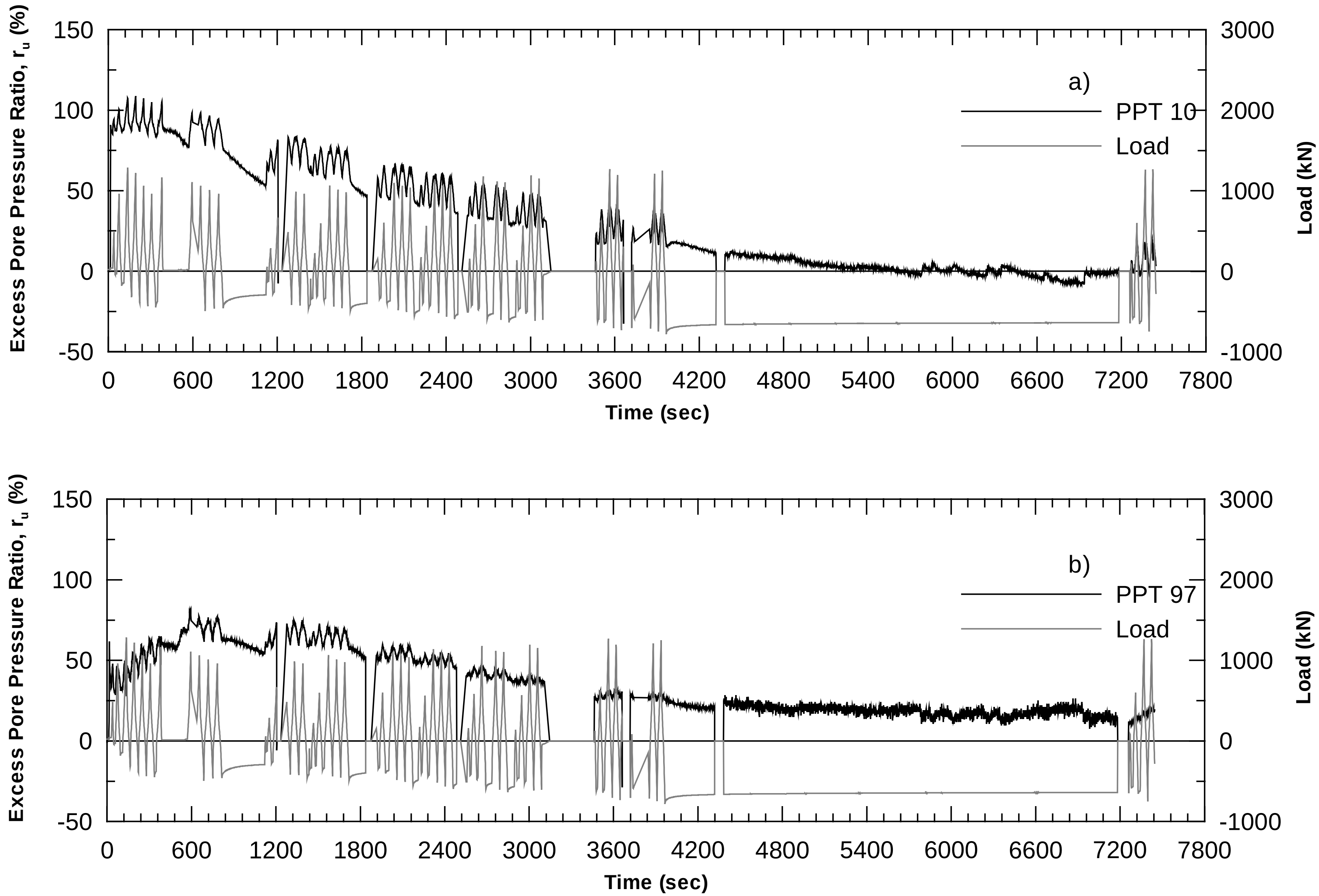


Figure 5.52 Excess Pore Pressure Ratio for 9-Pile Group/0.9m CISS Pile 2nd Blast a)PPT10 b)PPT97

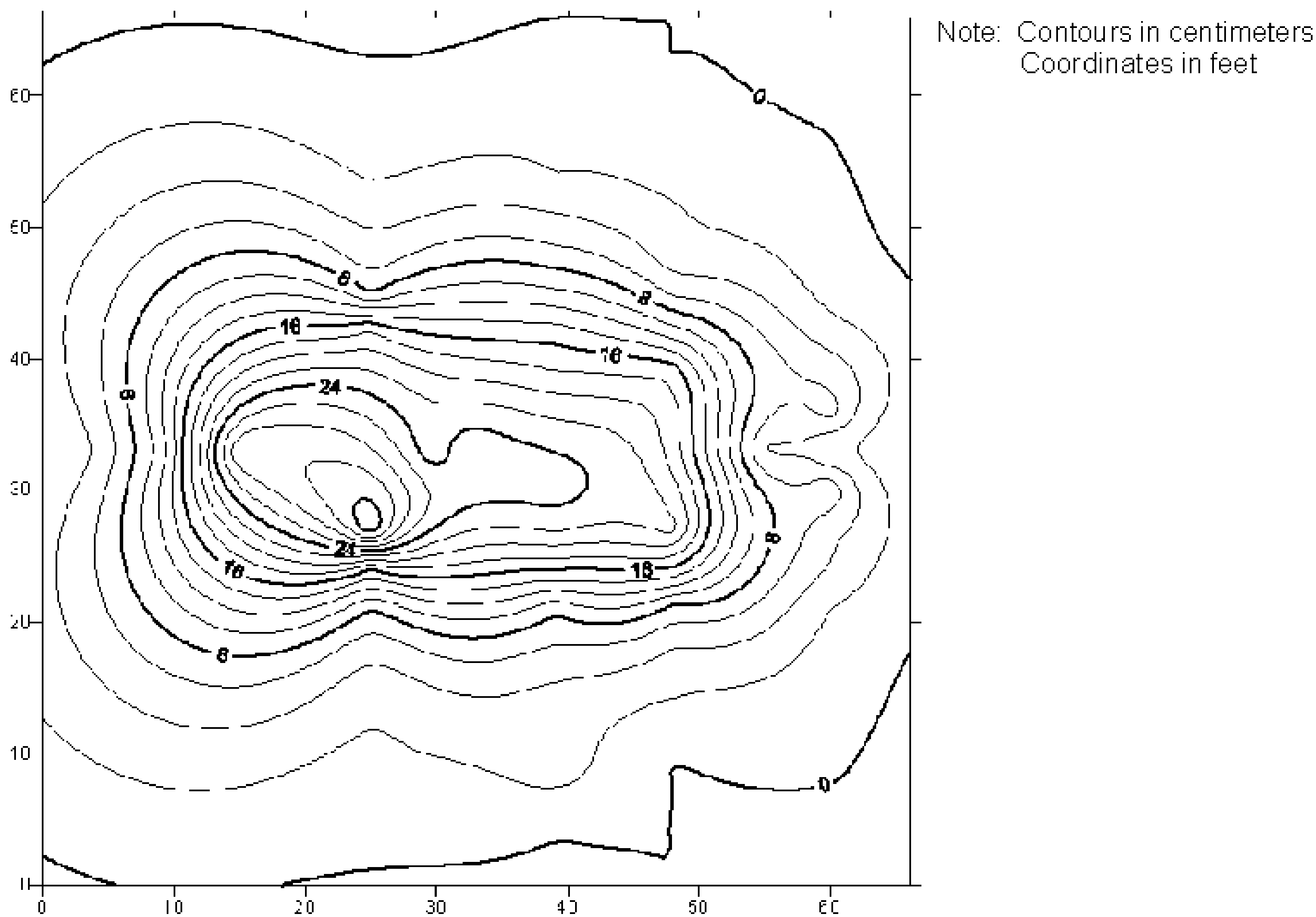


Figure 5.53 Contour of Settlement for 9 Pile-Group/0.9m CISS Pile 1st Blast

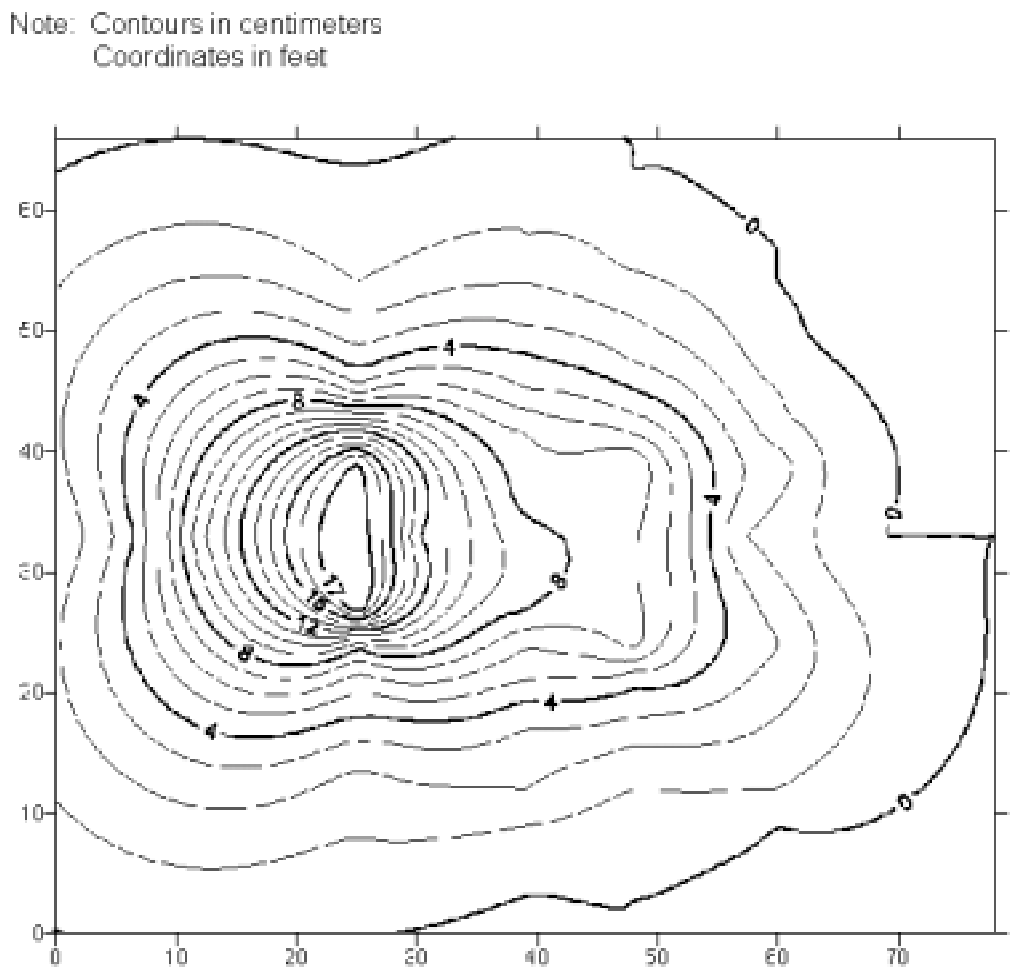


Figure 5.54 Contour of Settlement for 9 Pile-Group/0.9m CISS Pile 2nd Blast