FINAL REPORT

TIME DEPENDENT LOAD RESPONSE OF FLEXIBLE PIPE SUBJECTED TO SUSTAINED LOADING

FDOT Contract No. BDK75 977-21
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Submitted By:

Raphael W. Crowley, Ph.D., P.E.
College of Computing, Engineering, and Construction
Department of Construction Management
University of North Florida
Building 50, Room 2400
Jacksonville, FL 32224

David Bloomquist, Ph.D., P.E.
Engineering School of Sustainable Infrastructure and Environment
Department of Civil and Coastal Engineering
University of Florida
365 Weil Hall
Gainesville, FL 32611

Developed for the

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Rick Renna, P.E., Project Manager

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DISCLAIMER

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SI (MODERN METRIC) CONVERSION FACTORS (FROM FHWA)

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
FORCE and PRESSURE or STRESS				
lbf	pound force	4.45	newtons	N
lbf/in^2	pound force per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS TO ENGLISH UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C+32	Fahrenheit	°F
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
FORCE and PRESSURE or STRESS				
N	newtons	0.225	pound force	lbf
kPa	kilopascals	0.145	pound force per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.(Revised March 2003)

TECHNICAL REPORT DOCUMENTATION PAGE

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16. Abstract

As part of a previous FDOT project, a large box was designed and built to simulate overburden pressures and measure deflections on buried culvert pipe (Project No. BD-545). Using this Soil Box, a series of tests were conducted on HDPE, PVC, and steel pipes of varying diameters. A computer model was developed using LS-DYNA to simulate overburden as well. Results were mixed. While the Soil Box does appear to be capable of simulating overburden pressures on culvert pipes, the simulated pressures were not nearly as large as hoped. This discrepancy is believed to be caused by variable contact areas between the Soil Box's lift bags and the soil surface. A new testing technique was developed to develop higher simulated overburden stresses. However, results from this test appeared to indicate that adding more soil to the box was more effective than burying the lift bags. An ABAQUS finite element model was developed by Simpson, Gumpertz, and Heger (SGH) to simulate testing conditions in the Soil Box. While modeled data failed to match measured readings, modeled data were consistent with AASHTO specifications for buried pipe. Based upon modeled results and analysis of other pipe-types, SGH recommended a 30-day acceptance criteria of 3.5% deflection. However, SGH acknowledged that quantifying densification over the lifetime of a pipe was difficult. Additionally, time was not explicitly taken into account in the SGH model. UF/UNF investigators developed a preliminary three-dimensional model using LS-DYNA. Results appeared to indicate that it may be possible to explicitly add a time component to modeled results – but only if the pipe itself could be modeled properly.

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EXECUTIVE SUMMARY

As part of a previous FDOT project, a large box was designed and built to simulate overburden pressures and measure deflections on buried culvert pipe (Project No. BD-545). Using this Soil Box, a series of tests were conducted on HDPE, PVC, and steel pipes of varying diameters. A computer model was developed using LS-DYNA to simulate overburden as well. Results were mixed. While the Soil Box does appear to be capable of simulating overburden pressures on culvert pipes, the simulated pressures were not nearly as large as hoped. This discrepancy is believed to be caused by variable contact areas between the Soil Box's lift bags and the soil surface. A new testing technique was developed to develop higher simulated overburden stresses. However, results from this test appeared to indicate that adding more soil to the box was more effective than burying the lift bags. An ABAQUS finite element model was developed by Simpson, Gumpertz, and Heger (SGH) to simulate testing conditions in the Soil Box. While modeled data failed to match measured readings, modeled data were consistent with AASHTO specifications for buried pipe. Based upon modeled results and analysis of other pipe-types, SGH recommended a 30-day acceptance criteria of 3.5% deflection. However, SGH acknowledged that quantifying densification over the lifetime of a pipe was difficult. Additionally, time was not explicitly taken into account in the SGH model. UF/UNF investigators developed a preliminary three-dimensional model using LS-DYNA. Results appeared to indicate that it may be possible to explicitly add a time component to modeled results – but only if the pipe itself could be modeled properly.

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CHAPTER 1
INTRODUCTION

1.1 Background

For pipes installed under roadways, the Florida Department of Transportation (FDOT) requires inspection when backfill reaches 3 feet above the pipe crown or after placement of the stabilized subgrade (FDOT 2013). The purpose of this project was to assist the Florida Department of Transportation in the understanding of pipe-soil interaction by gathering deflection data of flexible pipes when subjected to overburden stress. These data will ultimately be used to identify pipes whose deflection are within the 5% specification tolerance at the time of inspection but may exceed this tolerance at the time of project acceptance.

1.2 Soil Box Design

The steel Soil Box (Figure 1-1) is 20 feet long, 10 feet wide, 8 feet tall, and was constructed under FDOT Project BD-530 (Bloomquist et al. 2009). It is comprised of multiple partitions bolted together and reinforced with wide flange beams to prevent deflections of less than two millimeters when subjected to a geostatic earth pressure of 118 pounds per square inch (150 ft. of overburden). This box was designed to eliminate boundary effects when testing dual pipes up to 36” in diameter. Observation portals located along the length of the Soil Box allow for monitoring of the pipe’s diameter during loading.

1.3 Basis of Development

Brachman et al. (2001) performed testing of small-diameter buried pipes subject to biaxially compressive earth pressures in a laboratory facility. Issues such as simulation of overburden pressures, boundary stiffness of the testing chamber, and influence of side wall friction were discussed. He reported the overall effects on the pipe from the idealized laboratory model were small. The development of the Soil Box design and testing approach were based on these previous tests (Brachman et al. 2000; Brachman et al. 2001). However, the unique feature of the Soil Box when compared with the Brachman devices is that it allows for testing of full-scale pipes up to 36 inches (the Brachman device only accommodated smaller diameter pipes).

1.4 Testing Overview and Organization

Throughout this project, several tests were conducted in the Soil Box on high density polyethylene (HDPE), polyvinylchloride (PVC), and steel pipes. Throughout this project, testing techniques were often modified as investigators learned more about how to effectively utilize the Soil Box to induce overburden pressures on these pipes. This report is presented chronologically in an effort to illustrate how the “current testing standard” became developed (Chapter 2 through Chapter 11). Ultimately, data from these tests were to be used as a basis of comparison for a finite element (FE) model. While the FE model was developed, results from the model may be questionable. These results are discussed in Chapter 12.

Figure 1-1. The Soil Box located at the University of Florida.

CHAPTER 2
INITIAL DESIGN FOR TESTING

2.1 Interface Between Soil Box and Pipe Ends

A sealing system for the interface between the pipe ends and the Soil Box side walls was designed to prevent soil and water from escaping through the observation portals while allowing the pipe to deflect freely. There were two design concepts for this system.

The first design concept (Figures 2-1 and 2-2) consisted of an inflatable tube sandwiched between two 2-inch wide PVC strips. One PVC strip wraps around the pipe end and the other encompasses the inflatable tubing.

The second design concept (Figures 2-3 and 2-4) is comprised of a rigid steel angle bolted to the Soil Box wall, two steel hose clamps, and a flexible rubber membrane. The rubber membrane was fastened to the rigid angle and the pipe-end via hose clamps.

After evaluating both design concepts, the second design concept was selected. Concerns arose with the cost associated with the first design concept due to the custom nature of the parts required for this type of system. Another issue with this design was that it would have required relatively large inflation pressures to achieve an adequate seal. These large pressures could cause a point of fixity at the pipe ends (as opposed to allowing for free deflection).

The second design concept was cost effective because only the rubber membrane required purchasing from an outside vendor. The second design concept also allows the pipe to deflect freely along its length.

2.2 Earth Pressure Cells

Earth pressure cells (Figure 2-5) were strategically placed throughout the Soil Box to measure the stress distribution within the soil during testing. Their locations can be seen in Figures 2-6 through 2-11.

2.3 Measurement of Pipe Deflection

Originally, an industry contact offered to provide a three dimensional digital measurement system to measure pipe deflection, but later informed researchers that it would not be available. Instead, a displacement laser, used on a previous FDOT/UF project, was provided by the FDOT State Material Office (SMO). This displacement laser (Figure 2-12) was mounted to a fixed trolley and directed through the pipe such that it measured vertical deflection during testing. The trolley system was fabricated at the University of Florida Coastal Lab and can be seen in Figures 2-13 and 2-14.

2.4 Loading Mechanism

The loading mechanism consisted of ten three-quarter inch thick steel plates that laid on the surface of the soil. These plates were overlain by pressurized lift bags that applied simulated overburden pressures (Figures 2-15 and 2-16).

2.5 Pipe Pre-deflection

The pipes were pre-deflected 4% of their nominal diameters by extending turnbuckles between two channels placed along the interior length of the pipe. As the turnbuckles were extended, measurements of the pipe diameter were taken until the diameter reached 4% deflection. This pre-deflection helped to promote deflection of the pipe during loading; without pre-deflection, it was unlikely that any noticeable deflection would be measured during testing.

2.6 Soil Saturation

To simulate variable high groundwater condition expected in Florida, the soil backfill was saturated with water and drained prior to loading. This allowed soil movement around the pipes's corrugations, haunches, and spring lines.

The fill material used for this project had a moisture content of approximately three percent. Using soil phase diagram relationships and dry unit weights obtained through triaxial testing, researchers estimated that 3,200 gallons of water would be required to achieve saturation.

The saturation procedure was completed using a lawn sprinkler. A trial was performed outside of the UF Coastal Engineering Laboratory to estimate the amount of time required for the lawn sprinkler to discharge 3,200 gallons. The footprint of the soil box was measured, outlined with orange chalk, and the lawn sprinkler was then adjusted until the water fell within the chalk boundary (Figures 2-17 and 2-18). The sprinkler was held over a container of known volume and its fill time measured. It took approximately 34 hours for the lawn sprinkler to discharge 3,200 gallons. At the time, the lawn sprinkler was thought to be the best way to uniformly distribute the water to the Soil Box. A French drain was placed in the Soil Box to allow drainage after saturation.

2.7 Boundary Friction

Boundary friction between the soil mass and side walls was minimized by placing two layers of lubricated polyethylene sheet in between the soil-wall interface (Figure 2-19).

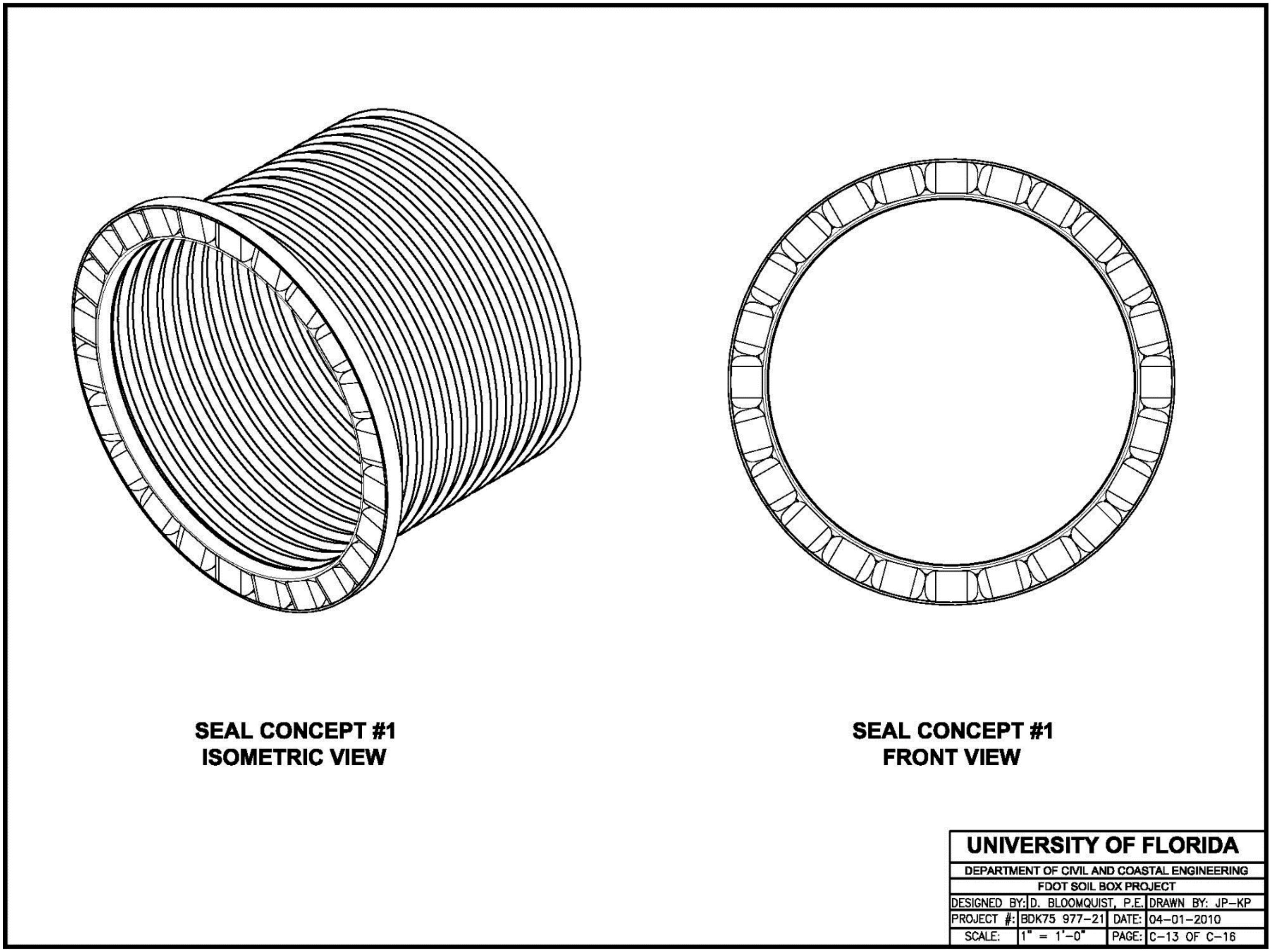


Figure 2-1. Seal concept #1, isometric and front views.

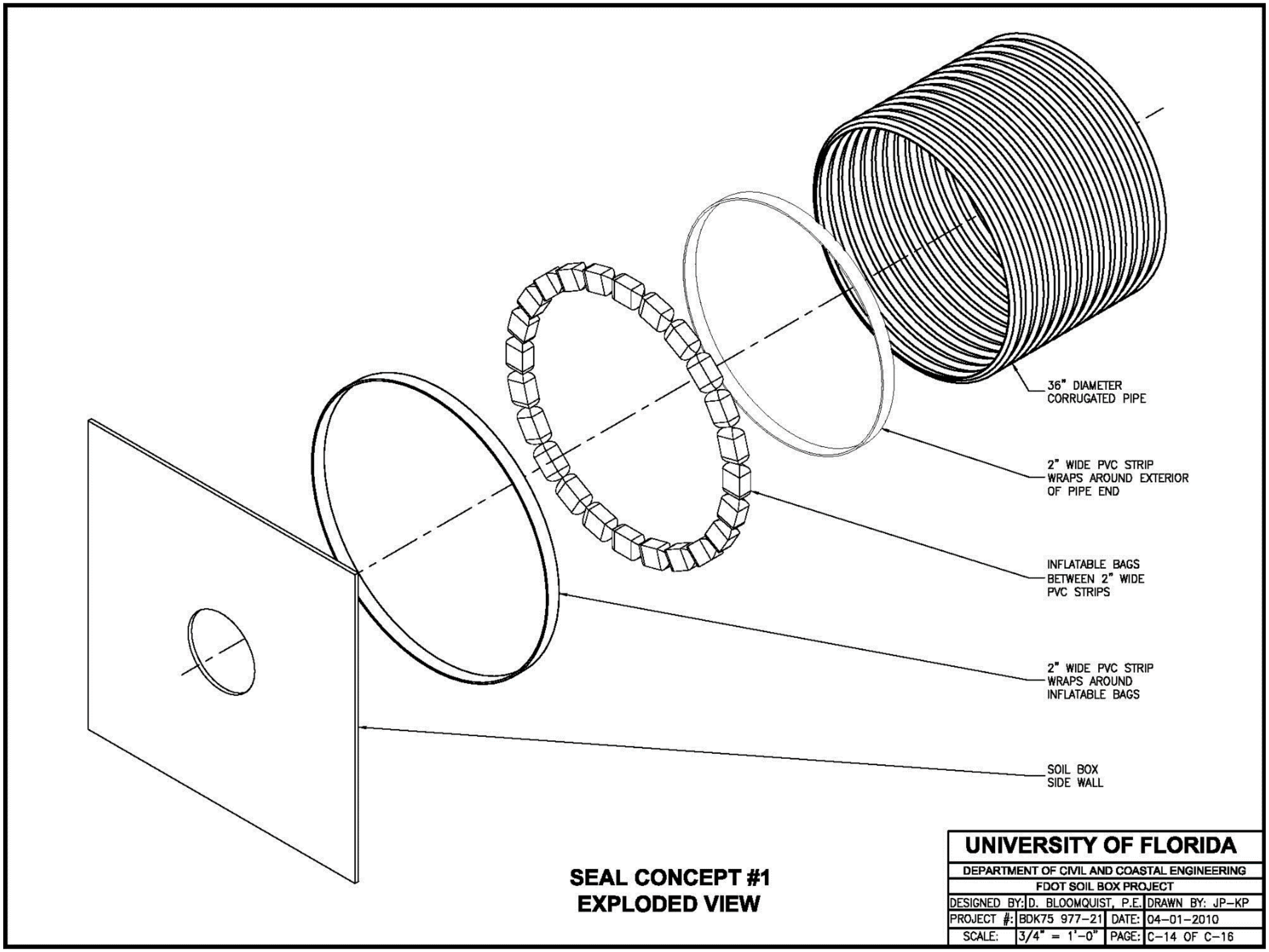


Figure 2-2. Seal concept #1, exploded view.

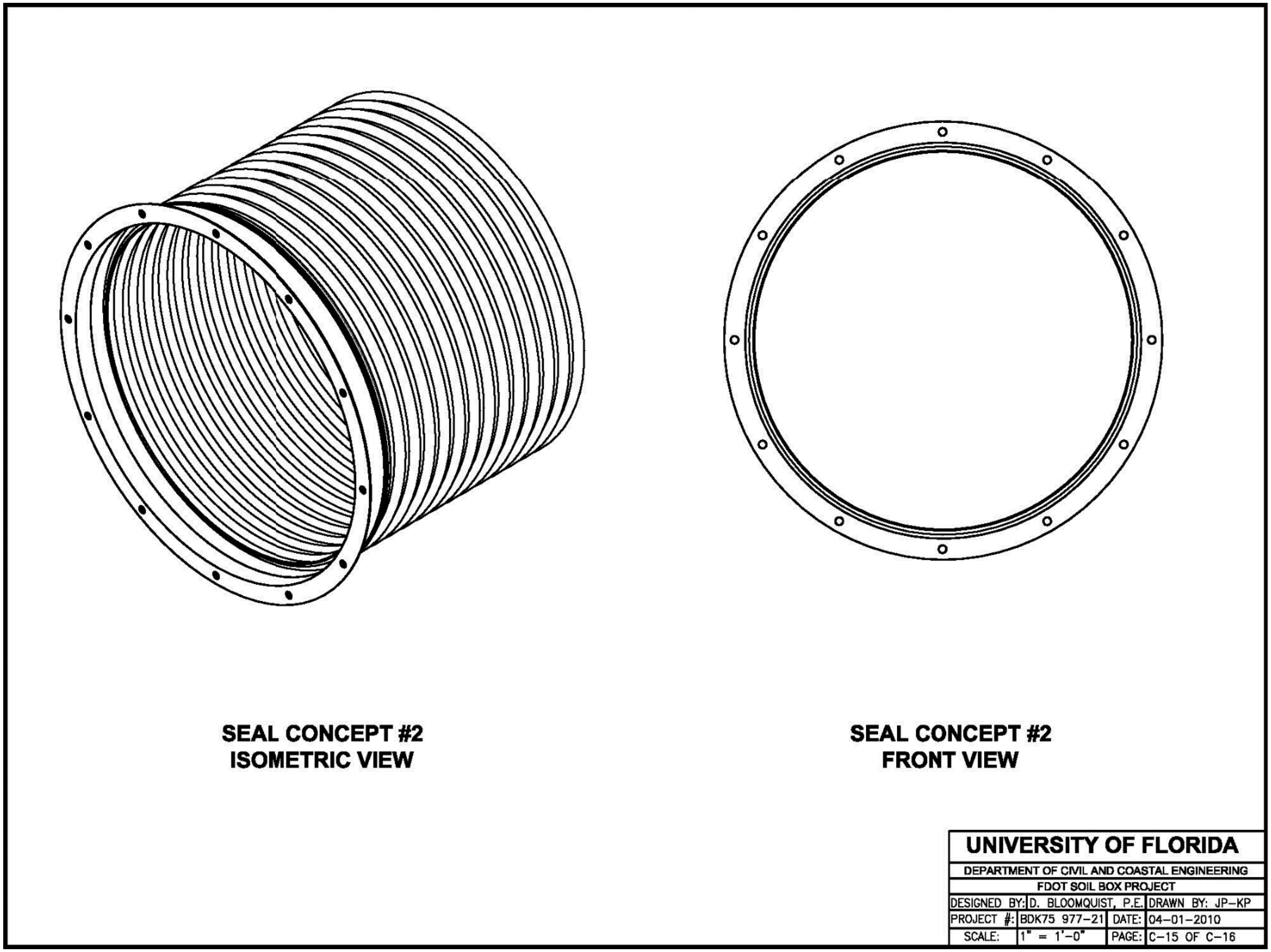


Figure 2-3. Seal concept #2, isometric and front views.

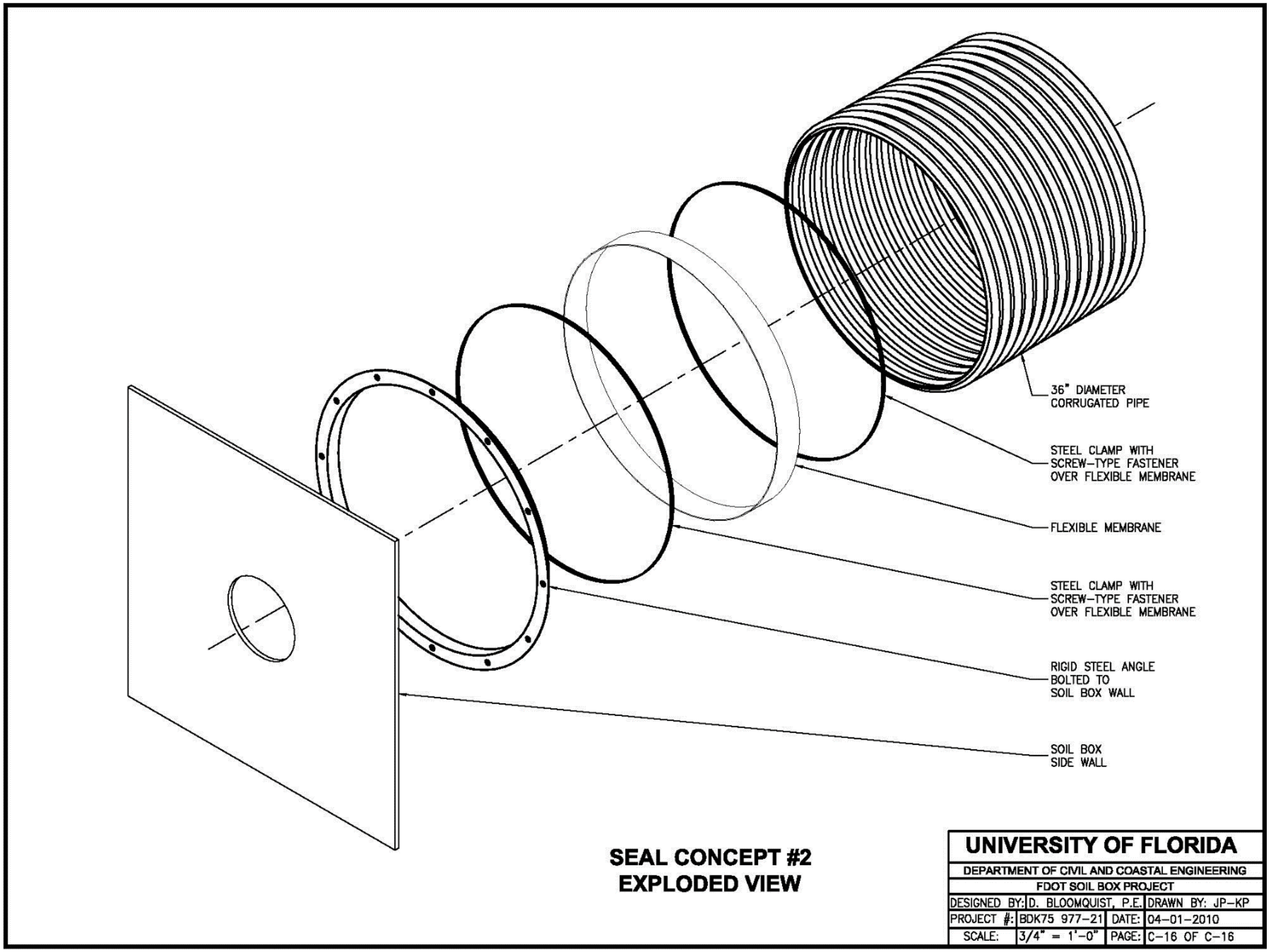


Figure 2-4. Seal concept #2, exploded view.

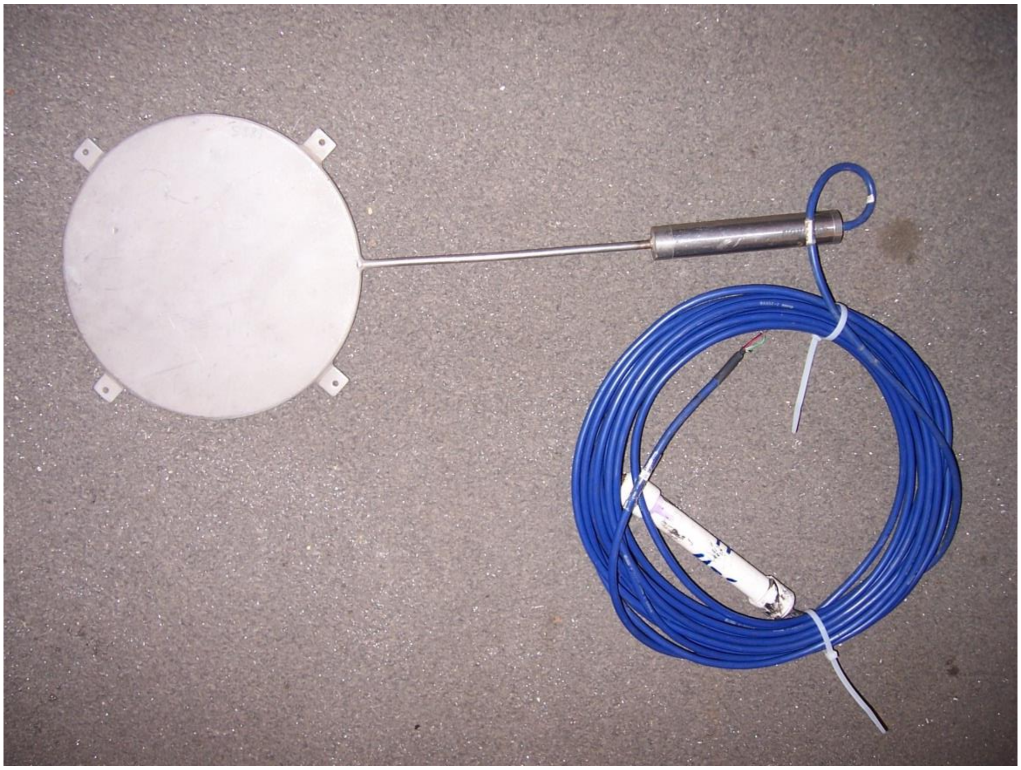


Figure 2-5. Earth pressure cell.

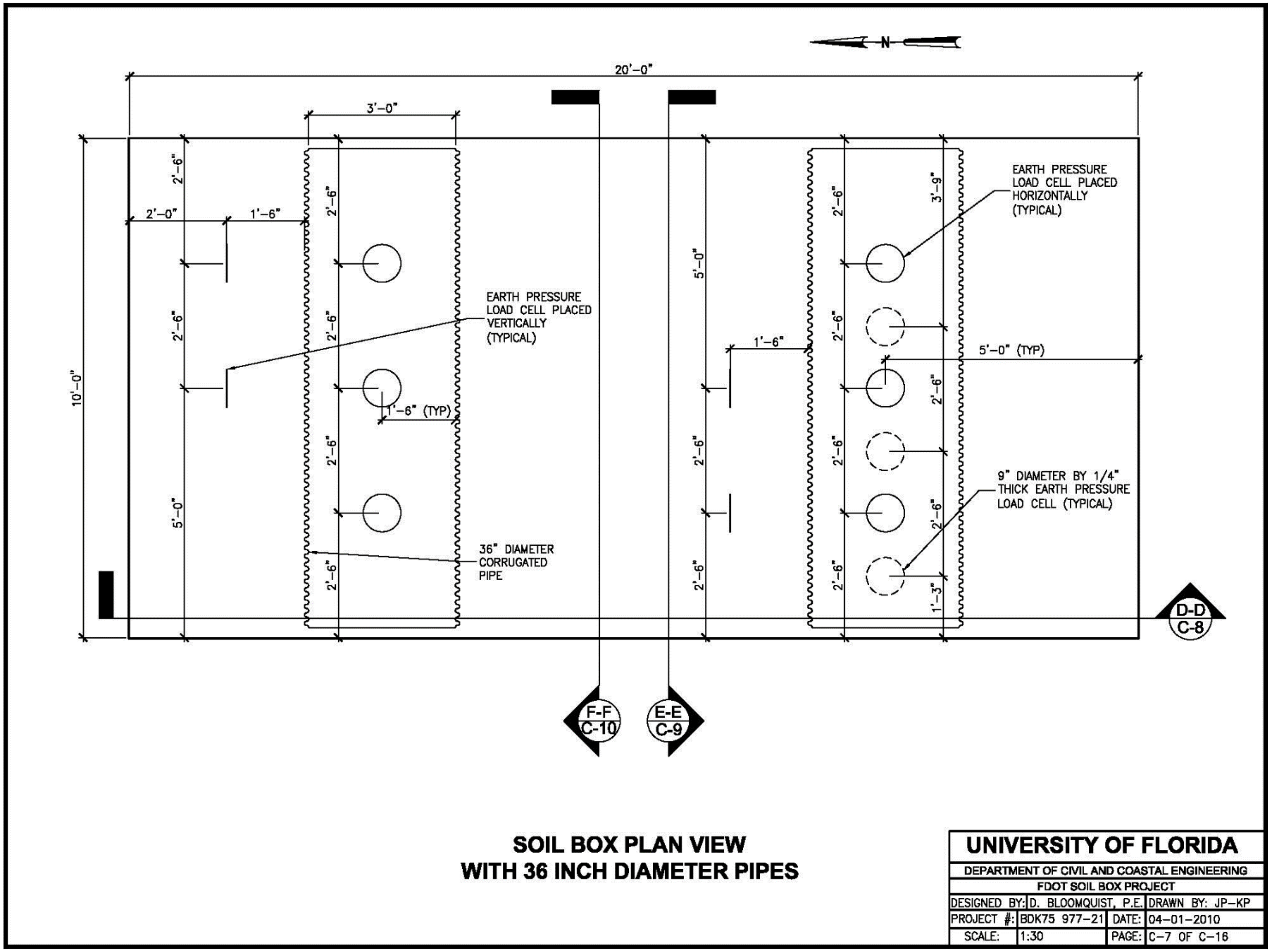


Figure 2-6. Earth pressure cell layout, plan view.

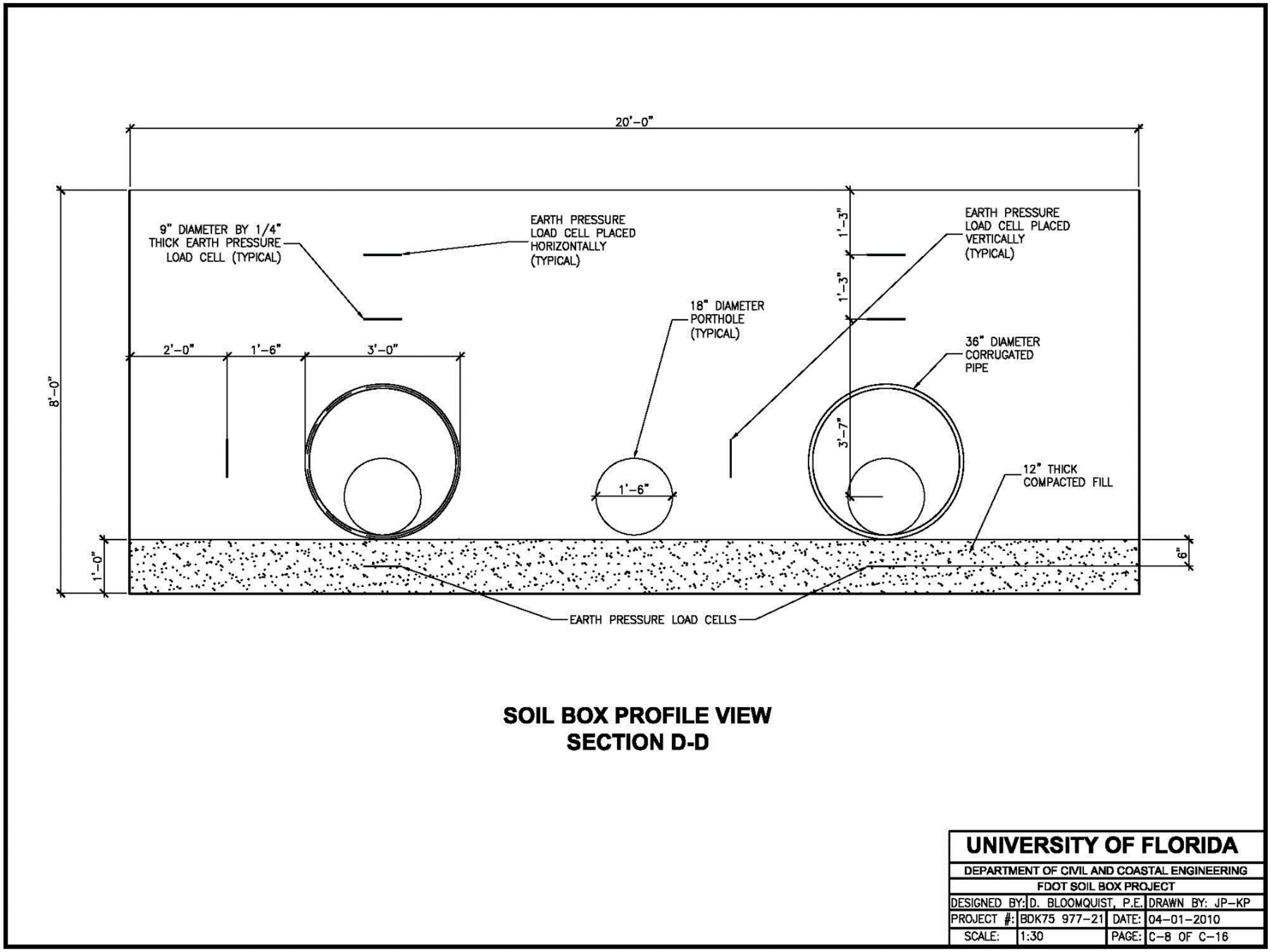


Figure 2-7. Earth pressure cell layout, section D-D.

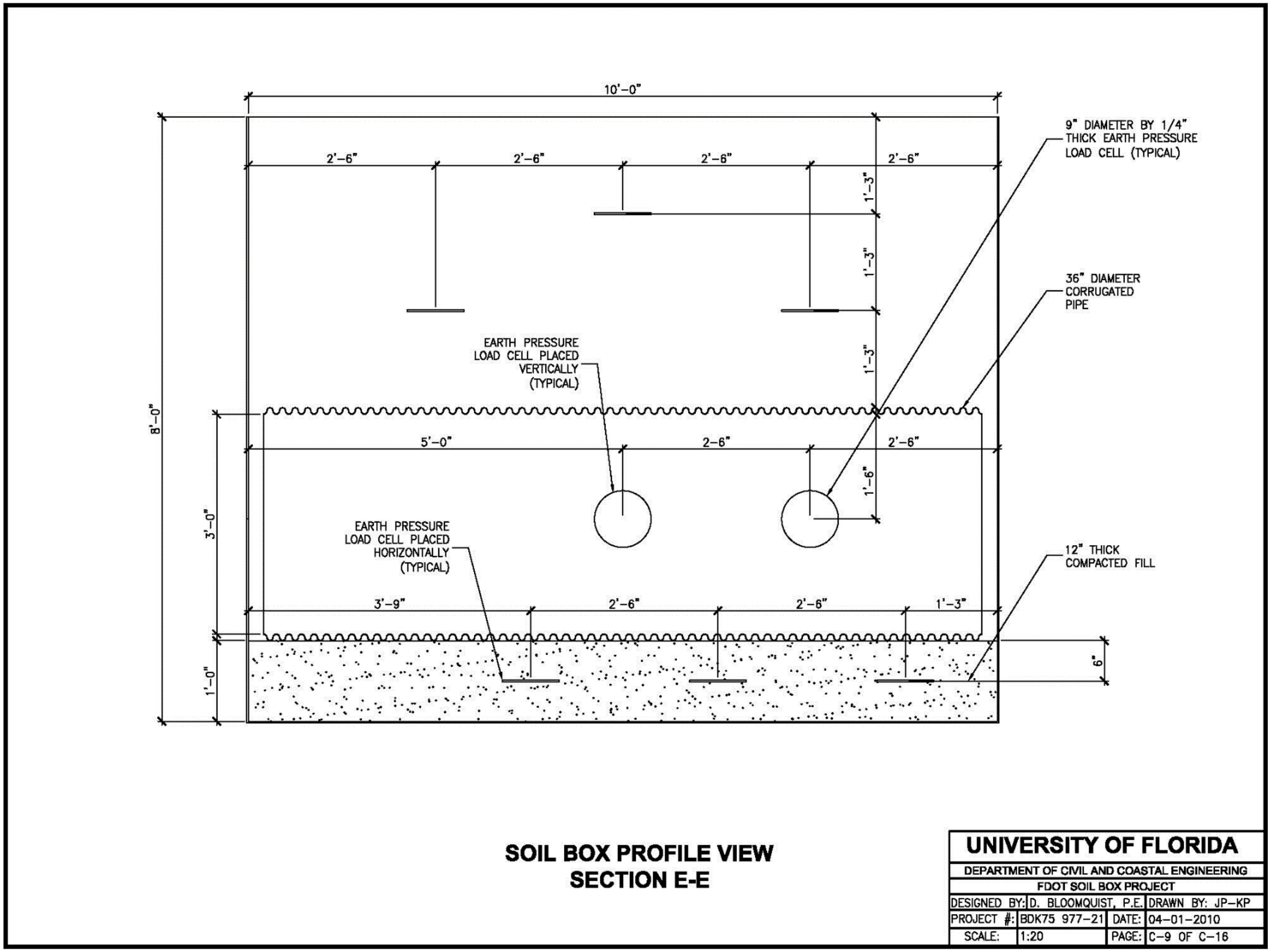


Figure 2-8. Earth pressure cell layout, section E-E.

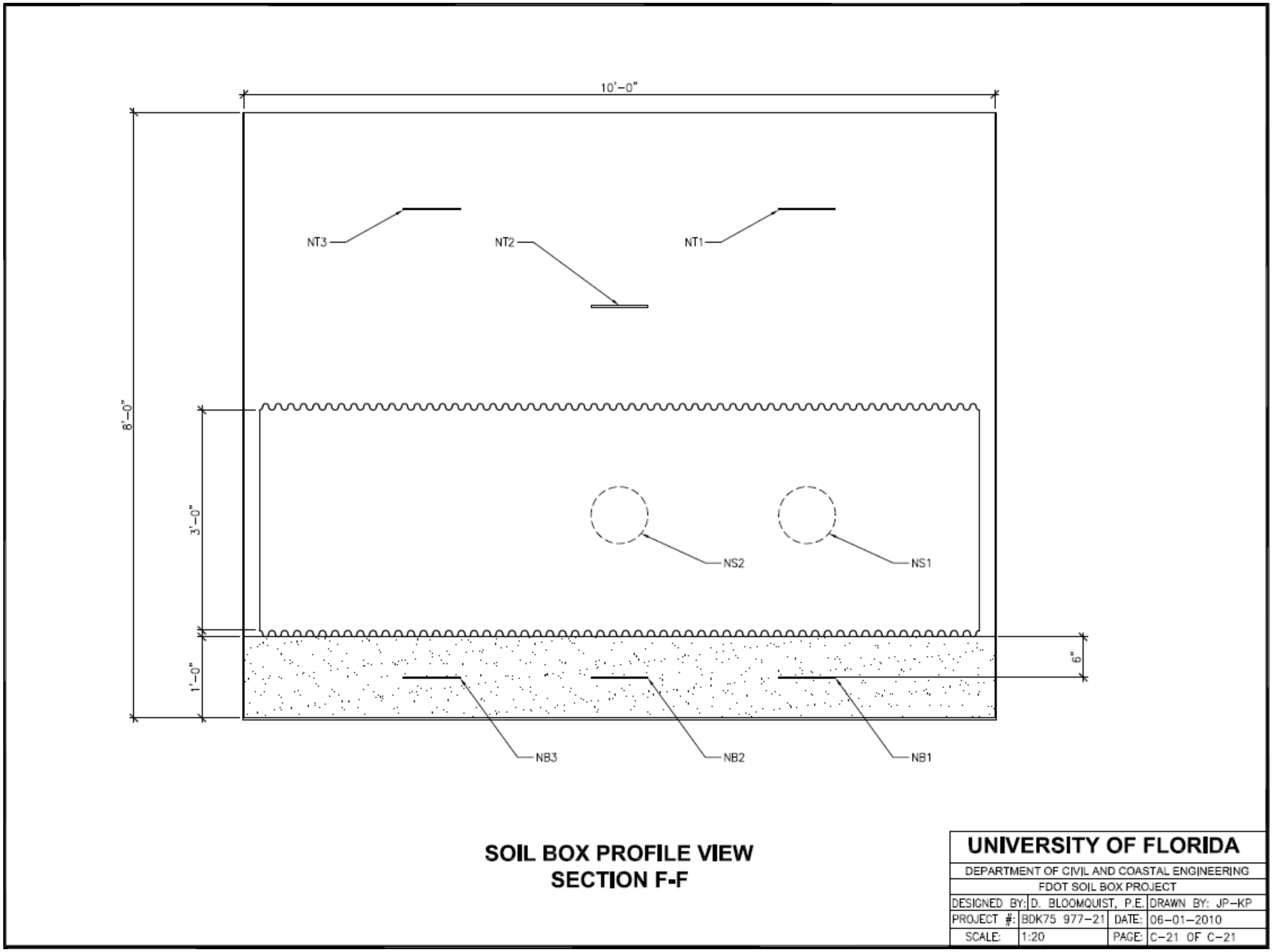


Figure 2-9. Earth pressure cell naming convention, section E-E.

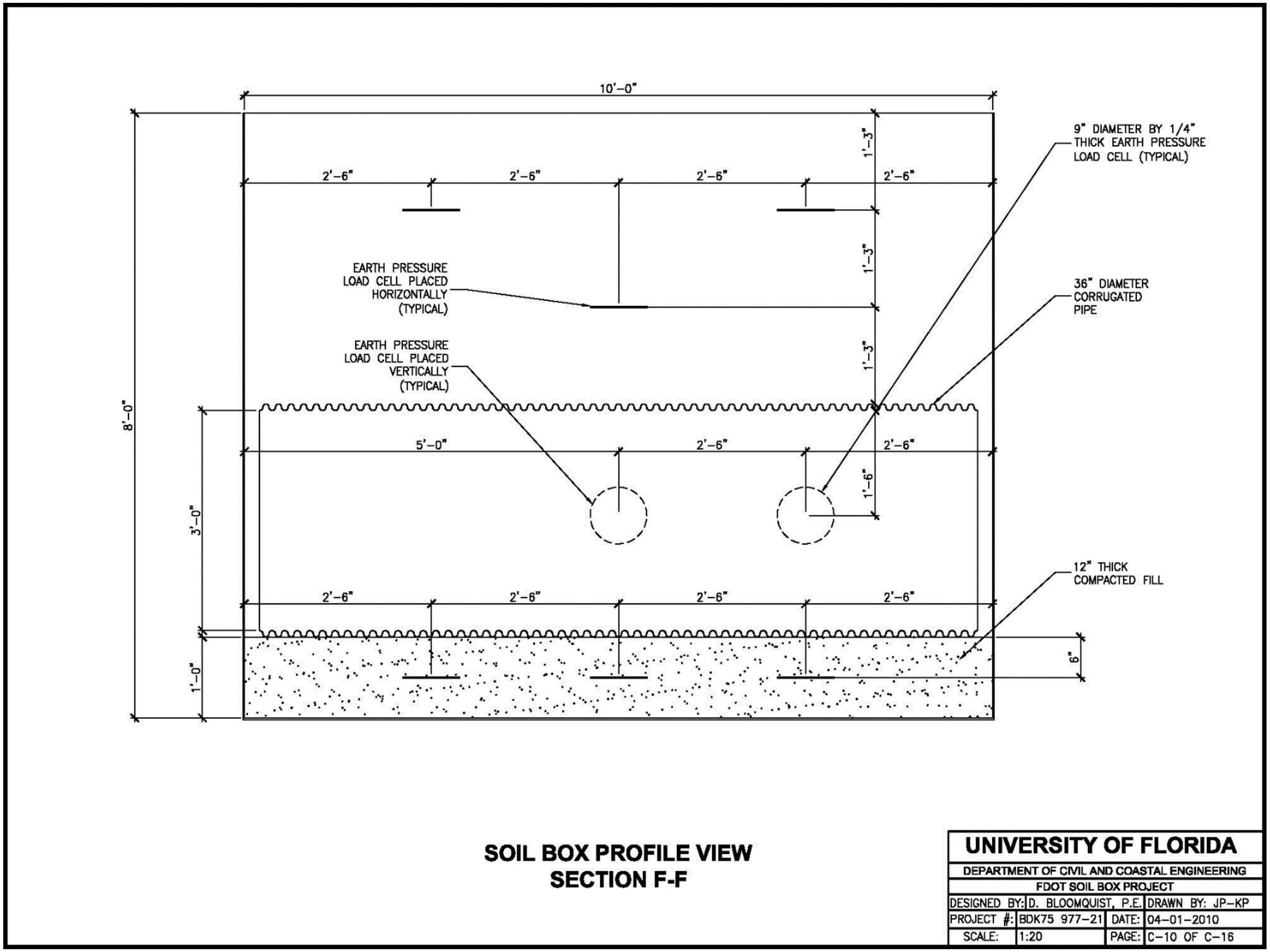


Figure 2-10. Earth pressure cell layout, section F-F.

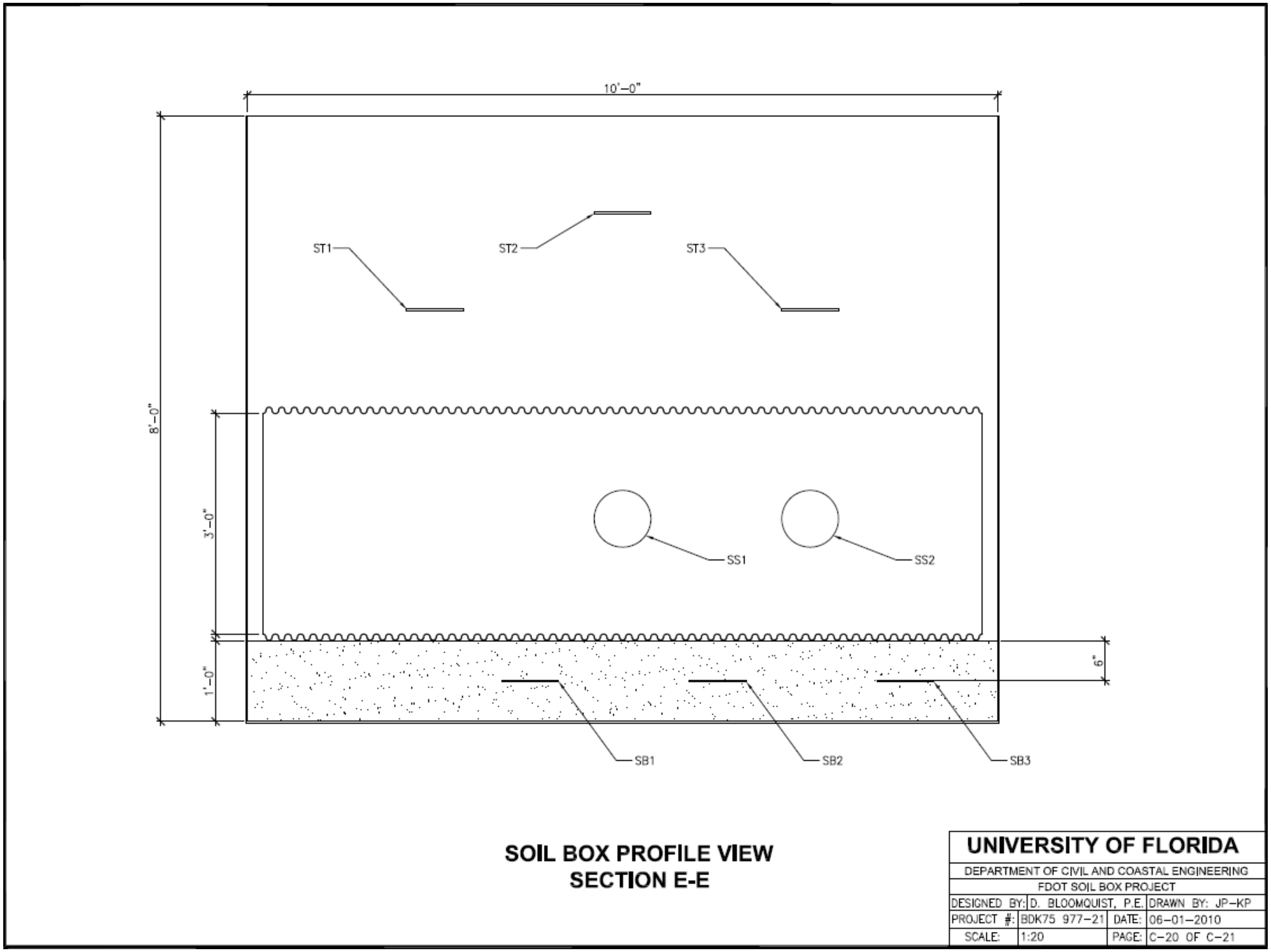


Figure 2-11. Earth pressure cell naming convention, section F-F.

Figure 2-12. Displacement laser and cabling.

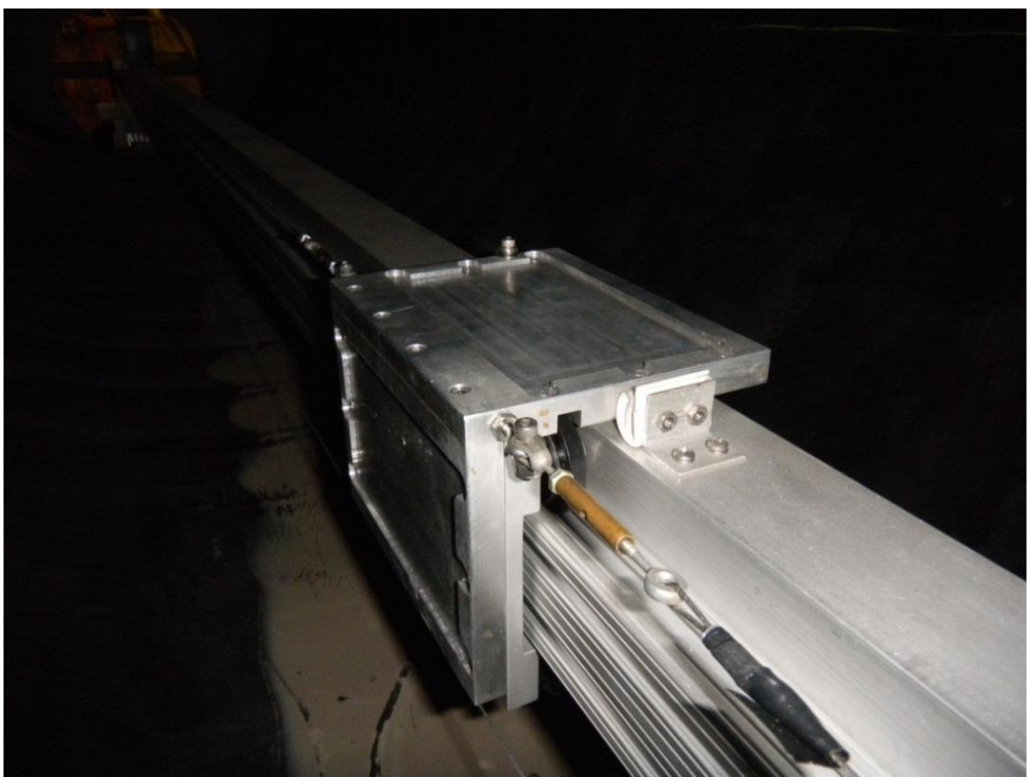


Figure 2-13. Laser mount on trolley system.

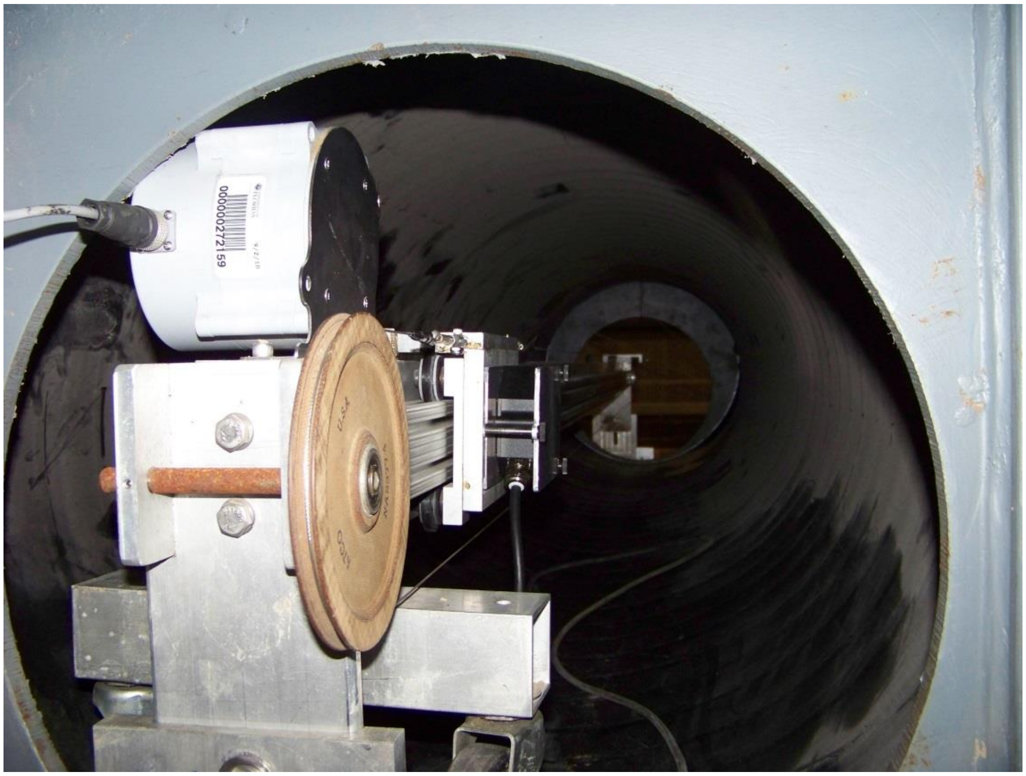


Figure 2-14. Laser trolley system.

Figure 2-15. Loading mechanism.

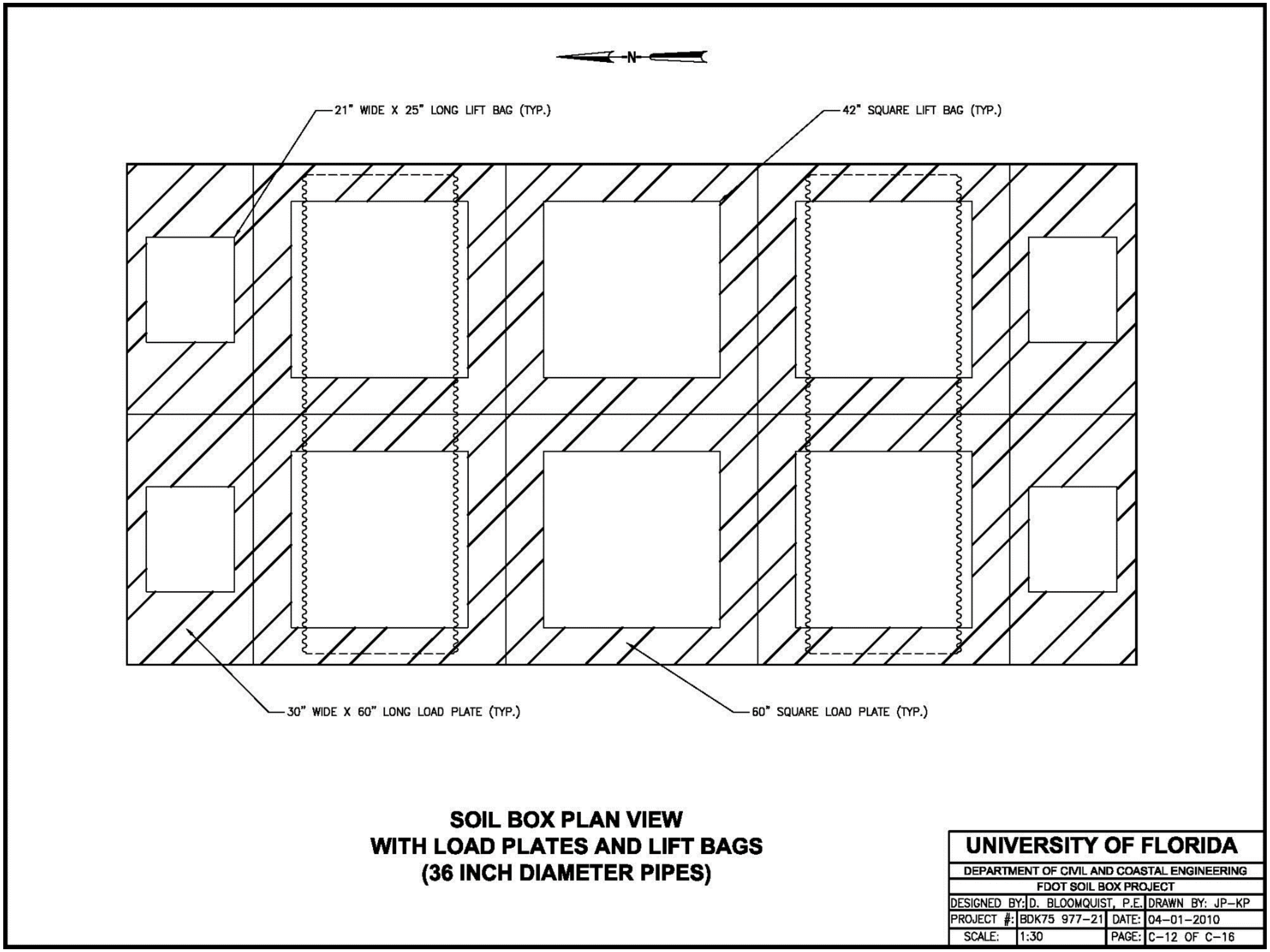


Figure 2-16. Plan view of loading mechanism.

Figure 2-17. Footprint of Soil Box outlined with chalk.

Figure 2-18. Adjusted water pressure to enclose Soil Box footprint.

Figure 2-19. Two layers of lubricated polyethylene sheets.

CHAPTER 3
36 INCH HDPE PIPE TEST

3.1 Pipe Preparation

The first pipes tested in the Soil Box were 36 inch HDPE pipes. The pipes were cut slightly smaller than the ten foot Soil Box width, and their ends were modified to enable assembly to the flexible membrane system by removing a single corrugation on each end of the HDPE pipes. The pipes were then pre-deflected (as discussed in Chapter 2) to approximately 4% (Figure 3-1).

3.2 Soil Box Preparation

First, the interior joints of the Soil Box were sealed with marine-grade sealant. Then, the rigid steel angles, were fastened to the Soil Box (Figure 3-2). Next, a polyethylene sheet was placed on the side walls, lubricated with silicone grease, and followed by a second polyethylene sheet.

A 12 inch layer of soil was placed into the Soil Box, leveled manually, and compacted with a vibratory plate compactor (Figure 3-3). Nuclear density tests were performed at multiple locations in the first layer for use in FEM analysis. The first set of earth pressure cells were then placed in their appropriate locations below the pipes. The pipes were then lifted into the Soil Box, rolled into their proper position, and the flexible membrane system was installed on each pipe end (Figure 3-4).

A concrete container was filled with soil, hoisted above the box, emptied into the box, and manually evenly distributed with shovels. This was repeated for three separate lifts. The earth pressure cells were placed while soil was added and nuclear density tests were performed after each lift. The Soil Box was then saturated and a final nuclear density test was conducted. Finally, the loading mechanism was installed (Figure 3-5).

After installing the loading mechanism, the three top Soil Box partitions were fastened to the Soil Box. Each section was hoisted onto the box via a lift truck and secured with nuts and bolts (Figure 3-6). The turnbuckles were removed and the laser profiling system was installed in each pipe (Figure 3-7).

3.3 Testing Procedure

The loading sequence was comprised of increments equivalent to a corresponding uniform surcharge. A summary of these increments and corresponding equivalent surcharges are displayed in Table 3-1. A maximum pressure of 66.67 psi was selected because of the capacity of the air compressor. As the load was increased, pipe deflection was monitored.

After the loading sequence was concluded, the unloading sequence began. This sequence consisted of reducing the pressure in the same increments in which it was increased. Throughout the loading and unloading sequence, pressure cell readings and soil deflection readings were recorded.

3.4 Discussion

The results for this test produced lesser simulated overburden pressures than expected. During the loading sequence, the lift bags were inflated, reducing the contact area between the lift bags and load plates. This reduction in contact area is believed to have decreased the simulated overburden pressure applied to the soil.

The existing air compressor (Figure 3-8) was replaced by a higher capacity air compressor (Figure 3-9) to aid in achieving larger simulated overburdens for future tests.

After the soil box was unloaded, two additional earth pressure cells were placed directly beneath the center load plates and the Soil Box was re-pressurized. The purpose of this was to more accurately assess the pressure applied to the soil (as opposed to just monitoring bag pressure). Ultimately, based upon these results, investigators concluded that a calibration test should be conducted.

Table 3-1. Loading sequence and simulated overburdens for 36 inch HDPE pipe test.

Lift Bag Pressure (psi)	Equivalent Simulated Surcharge (feet)	Measured Simulated Overburden on North Pipe (feet)	Measured Simulated Overburden on South Pipe (feet)
0	0	3.79	3.75
2	2.4	4.18	4.20
4	4.8	5.16	4.94
6	7.2	6.10	6.08
8	10	7.53	7.34
11	13.2	9.00	8.89
14	16.8	10.69	10.62
16.67	20	14.24	14.38
21.67	26	13.93	14.06
26.67	32	16.04	16.27
31.67	38	17.63	18.05
36.67	44	20.84	22.16
41.67	50	20.72	21.29
46.67	56	22.38	23.34
51.67	62	24.17	25.48
56.67	68	25.40	26.79
61.67	74	26.42	28.02
66.67	80	28.07	29.54

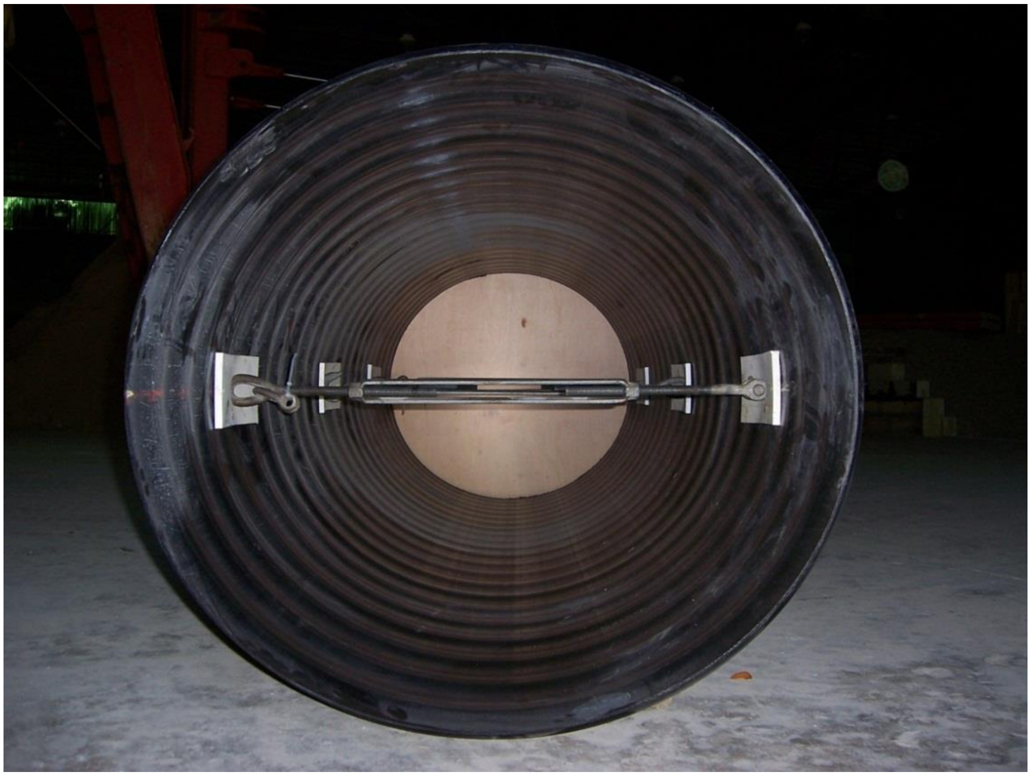


Figure 3-1. Pre-deflected HDPE pipe.

Figure 3-2. Rigid steel angle, part of the flexible membrane system.

Figure 3-3. 12 inch thick layer of compacted soil.

Figure 3-4. Installed flexible membrane system.

Figure 3-5. Installed loading mechanism.

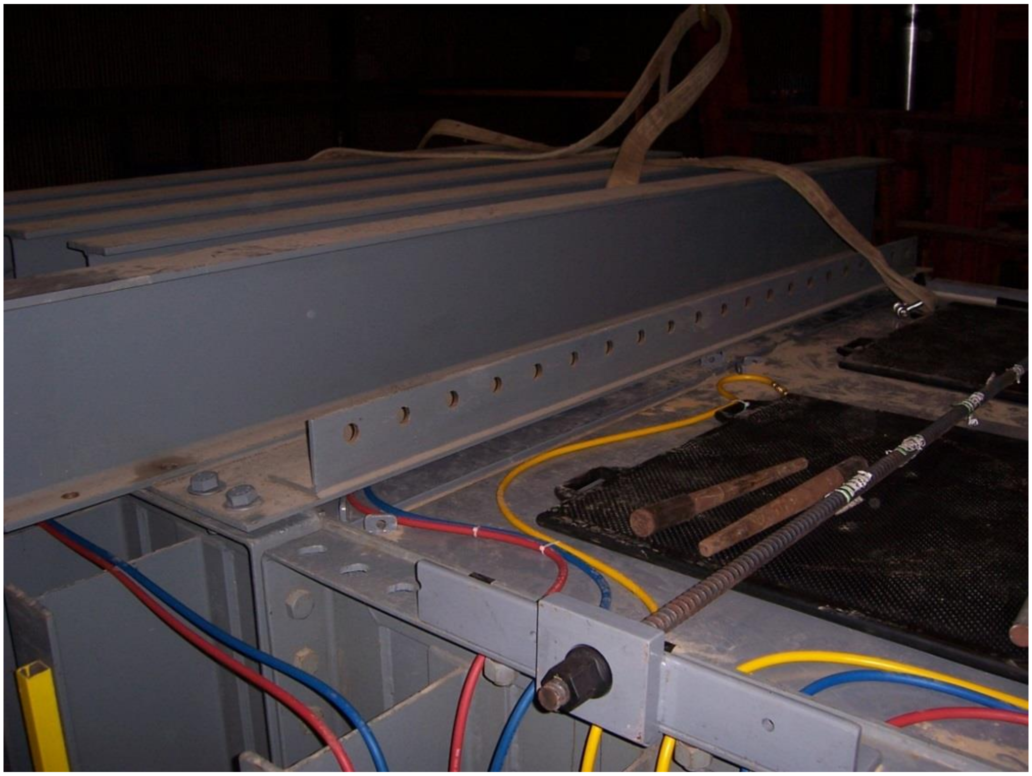


Figure 3-6. Top partition bolted onto the Soil Box.

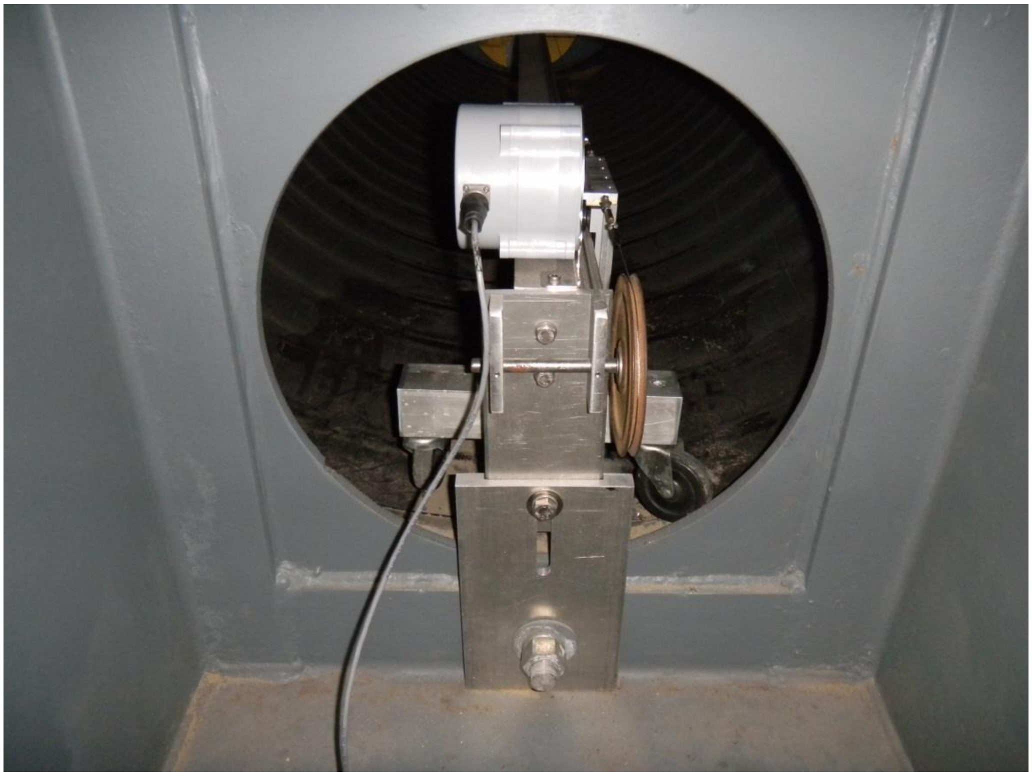


Figure 3-7. Installed laser profiling system.

Figure 3-8. Old, existing air compressor.

Figure 3-9. New, replacement air compressor.

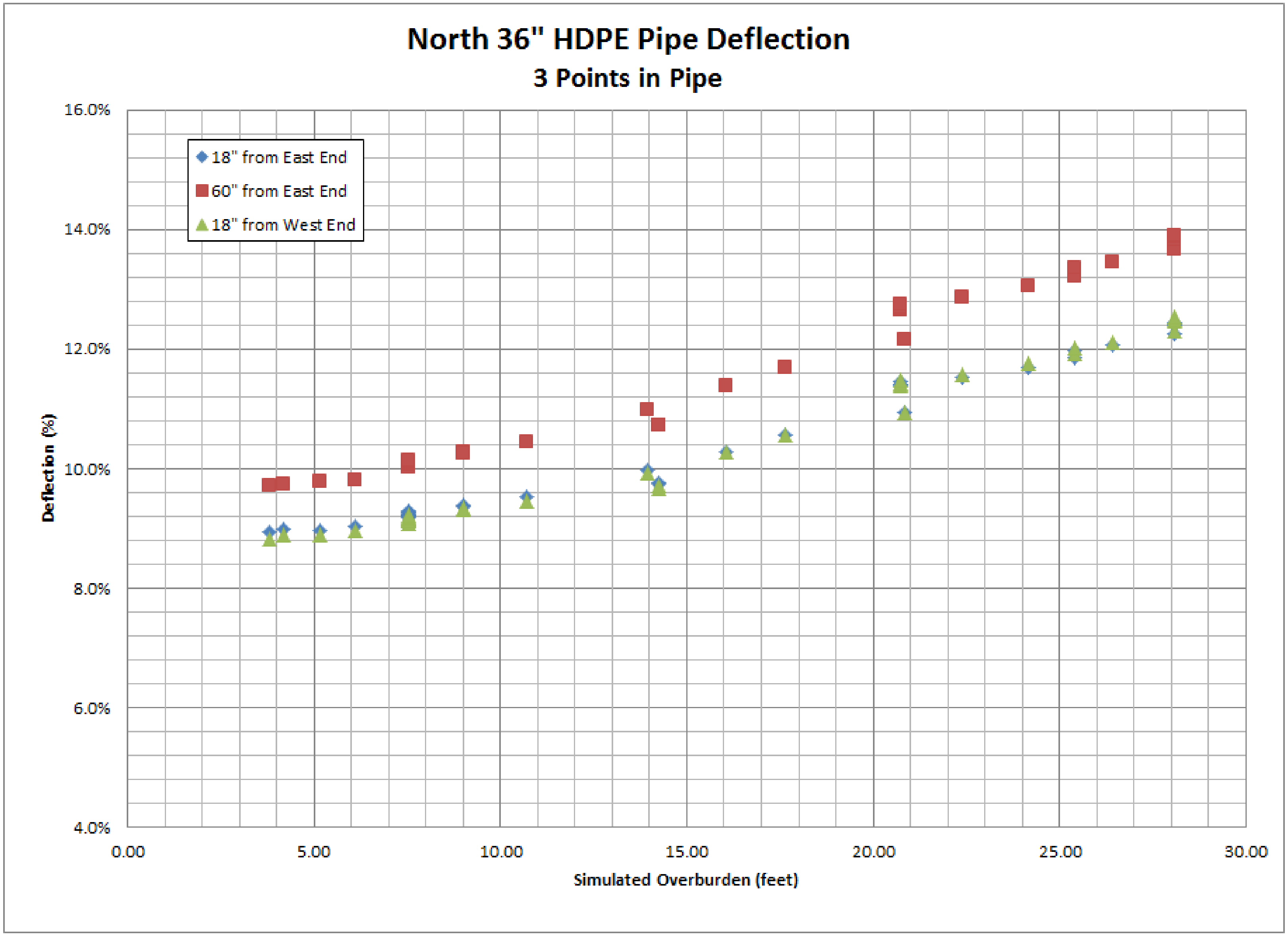


Figure 3-10. Three points in North 36" HDPE pipe, deflection versus simulated overburden.

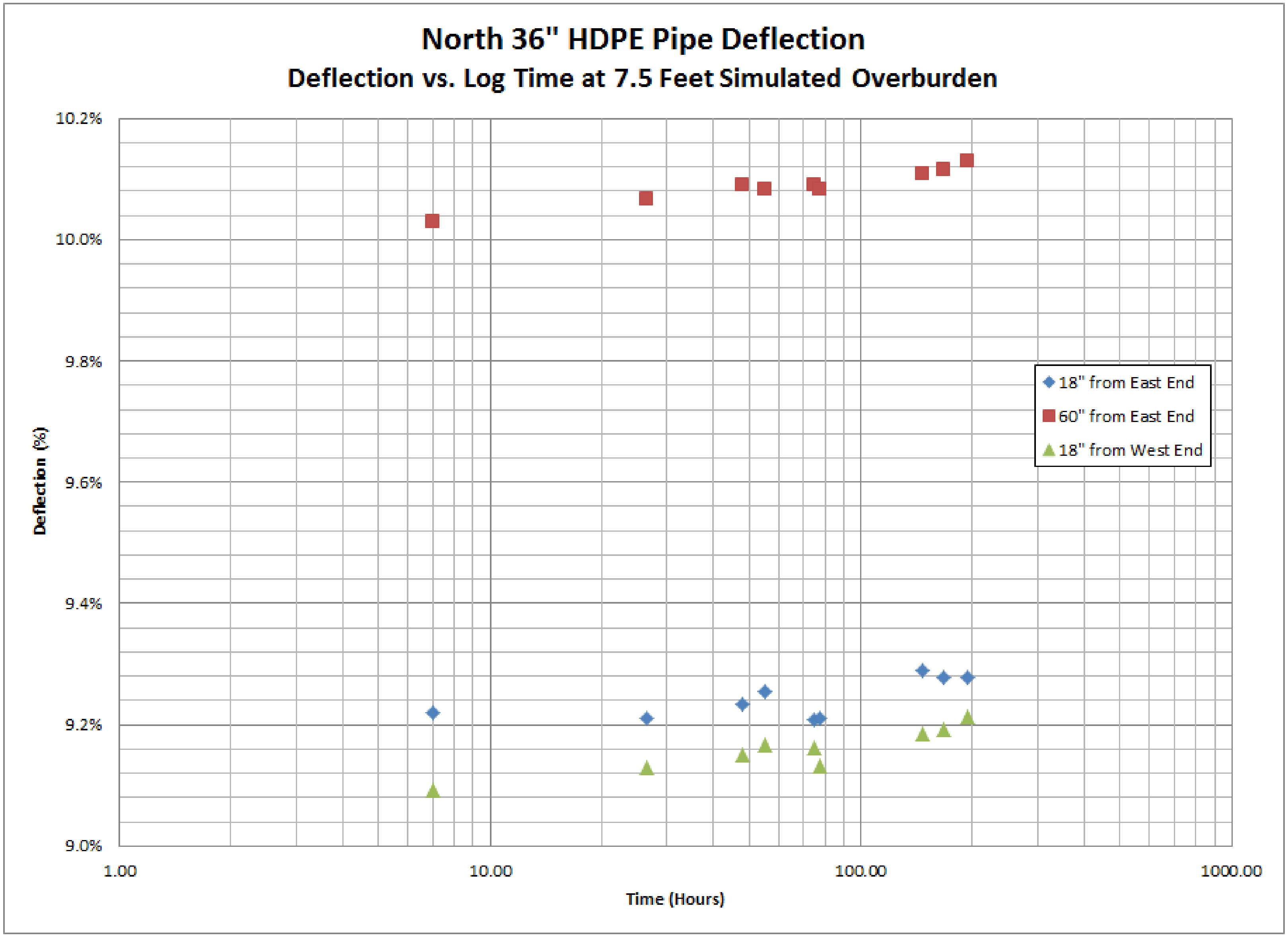


Figure 3-11. Deflection of North 36" HDPE pipe under 7.5 ft of simulated overburden over time.

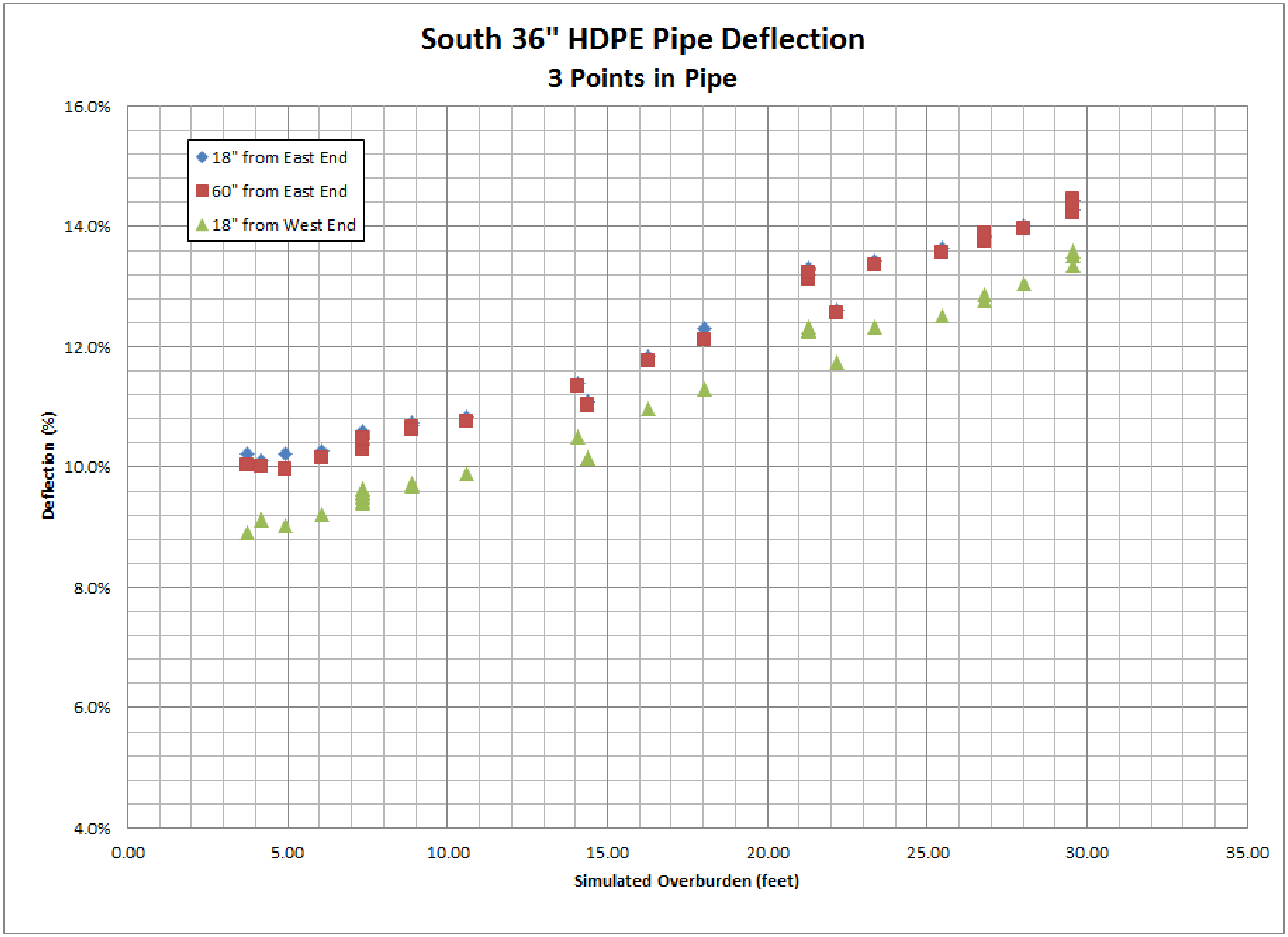


Figure 3-12. Three points in South 36" HDPE pipe, deflection versus simulated overburden.

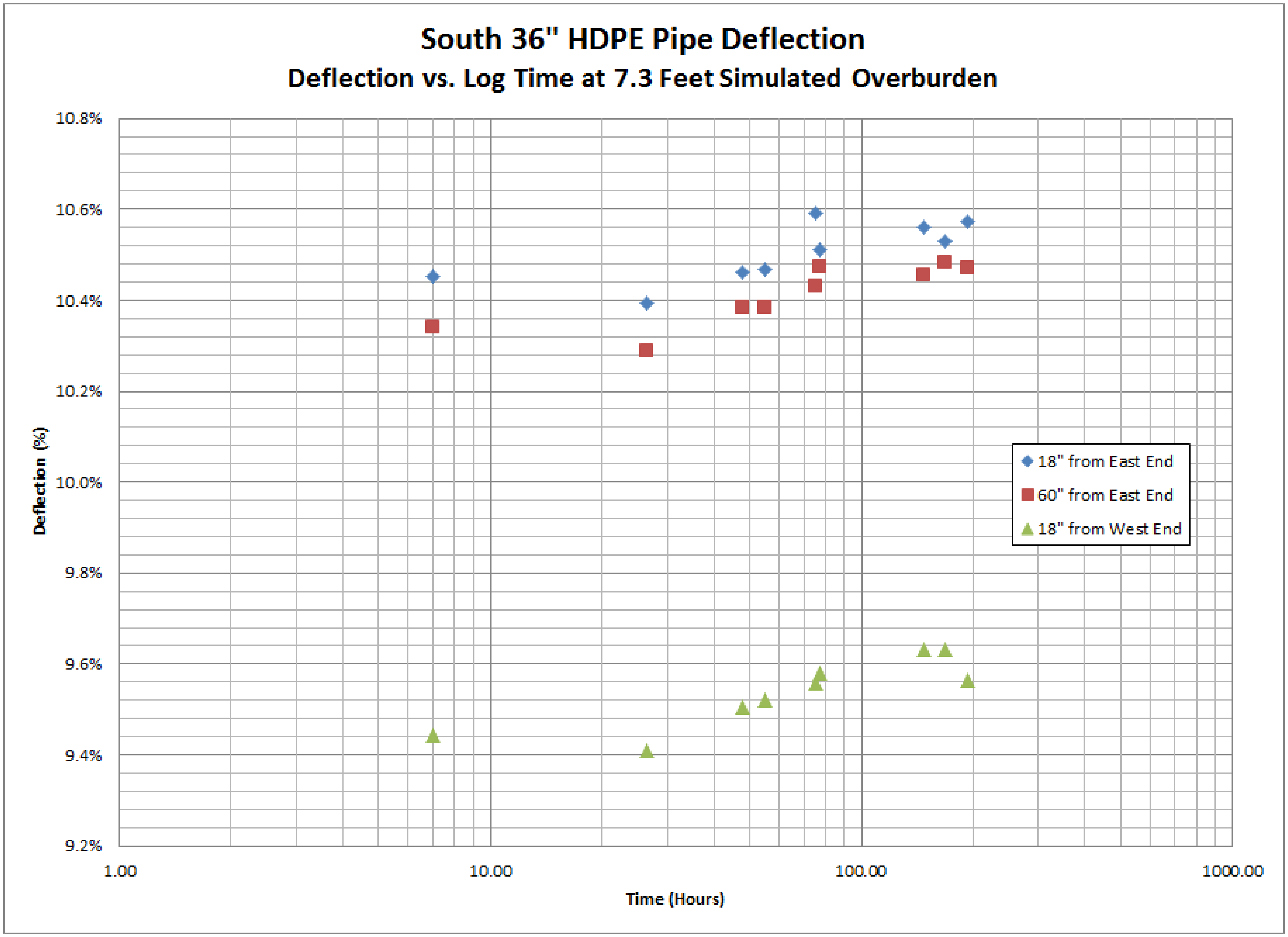


Figure 3-13. Deflection of South 36” HDPE pipe under 7.3 ft of simulated overburden over time.

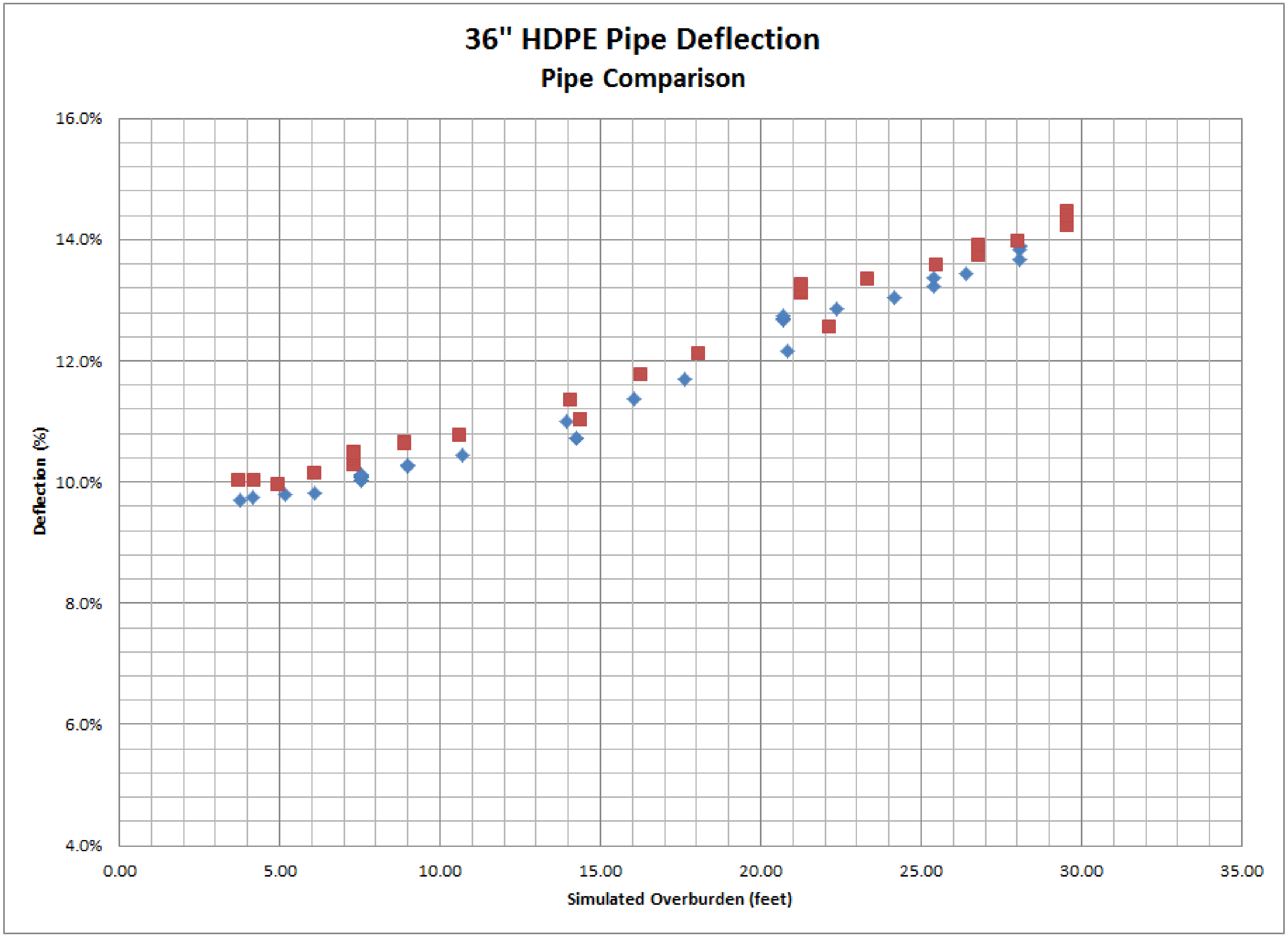


Figure 3-14. Deflection comparison of 36" HDPE pipes.

CHAPTER 4
CALIBRATION TEST

4.1 Design

As discussed in Chapter 3, a calibration test was performed to better assess the pressure distribution within the Soil Box. The pressure cell locations for this test can be seen in Figure 4-1. Ten pressure cells (not shown in Figure 4-1) were placed six inches below each loading plate to measure the stress transfer from the lift bags to the soil. The naming convention for the load plates and their corresponding earth pressure cells can is illustrated in Figure 4-2.

A separate regulator was connected to the smaller lift bags (Figure 4-3) to independently control their pressures. These lift bag pressures were adjusted to create a load similar to the load caused by the larger lift bags. The large lift bags have a larger footprint on the load plates and therefore a higher load for the same bag pressure. The separate regulator helped adjust for the reduction in the footprint of the smaller lift bags on the load plates and aided in creating a more uniformly distributed load across the soil surface.

Vertical displacement in the soil was measured for the calibration test. A settlement plate was placed at the location of each pipe invert. The plate was connected to a string potentiometer that recorded the vertical movement of the soil mass. The earth pressure cells were tested in an air-pressurized enclosure to confirm they were within factory calibration (Figure 4-4). To confirm the validity of the pressurized enclosure and the data-logging system, a new earth pressure cell was purchased and tested successfully.

4.2 Testing Procedure

The loading sequence was comprised of the same increments used for the 36-inch HDPE pipe test, but with maximum pressure increment of 101.67 psi. A summary of these increments are displayed in Table 4.1. After the loading sequence was concluded, the Soil Box was rapidly unloaded. Throughout the loading sequence, pressure cell readings and soil deflection readings were recorded.

4.3 Discussion

The pressure cell readings in the North end of the Soil Box were much greater than that of South end. This was attributed to a faulty hose fitting for the large lift bag located on plate 7 (Figure 4-2). The pressure readings under the small lift bags were consistently less than the readings under the large lift bags. This was most likely due to the difference in footprint between the large and small lift bags. To account for this, two small lift bags per small load plate were used for all subsequent tests (Figure 4-6 and Figure 4-7). Upon conclusion of the calibration test, a testing procedure was created for future testing. This procedure will be discussed in Chapter 5.

Table 4-1. Loading sequence for calibration test.

Large Lift Bag Pressure (PSI)	Small Lift Bag Pressure (PSI)
0	0
2	0
4	2
6	4
8	6
11	8
14	12
16.67	17
21.67	23
26.67	28
31.67	37
36.67	50
41.67	60
46.67	70
51.67	77
56.67	86
61.67	94
66.67	100
71.67	108
76.67	115
81.67	123
86.67	130
91.67	137
96.67	146
101.67	154

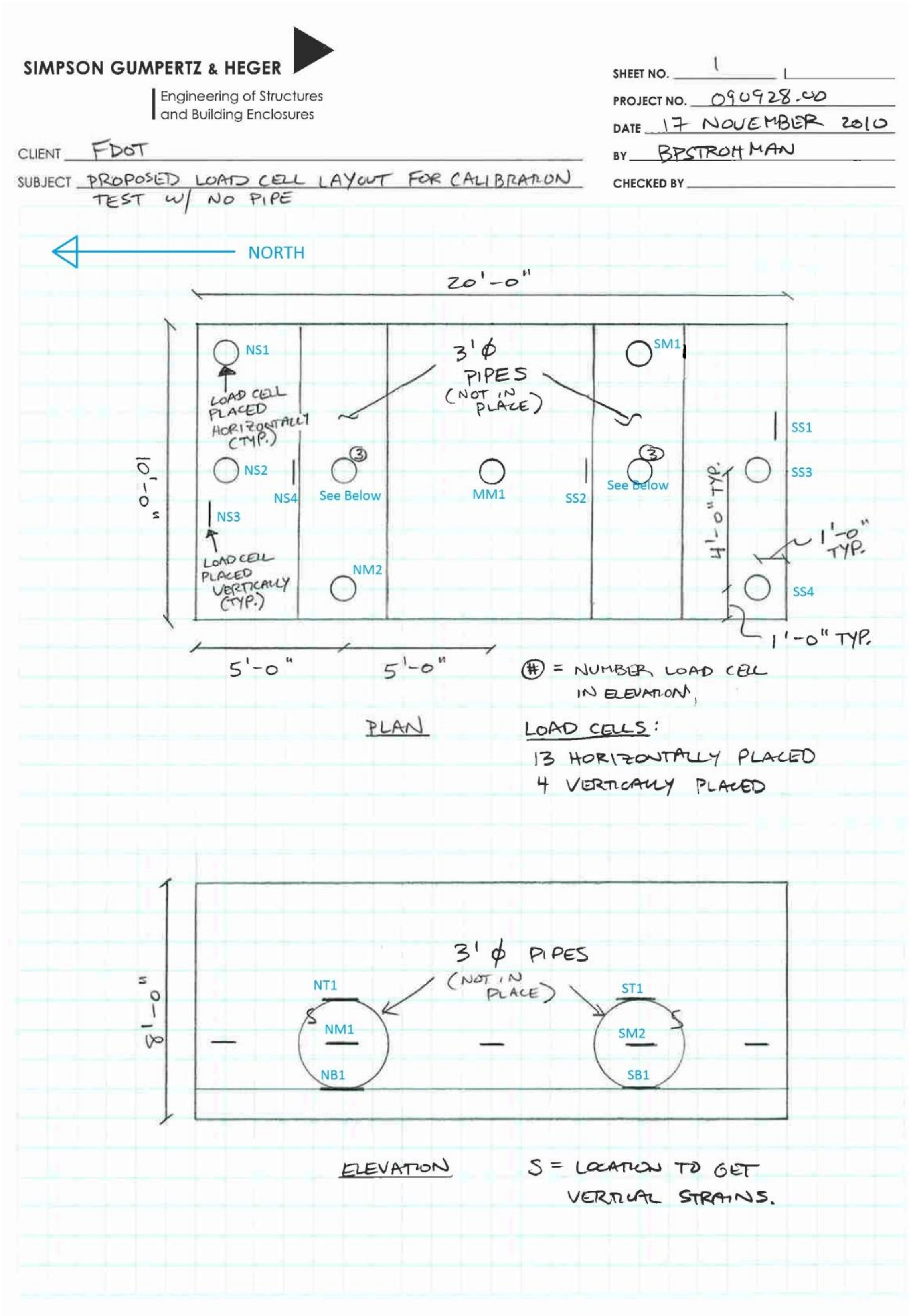


Figure 4-1. Earth pressure cell layout for calibration test.

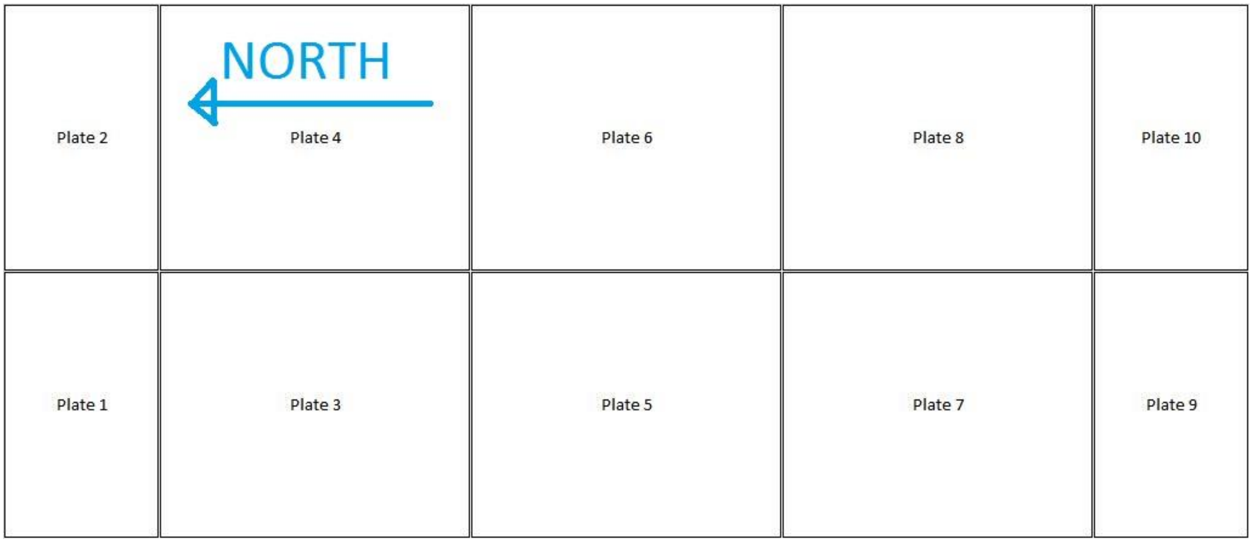


Figure 4-2. Load plate naming convention.

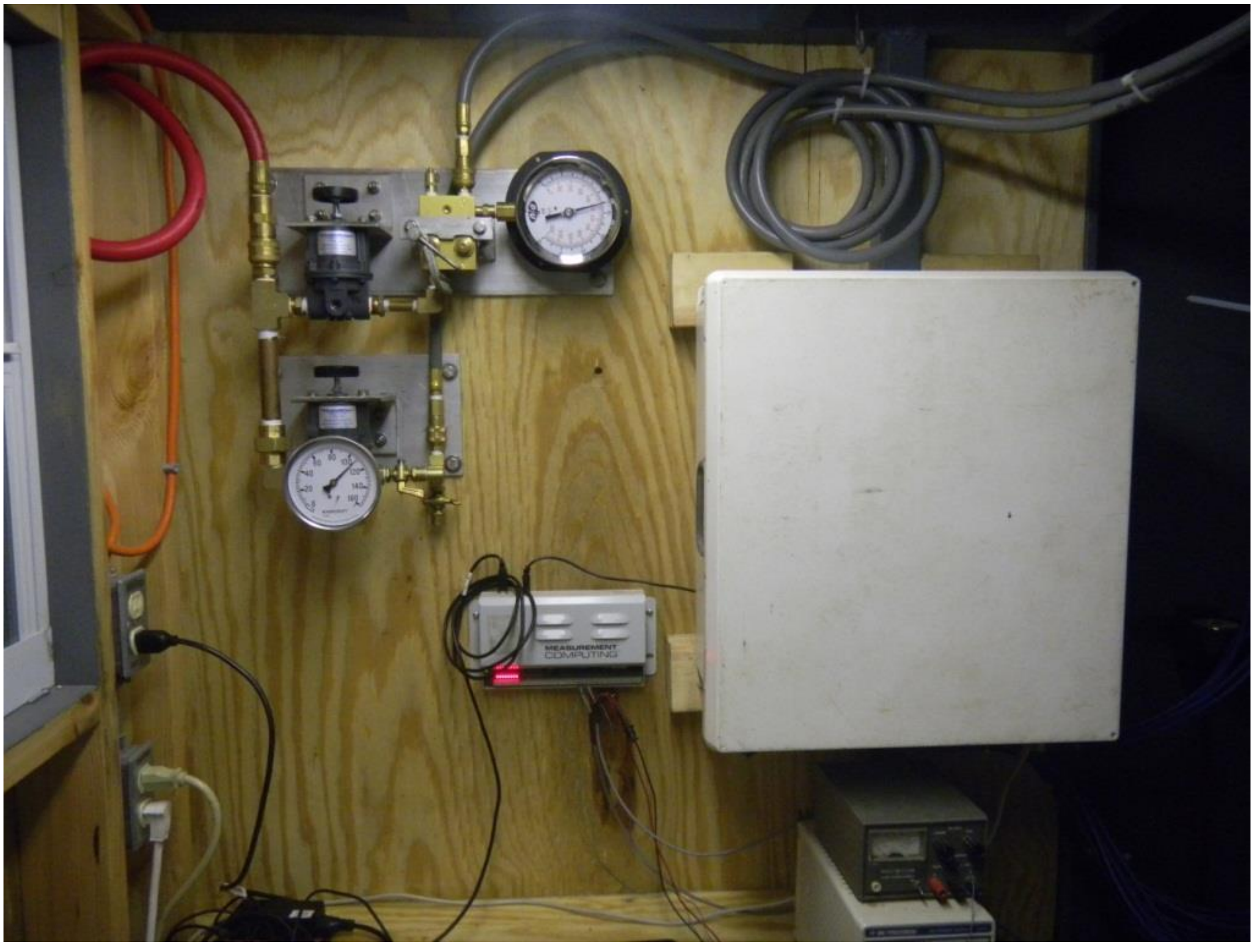


Figure 4-3. Additional regulator for smaller lift bags.

Figure 4-4. Earth pressure cells inside pressure vessel.

Figure 4-5. Pressure vessel checking factory calibration of pressure cells.

Figure 4-6. Addition of one small lift bag to small load plate.

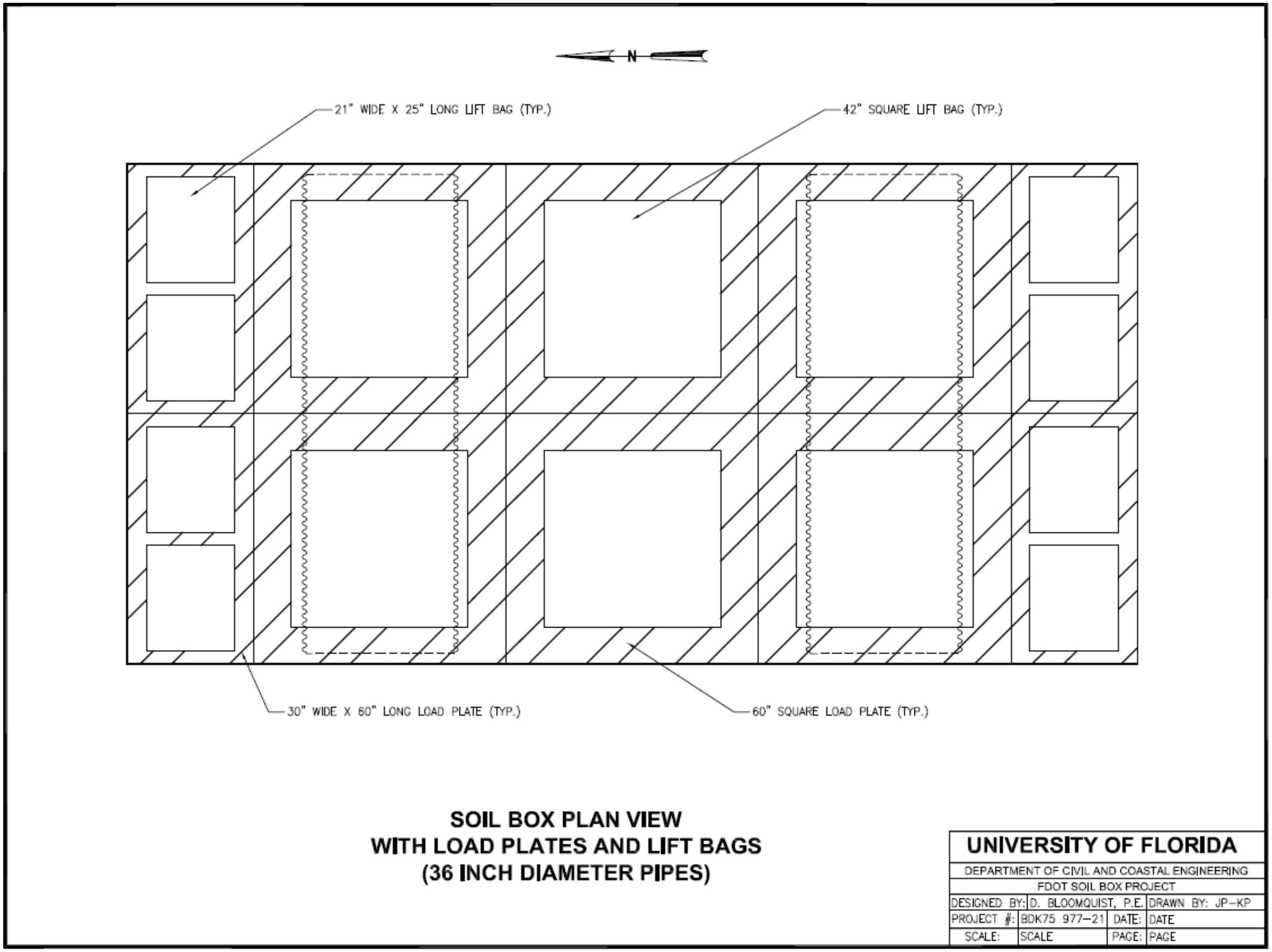


Figure 4-7. Adjusted loading mechanism.

CHAPTER 5
PROCEDURE

5.1 Pipe Preparation

As discussed in Chapter 4, based upon results from the calibration test, a new testing procedure was developed for all subsequent tests. Pipe preparation was almost identical to the method described in Chapter 2. For completeness, the full procedure is repeated below:

The pipes were cut slightly smaller than the ten foot Soil Box width, and their ends were modified to enable assembly to the flexible membrane system. The details of each pipe-end modification are discussed in the chapters that follow. The pipes were pre-deflected 4% of their nominal diameters by extending turnbuckles between two channels placed along the interior length of the pipe. As the turnbuckles were extended, measurements of the pipe diameter were taken until the diameter reached 4% deflection. An example pre-deflected pipe can be seen in Figure 5-1.

5.2 Soil Box Preparation

The Soil Box was prepared almost identically to the procedure described in Chapter 2 as well. The difference between this procedure and the procedure described in Chapter 2 was the locations of the large versus small lift bags. Again, the full procedure is repeated below:

First, the interior joints of the Soil Box were sealed with marine-grade sealant. The rigid steel angles, as part of the flexible membrane system, were then fastened to the Soil Box (Figure 5-2). Next, a polyethylene sheet was placed on the side walls, lubricated with silicone grease, and followed by a second polyethylene sheet (Figure 5-3).

A 12 inch layer of soil was placed into the Soil Box via a front-end loader (Figure 5-4). The soil was leveled manually and compacted with a vibratory plate compactor. The soil during and after compaction are shown in Figures 5-5 and 5-6, respectively. Nuclear density tests were performed at multiple locations in the first layer for use in FEM analysis. The first set of earth pressure cells were then placed in their appropriate locations below the pipes.

The pipes were placed in the Soil Box via fork lift, rolled into their proper position, and the flexible membrane system is installed on each pipe end (Figures 5-7 and 5-8). The Soil Box end partition was then fastened to the Soil Box and its joints were sealed as shown in Figure 5-9. Two lubricated polyethylene sheets were placed on the inside wall partition, and the remaining soil for the first 12 inch layer was placed and manually compacted.

A concrete bucket was filled with soil, hoisted above the box, emptied into the box, and manually evenly distributed with shovels (Figures 5-10 and 5-11). This was repeated in 18 inch lift-increments until the Soil Box was filled. The earth pressure cells were placed while soil was added and nuclear density tests were performed after each lift. The locations of the nuclear density tests and earth pressure cells are displayed in chapters that follow. The Soil Box was then saturated and followed by a final nuclear density test. After saturation, the loading mechanism was installed (Figure 5-12).

After installing the loading mechanism, the three top Soil Box partitions were fastened to the Soil Box. Each section was hoisted onto the box via a lift truck and secured with nuts and bolts (Figure 5-13). The turnbuckles were removed and the laser profiling system was installed in each pipe (Figure 5-14).

5.3 Testing Procedure

The loading sequence was comprised of five pound per square inch increments. Each increment was held for one hour before deflection readings were recorded. The increments were increased until they reached 130 pounds per square inch because this was the maximum amount of pressure the lift bags could tolerate without failure (note, these pressures are much higher than the pressures described in Chapter 2). For every third loading increment (45 pounds per square inch, 90 pounds per square inch, and 130 pounds per square inch, respectively), the load was held for 24 hours and deflection readings were taken at one hour, four hour, eight hour, and 24 hours to test the creep characteristics of the pipes. Table 5-1 displays the loading sequence and deflection reading times. As the load was increased, pipe deflection was monitored, and the lift bag regulators were adjusted to maintain uniform deflection along the pipes' length.

After the loading sequence was concluded, the unloading sequence began. This sequence consisted of reducing the pressure in the same increments in which it was increased. Each increment was held for an hour. Throughout the loading and unloading sequence, pressure cell readings and soil deflection readings were recorded.

Table 5-1. Loading sequence and deflection reading timing.

Pressure Applied to Main Regulator (PSI)	Deflection Readings taken x hours after pressure is applied
0	1
5	1
10	1
15	1
20	1
25	1
30	1
35	1
40	1
45	1, 4, 8, 24
50	1
55	1
60	1
65	1
70	1
75	1
80	1
85	1
90	1, 4, 8, 24
95	1
100	1
105	1
110	1
115	1
120	1
125	1
130	1, 4, 8, 24

Figure 5-1. Example pipe pre-deflected to 4% of its nominal diameter.

Figure 5-2. Side wall flange, as part of the flexible membrane system.

Figure 5-3. Installed lubricated polyethylene sheets.

Figure 5-4. Soil being placed with front end loader.

Figure 5-5. First lift during compaction.

Figure 5-6. First lift after compaction.

Figure 5-7. Properly positioned pipe before flexible membrane system installation.

Figure 5-8. Installed flexible membrane system.

Figure 5-9. Fastening of Soil Box end partition.

Figure 5-10. Filling concrete bucket with front end loader.

Figure 5-11. Soil being hoisted into Soil Box.

Figure 5-12. Loading mechanism.

Figure 5-13. Top partition hoisted onto Soil Box via lift truck.

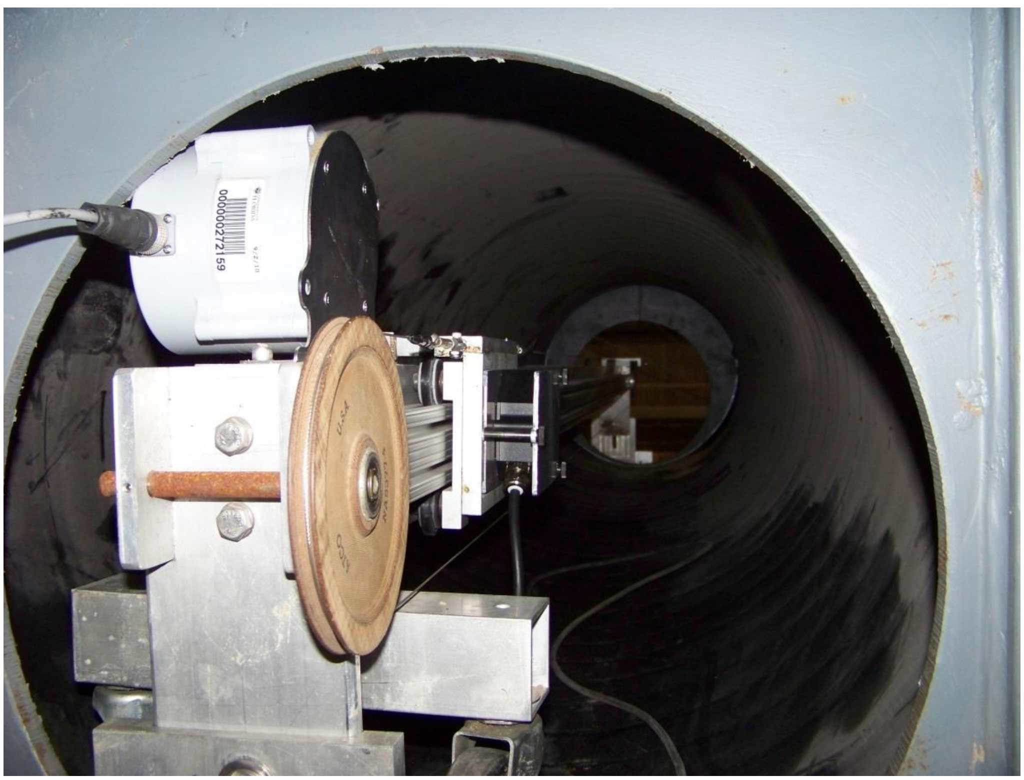


Figure 5-14. Installed laser profiling system.

CHAPTER 6
36 INCH PVC PIPE TEST

6.1 Test Modifications

Several modifications were required for the 36 inch PVC pipe test. Note, these modifications were very similar to the modifications used for the 36 inch HDPE pipe test (Chapter 3). One corrugation was removed on each pipe end (Figure 6-1). This allowed the pipes to connect to the flexible membrane system.

The loading sequence consisted of ten pound per square inch increments until noticeable pipe deflection was measured by the laser profiling system. After pipe movement was measured, the loading sequence continued as discussed in the above chapter. A summary of the loading sequence and deflection reading timing is displayed in Table 6-1.

An array of pressure cells located eight inches above the pipe crowns were closely monitored during testing and the lift bag pressures were adjusted accordingly to create approximately equal pressures across the array of pressure cells. These pressure cells were used to calculate simulated overburden. The pressure cell locations can be seen in Figures 6-2 through 6-7.

6.2 Results

A longitudinal crack formed along the South PVC pipe invert during testing. This crack extended along the entire pipe length (Figures 6-8 and 6-9) and occurred at approximately 14 feet of simulated overburden. Deflection versus overburden results are presented in Figure 6-10 and Figure 6-11.

Table 6-1. Loading sequence, deflection reading timing, and simulated overburden for PVC pipe test.

Pressure Applied to Large Lift Bags (PSI)	Deflection Readings taken x hours after pressure was applied	Simulated Overburden on North Pipe (feet)
0	1	5.16
10	1	7.68
15	1	8.85
20	1	10.90
25	1	12.04
30	1	13.22
35	1	14.64
40	1	17.12
45	1, 4, 8	18.39
50	1	19.30
55	1	20.41
60	1	21.67
65	1	23.33
70	1	24.43
75	1	25.56
80	1	26.75
85	1, 4, 8	27.96
90	1	28.83
95	1	30.53
100	1	31.77
105	1	32.97
110	1	34.32
115	1	35.05
120	1	36.22
125	1	36.97
130	1, 4, 8, 24	38.50

Figure 6-1. PVC pipe end modification.

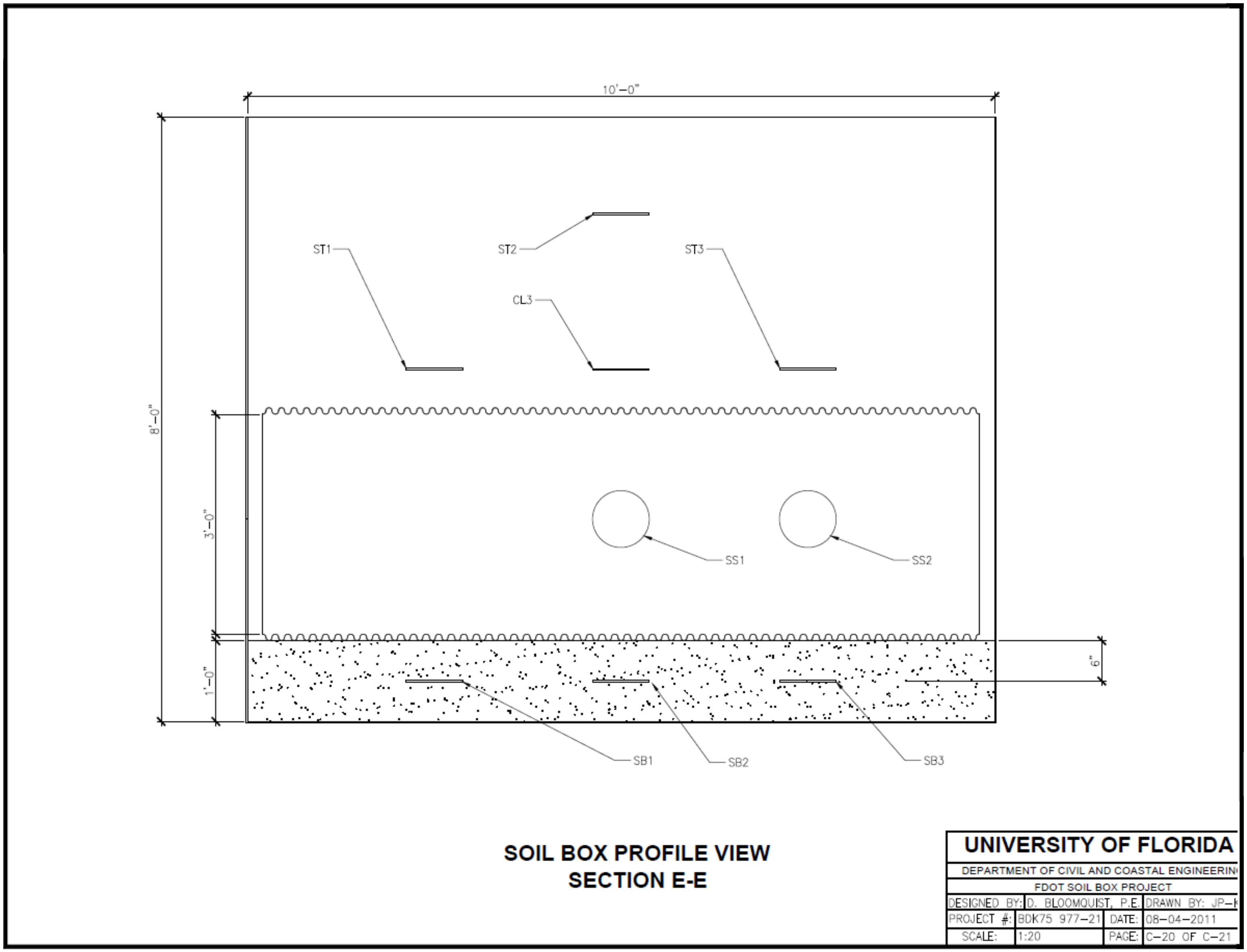


Figure 6-2. Earth pressure cell layout for PVC pipe test, section E-E.

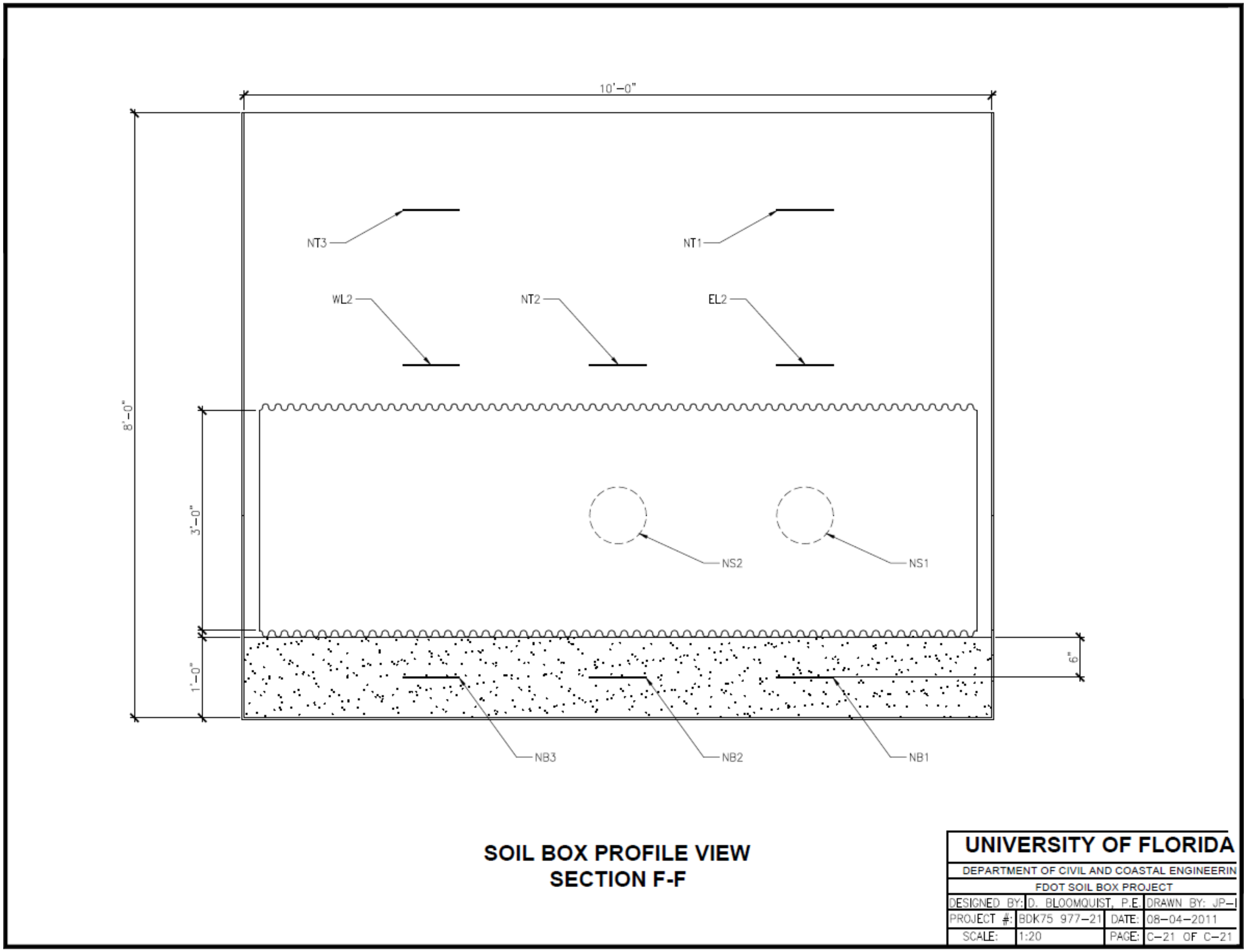


Figure 6-3. Earth pressure cell layout for PVC pipe test, section F-F.

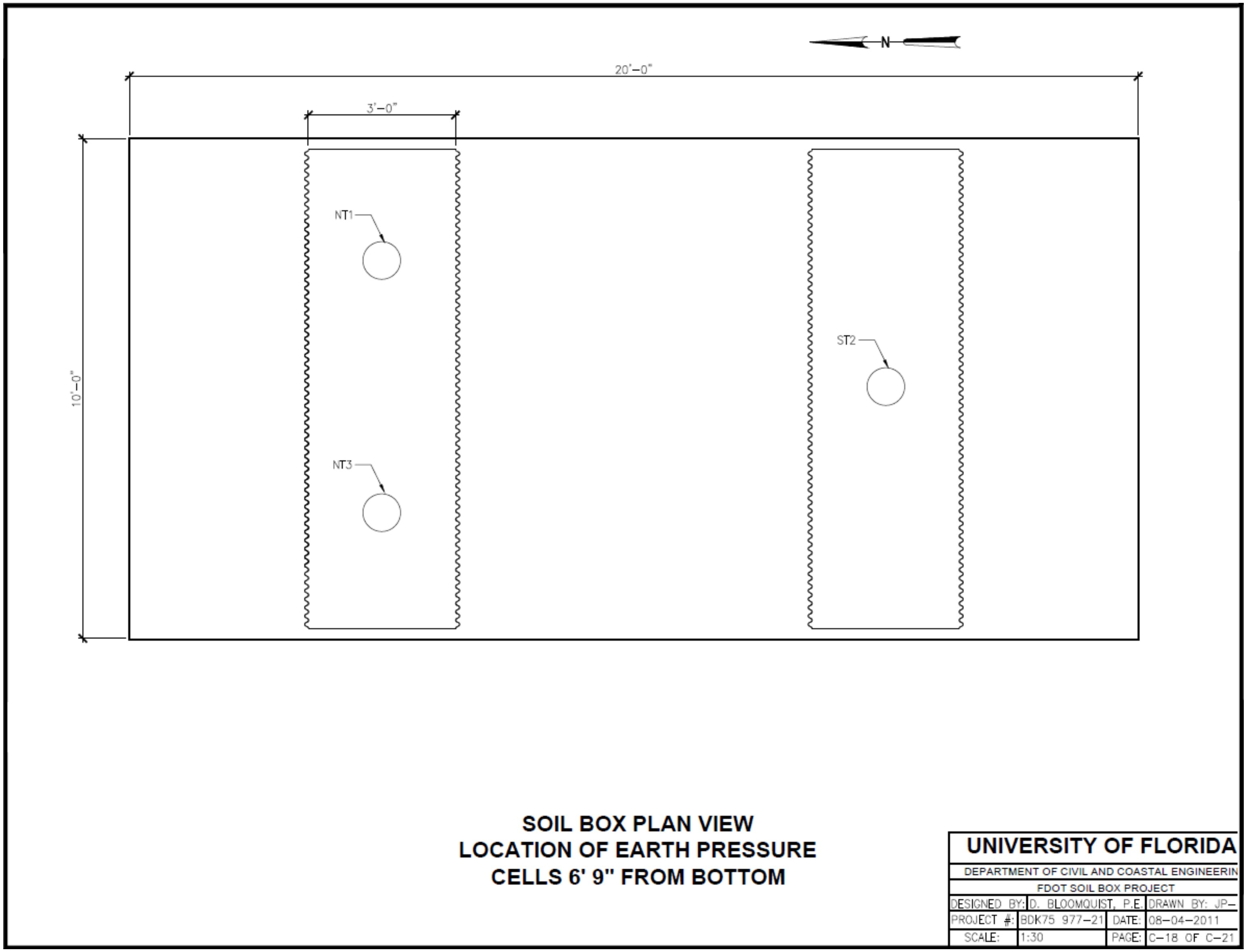


Figure 6-4. Earth pressure cell layout for PVC pipe test 6'-9" from bottom of Soil Box.

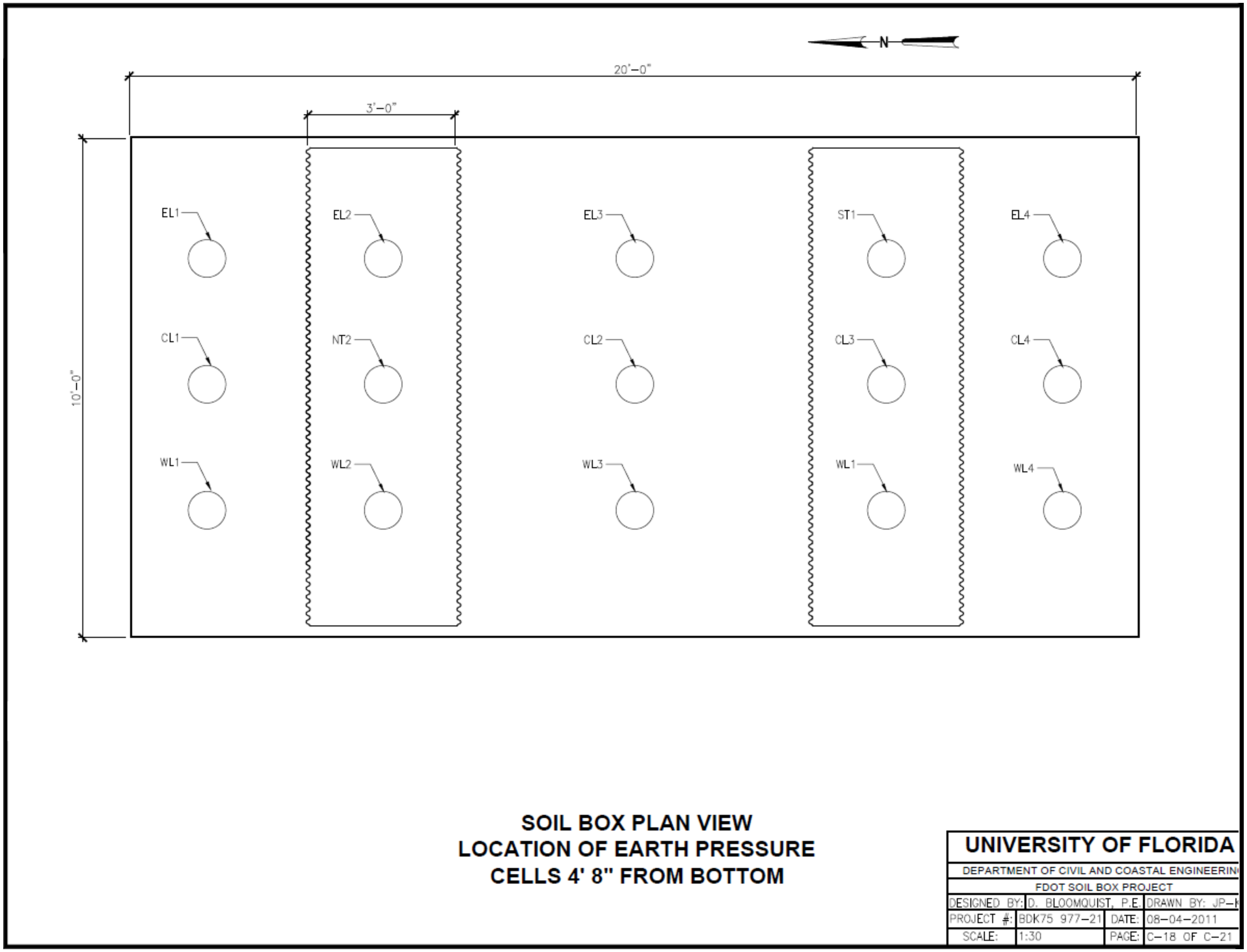


Figure 6-5. Earth pressure cell layout for PVC pipe test 4'-8" from bottom of Soil Box.

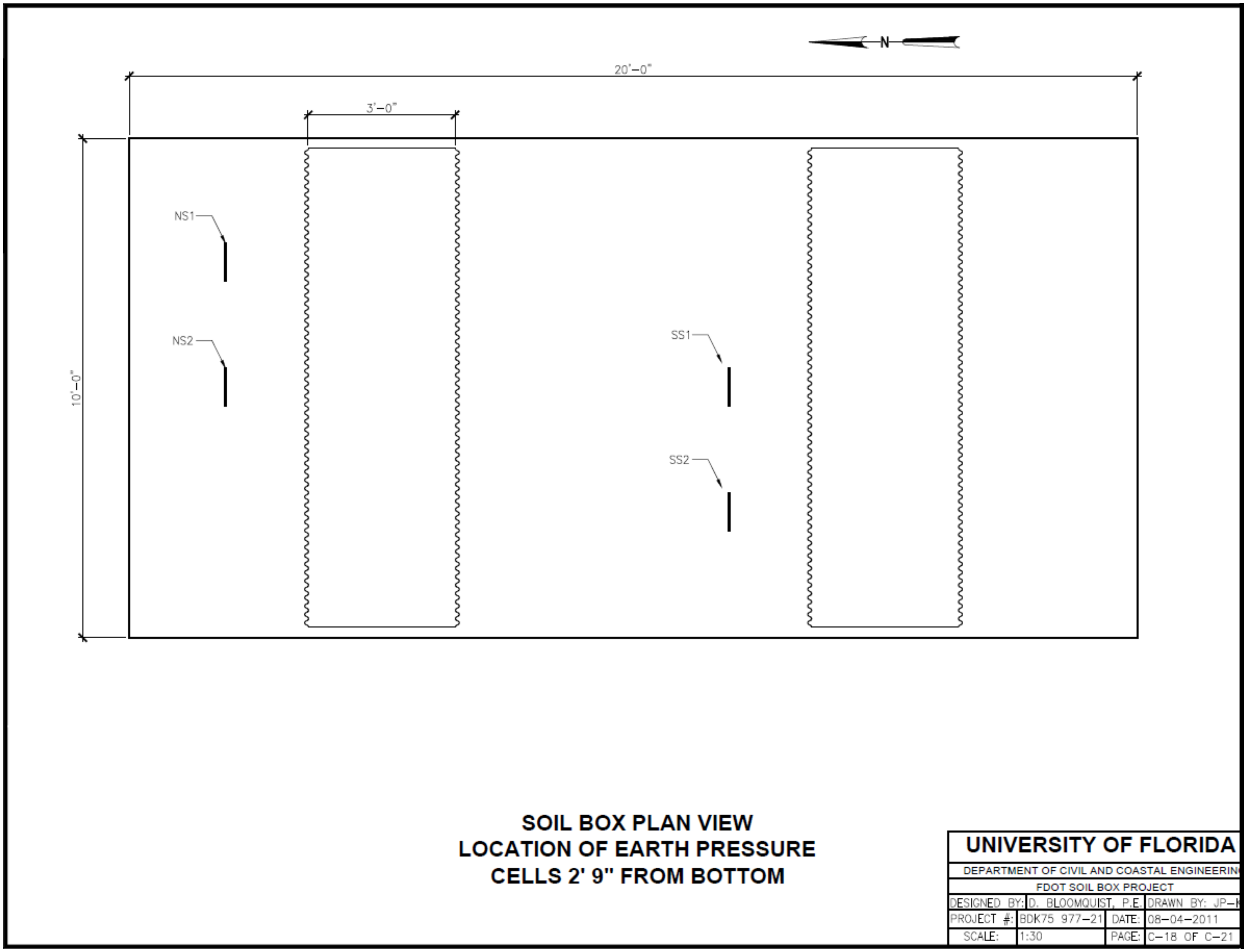


Figure 6-6. Earth pressure cell layout for PVC pipe test 2'-9" from bottom of Soil Box.

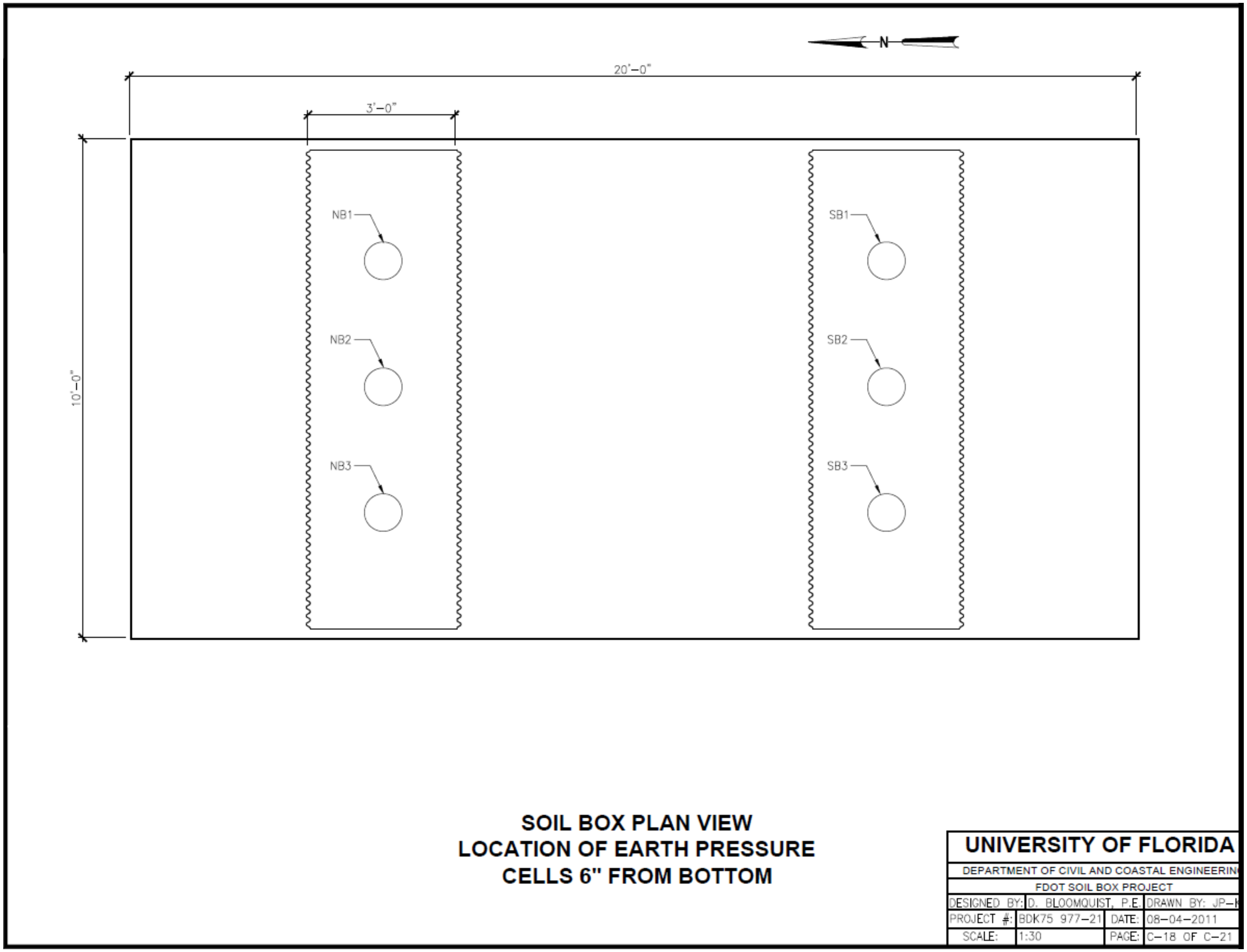


Figure 6-7. Earth pressure cell layout for PVC pipe test 6” from bottom of Soil Box.

Figure 6-8. South pipe failure.

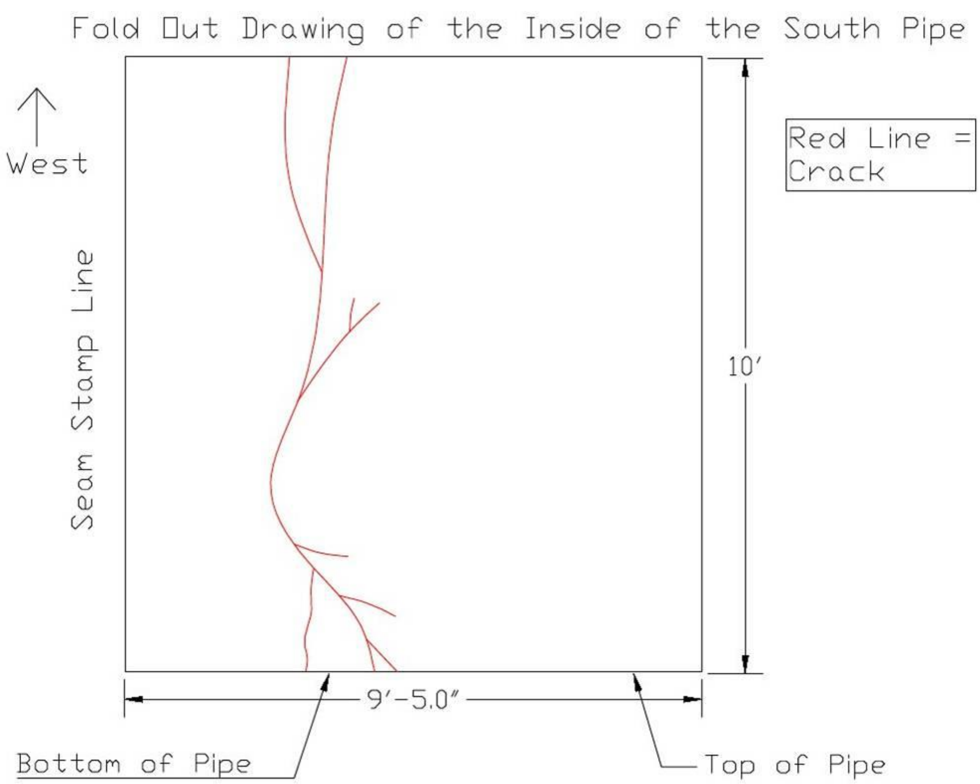


Figure 6-9. Fold out drawing of South PVC pipe.

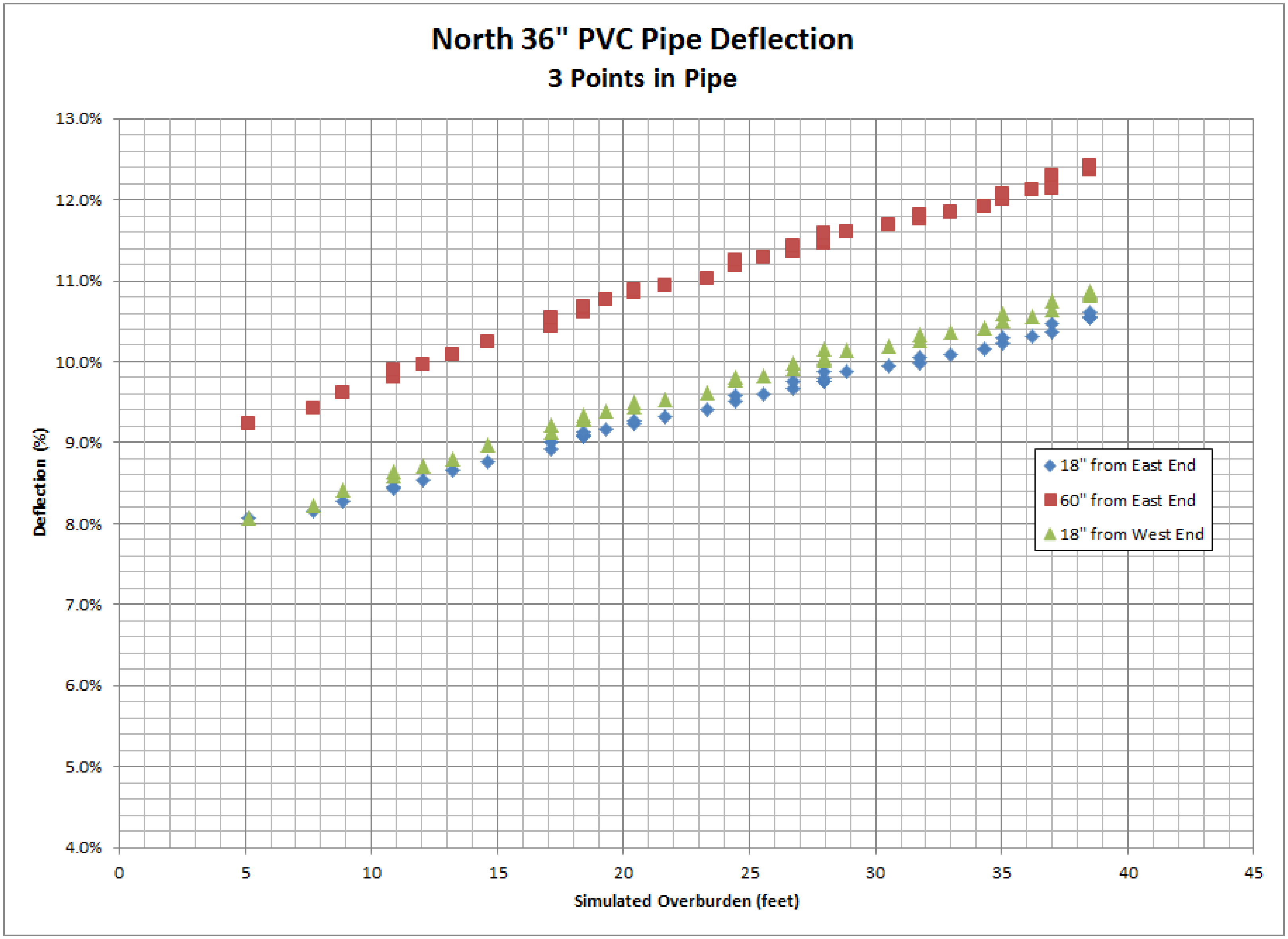


Figure 6-10. Three points in North 36” PVC pipe, deflection versus simulated overburden.

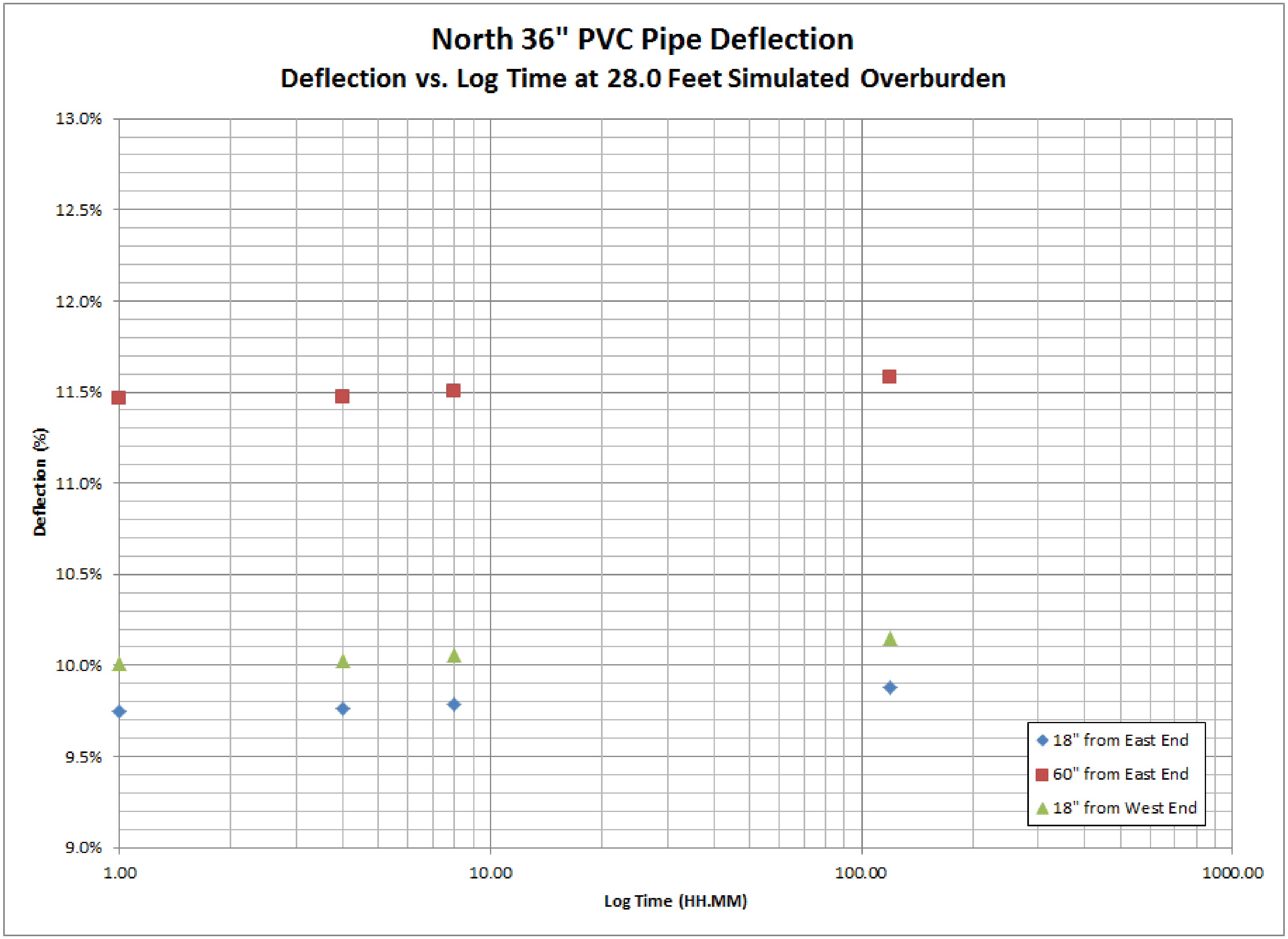


Figure 6-11. Deflection of North 36” PVC pipe under 28.0 ft of simulated overburden over time.

CHAPTER 7
36 INCH STEEL PIPE TEST

7.1 Test Modifications

Several modifications were required for the 36 inch steel pipe test. A steel ring was spot-welded on each pipe end (Figures 7-1 and 7-2). This allowed the pipes to connect to the flexible membrane system.

Ten regulators were installed to control the lift bag pressures over each individual load plate (Figure 7-3). Six regulators controlled the large lift bags and the remaining four controlled the small lift bags. These regulators were installed to provide better uniform pressure distribution to the soil surface.

An array of pressure cells located eight inches above the pipe crowns were closely monitored during testing and the lift bag pressures were adjusted accordingly to create approximately equal pressures across the array of pressure cells. These pressure cells were used to calculate simulated overburden. The locations of all the pressure cells are shown in Figures 7-4 through 7-9.

7.2 Results

A summary of the loading sequence, deflection reading timing, and simulated overburden for each pipe is displayed in Table 7-1. Results appear to indicate that while the modified testing procedure discussed in Chapter 5 represented an improvement when compared with the procedure discussed in Chapter 2, there was still more work to be conducted. Specifically, data appeared to indicate that the North pipe deflected more on its West end than its East end. Investigators concluded that this must be due to incorrect pressure cell readings. During testing, pressure was controlled through a feedback loop between the pressure regulators and the pressure cells. It was observed that only one lift bag inflated to full capacity (the lift bag above the East end of the pipe). In other words, due to an inaccurate pressure cell reading, the bag was over-inflated relative to the other bags. This in turn caused greater soil pressure and greater deflection under this bag.

To remedy this situation, a new series of pressure cells was acquired. Additionally, another steel pipe was tested (along with an HDPE pipe with a trench box; Chapter 8) to reconcile the data.

Table 7-1. Loading sequence, deflection reading timing, and simulated overburden for steel pipe test.

Pressure Applied to Main Regulator (PSI)	Deflection Readings taken x hours after pressure was applied	Simulated Overburden Over Pipes (feet)
0	1	3.3
5	1	3.5
10	1	3.7
15	1	5.3
20	1	6.3
25	1	7.5
30	1	7.8
35	1	8.9
40	1	9.9
45	1, 4, 8	10.3
50	1	10.7
55	1	11.3
60	1	12.6
65	1	12.9
70	1	13.4
75	1	14.5
80	1	15.3
85	1	16.0
90	1, 4, 8	16.7
95	1	17.9
100	1	18.1
105	1	18.7
110	1	19.4
115	1	20.3
120	1	20.8
125	1	21.3
130	1, 4, 8, 24	22.0
130 (80)	1	28.5
130 (100)	1	32.8
130 (120)	1	36.2
130 (130)	1	36.7

The numbers in parentheses indicate the increment increased in lift bags over the South pipe.

Figure 7-1. Before steel pipe end modification.

Figure 7-2. After steel pipe end modification.

Figure 7-3. Regulators used to control lift bag pressure over individual load plates.

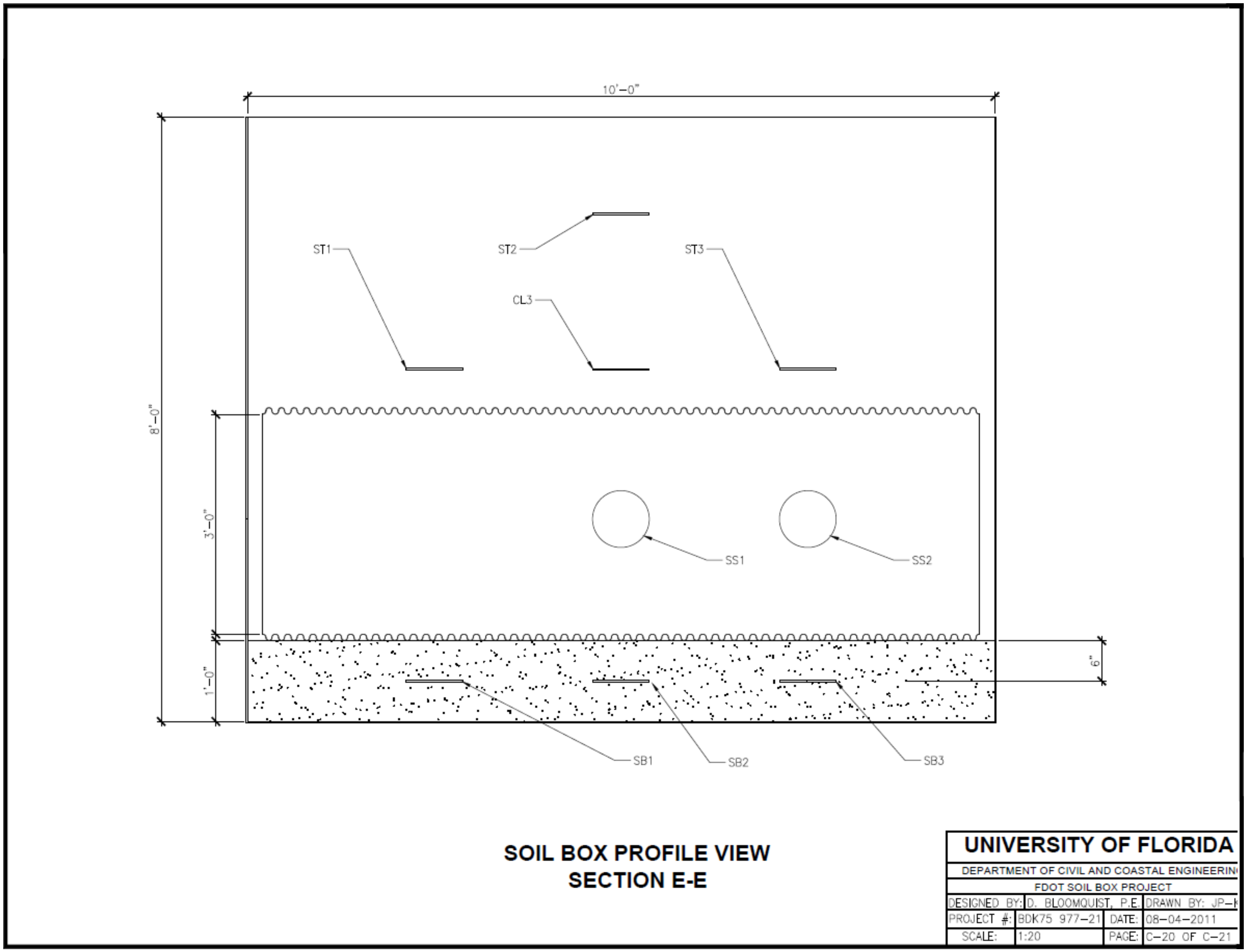


Figure 7-4. Earth pressure cell layout for steel pipe test, section E-E.

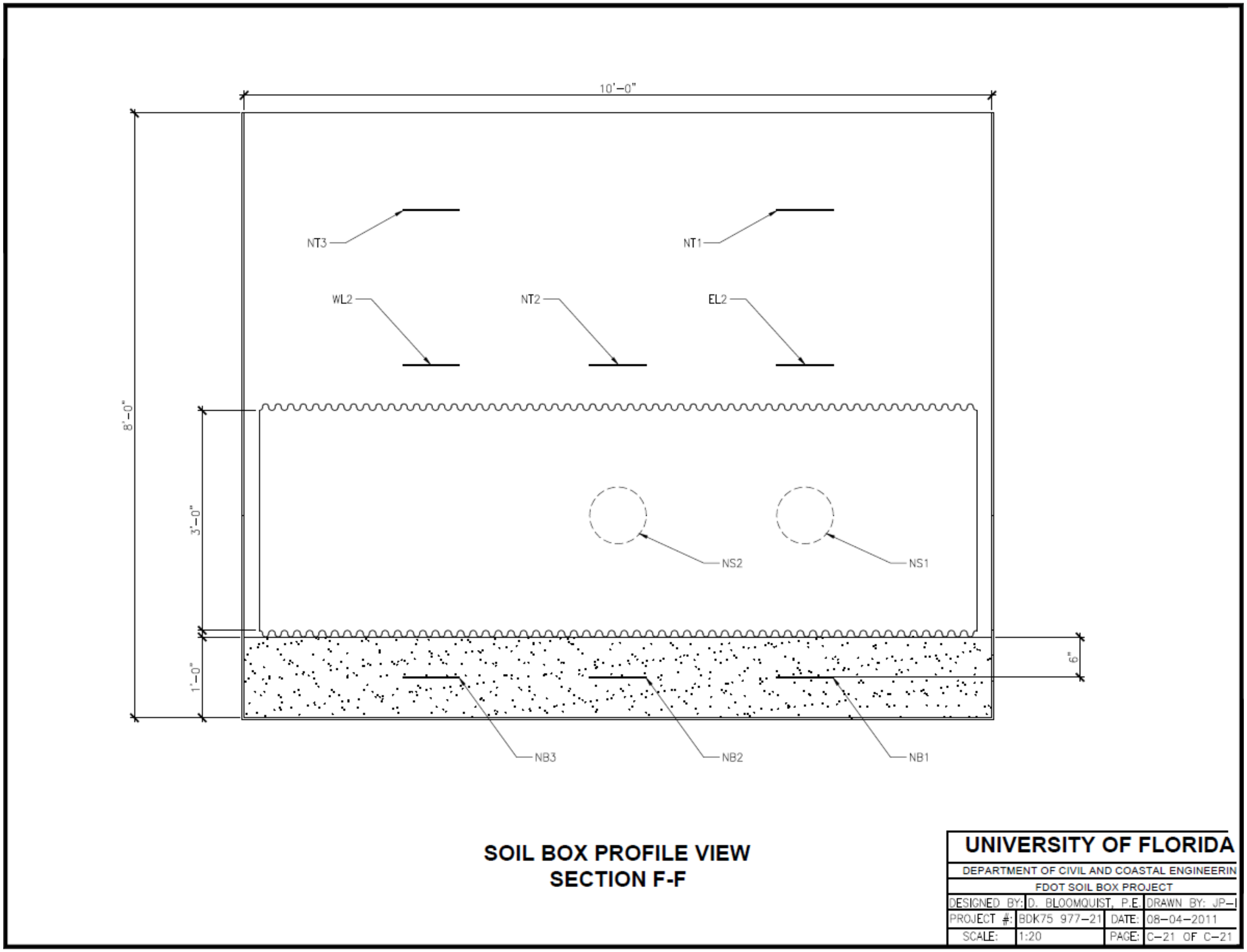


Figure 7-5. Earth pressure cell layout for steel pipe test, section F-F.

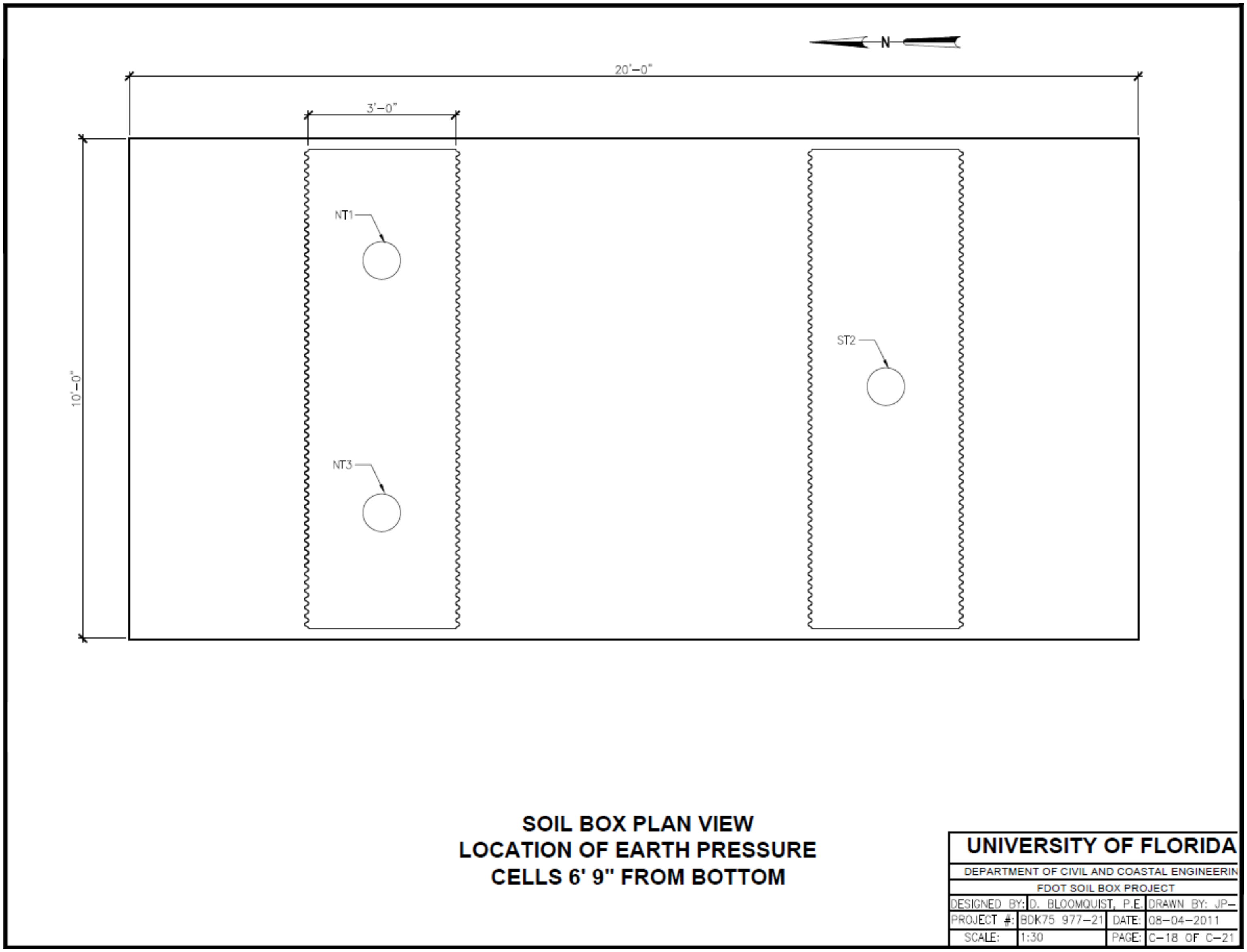


Figure 7-6. Earth pressure cell layout for steel pipe test 6'-9" from bottom of Soil Box.

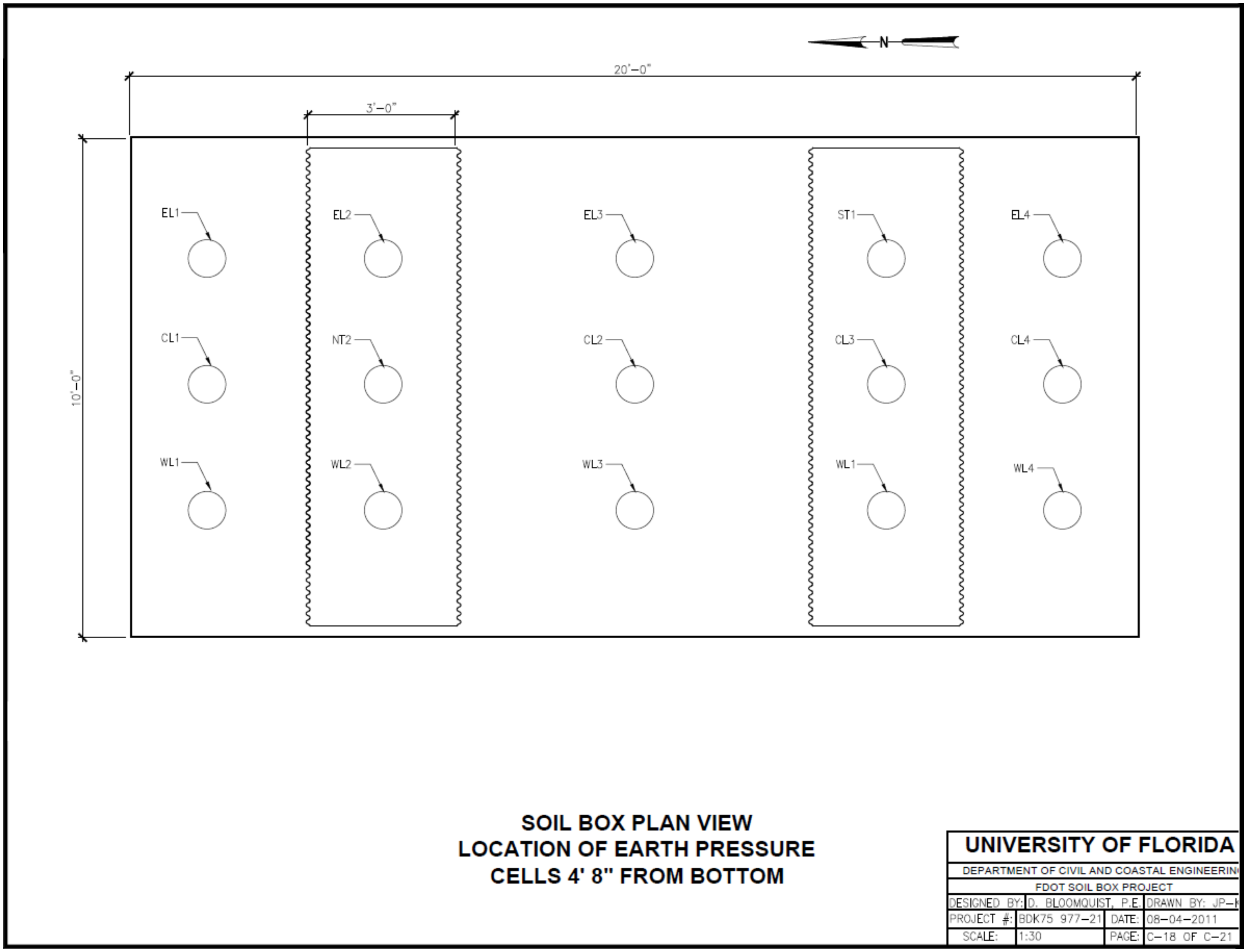


Figure 7-7. Earth pressure cell layout for steel pipe test 4'-8" from bottom of Soil Box.

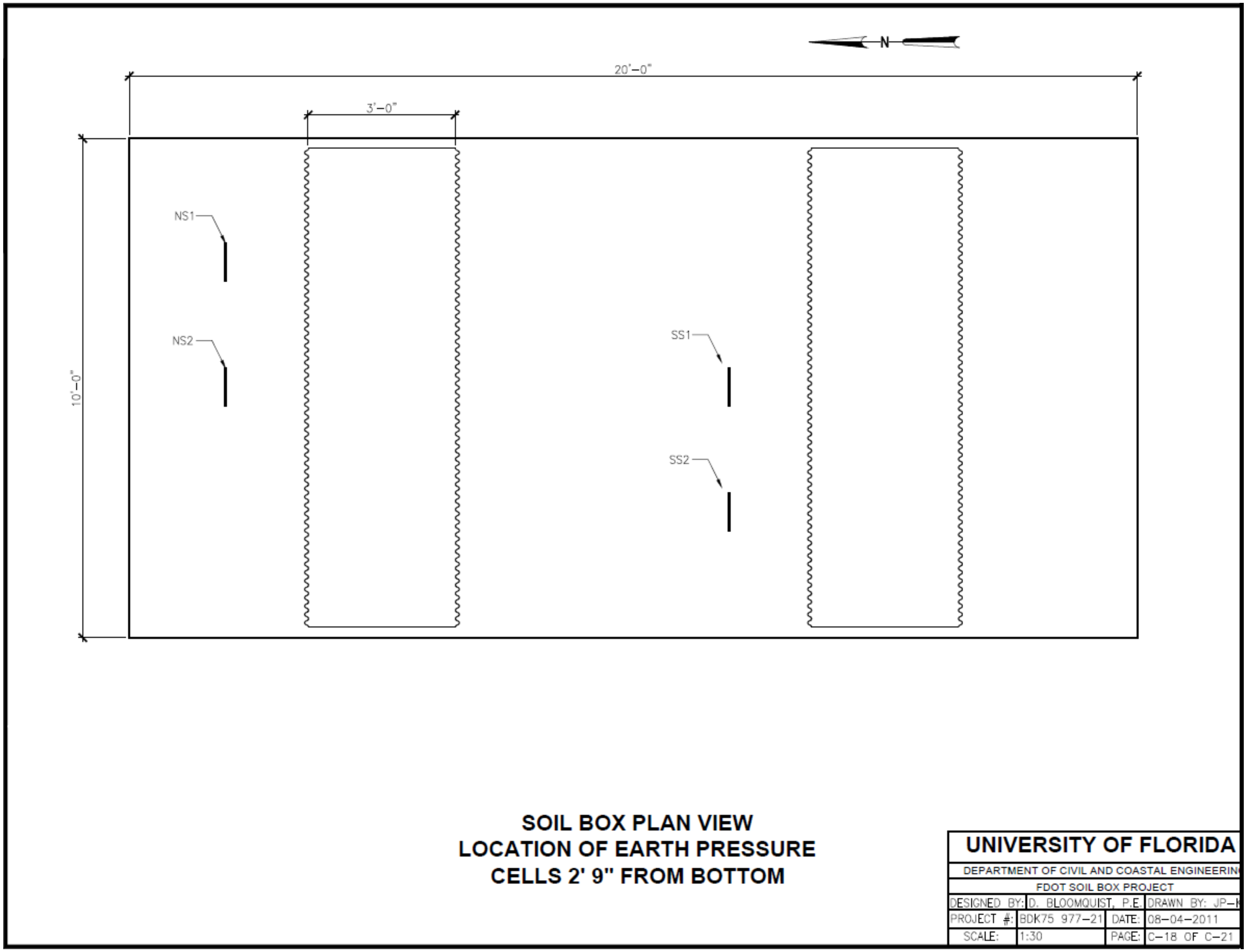


Figure 7-8. Earth pressure cell layout for steel pipe test 2'-9" from bottom of Soil Box.

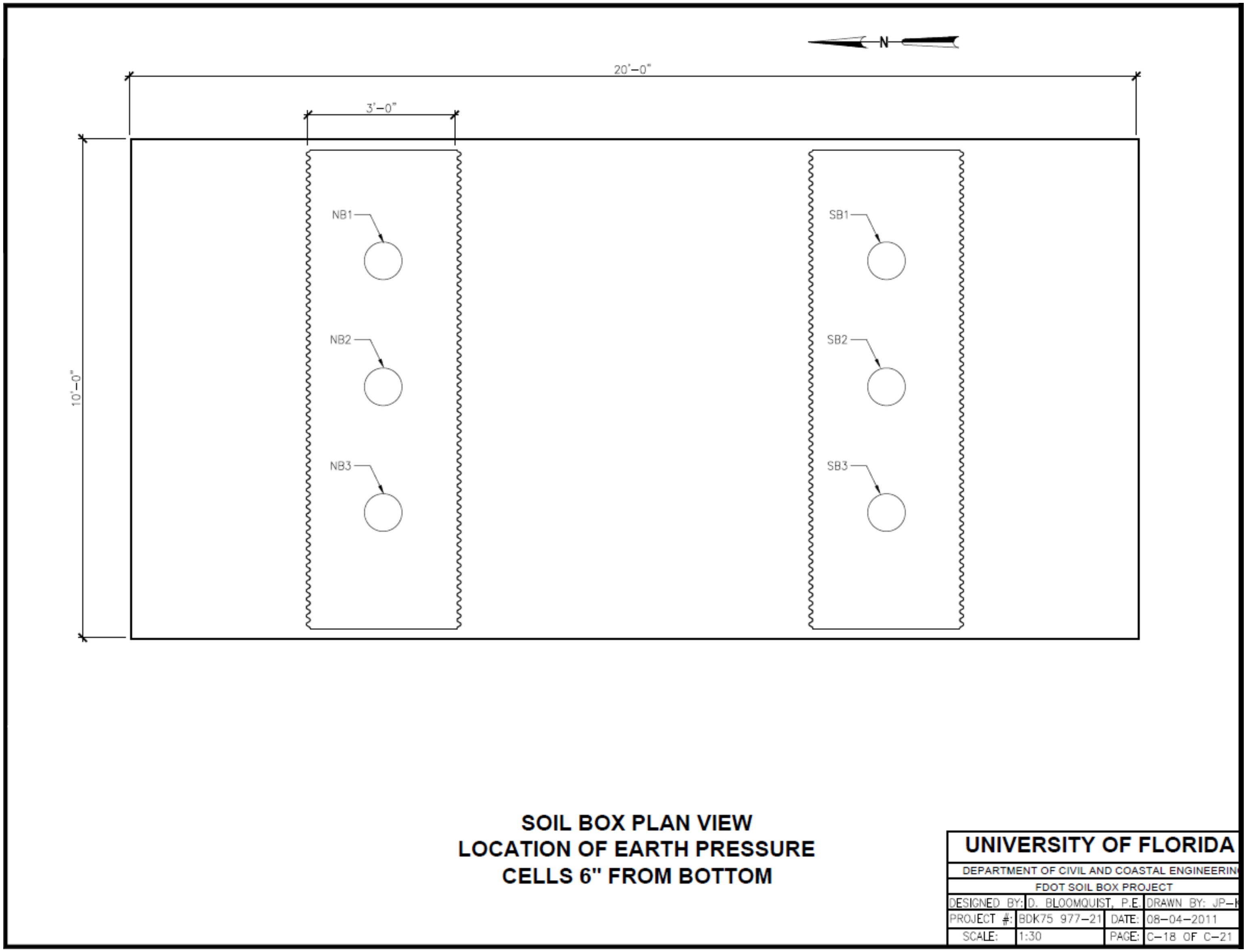


Figure 7-9. Earth pressure cell layout for steel pipe test 6" from bottom of Soil Box.

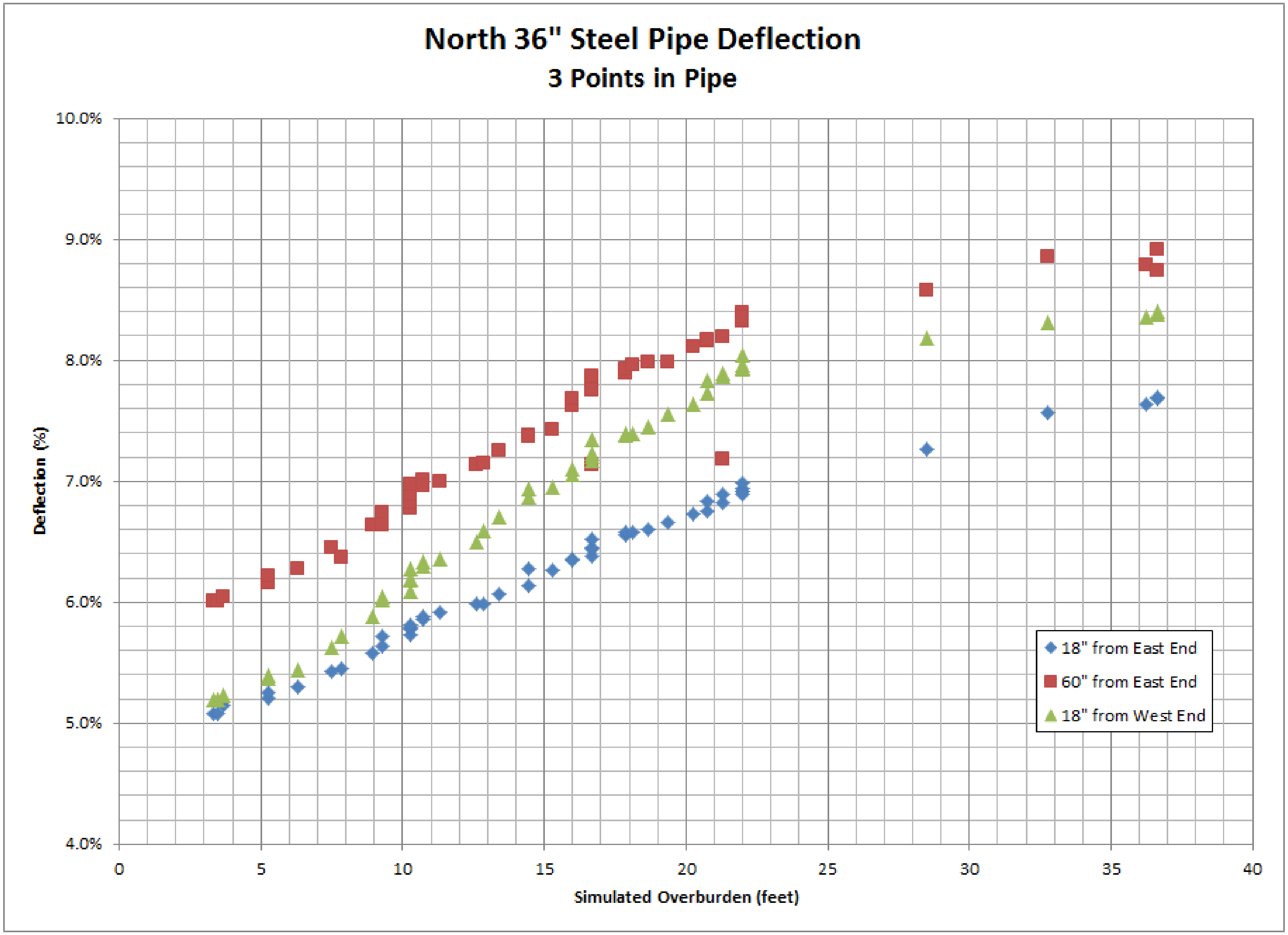


Figure 7-10. Three points in North 36” steel pipe, deflection versus simulated overburden.

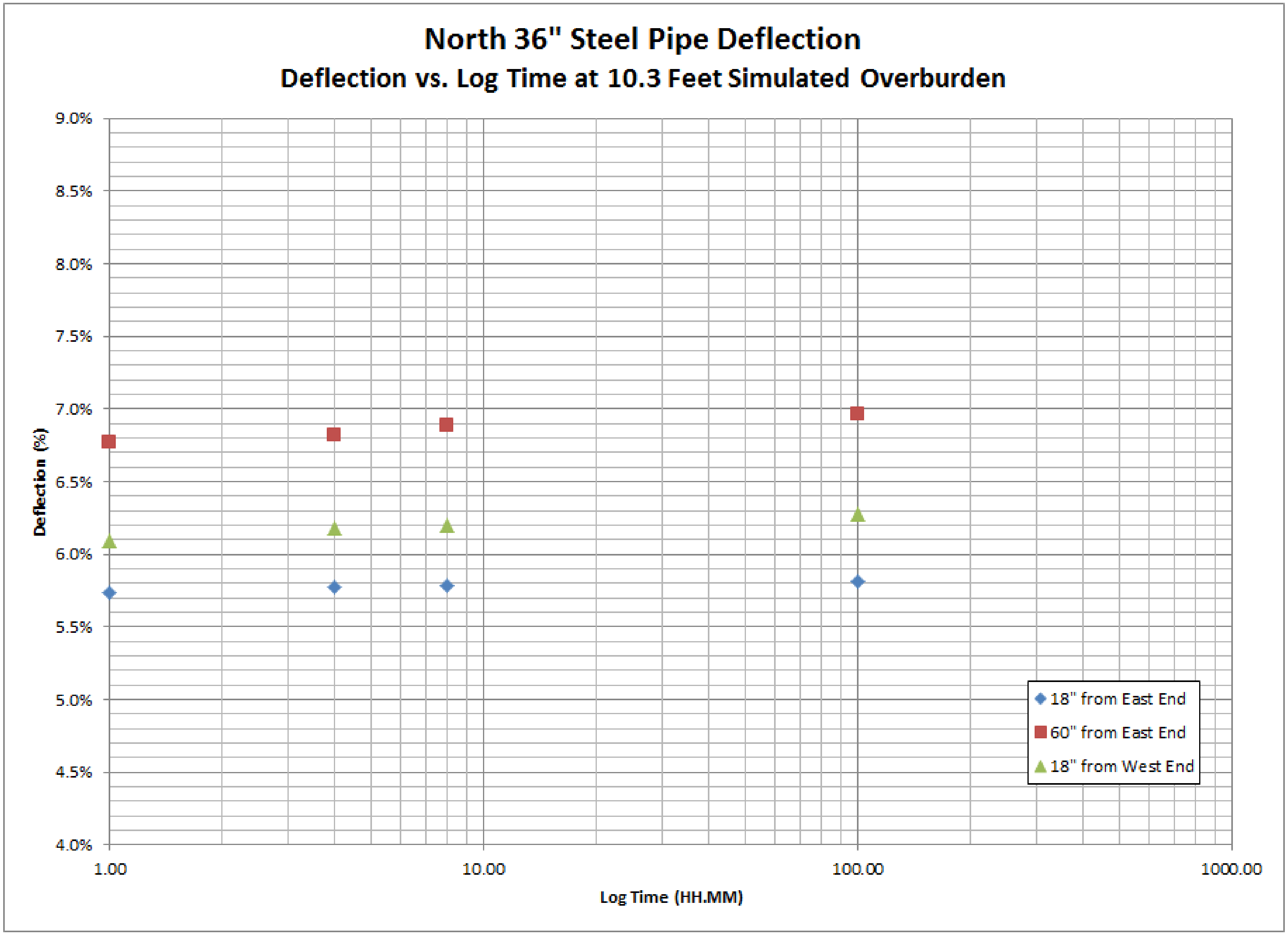


Figure 7-11. Deflection of North 36" steel pipe under 10.3 ft of simulated overburden over time.

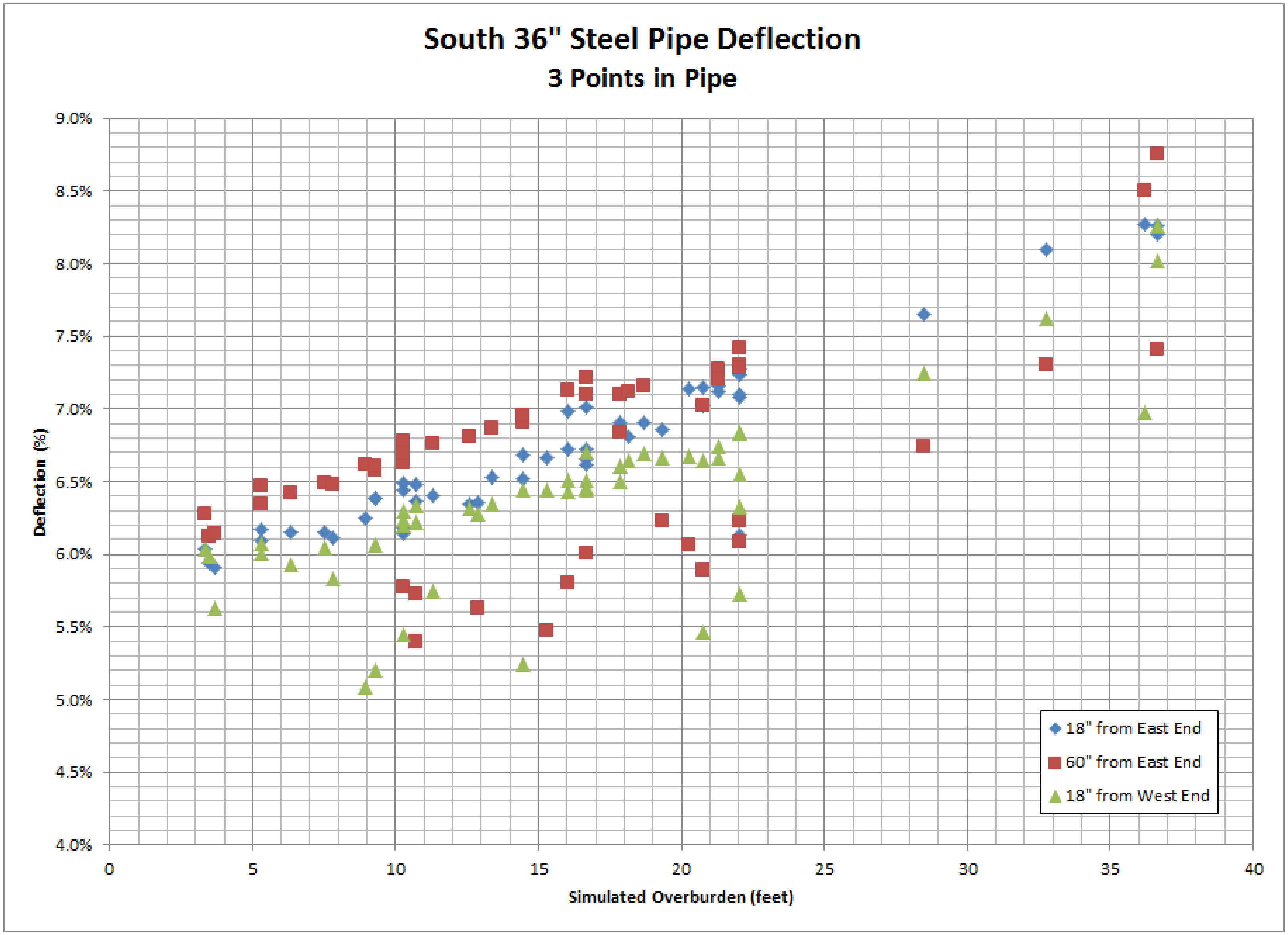


Figure 7-12. Three points in South 36” steel pipe, deflection versus simulated overburden.

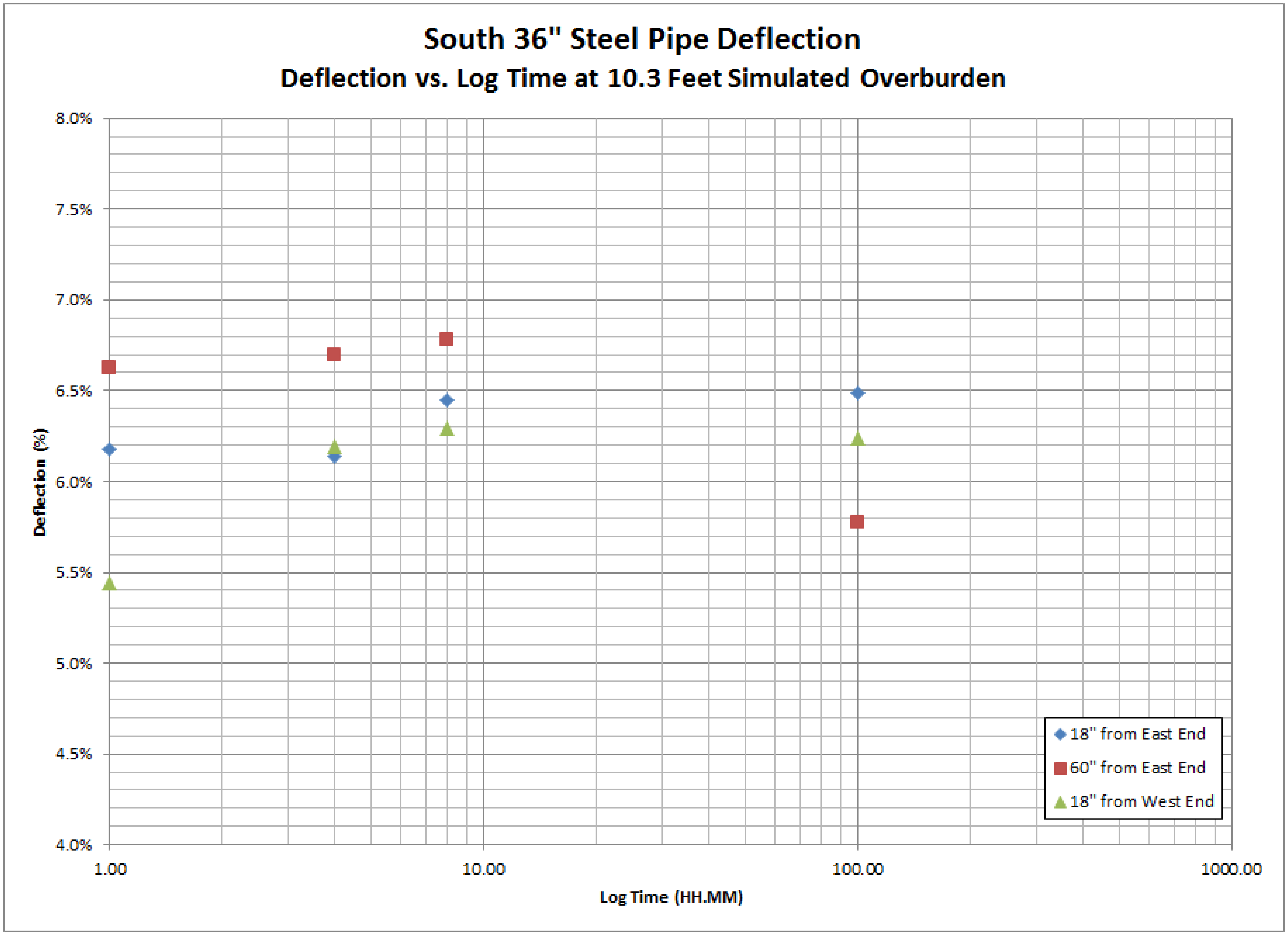


Figure 7-13. Deflection of South 36” steel pipe under 10.3 ft of simulated overburden over time.

CHAPTER 8
36 INCH HDPE PIPE WITH TRENCH BOX AND 36 INCH STEEL PIPE TEST

8.1 Trench Box Purpose

A trench box is a structure that protects workers within it and withstands the forces imposed on it by a cave-in during trench excavation (OSHA 1926.650). When laying pipe, a trench box is placed in the excavated trench while workers install a pipe section. The trench box is then pulled forward and this procedure is repeated until the pipe laying is complete.

Strength of flexible pipe is based upon passive soil pressures developed at the sides of the pipe when subjected to overburden stress. When pulling the trench box forward during pipe installation, the voids left by the trench box are supposed to be filled and compacted. This test aims to account for the impact of a contractor's use of a trench box for pipe installation on the most backfill sensitive pipe, HDPE.

8.2 Trench Box Fabrication

Because of the Soil Box's dimensions (it is not exactly 10 feet wide in all locations), it was not feasible to use a pre-fabricated trench box during testing. Therefore, a trench box was fabricated at the UF Coastal Engineering Laboratory. There was some question as to what the trench box's dimensions should be. The distance between trench box walls is standardized by the American Association of State Highway and Transportation Officials (AASHTO). According to AASHTO, distance between walls should be 73 inches. However, wall thickness is not was well-codified. FDOT uses trench boxes with wall thicknesses of 4 inches, 6 inches, and 8 inches.

Ultimately, investigators decided to use the 8 inch wall thickness because the largest wall dimension should have the largest effect on the surrounding soils' densities.

To construct the box, a frame was fabricated using six inch steel channels, and it was painted to mitigate corrosion (Figure 8-1). The trench box's walls were fabricated from three-quarter inch plywood that was screwed to each side of the steel frame. The plywood was painted and lacquered on both sides to prevent moisture intrusion into the trench box (Figure 8-2). Once the walls were constructed, they were connected using four steel pipe braces. A photograph of the completed trench box is shown in Figure 8-3.

8.3 Test Modifications

Several modifications to the testing procedure were required for the HDPE trench box/steel pipe test. A steel ring was spot welded on each end of the steel pipe and one corrugation was removed on each end of the HDPE pipe (Figures 8-4 and 8-5). This allowed the pipes to connect to the flexible membrane system.

To prevent damage to the Soil Box's electronics, the pressure cell cables were rerouted underneath the trench box. These cables were buried before the installation of the trench box (Figure 8-6). As implied the trench box was installed after placement of the first 12 inch soil layer. To install the trench box, a lift truck and crane were used to place it around the HDPE pipe (Figure 8-7 and Figure 8-8).

Once the trench box was installed, the Soil Box was filled using the same procedure outlined in Chapter 5. However, because the lift truck would not be capable of extracting the trench box after backfilling due to capacity issues, its crossbars were removed sequentially as the Soil Box was filled. Once the Soil box was full, each trench box wall was individually extracted using the lift truck (Figure 8-9 and Figure 8-10). Then, the voids left from trench box extraction were filled.

A chain-link fence was added to the loading mechanism. The fence was placed below the steel load plates to help distribute the load more uniformly to the soil surface (Figure 8-11).

A new array of pressure cells (Figure 8-12) were placed one half inch above the pipe crowns. These pressure cells were closely monitored during testing and the lift bag pressures were adjusted accordingly to create approximately equal pressures along the pipe's length. These pressure cells were used to calculate simulated overburden. The locations of all the pressure cells are shown in Figures 8-13 through 8-19.

8.4 Saturation Failure

During saturation, soil began flowing from the HDPE pipe observation portals. After examining the failure, it was apparent that the flexible membrane had breached. A void formed at the top of the Soil Box, through the observation portals, thereby exposing the HDPE pipe (Figure 8-20 and Figure 8-21). The performance of flexible pipes is directly related to lateral passive pressures developed by the soil-pipe system. These passive pressures were not developed due to the reduction in soil density from trench box removal. Therefore, as the HDPE pipe deflected (during backfilling), the stress on the flexible membrane increased, and ultimately failed. To remedy this issue, the membrane failure was sealed by placing an inflatable inner tube inside the HDPE pipe. The void was filled and saturation was completed.

8.5 Lift Bag Failure

A lift bag failure occurred in a large lift bag located in the Northeast corner of the Soil Box (Figure 3-10). The lift bag was leaking air through a small tear first observed during the early stages of the loading sequence. When the loading sequence reached 90 pounds per square inch, the lift bag appeared to be leaking significantly. To avoid overstressing the air compressor, the air supply to the lift bag was removed.

A large lift bag failure occurred during the 125 pounds per square inch load increment. This lift bag was located in the south west side of the Soil Box (Figure 8-22). This lift bag was leaking significantly, and as a result, pressure in the remaining large lift bags dropped. To remedy this issue, the air supply to the breached lift bag was removed. This failure caused a discontinuity of surcharge in the Soil Box, which ultimately reduced the simulated overburden pressure. These lift bags were utilized through several loading and unloading sequences and probably failed due to fatigue.

8.6 Results

Plots of simulated overburden pressure versus deflection are presented in from Figure 8-23 through Figure 8-27. As shown, the HDPE pipe with trench box appeared to significantly deflect. Qualitatively, during testing, the pipe appeared to fail (as evidenced by the appearance of “bubbles” inside the pipe). As data shows, at the beginning of the loading sequence, deflection was approximately 10.25%; by the end of the lading sequence, a deflection of 18% was achieved using a simulated overburden of approximately 20 feet. Note that data in Figure 8-24 appears to indicate that creep (at approximately 12.4 feet of simulated overburden) may also have been an issue during testing.

Steel pipe results (Figure 8-25 and Figure 8-26) were relatively consistent with results from previous tests. Note, that as shown in Figure 8-27, the rate of deflection as a function of simulated overburden was similar for both steel pipes. The “shift” in data is due to a lower initial overburden, which could be caused by the adjacent trench box removal.

Table 8-1. Loading sequence, deflection reading timing, and simulated overburden for HDPE with trench box and steel pipe test.

Pressure Applied to Main Regulator (PSI)	Deflection Readings taken x hours after pressure was applied	Simulated Overburden Over HDPE Pipe (feet)	Simulated Overburden Over Steel Pipe (feet)
0	1	4.4	4.3
5	1	5.2	5.2
10	1	6.6	6.7
15	1	8.2	8.4
20	1	9.3	9.8
25	1	10.0	10.7
30	1	9.6	11.2
35	1	11.3	12.7
40	1	12.0	13.2
45	1, 4, 8, 24	12.4	14.3
50	1	12.7	14.9
55	1	14.3	15.9
60	1	15.2	16.5
65	1	16.4	18.5
70	1	17.2	20.4
75	1	14.7	23.2
80	1	16.7	26.3
85	1	18.0	27.6
90	1, 4, 8, 24	17.4	28.3
95	1	17.7	29.3
100	1	18.7	30.5
105	1	18.0	31.4
110	1	18.9	32.5
115	1	19.8	33.9
120	1	18.9	34.6
125	1	19.6	35.6
130	1, 4, 8, 24	19.8	24.6

Figure 8-1. Painted trench box frames.

Figure 8-2. Trench box wall with first side of plywood.

Figure 8-3. Completed trench box.

Figure 8-4. Steel pipe end modification.

Figure 8-5. HDPE pipe end modification.

Figure 8-6. Rerouted pressure cell cabling.

Figure 8-7. Hoisting trench box into Soil Box.

Figure 8-8. Installed trench box.

Figure 8-9. Trench box removal

Figure 8-10. Installed trench box.

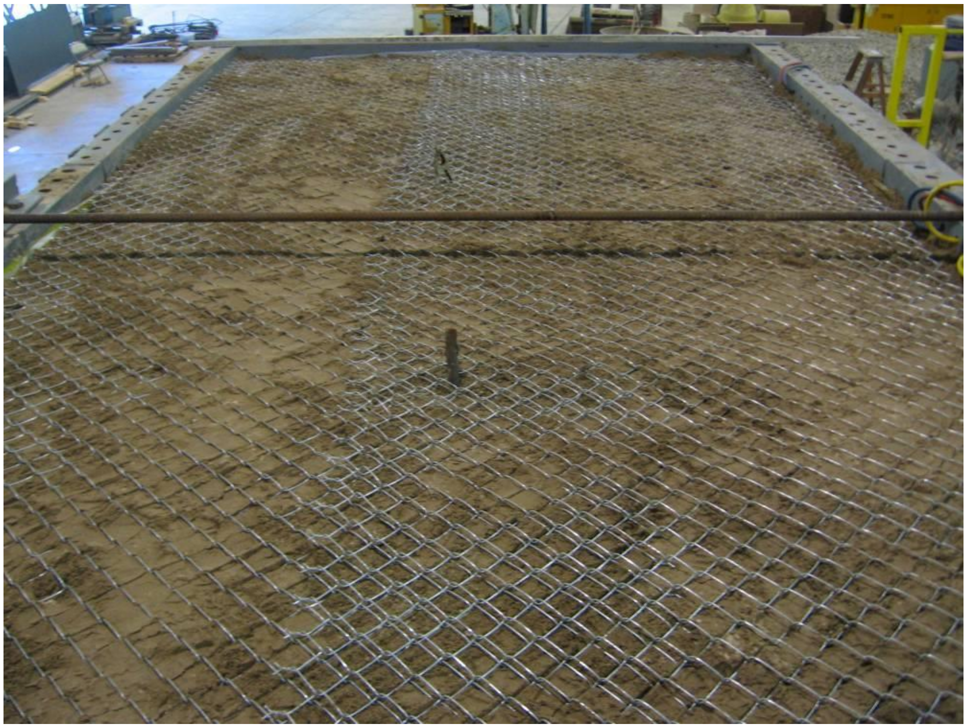


Figure 8-11. Addition of chain link fence to loading mechanism.

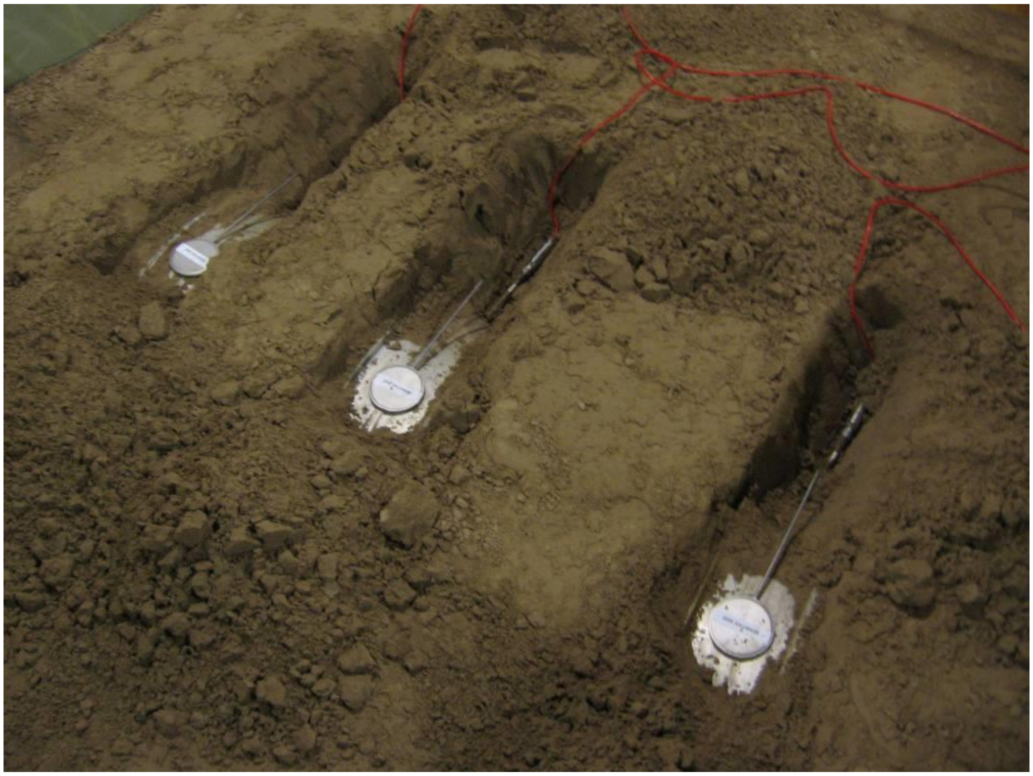


Figure 8-12. Earth pressure cells located one half inch above pipe crown.

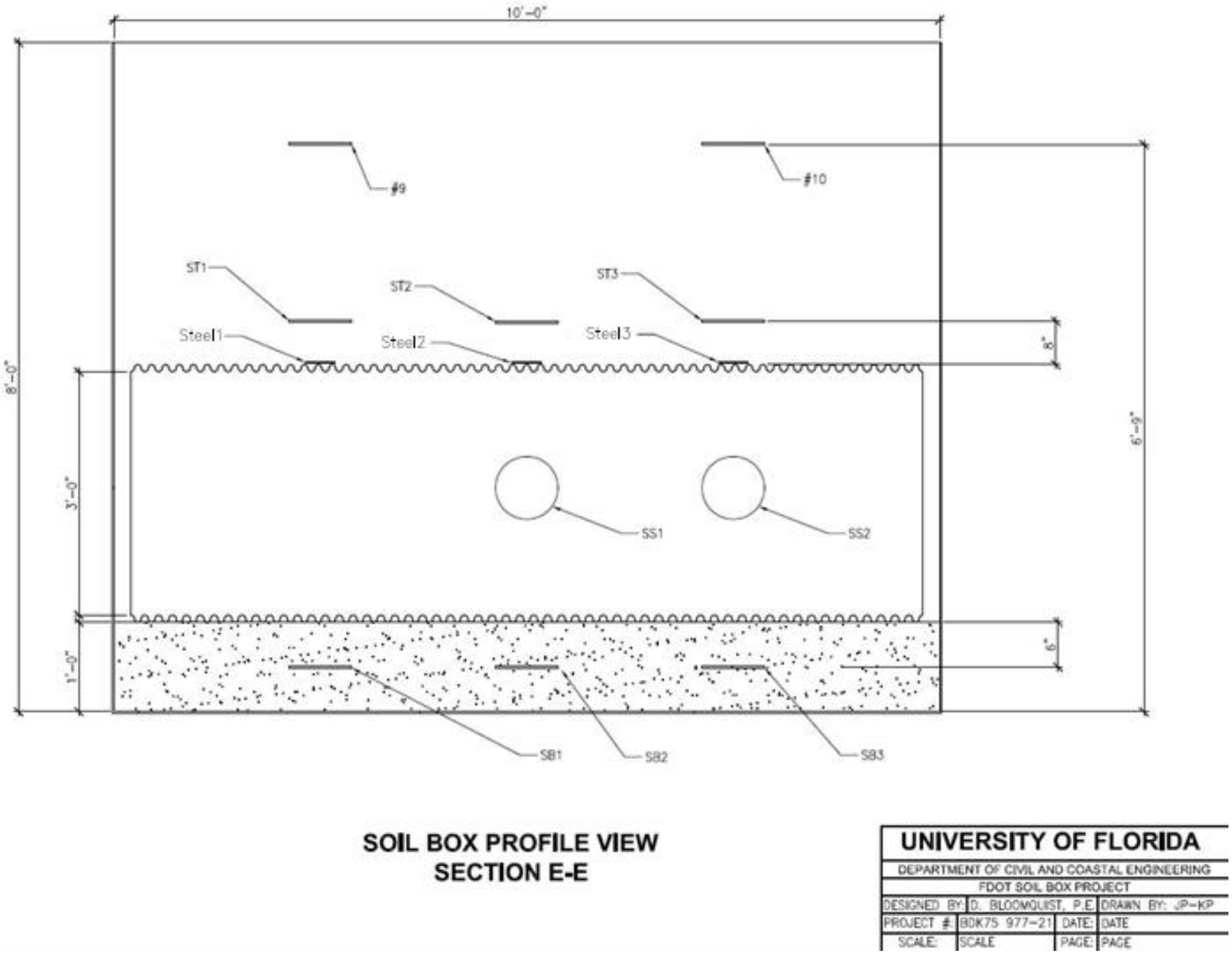


Figure 8-13. Earth pressure cell layout for trench box test one, section E-E.

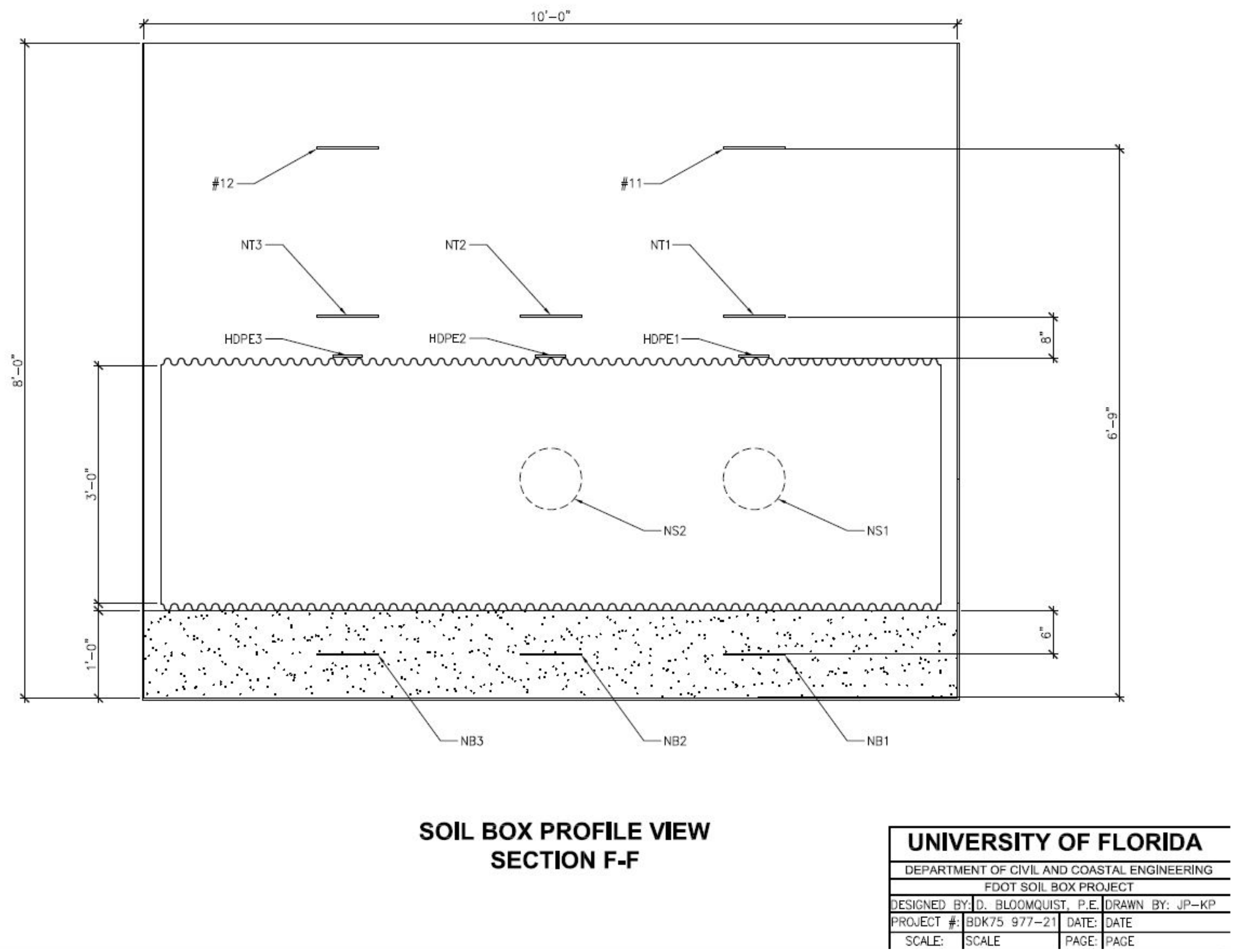


Figure 8-14. Earth pressure cell layout for trench box test one, section F-F.

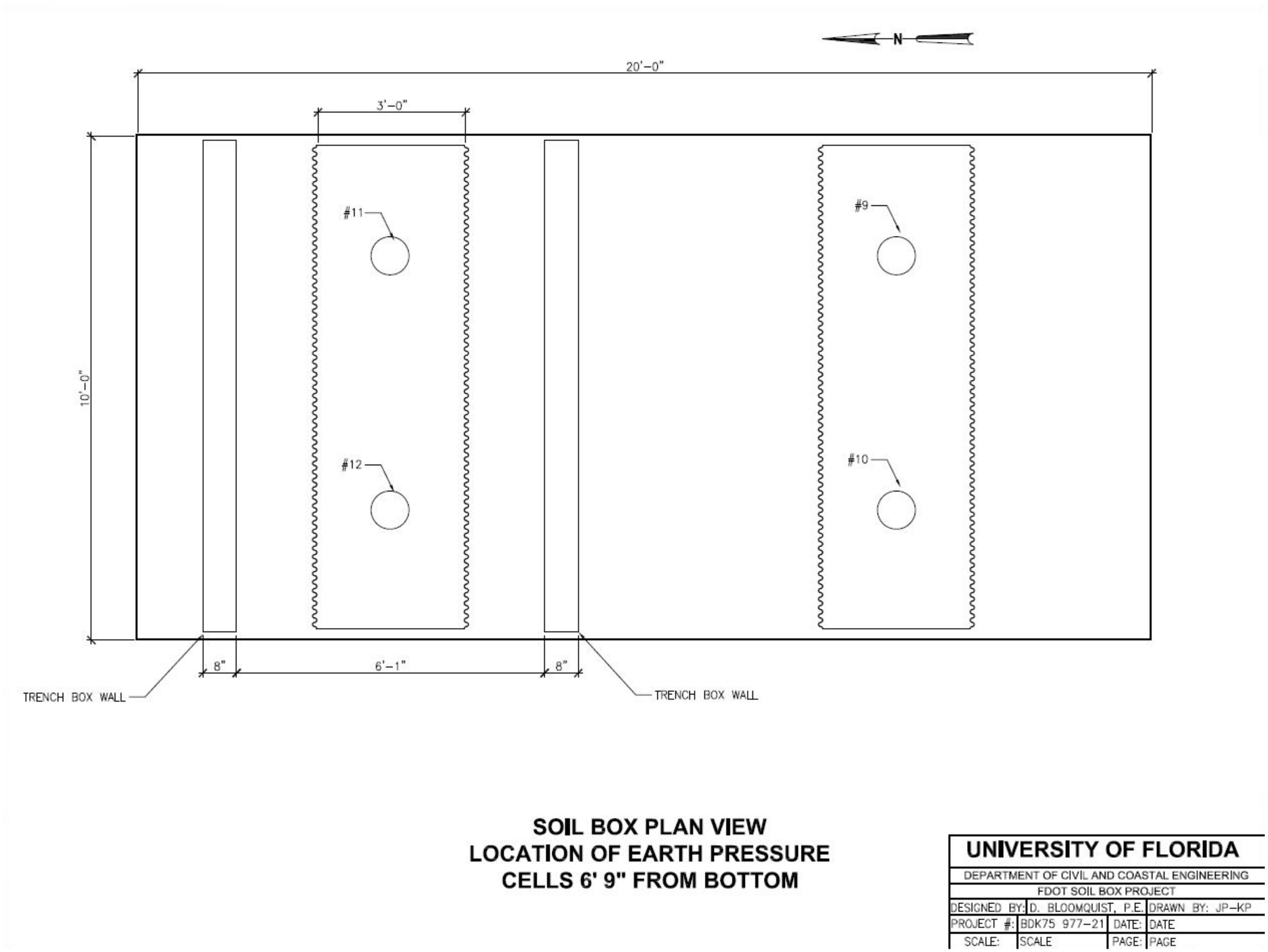


Figure 8-15. Earth pressure cell layout for trench box test one 6'-9" from bottom of Soil Box.

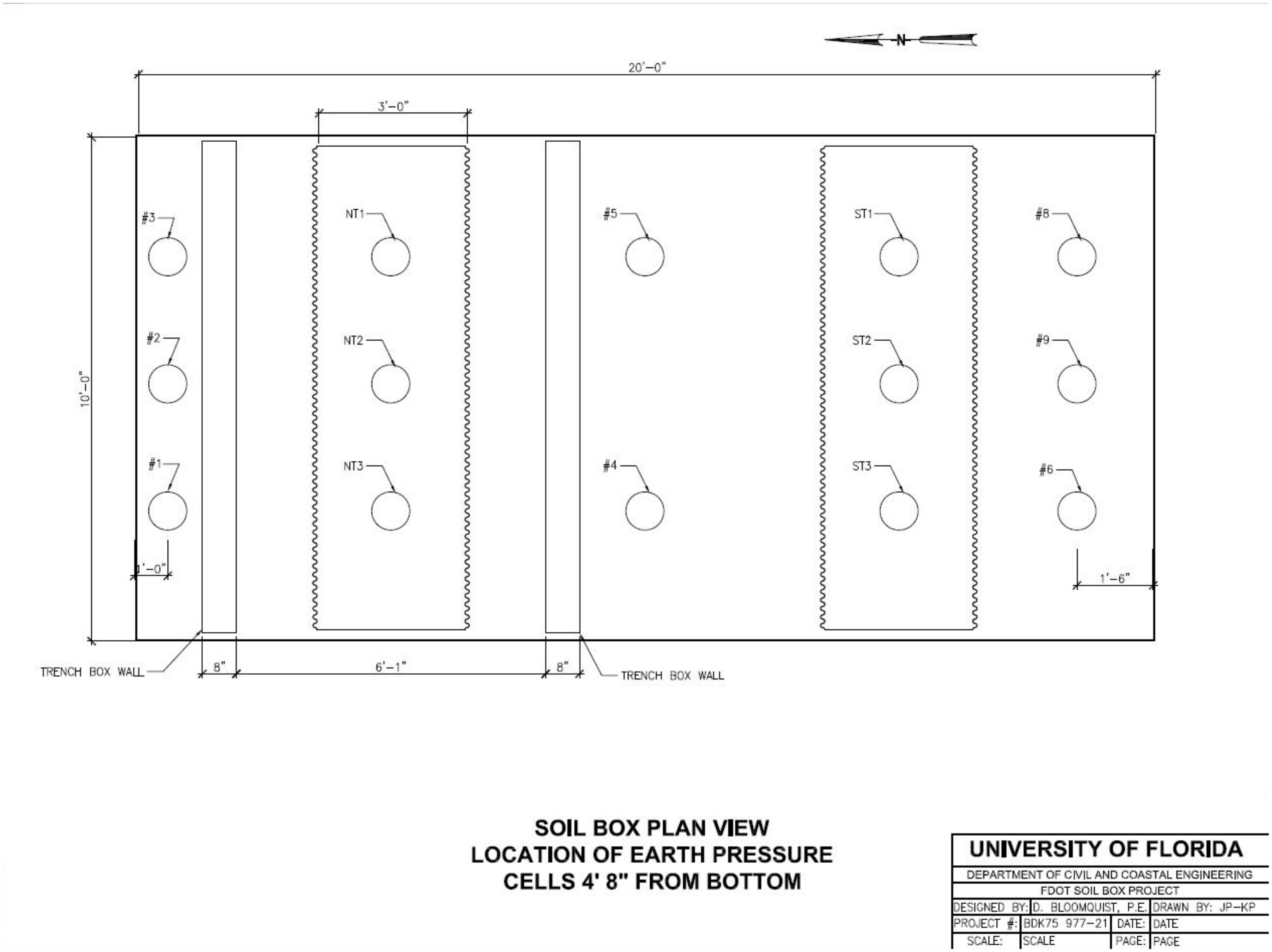


Figure 8-16. Earth pressure cell layout for trench box test one 4’-8” from bottom of Soil Box.

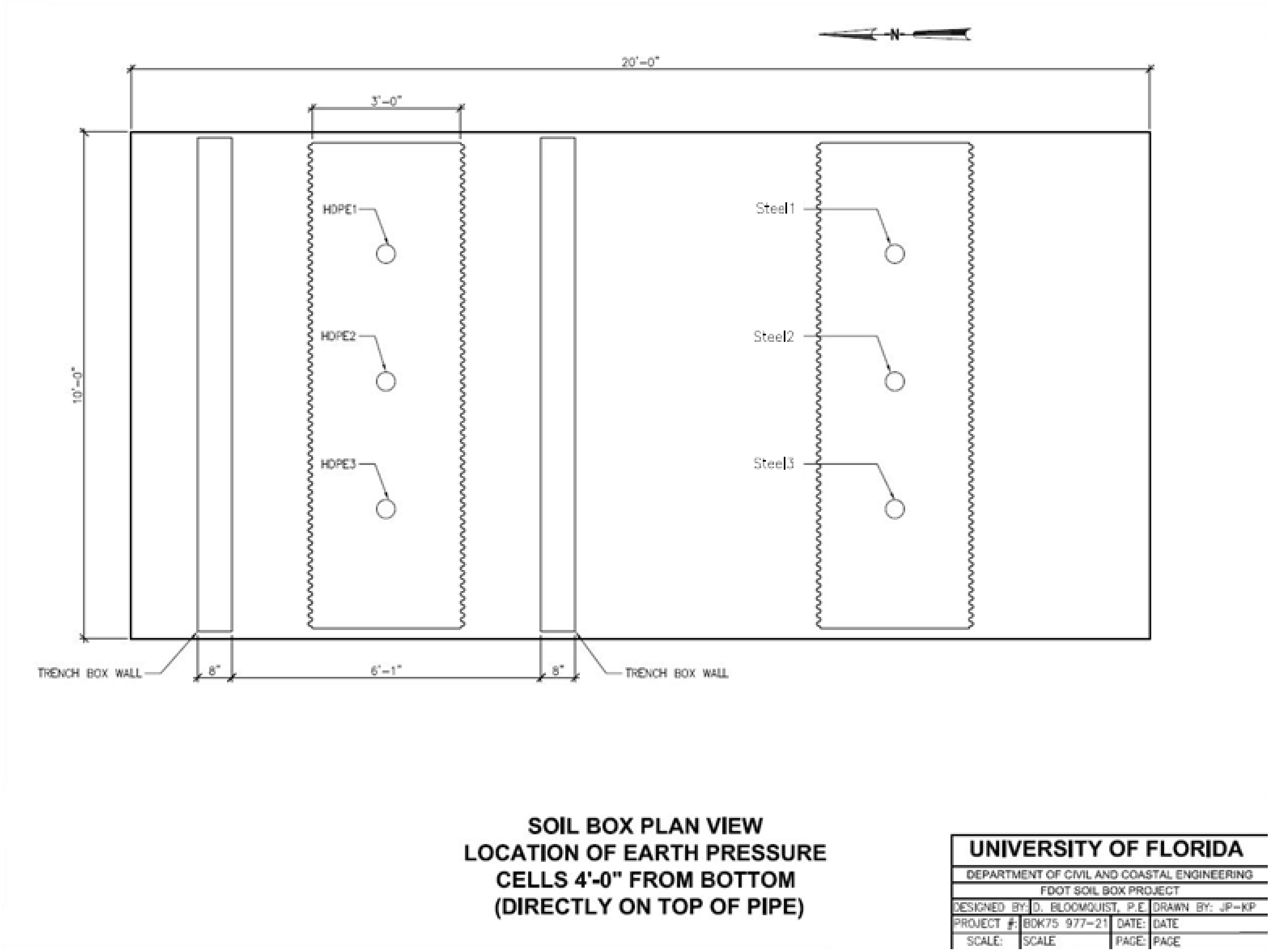


Figure 8-17. Earth pressure cell layout for trench box test one 4'-0" from bottom of Soil Box.

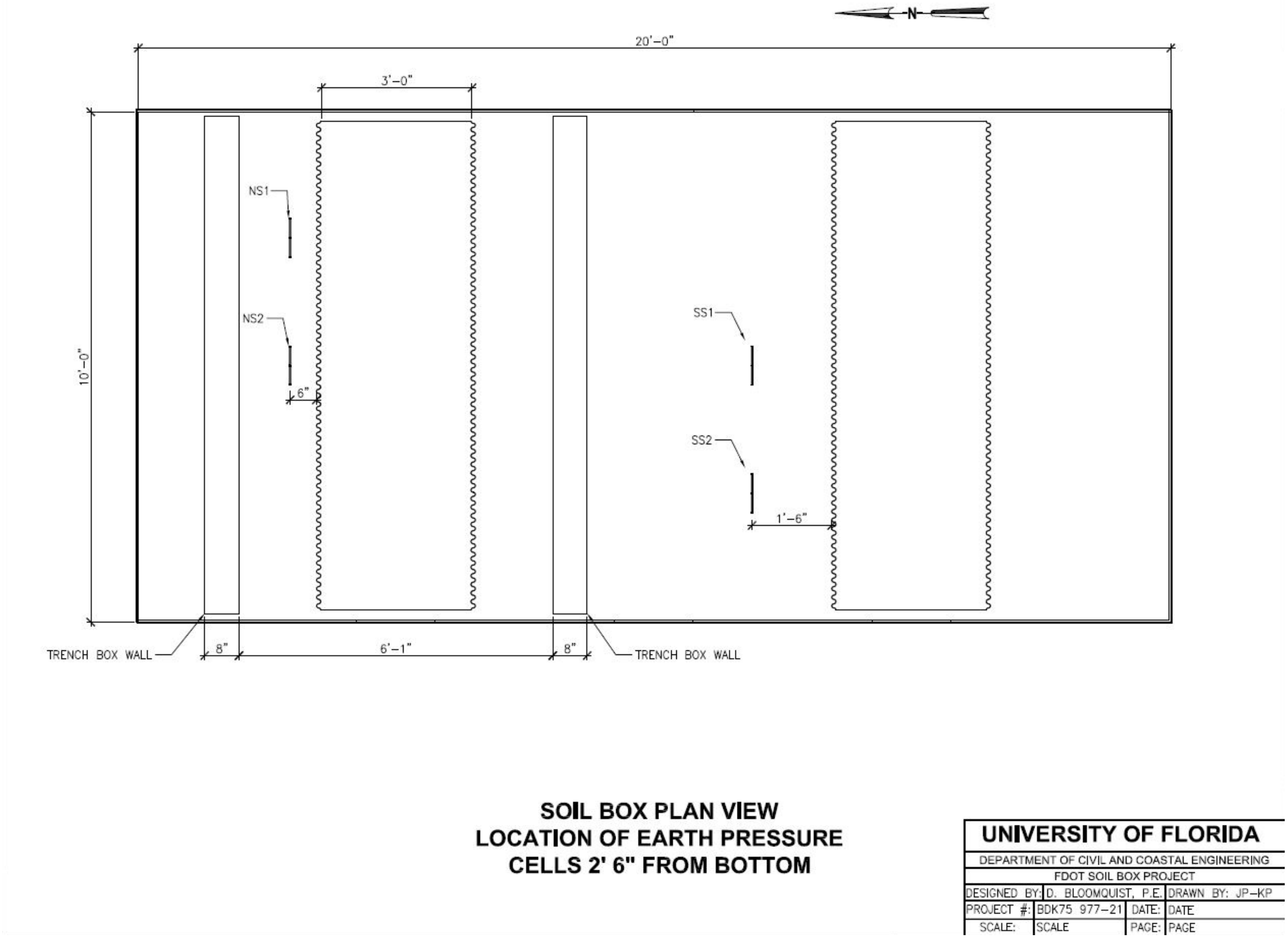


Figure 8-18. Earth pressure cell layout for trench box test one 2'-6" from bottom of Soil Box.

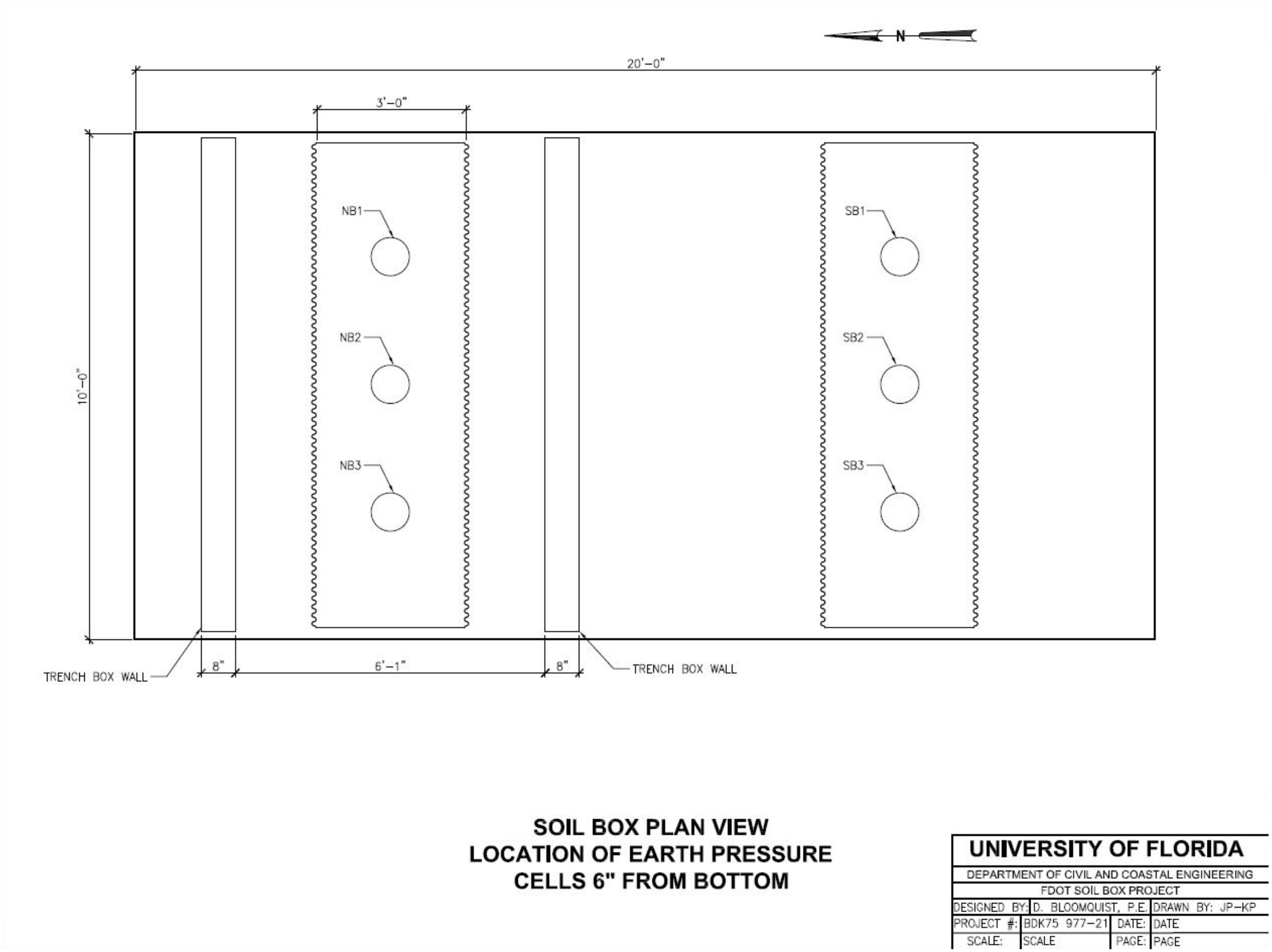


Figure 8-19. Earth pressure cell layout for trench box test one 0'-6" from bottom of Soil Box.

Figure 8-20. Top view of void formed above HDPE pipe end.

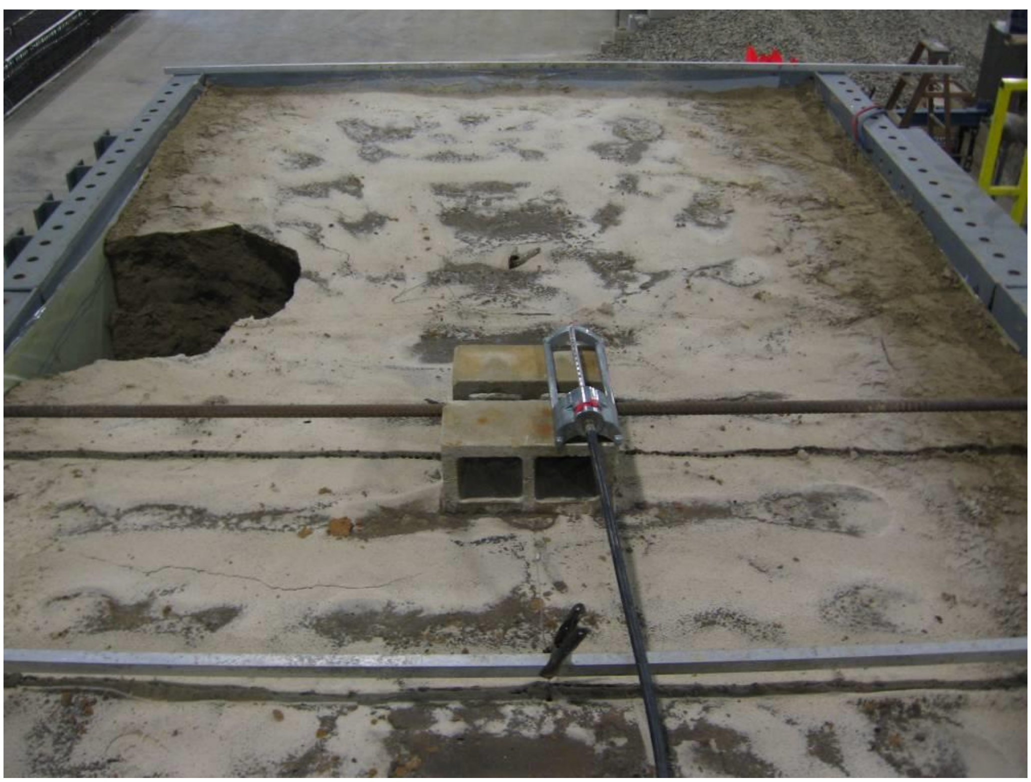


Figure 8-21. Void formed from flexible membrane breach.

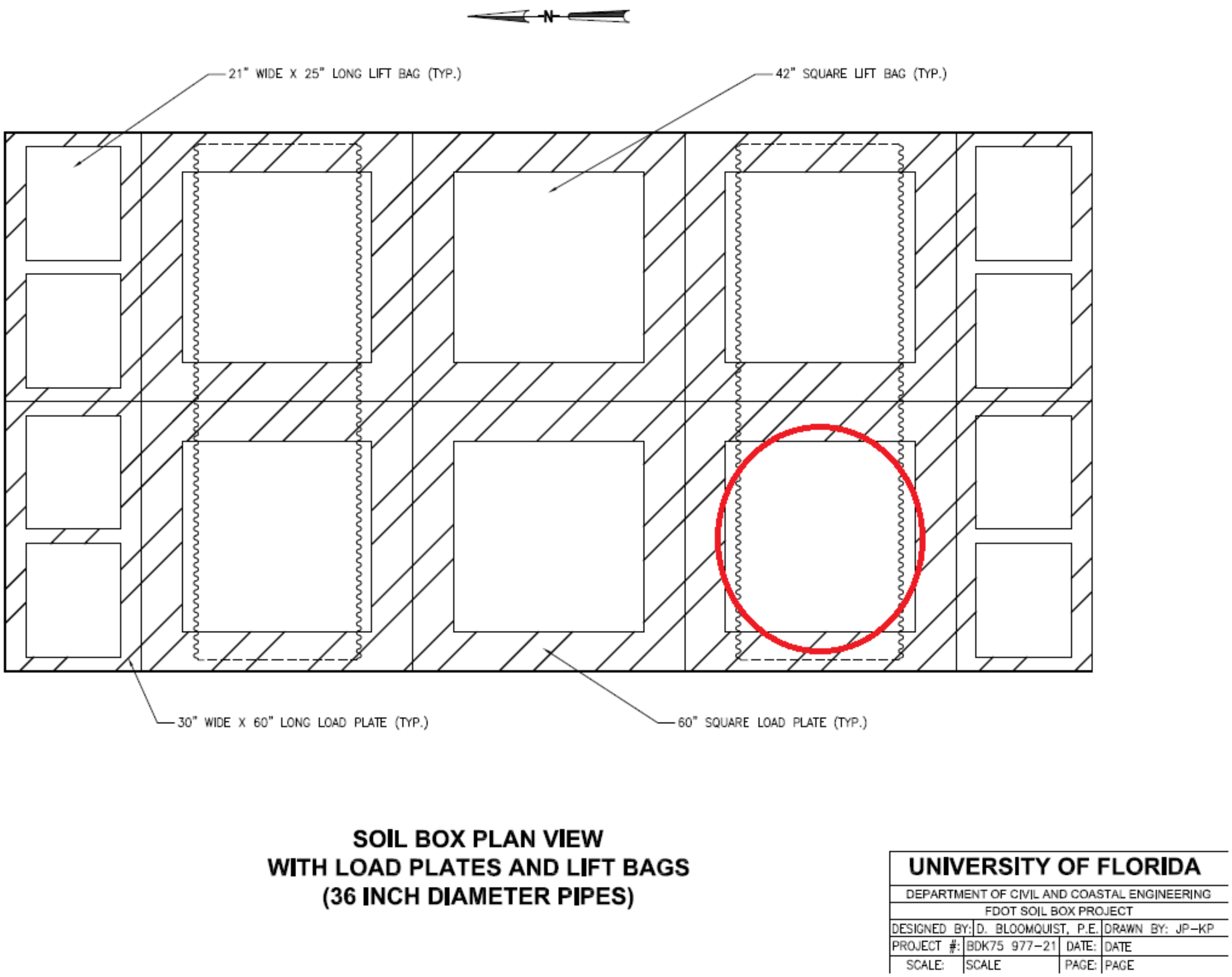


Figure 8-22. Location of large lift bag failure

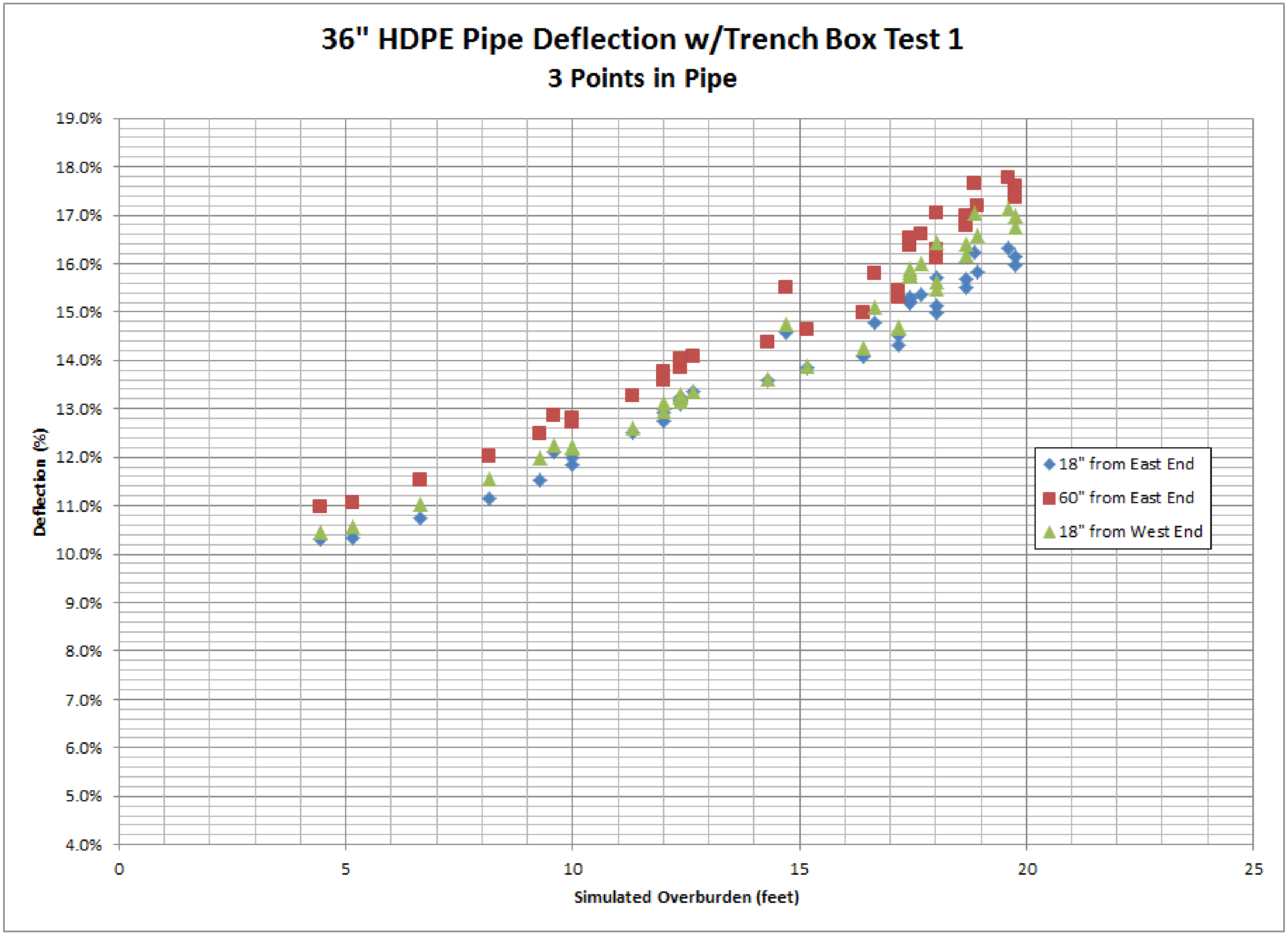


Figure 8-23. Three points in 36” HDPE pipe with trench box, deflection versus simulated overburden.

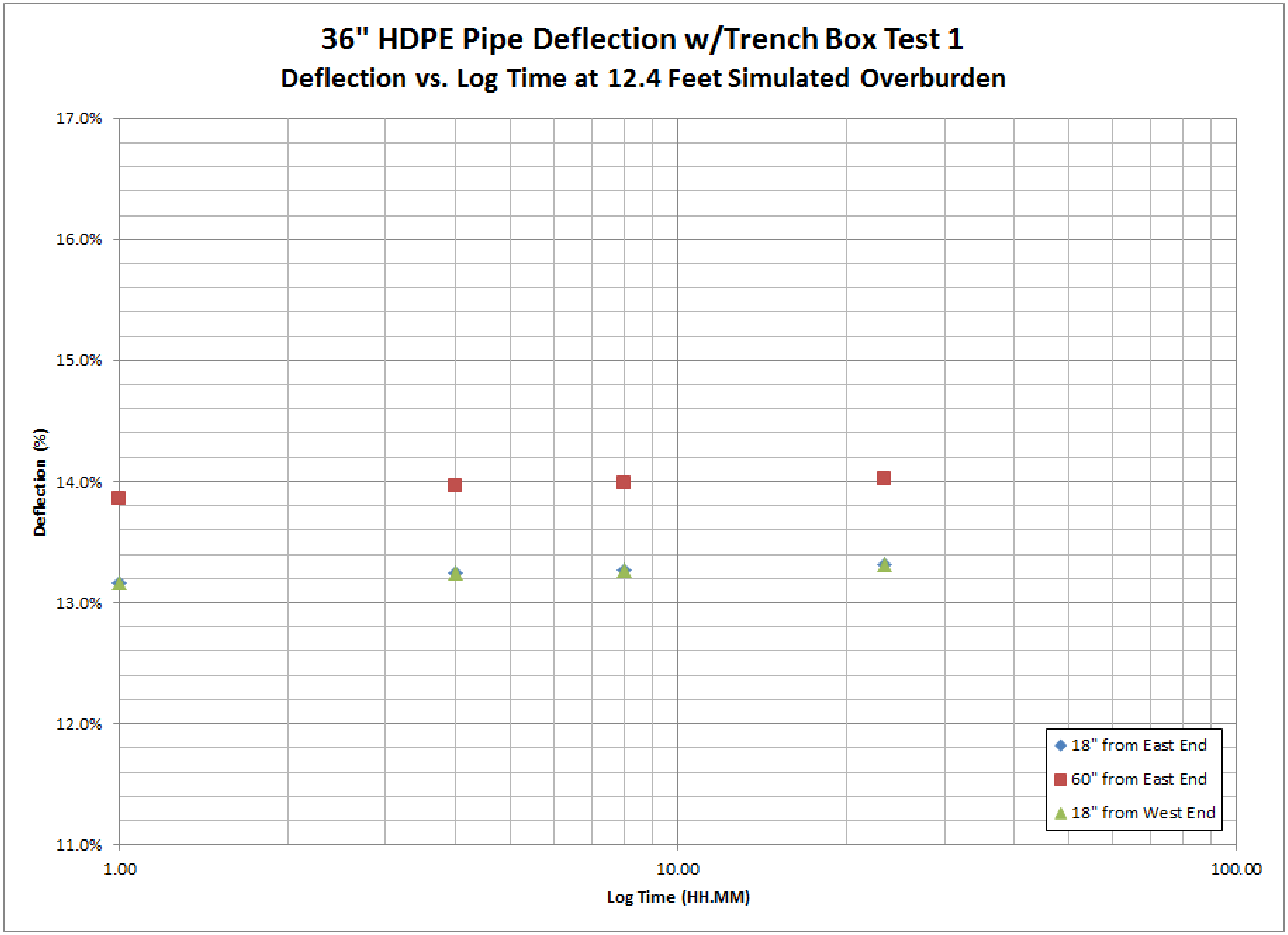


Figure 8-24. Deflection of 36” HDPE pipe with trench box under 12.4 ft of simulated overburden over time.

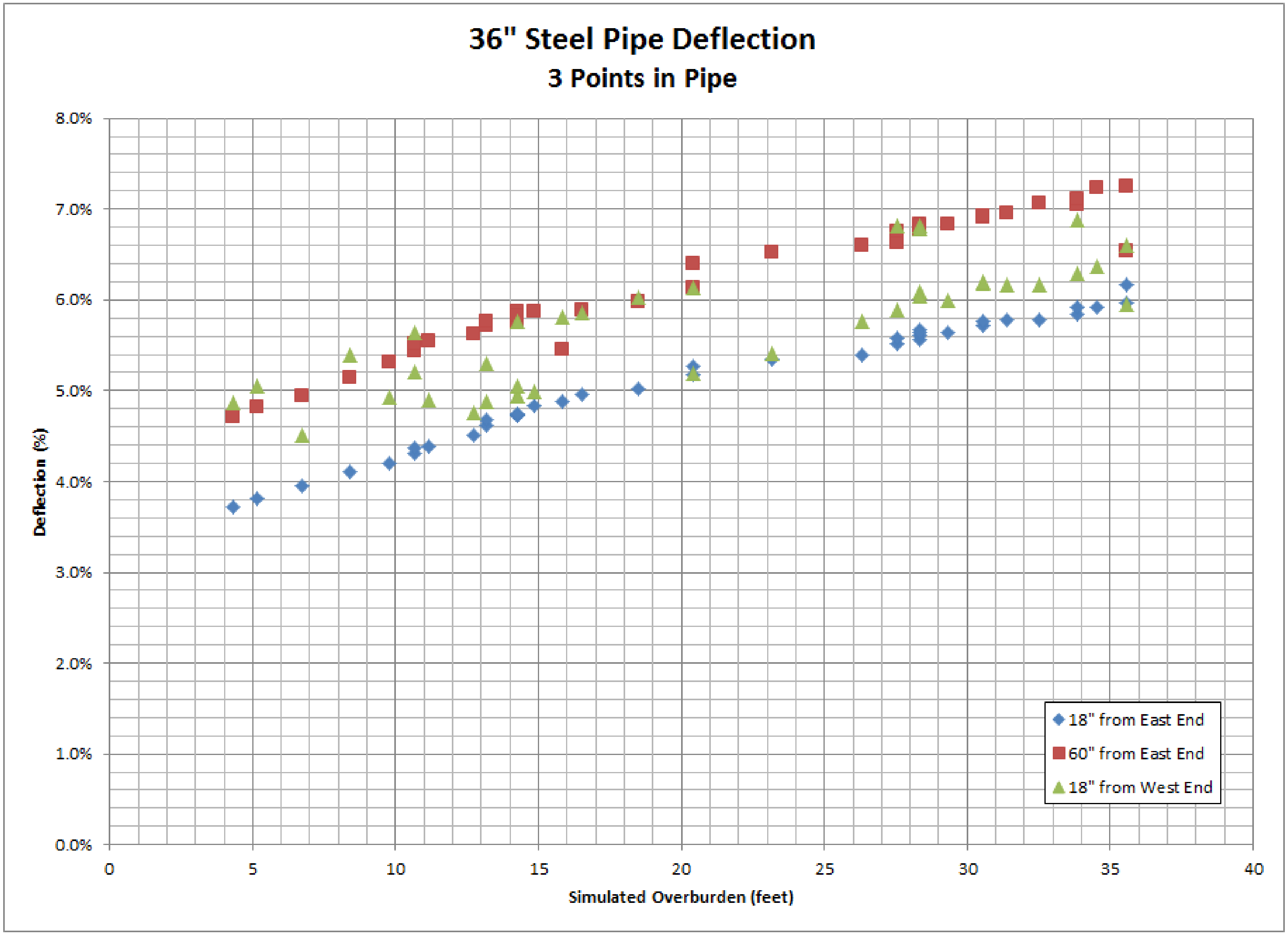


Figure 8-25. Three points in 36” steel pipe, deflection versus simulated overburden.

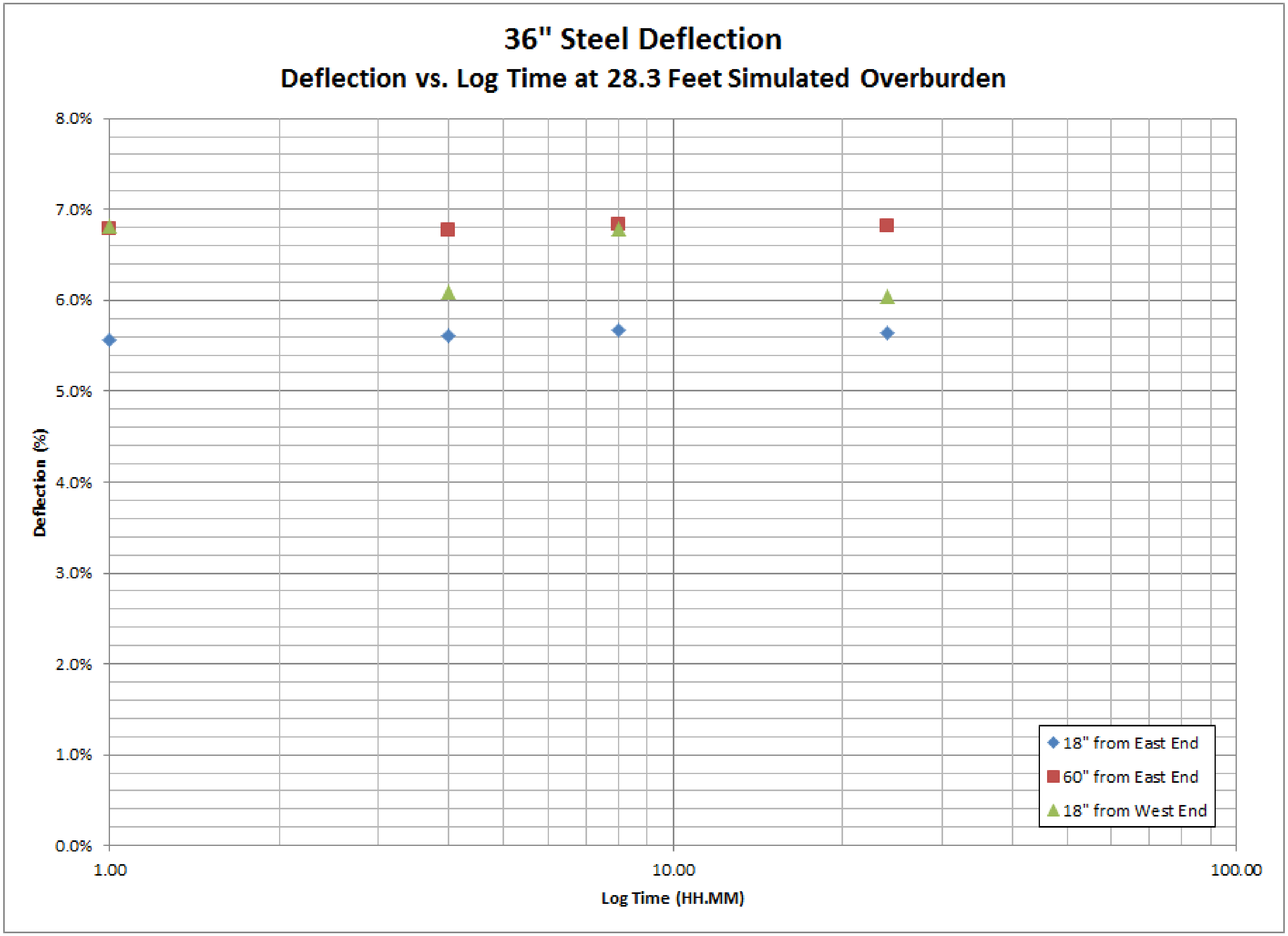


Figure 8-26. Deflection of 36” steel pipe under 28.3 ft of simulated overburden over time.

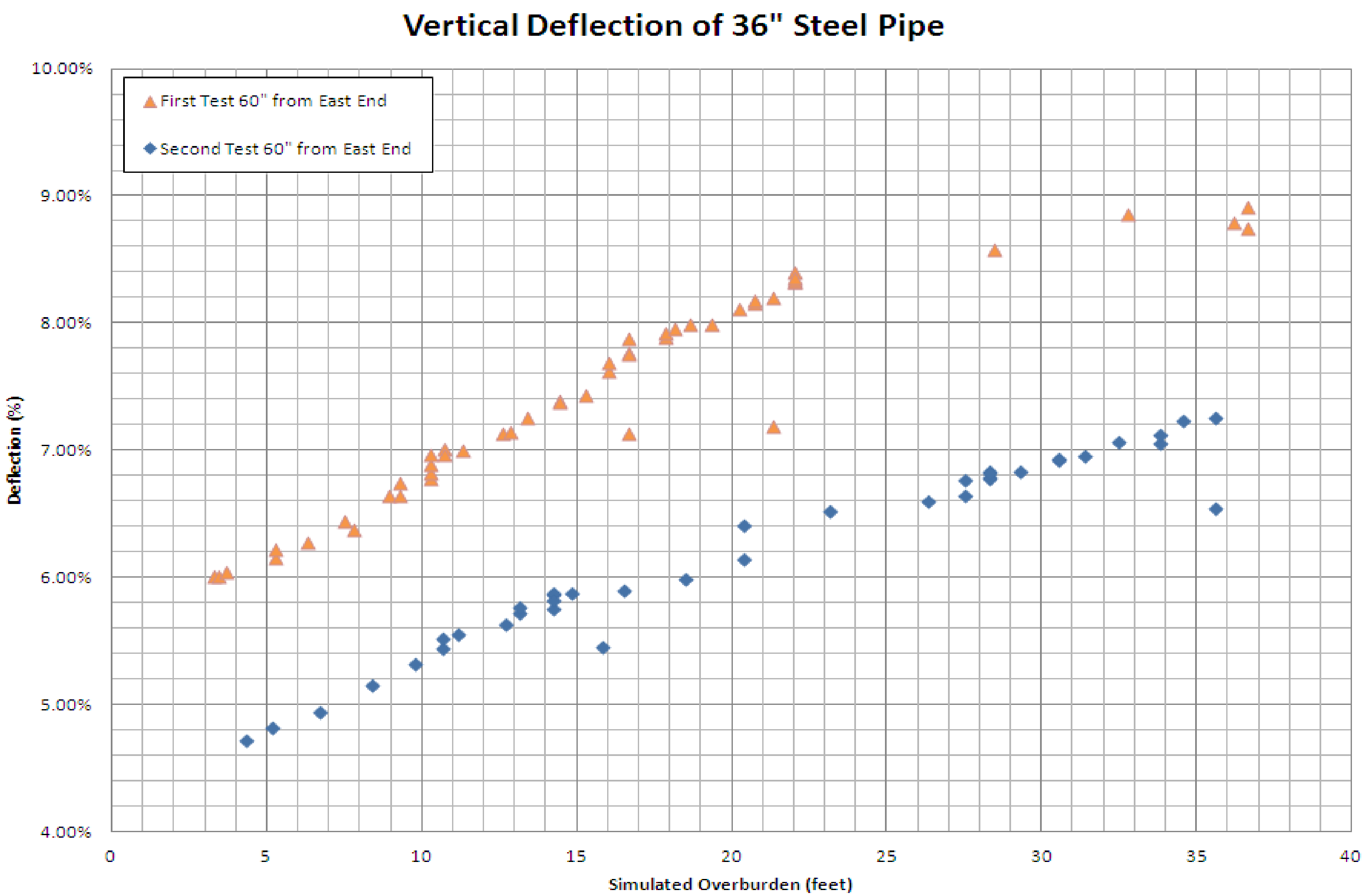


Figure 8-27. Deflection comparison between previous steel pipes test,

CHAPTER 9
36 INCH HDPE PIPE WITH TRENCH BOX AND 36 INCH ALUMINUM PIPE TEST

9.1 Test Modifications

Several modifications were required for the second trench box test. Similar to the first trench box tests, an aluminum ring was riveted on each end of the aluminum pipe and one corrugation was removed on each end of the HDPE pipe (Figures 9-1 and 9-2). This allowed the pipes to connect to the flexible membrane system. Trench box installation and extraction procedures were identical to the previous trench box test (Chapter 8). The locations of the earth pressure cells are shown in Figures 9-3 through 9-9.

9.2 Saturation Failure

As with the previous trench box test, during saturation, soil began flowing from the HDPE pipe's observation portal (Figure 9-10 and Figure 9-11). Again, an inflatable inner tube was used to seal the breach, the void was refilled, and saturation was completed.

9.3 Lift Bag Failures

Two lift bag failures occurred during testing (Figure 9-12). The first failure occurred in a small lift bag located in the Northeast corner of the Soil Box. The lift bag was leaking air through a small tear first observed during the early stages of the loading sequence. When the loading sequence reached 90 pounds per square inch, the lift bag appeared to be leaking significantly. To avoid overstressing the air compressor, the air supply to the lift bag was removed. The second lift bag failure occurred during the 110 pounds per square inch load increment in a large lift bag located in the center east side of the Soil Box. This lift bag ruptured without warning. The rupture resulted in immediate pressure loss. Although the failed lift bags were not located directly over the pipes, they caused a discontinuity of surcharge in the Soil Box, which ultimately reduced the simulated overburden pressure. As mentioned in Chapter 8, these lift bags were utilized through several loading and unloading sequences and most likely failed due to fatigue.

9.4 Results

The HDPE pipe with trench box appeared to show the largest deflections of the pipes tested. A plot of deflection versus simulated overburden for three points (mid-span and 18 inches from each end) in the pipe can be seen in Figure 9-13. As shown, the mid-span point appears to indicate the highest deflection reading. This may indicate the flexible membrane system did not perform as expected. The flexible membrane was designed to prevent soil from exiting the Soil Box while allowing for free deformation of the pipe. This difference of deflection between the three points may be caused by a partially fixed boundary condition at the pipe and wall interface. There is a small amount of creep deflection in the HDPE pipe (Figure 9-14).

A plot of the two tested HDPE pipes with trench box can be seen in Figure 9-15. The simulated overburden for this test yielded lesser depths than the previously tested HDPE pipe with trench box. This result is most likely due to the lift bag failures during testing.

The aluminum pipe yielded deflections less than the HDPE pipe with trench box. This was expected because aluminum is a stiffer material than HDPE. The deflection of three points in the aluminum pipe versus simulated overburden is shown in Figure 9-16. There does not appear to be any significant creep of the aluminum pipe (Figure 9-17). Table 9-1 displays the simulated overburden depth results HDPE pipe with trench box and aluminum pipe.

Table 9-1. Loading sequence, deflection reading timing, and simulated overburden for HDPE with trench box and aluminum pipe test.

Pressure Applied to Main Regulator (PSI)	Deflection Readings taken x hours after pressure was applied	Simulated Overburden Over HDPE Pipe (feet)	Simulated Overburden Over Aluminum Pipe (feet)
0	1	2.2	4.9
5	1	2.5	5.5
10	1	3.3	7.0
15	1	3.9	8.3
20	1	4.6	9.7
25	1	4.9	10.7
30	1	5.6	12.1
35	1	6.2	13.3
40	1	6.8	14.5
45	1, 4, 8, 24	6.8	15.2
50	1	7.0	15.9
55	1	7.8	16.6
60	1	8.2	17.0
65	1	8.6	17.8
70	1	8.4	18.1
75	1	9.0	18.6
80	1	9.4	19.1
85	1	9.8	19.5
90	1, 4, 8, 24	9.3	19.5
95	1	9.2	20.6
100	1	9.6	21.4
105	1	9.9	22.0
110	1	10.2	22.0
115	1	9.6	21.3
120	1	9.8	21.7
125	1	10.1	22.2
130	1, 4, 8, 24	9.9	22.8

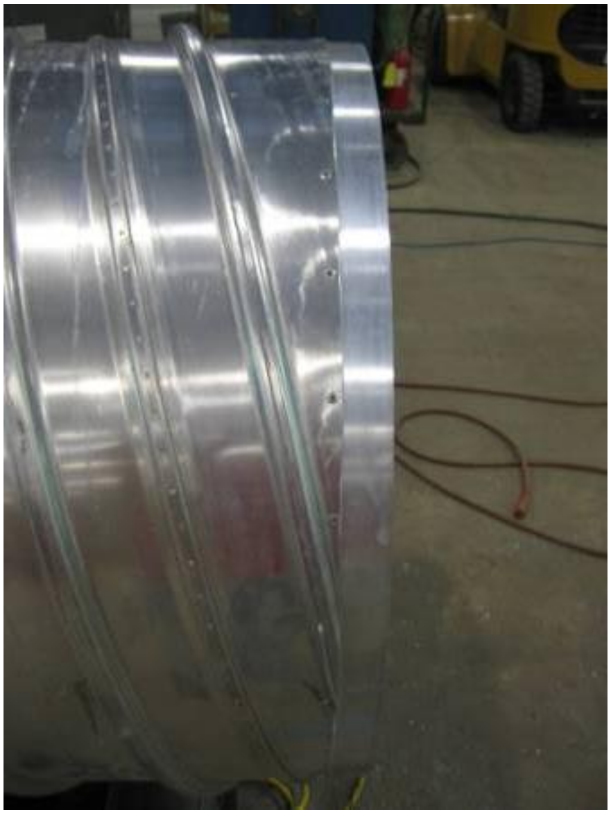


Figure 9-1. Aluminum pipe end modification


Figure 9-2. HDPE pipe end modification

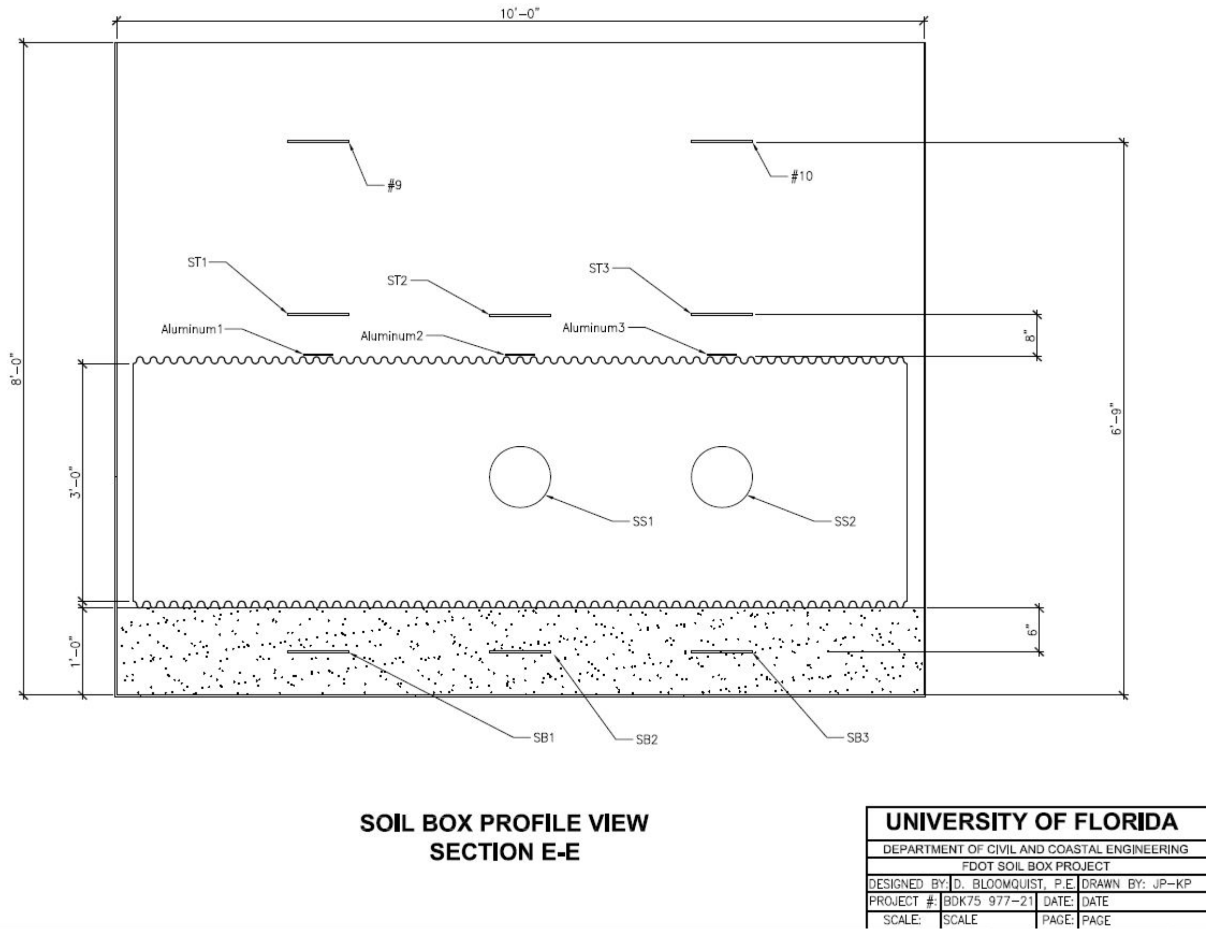


Figure 9-3. Earth pressure cell layout through Section E-E for Trench Box Test Two

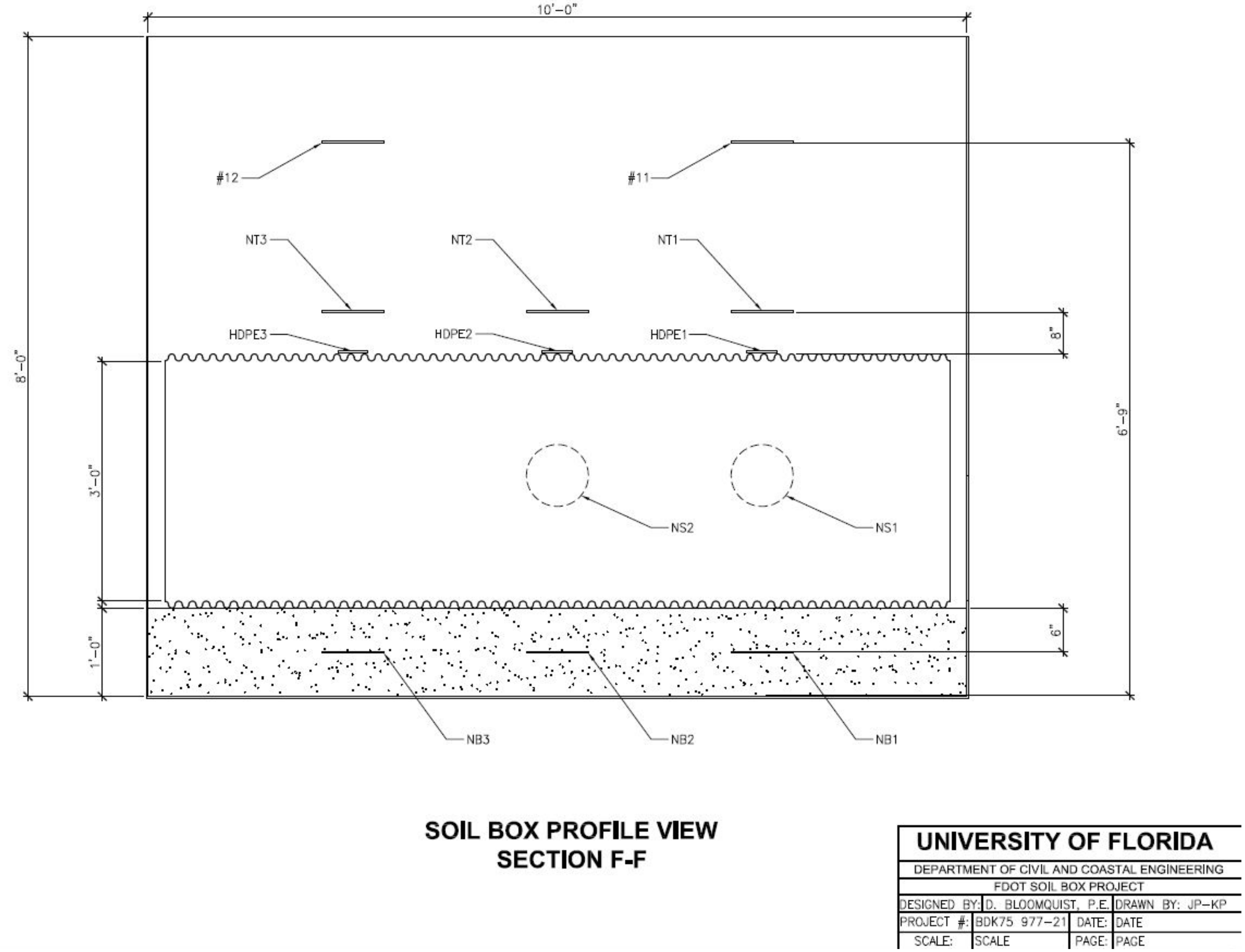


Figure 9-4. Earth pressure cell layout through Section F-F for Trench Box Test Two

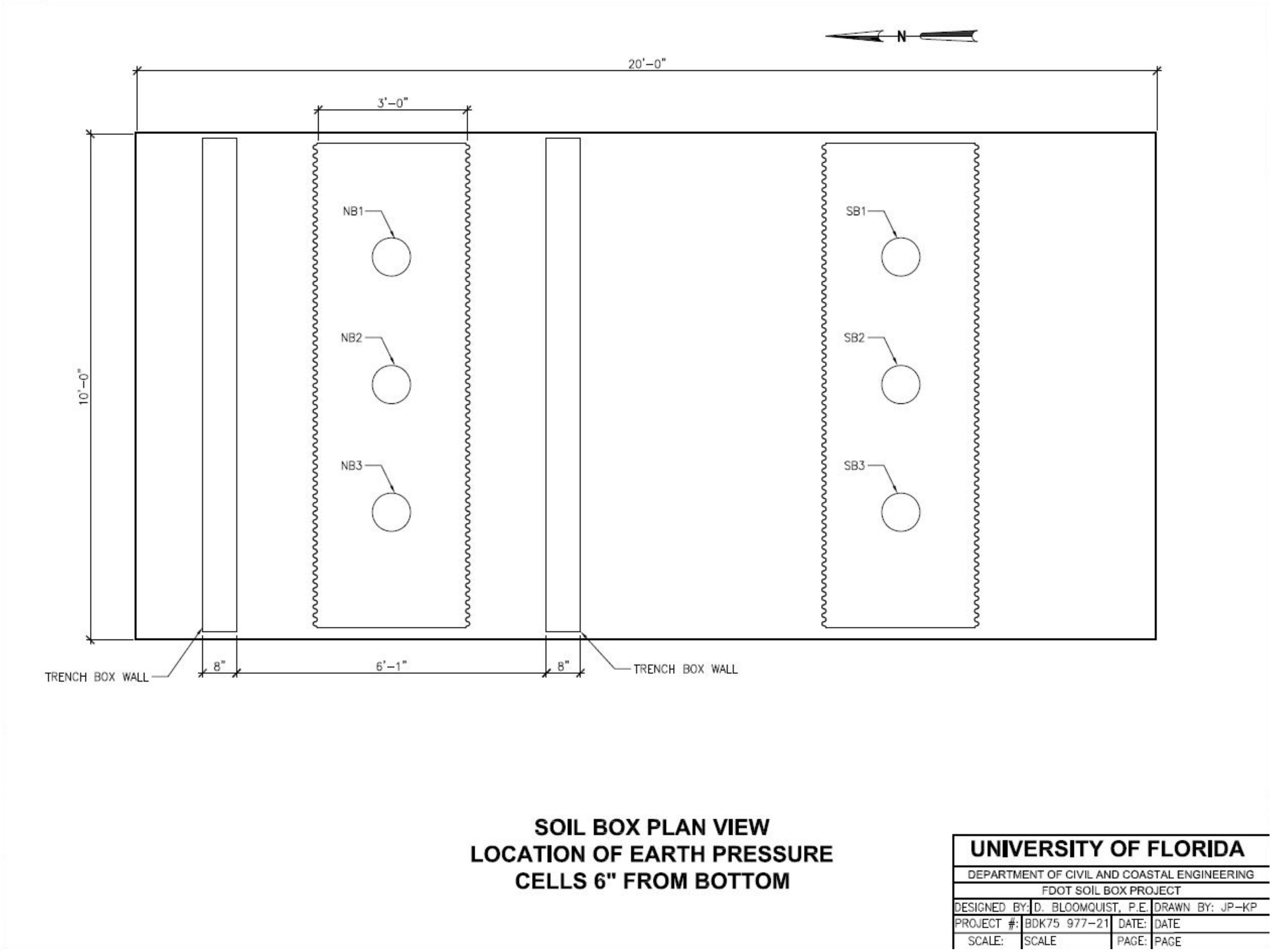


Figure 9-5. Earth pressure cell layout for trench box test two 0’-6” from bottom of Soil Box

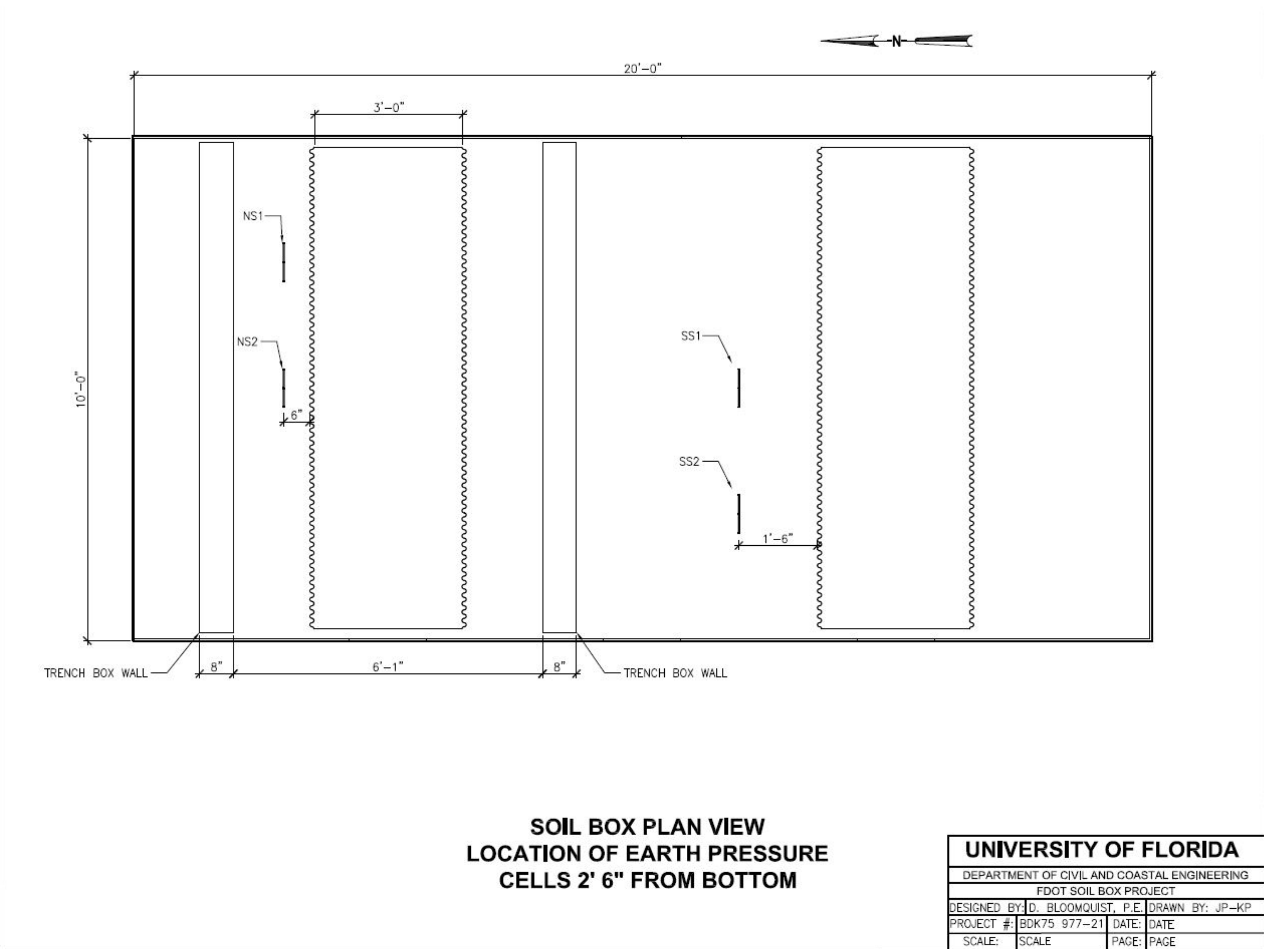


Figure 9-6. Earth pressure cell layout for trench box test two 2'-6" from bottom of Soil Box

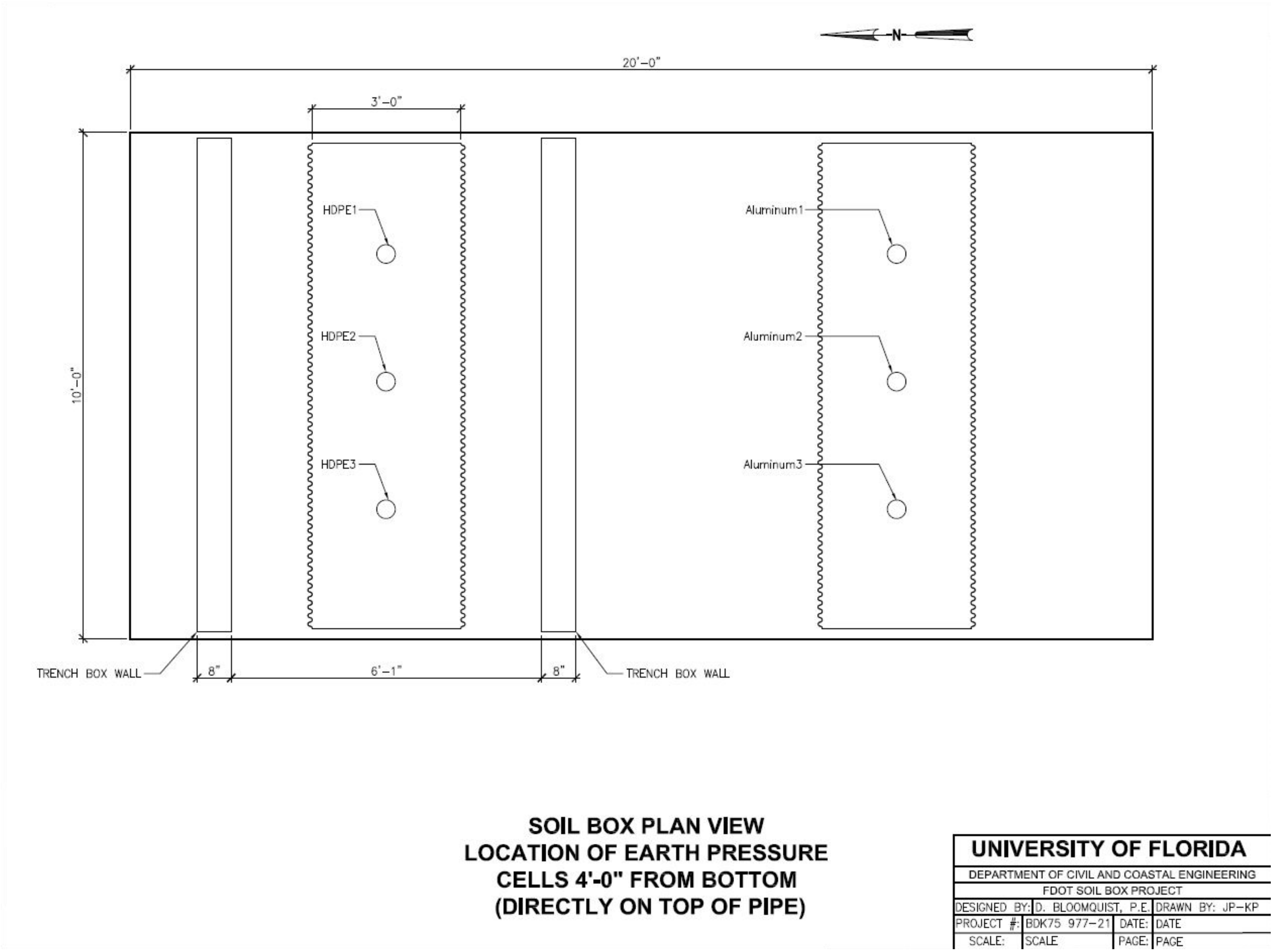


Figure 9-7. Earth pressure cell layout for trench box test two 4'-0" from bottom of Soil Box

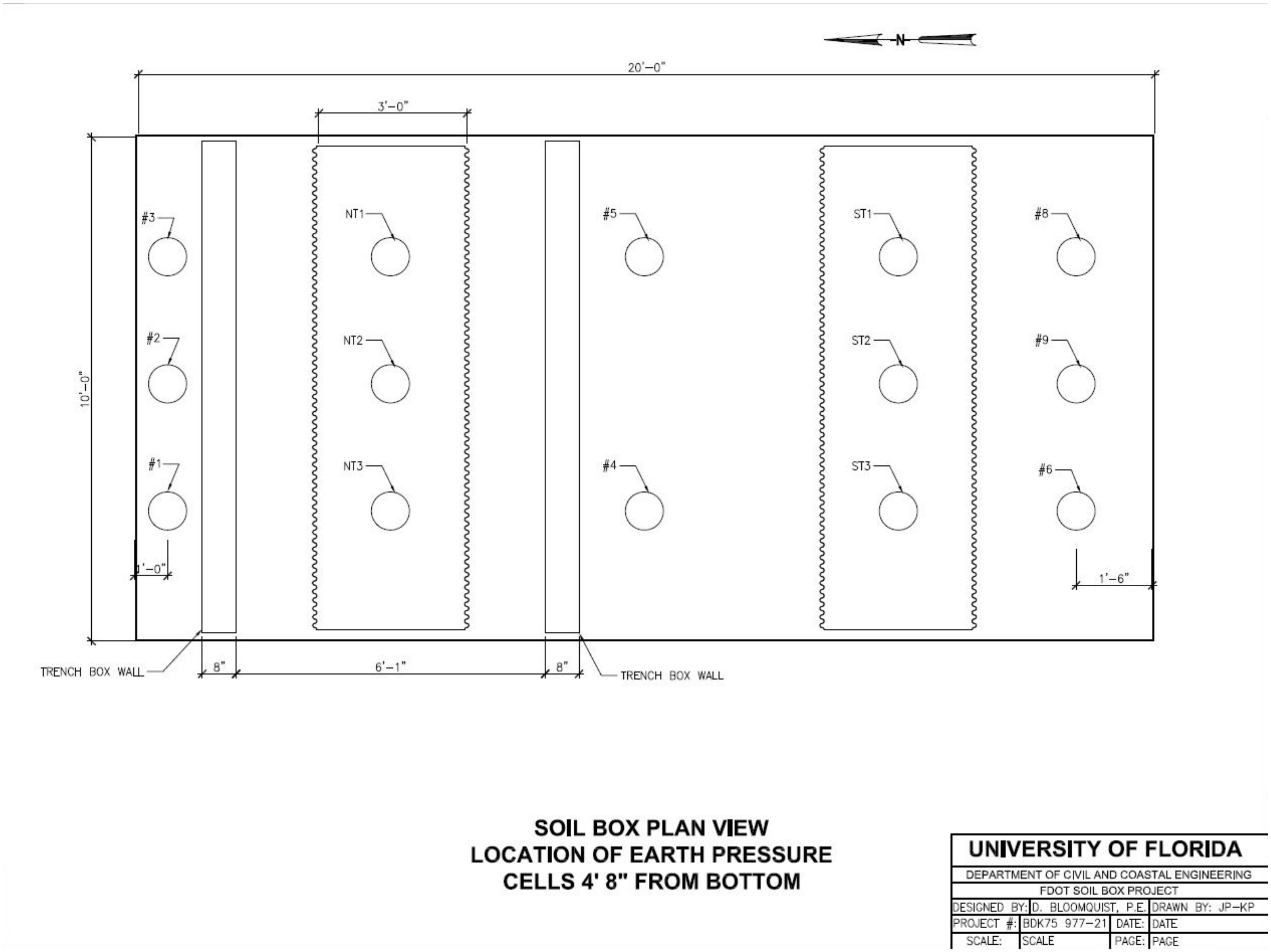


Figure 9-8. Earth pressure cell layout for trench box test two 4'-8" from bottom of Soil Box

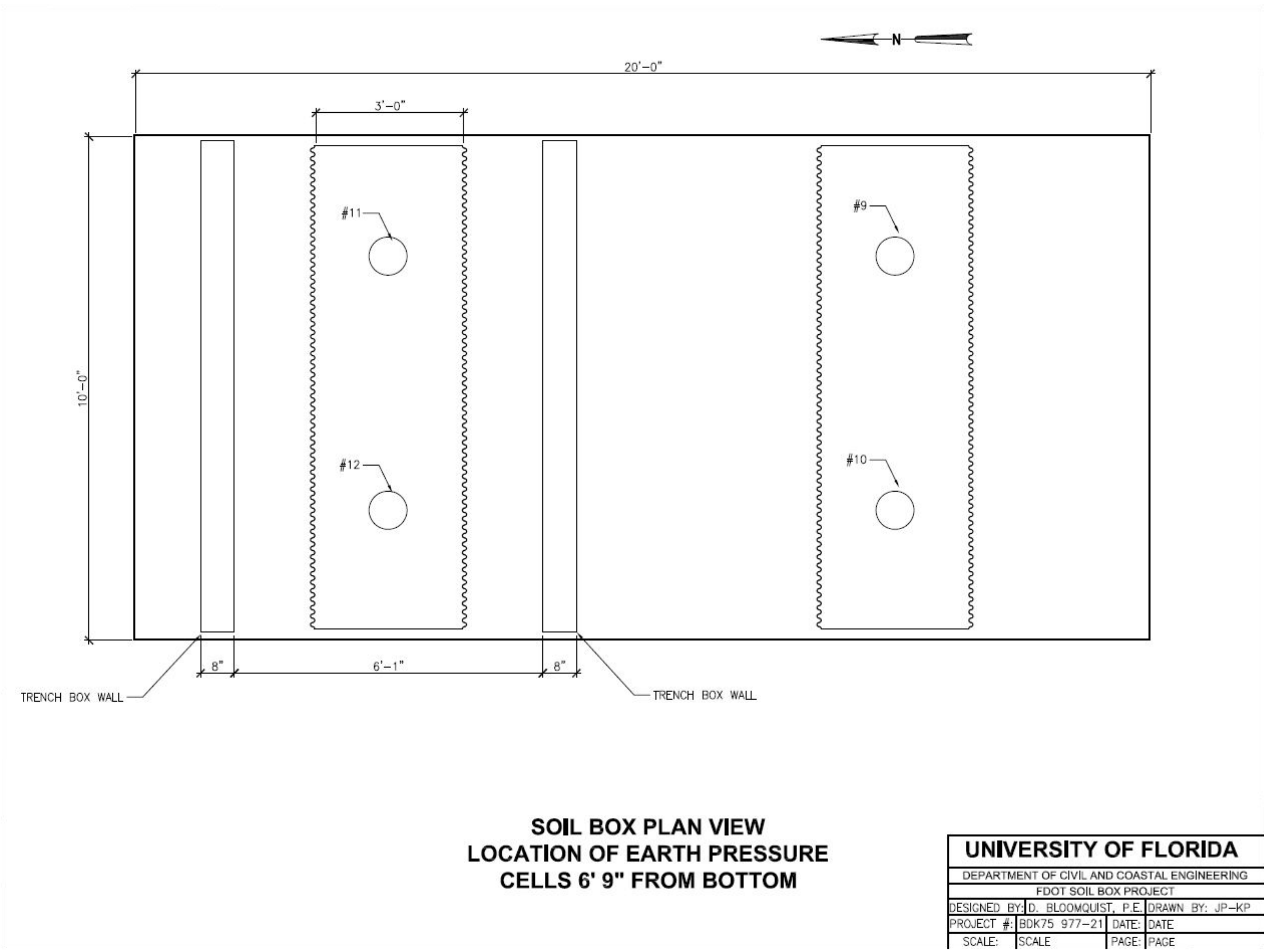


Figure 9-9. Earth pressure cell layout for trench box test two 6'-9" from bottom of Soil Box

Figure 9-10. Void formed from flexible membrane breach

Figure 9-11. Top view of void (circle showing HDPE pipe)

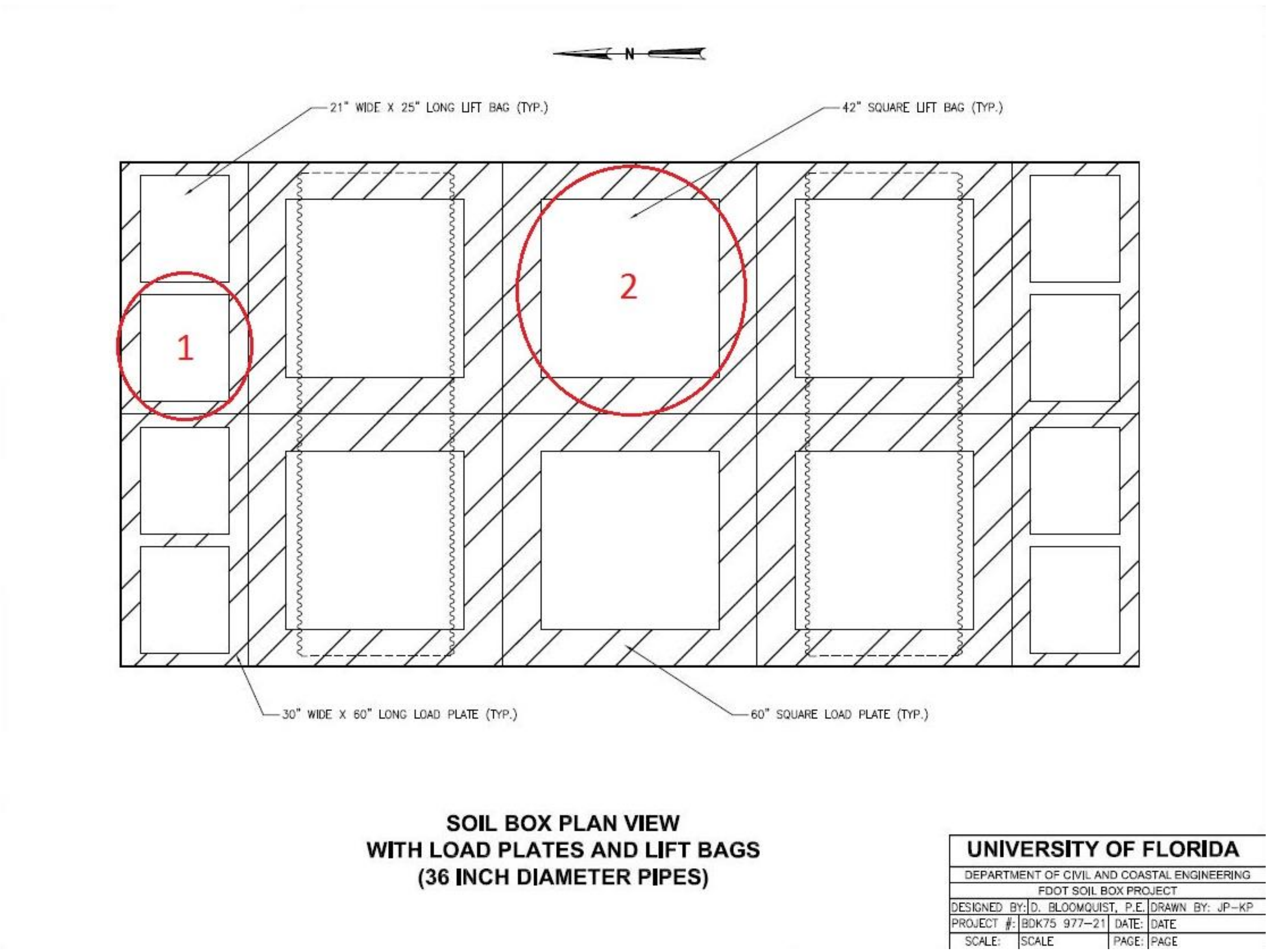


Figure 9-12. Lift bag failure locations.

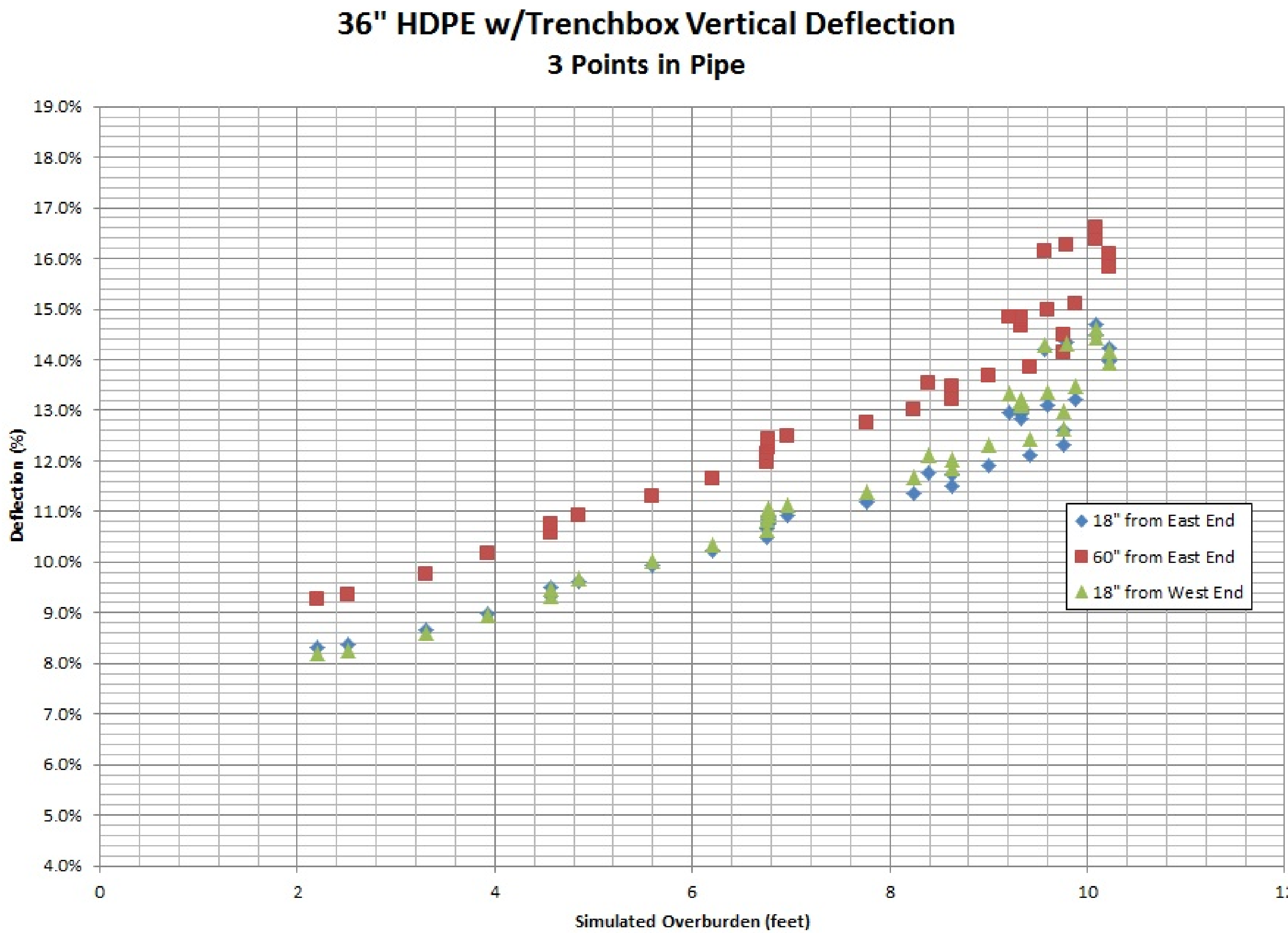


Figure 9-13. Three points in 36” HDPE pipe with trench box, deflection versus simulated overburden.

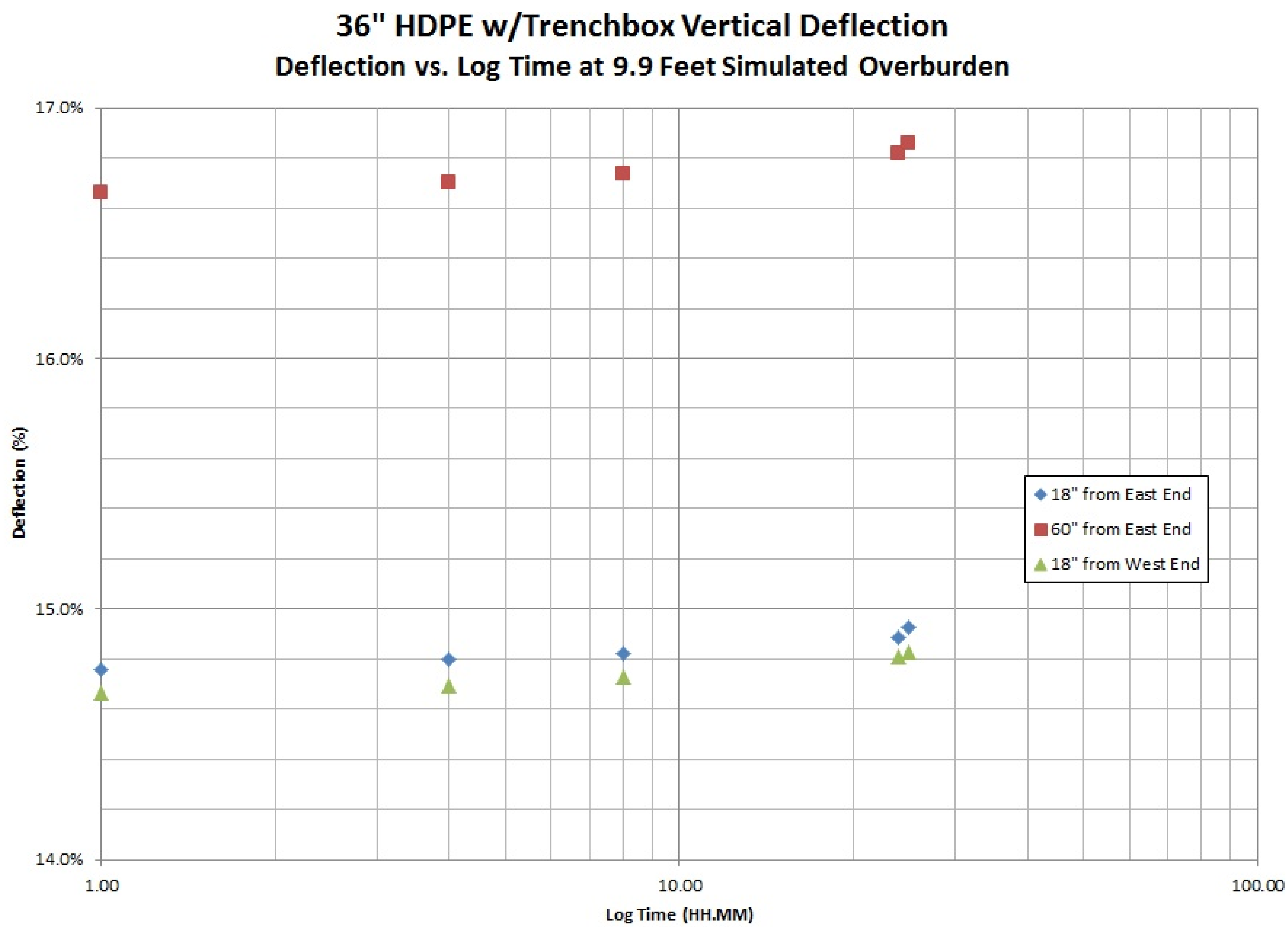


Figure 9-14. Deflection of 36” HDPE pipe under 9.9ft of simulated overburden over time.

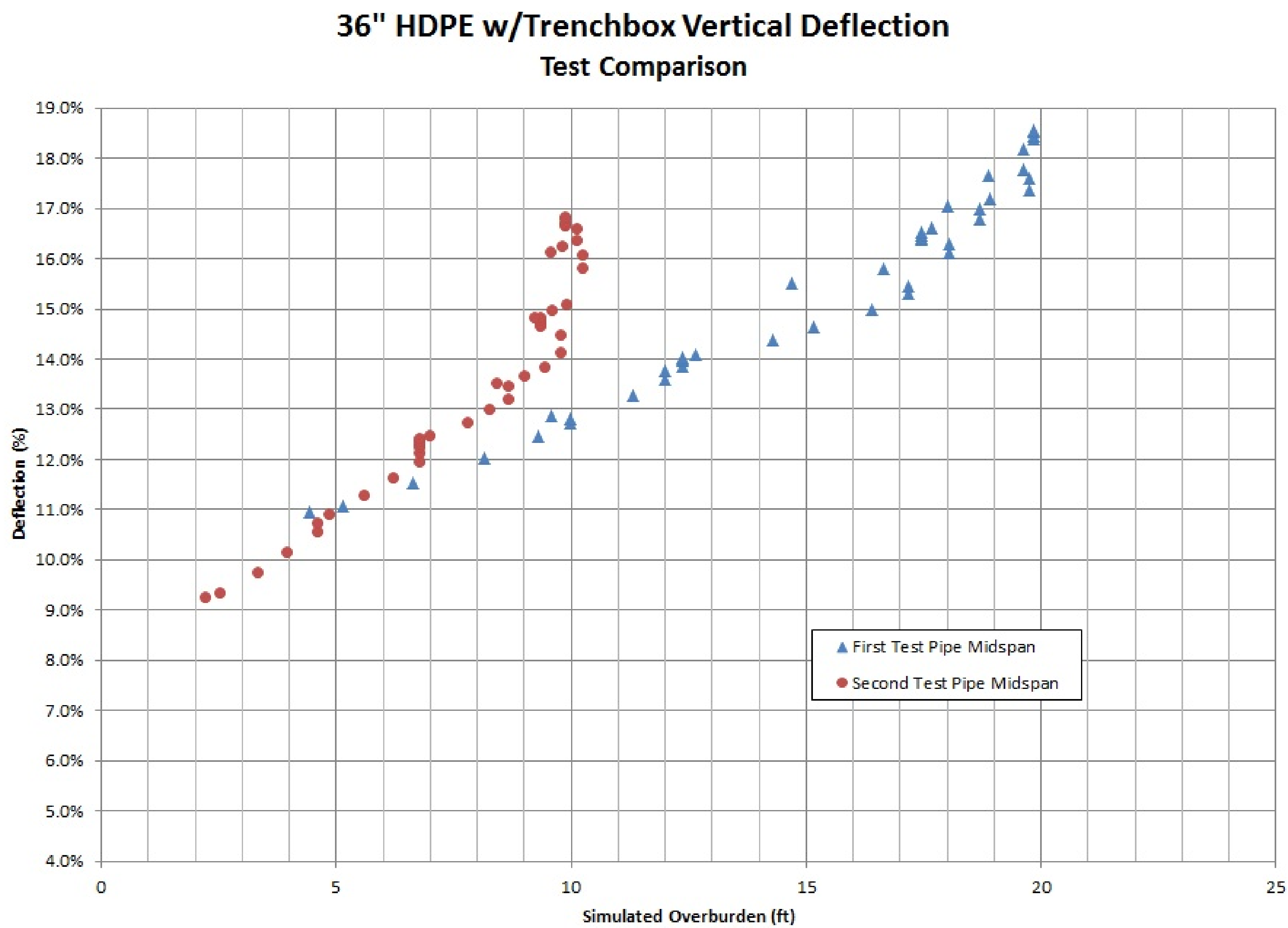


Figure 9-15. Deflection comparison of 36” HDPE pipe with trench box tests.

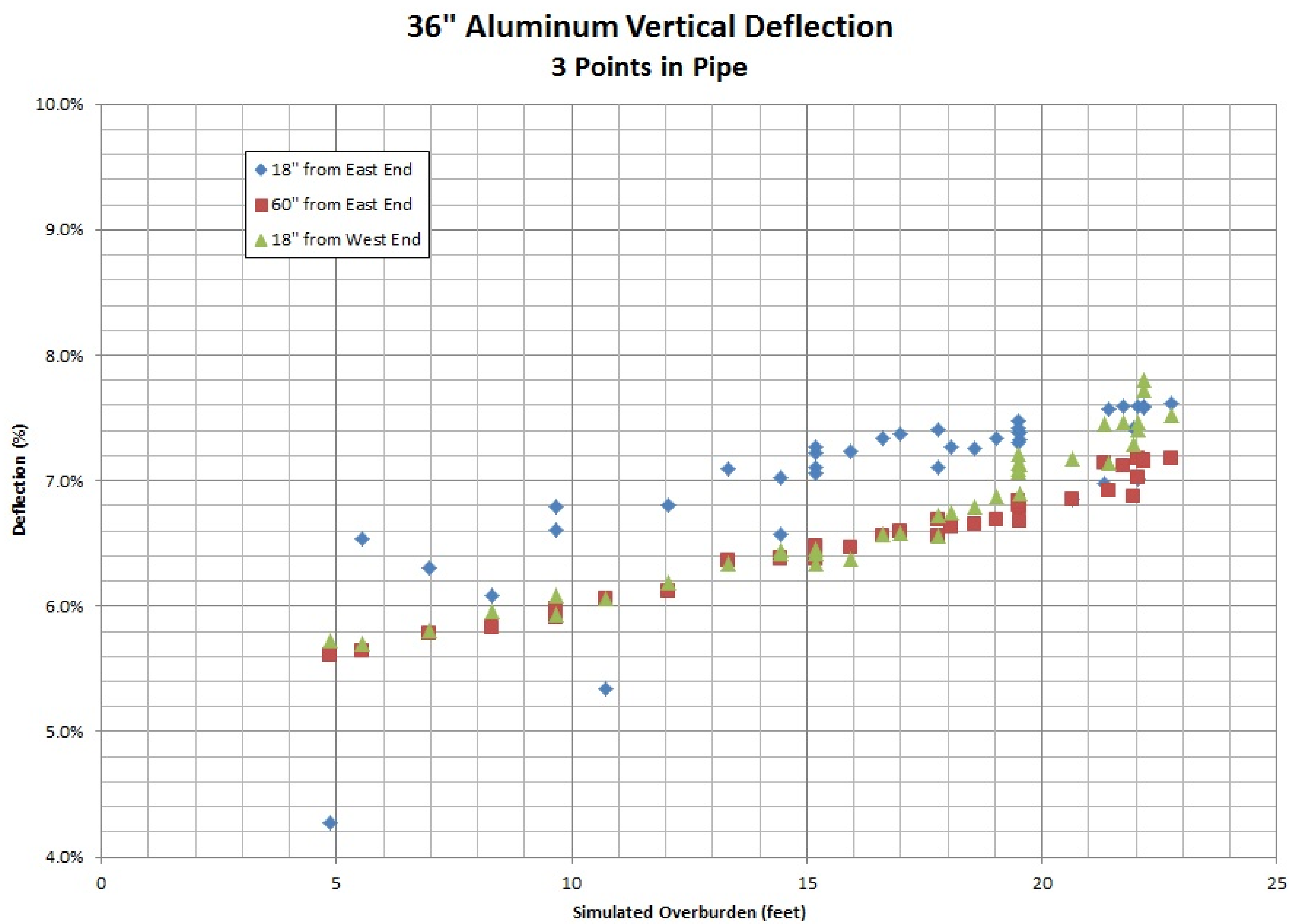


Figure 9-16. Three points in 36” aluminum pipe, deflection versus simulated overburden.

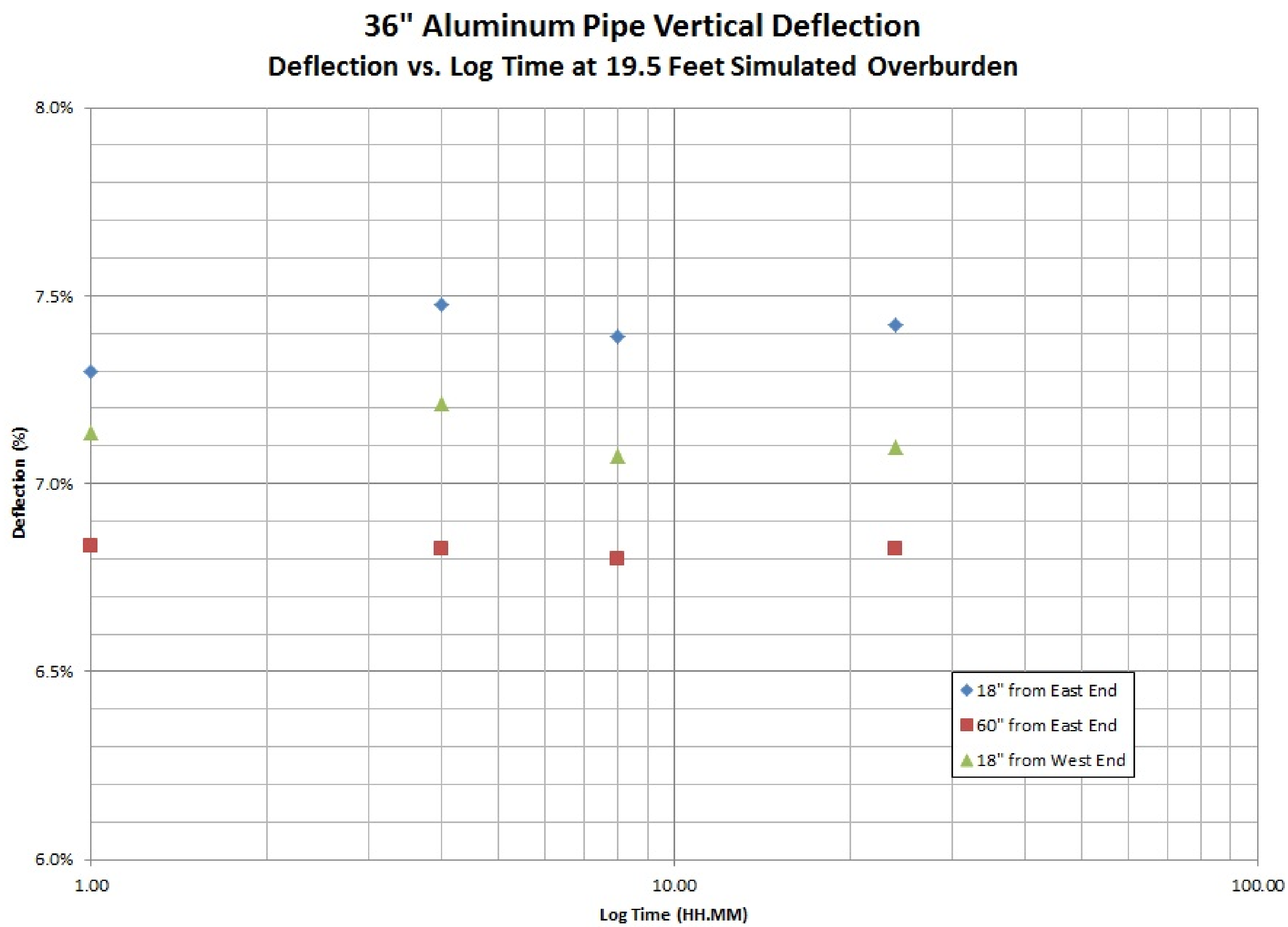


Figure 9-17. Deflection of 36” aluminum pipe under 19.5ft of simulated overburden over 24 hour

CHAPTER 10
24 INCH HDPE PIPE TEST

10.1 Test Modifications

Several modifications were required for the 24 inch diameter HDPE pipe test. The locations of the pressure cells were modified to account for the smaller pipe diameter; these locations are shown from Figure 10-1 through Figure 10-7.

The flexible membrane system was modified to fit the 24 inch pipes. A replacement rigid steel angle was fabricated, painted, and bolted to the Soil Box wall. As before, a single corrugation on each end of the HDPE pipes was removed to allow assembly to the flexible membrane system (Figures 10-8 and 10-9).

The laser previously used to measure displacements of the 36 inch pipes did not have adequate precision to measure displacements of the 24 inch pipes. A new ILD1700-250VT laser was purchased from Micro-Epsilon. Two aluminum plates were attached to the laser's exterior casing so that it could be properly mounted on the laser profiling system (Figures 10-10 and 10-11).

10.2 Lift Bag Failure

A lift bag failure occurred during the 110 pounds per square inch load increment in a small lift bag located in the Northeast corner of the Soil Box (Figure 10-12). Although the failed lift bag was not located directly over the pipes, it caused a discontinuity of surcharge in the Soil Box, which ultimately reduced the simulated overburden. This may explain the difference in simulated overburden of the two HDPE pipes shown in Table 10-1. Like the previous bags, investigators believe this bag failed due to fatigue.

10.3 Results

The North HDPE pipe appeared to produce approximately uniform deflection along its length (Figure 10-13). Results appear to show creep deflections in this pipe over a 24 hour time period (Figure 10-14). A deflection versus simulated overburden plot of three points in the South HDPE pipe displays a 2% deflection discrepancy between each end of the pipe (Figure 10-15). The creep characteristics for the South HDPE pipe over a 24 hour time period displays a similar upward trend (Figure 10-16).

The North pipe consistently deflected about 2% more than the South pipe during the loading sequence (Figure 10-17). This deflection appears to conflict with the overburden simulations calculated for both pipes (Table 10-1). The simulated overburdens for the South pipe versus the North pipe are higher, but the deflections recorded in the South pipe versus the North pipe are smaller. A possible reason for this discrepancy is the variation of the soil density in the Soil Box and variable contact areas between the lift bags and their corresponding load plates. During the loading sequence, the load plate settles with the soil causing a reduction in contact area between the lift bag and load plate. This reduction in contact area decreases the simulated overburden pressure applied to the soil.

Table 10-1. Simulated overburden for 24 inch HDPE pipe test.

Pressure Applied to Main Regulator (PSI)	Deflection Readings taken x hours after pressure was applied	Simulated Overburden Over North HDPE Pipe (feet)	Simulated Overburden Over South HDPE Pipe (feet)
0	1	3.8	4.2
5	1	4.4	4.7
10	1	5.6	6.3
15	1	5.7	6.8
20	1	6.4	7.5
25	1	8.9	10.3
30	1	9.5	11.0
35	1	11.0	12.9
40	1	12.4	14.6
45	1, 4, 8, 24	12.9	15.0
50	1	13.3	15.5
55	1	14.7	17.3
60	1	15.0	17.5
65	1	15.4	17.8
70	1	16.6	19.4
75	1	16.6	19.2
80	1	17.2	20.0
85	1	18.6	21.6
90	1, 4, 8, 24	19.1	22.0
95	1	19.0	21.8
100	1	19.8	22.8
105	1	20.8	23.9
110	1	21.8	25.1
115	1	21.6	23.7
120	1	22.3	24.6
125	1	23.0	25.3
130	1, 4, 8, 24	23.4	26.2

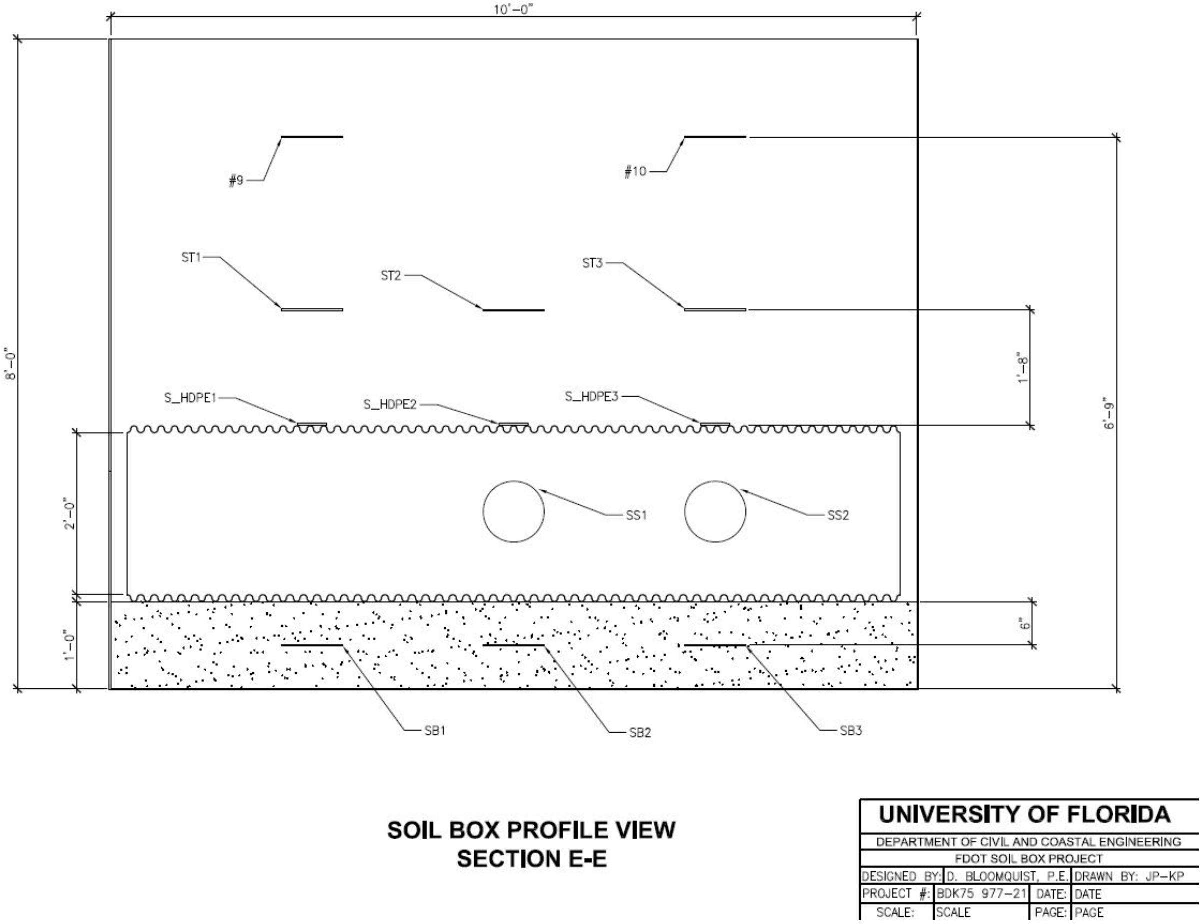


Figure 10-1. Earth pressure cell layout through Section E-E for 24 inch HDPE pipe test

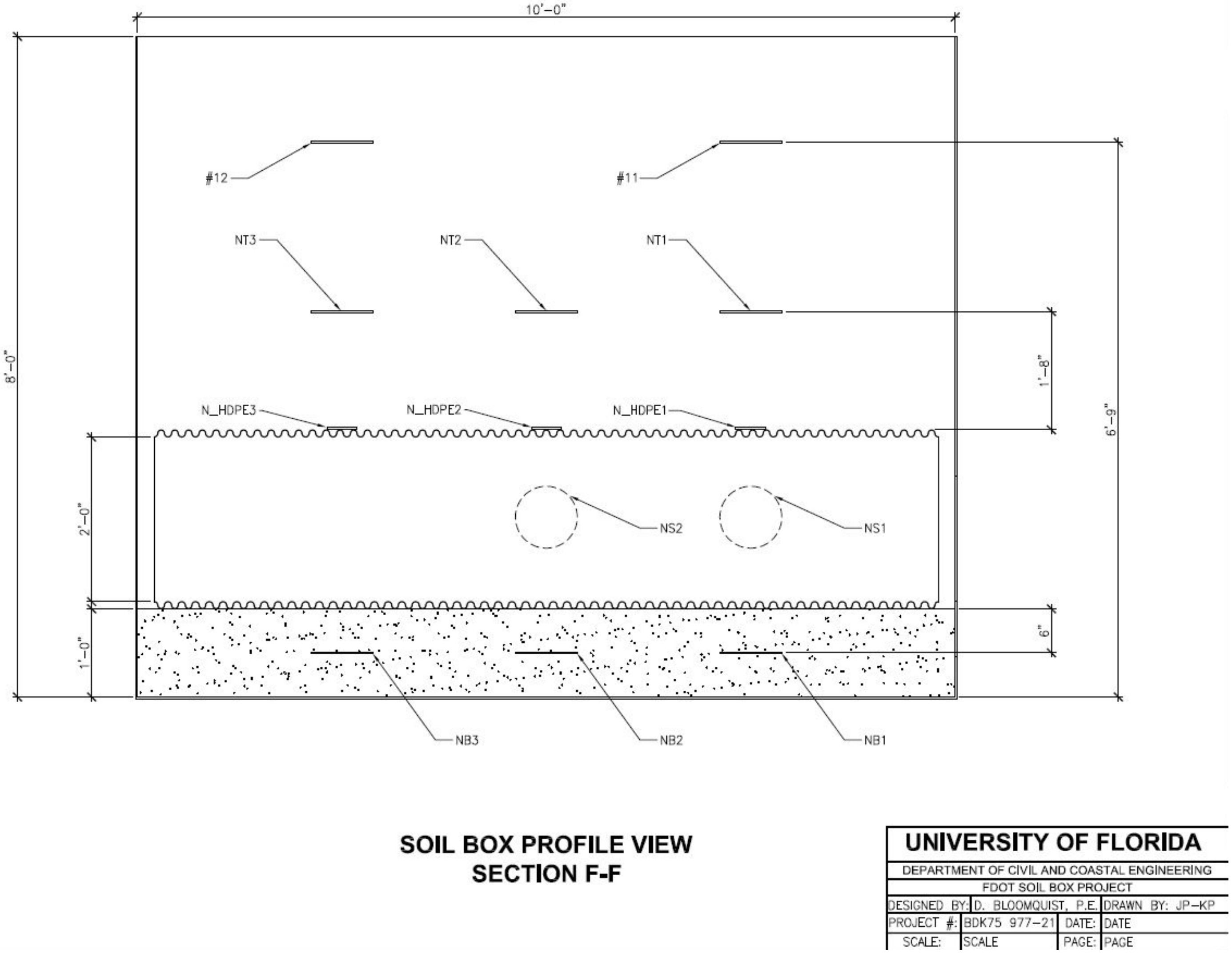


Figure 10-2. Earth pressure cell layout through Section F-F for 24 inch HDPE pipe test

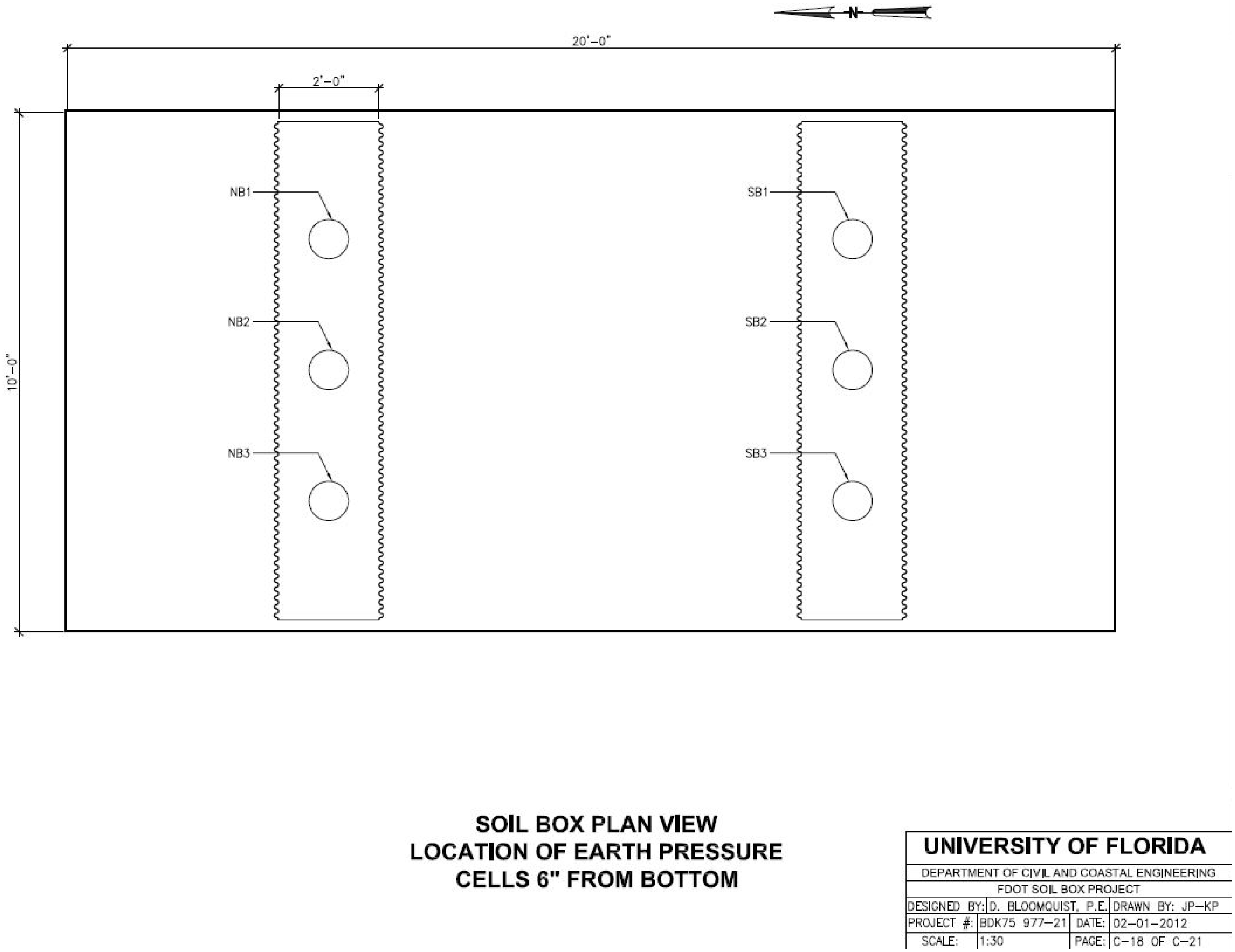


Figure 10-3. Earth pressure cell layout for 24 inch HDPE pipe test 0'-6" from bottom of Soil Box

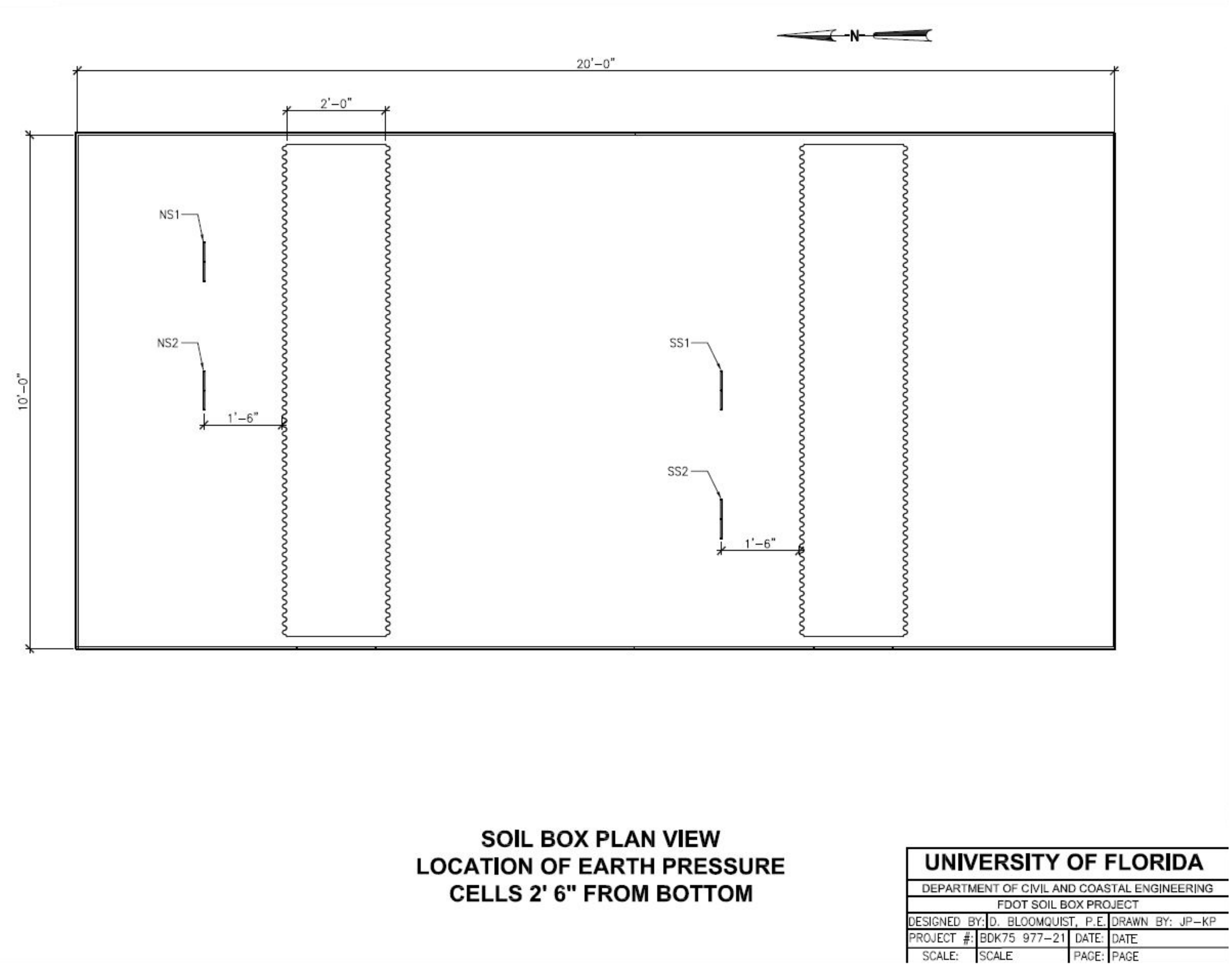


Figure 10-5. Earth pressure cell layout for 24 inch HDPE pipe test 2’-6” from bottom of Soil Box

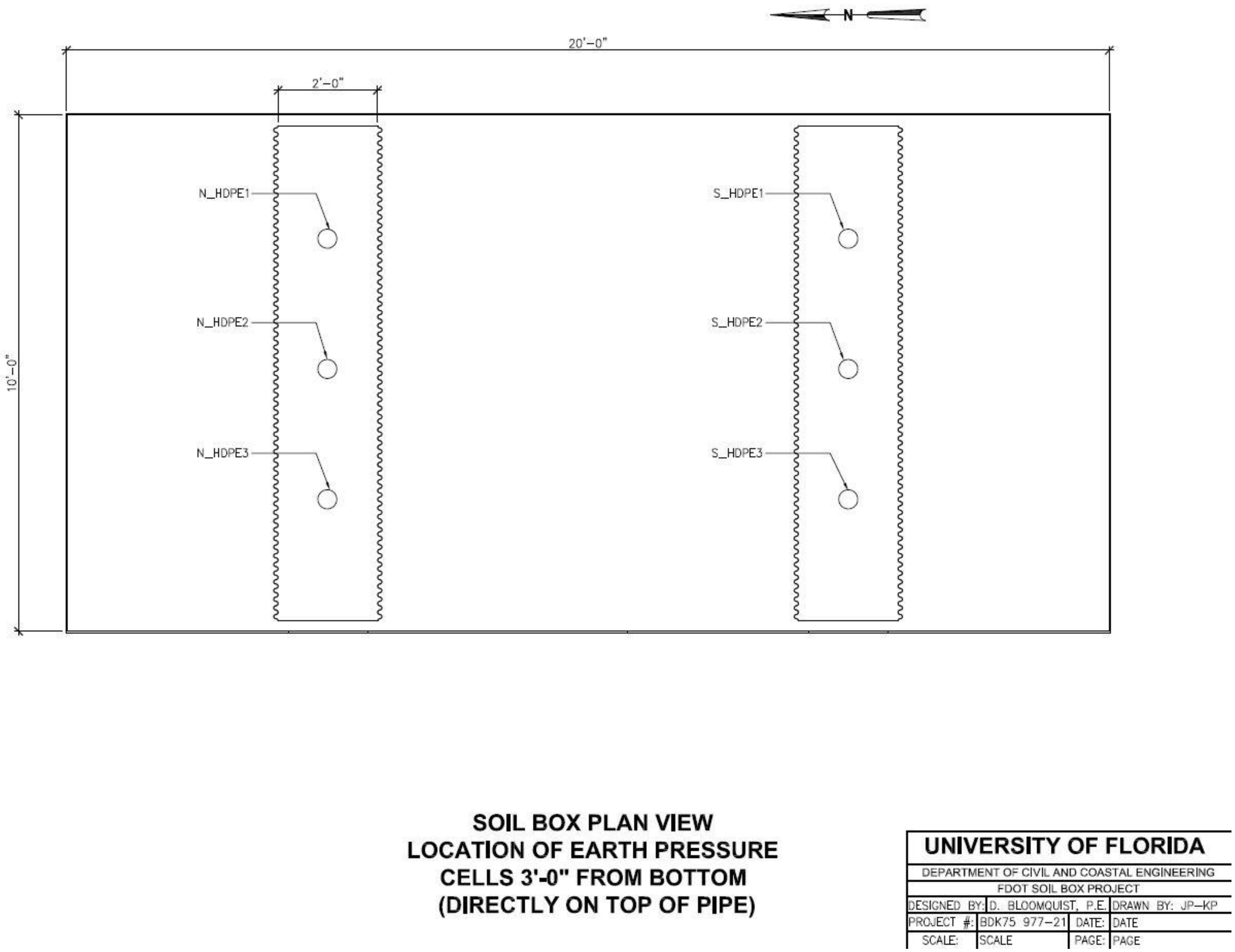


Figure 10-6. Earth pressure cell layout for 24 inch HDPE pipe test 3'-0" from bottom of Soil Box

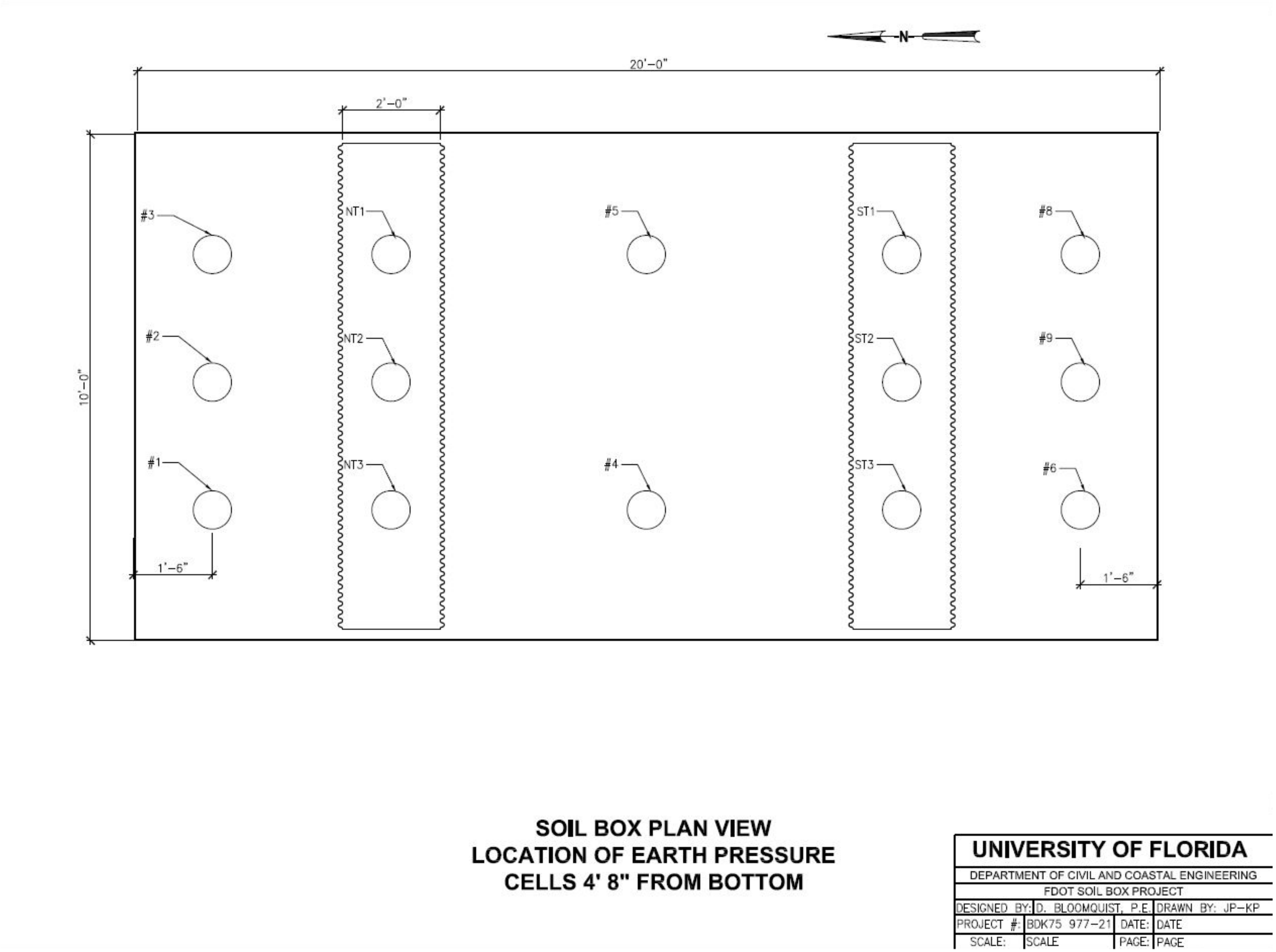


Figure 10-7. Earth pressure cell layout for 24 inch HDPE pipe test 4'-8" from bottom of Soil Box

Figure 10-8. Shaved corrugation on HDPE pipe prior to flexible membrane installation

Figure 10-9. Installed flexible membrane system

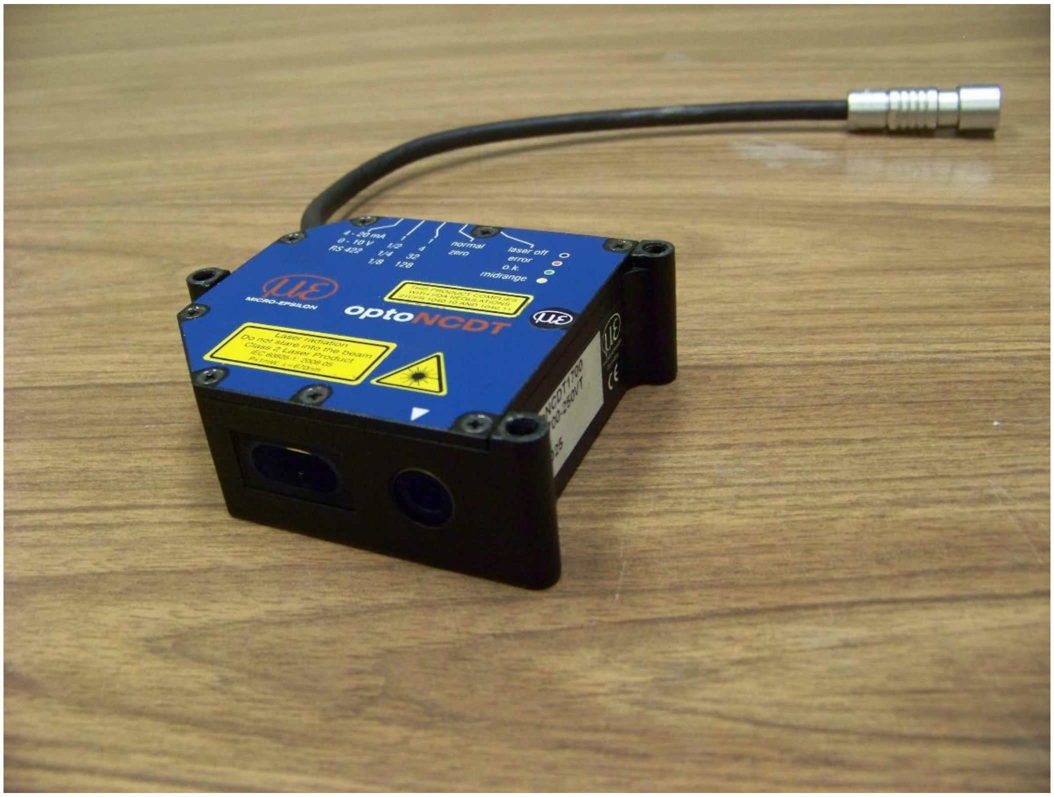


Figure 10-10. Purchased laser for 24 inch pipe testing

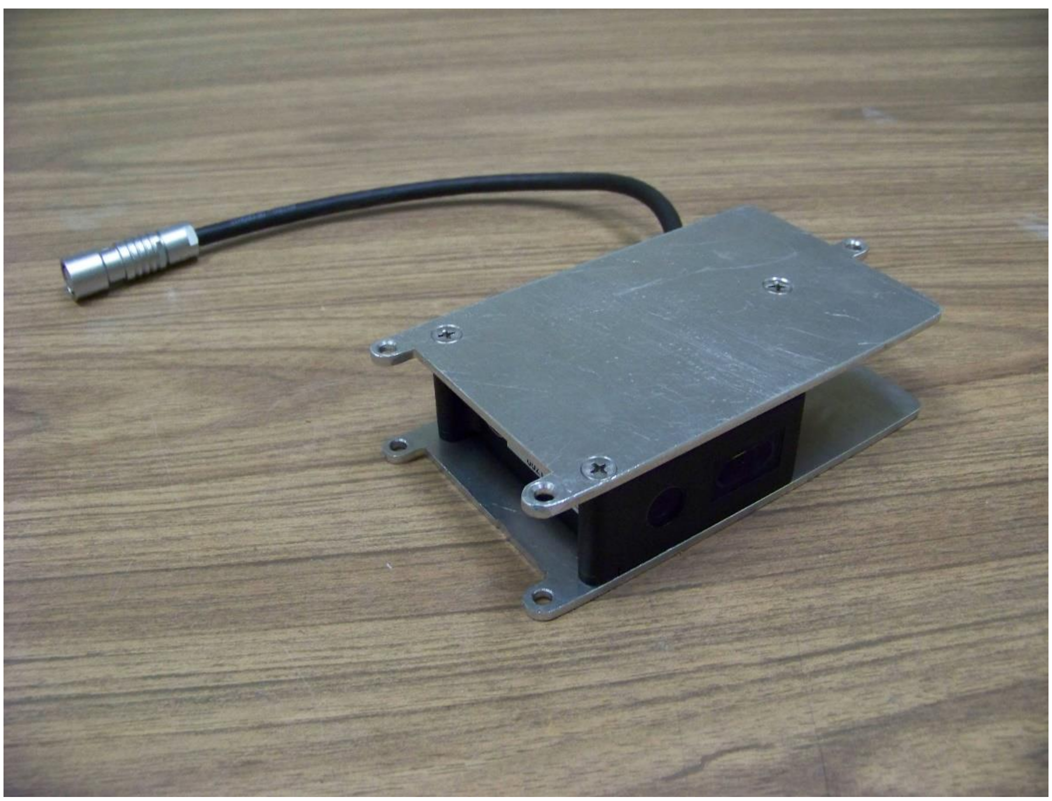


Figure 10-11. Modified laser

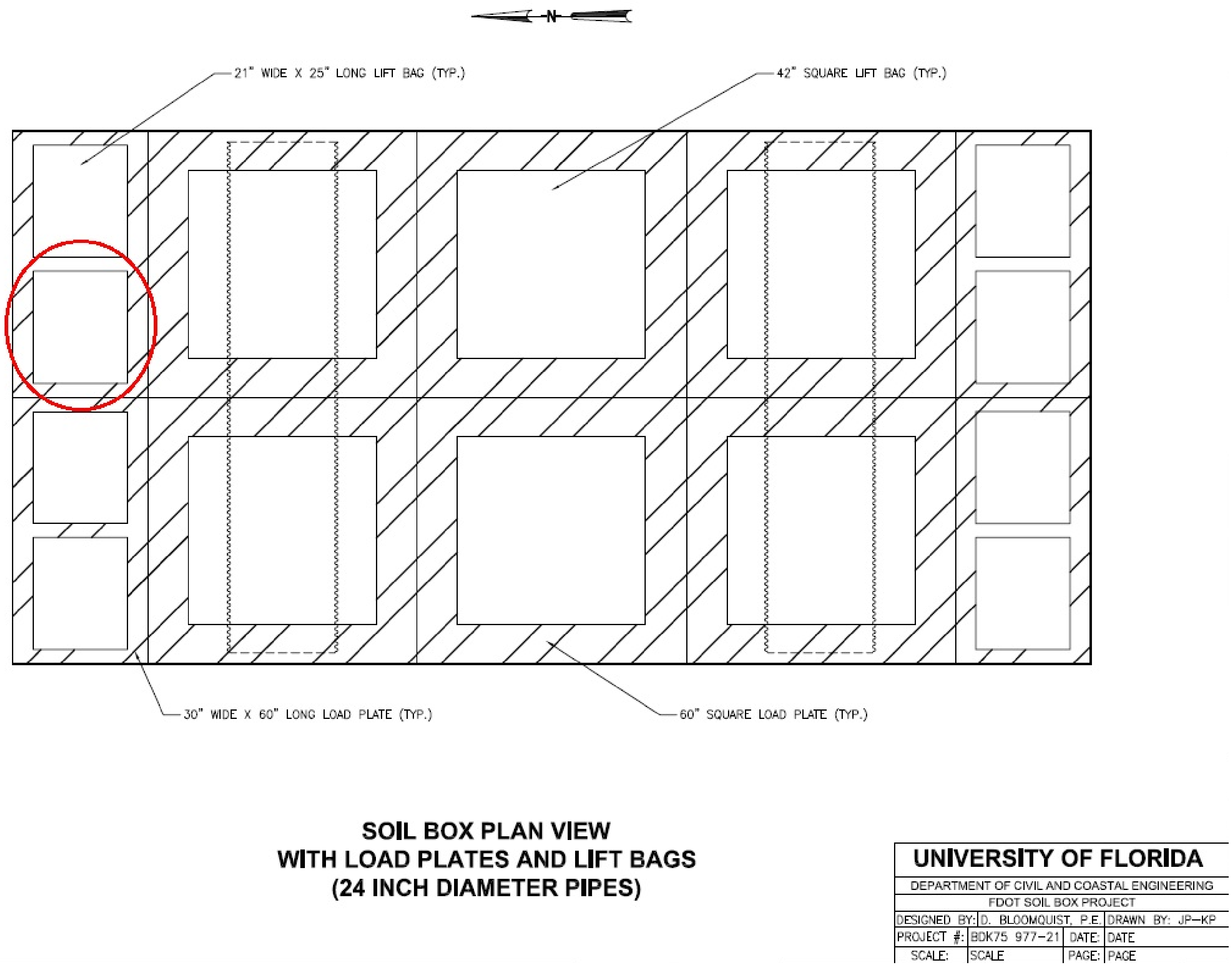


Figure 10-12. Lift bag failure location during 24 inch HDPE pipe test

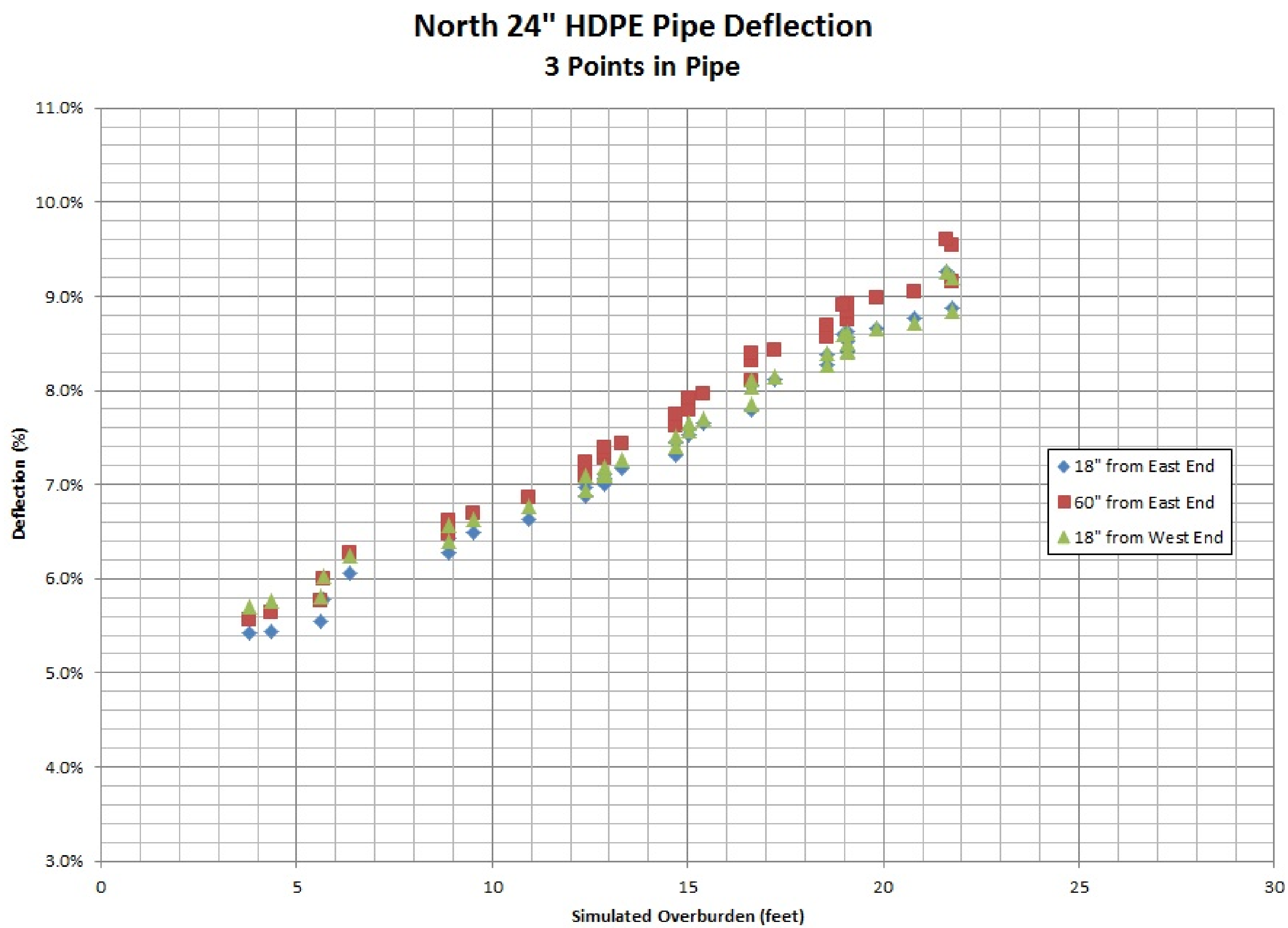


Figure 10-13. Three points in North 24" HDPE pipe, deflection versus simulated overburden

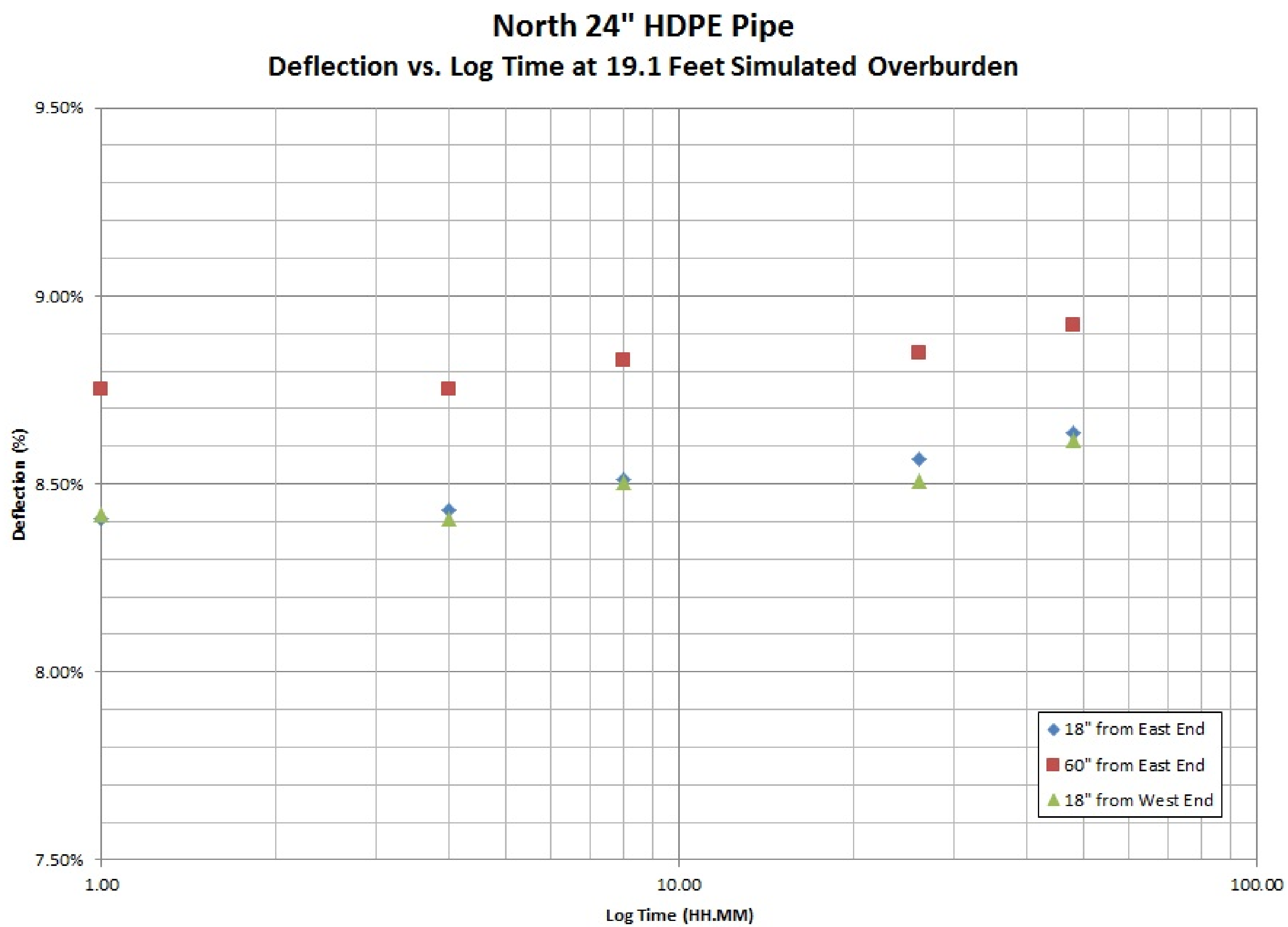


Figure 10-14. Deflection of North 24” HDPE pipe under 19.1ft of simulated overburden over 24 hours

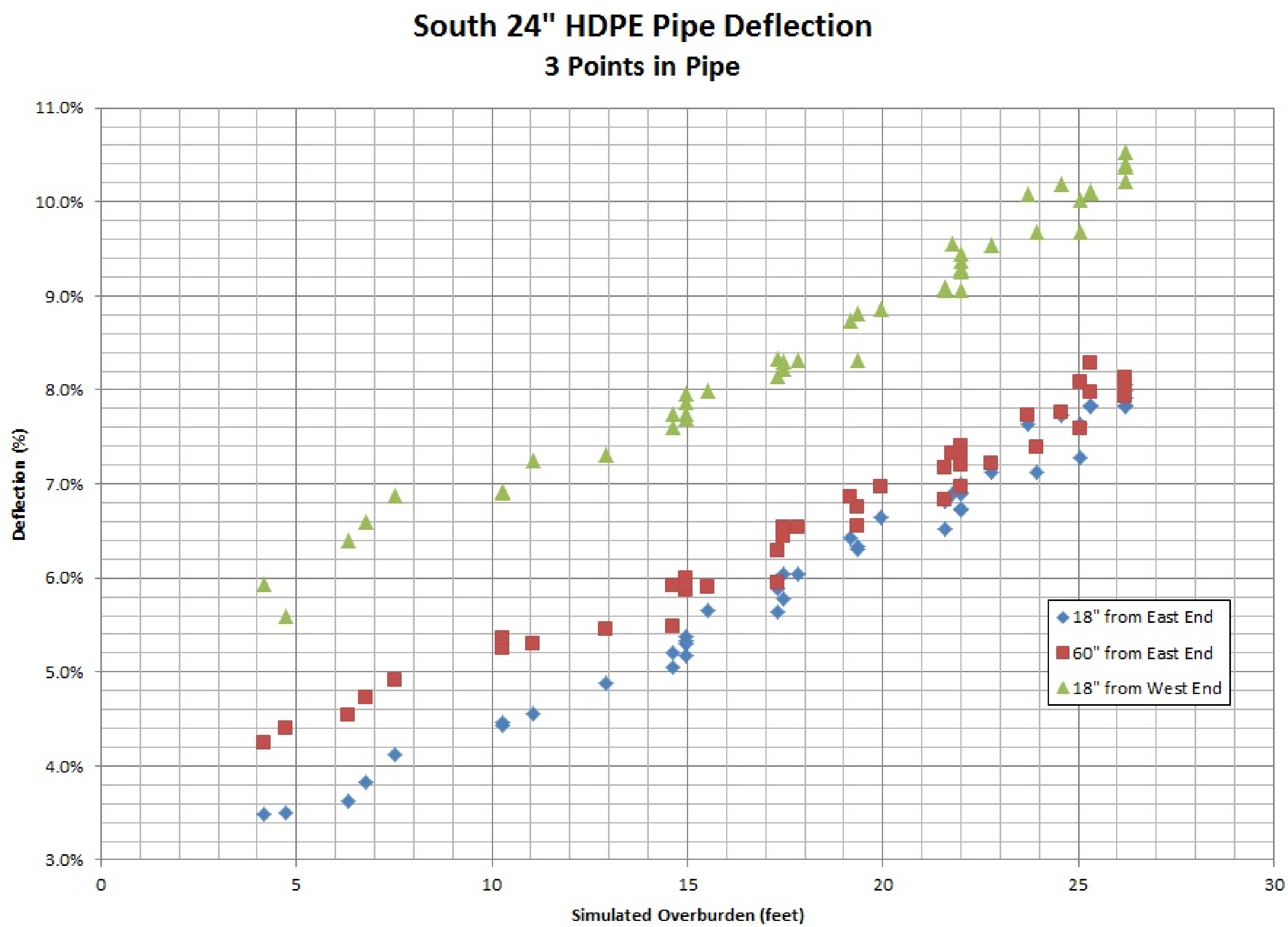


Figure 10-15. Three points in South 24” HDPE pipe, deflection versus simulated overburden

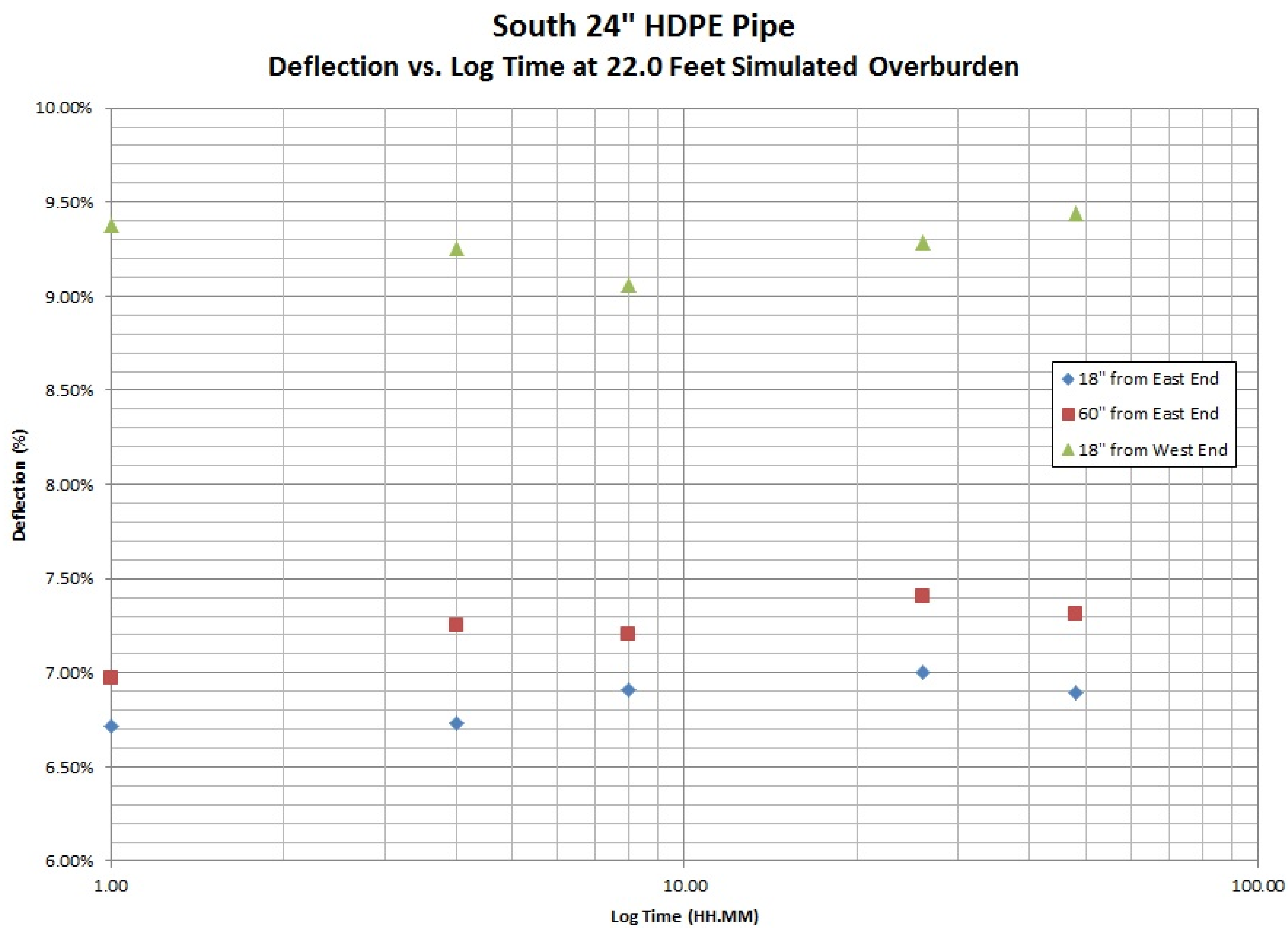


Figure 10-16. Deflection of South 24” HDPE pipe under 22.0ft of simulated overburden over 24 hours

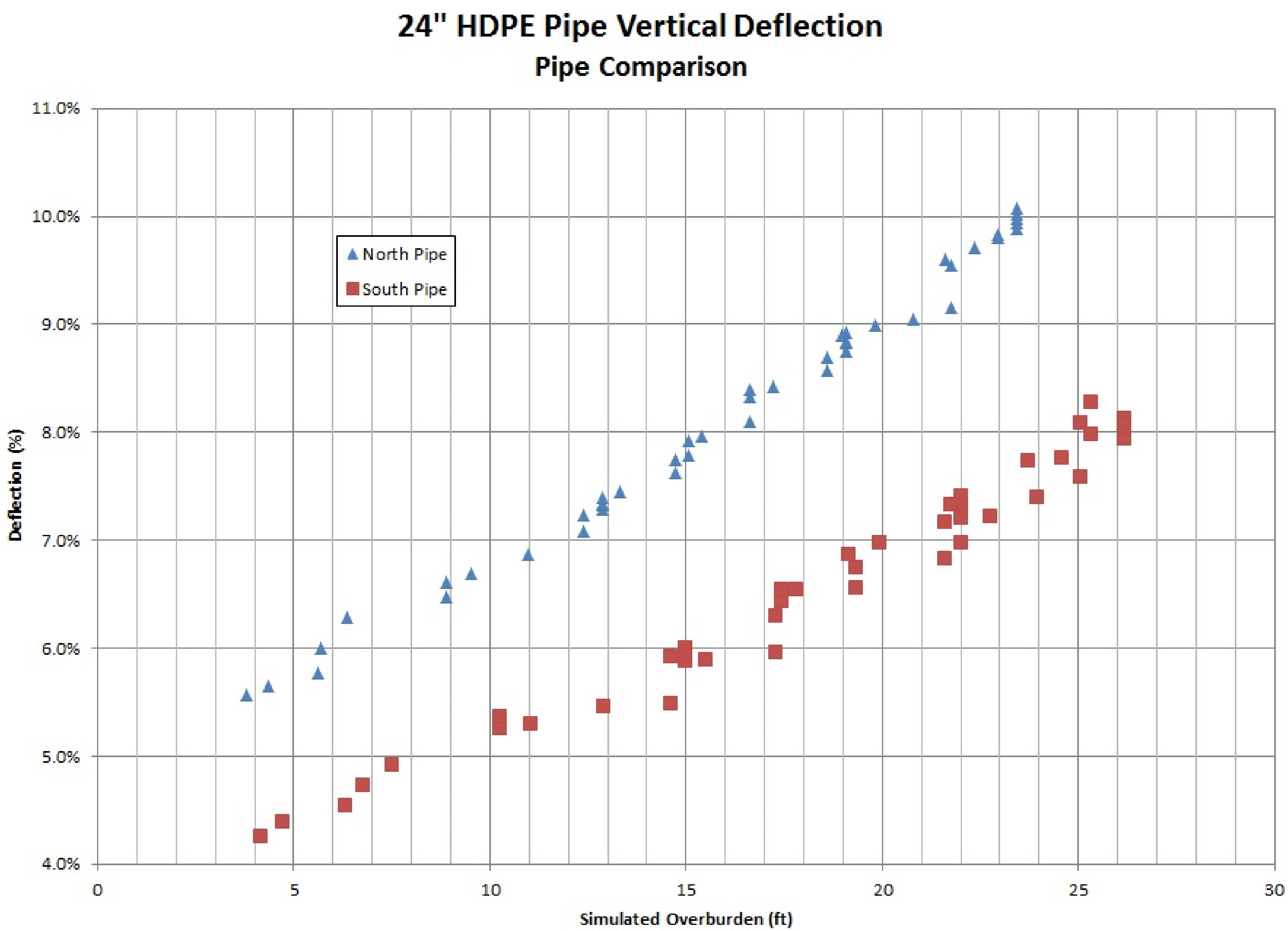


Figure 10-17. Deflection comparison of 24” HDPE pipes

CHAPTER 11
24 INCH STEEL PIPE TEST

11.1 Test Modifications

Similar modifications to the 24 inch diameter HDPE pipe test were required for the 24 inch diameter steel pipe test. The locations of the pressure cells were modified to account for the smaller pipe diameter; these locations are shown from Figure 11-1 through Figure 11-7. A steel ring was spot welded onto each end of the steel pipes to allow assembly to the flexible membrane system. Additionally, more soil was added to the box than normal in an attempt to generate higher simulated overburdens.

11.2 Results

The North steel pipe appeared to produce approximately uniform deflection along its length (Figure 11-8). Results appear to show creep deflections in this pipe over a 24 hour time period (Figure 11-9). A deflection versus simulated overburden plot of three points in the South steel pipe displays a 1.5% deflection discrepancy between each end of the pipe (Figure 11-10). The creep characteristics for the South HDPE pipe over a 24 hour time period displays a similar upward trend (Figure 11-11). A comparison of the North and South pipe deflections appears to be approximately equal during the loading sequence (Figure 11-12).

11.3 New Testing Method

In an effort to utilize the Soil Box to simulate higher overburdens, a new testing methods was developed after the final steel pipe test. Once the Box was unloaded, its lid was opened, and sand was packed around the box's lift bags (Figure 11-13 and Figure 11-14). The purpose of this procedure was to prevent the bags from "bulging" as severely when they were inflated.

Using the newly-packed bags, lift bag pressure was increased in increments to 130 psi. As shown in Figure 11-15 and Figure 11-16, using this technique, maximum simulated overburden pressures were similar to maximum simulated overburden pressures using non-buried lift bags. However, note that maximum simulated overburden for all tests discussed in this chapter reached approximately 50 ft., while maximum simulated overburdens for previous tests were only approximately 25 ft. It appears that adding more fill to the box affected maximum simulated overburden more than burying the lift bags. Additionally, burying the lift bags appears to cause more variability in deflection when compared to non-buried results. This may be due to the subjective nature of what constitutes a "buried" lift bag. This information is useful if strong pipes (concrete for example) are to be tested in the future.

Table 11-1. Simulated overburden for 24 inch steel pipe test.

Pressure Applied to Main Regulator (PSI)	Deflection Readings taken x hours after pressure was applied	Simulated Overburden Over North Steel Pipe (feet)	Simulated Overburden Over South Steel Pipe (feet)
0	1	6.6	7.3
5	1	8.3	9.3
10	1	10.0	11.3
15	1	12.0	13.4
20	1	13.5	14.9
25	1	15.3	16.8
30	1	17.2	18.9
35	1	19.0	20.7
40	1	20.8	22.8
45	1, 4, 8, 24	22.7	25.0
50	1	24.4	26.5
55	1	25.9	28.2
60	1	27.8	30.3
65	1	29.7	32.4
70	1	30.5	33.1
75	1	32.2	35.0
80	1	33.3	36.1
85	1	35.0	38.0
90	1, 4, 8, 24	37.2	40.4
95	1	39.0	42.1
100	1	40.4	43.5
105	1	42.1	45.2
110	1	43.5	46.6
115	1	44.5	47.7
120	1	45.9	49.1
125	1	47.6	51.0
130	1, 4, 8, 24	48.5	51.6

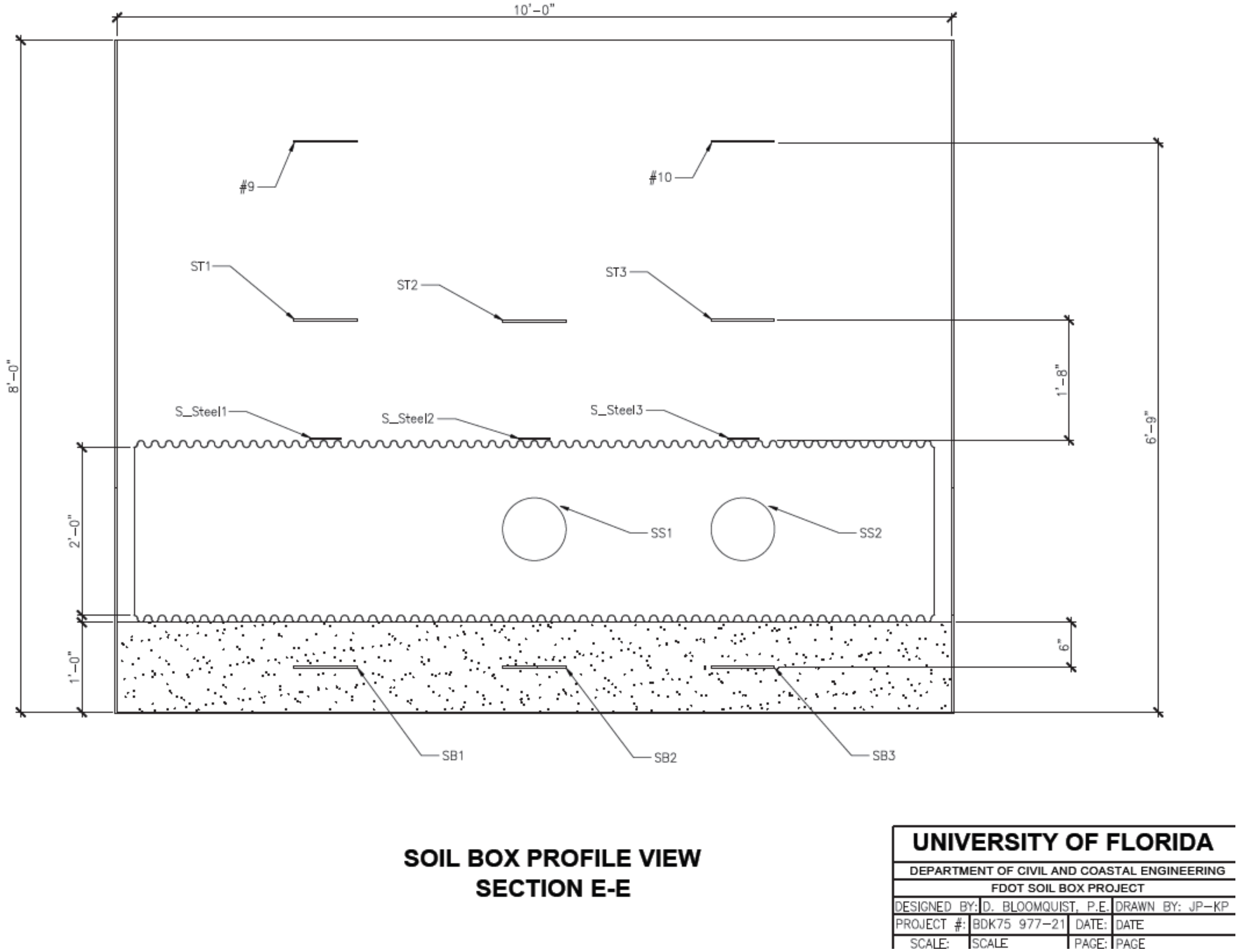


Figure 11-1. Earth pressure cell layout through Section E-E for 24 inch steel pipe test

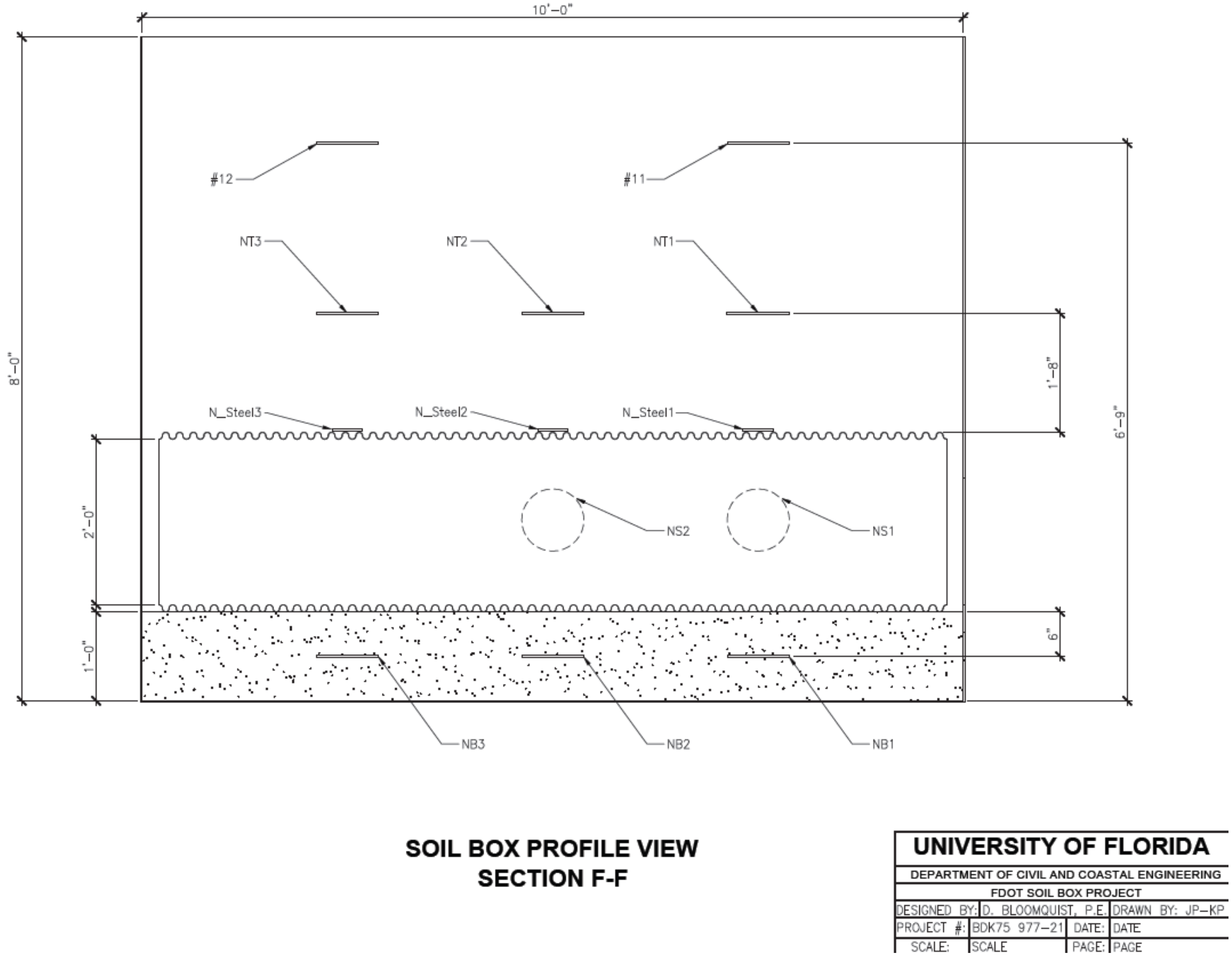


Figure 11-2. Earth pressure cell layout through Section F-F for 24 inch steel pipe test

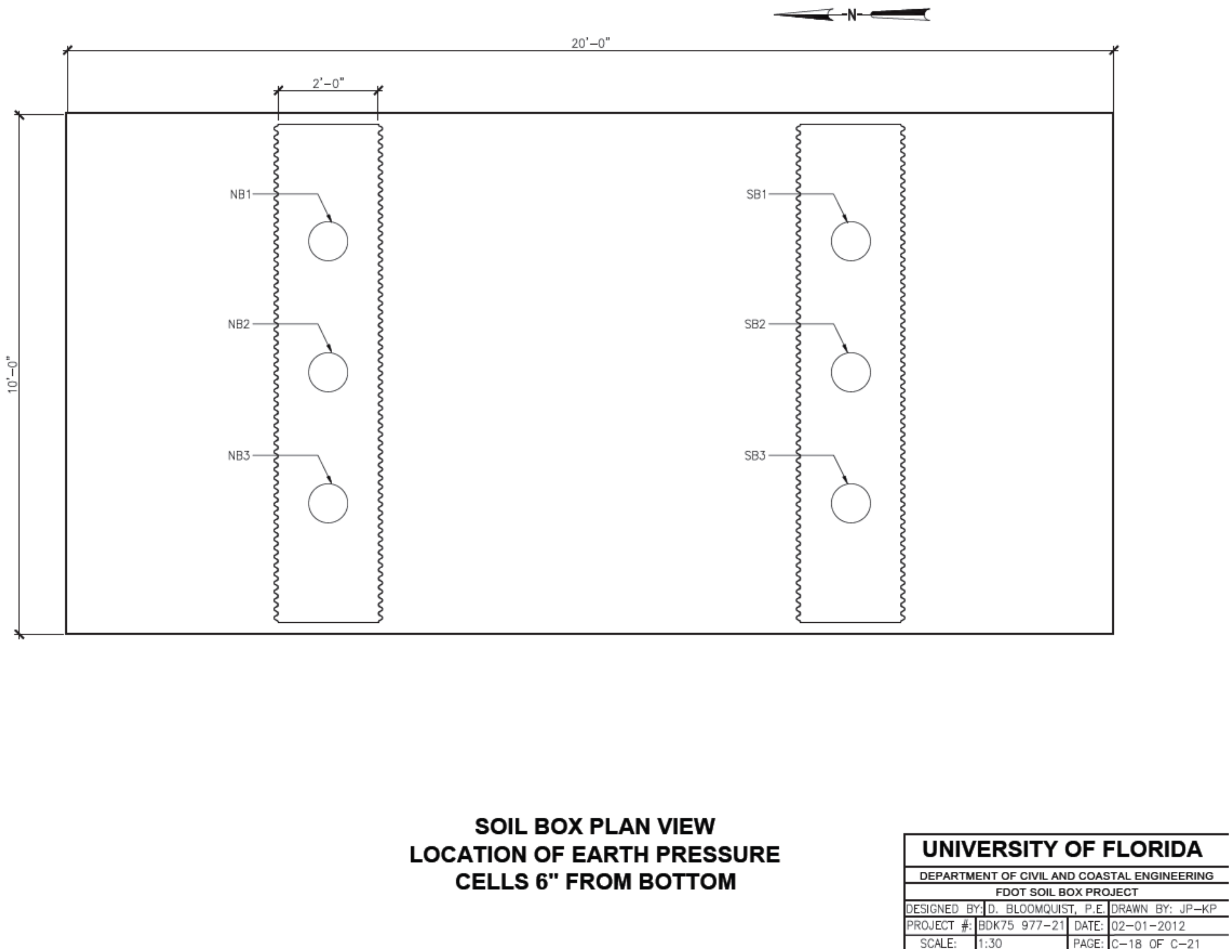


Figure 11-3. Earth pressure cell layout for 24 inch steel pipe test 0'-6" from bottom of Soil Box

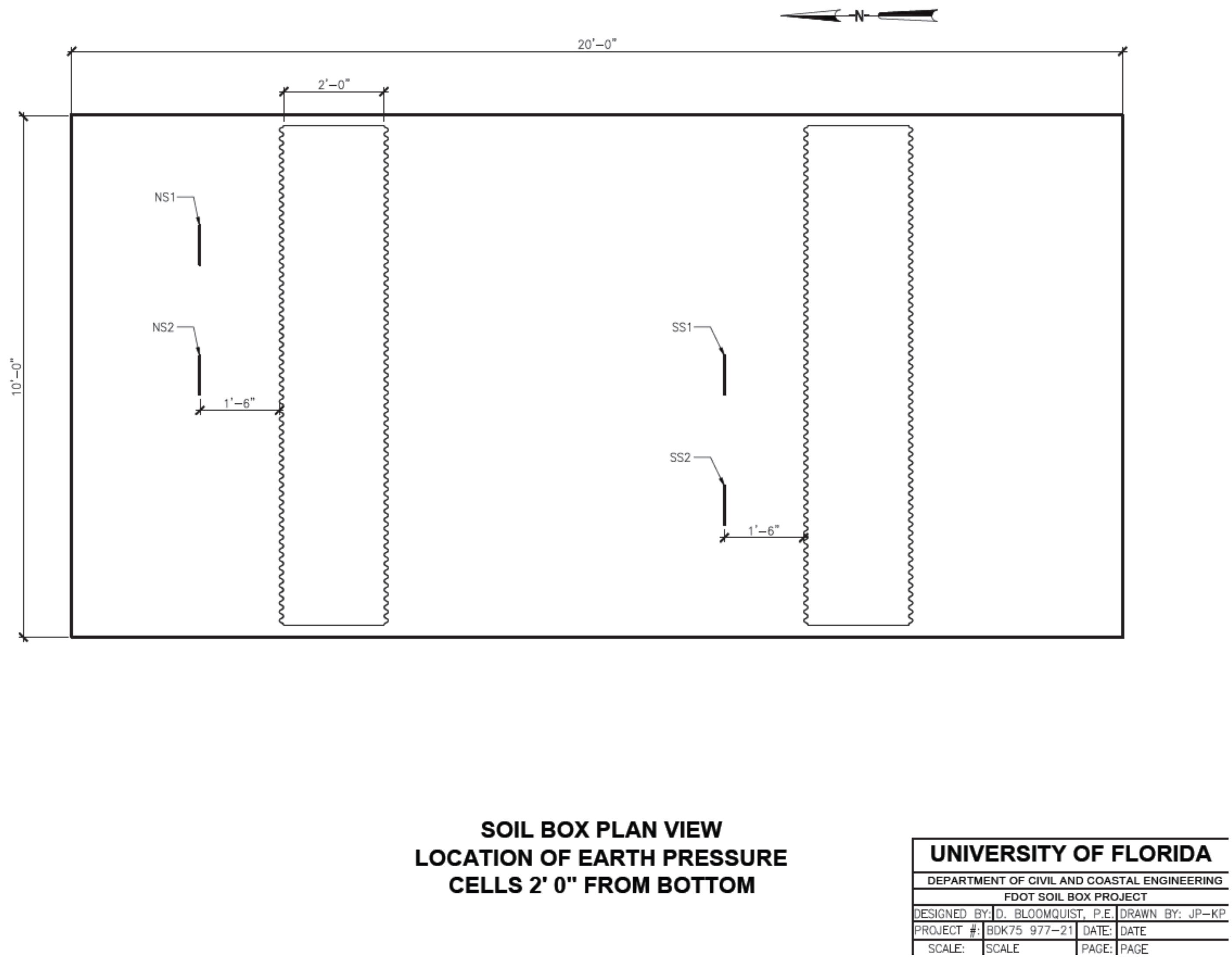


Figure 11-4. Earth pressure cell layout for 24 inch steel pipe test 2'-0" from bottom of Soil Box

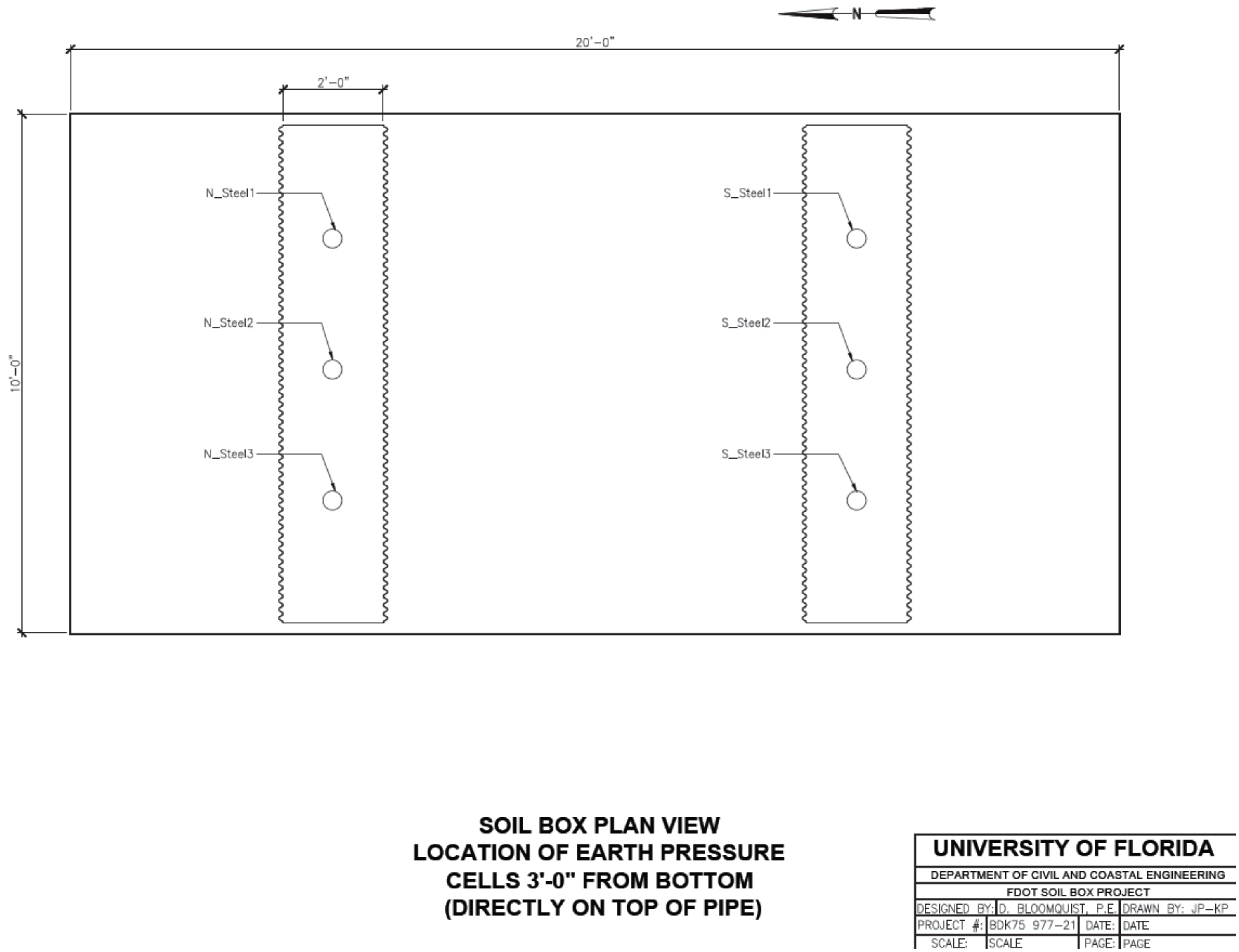


Figure 11-5. Earth pressure cell layout for 24 inch steel pipe test 3'-0" from bottom of Soil Box

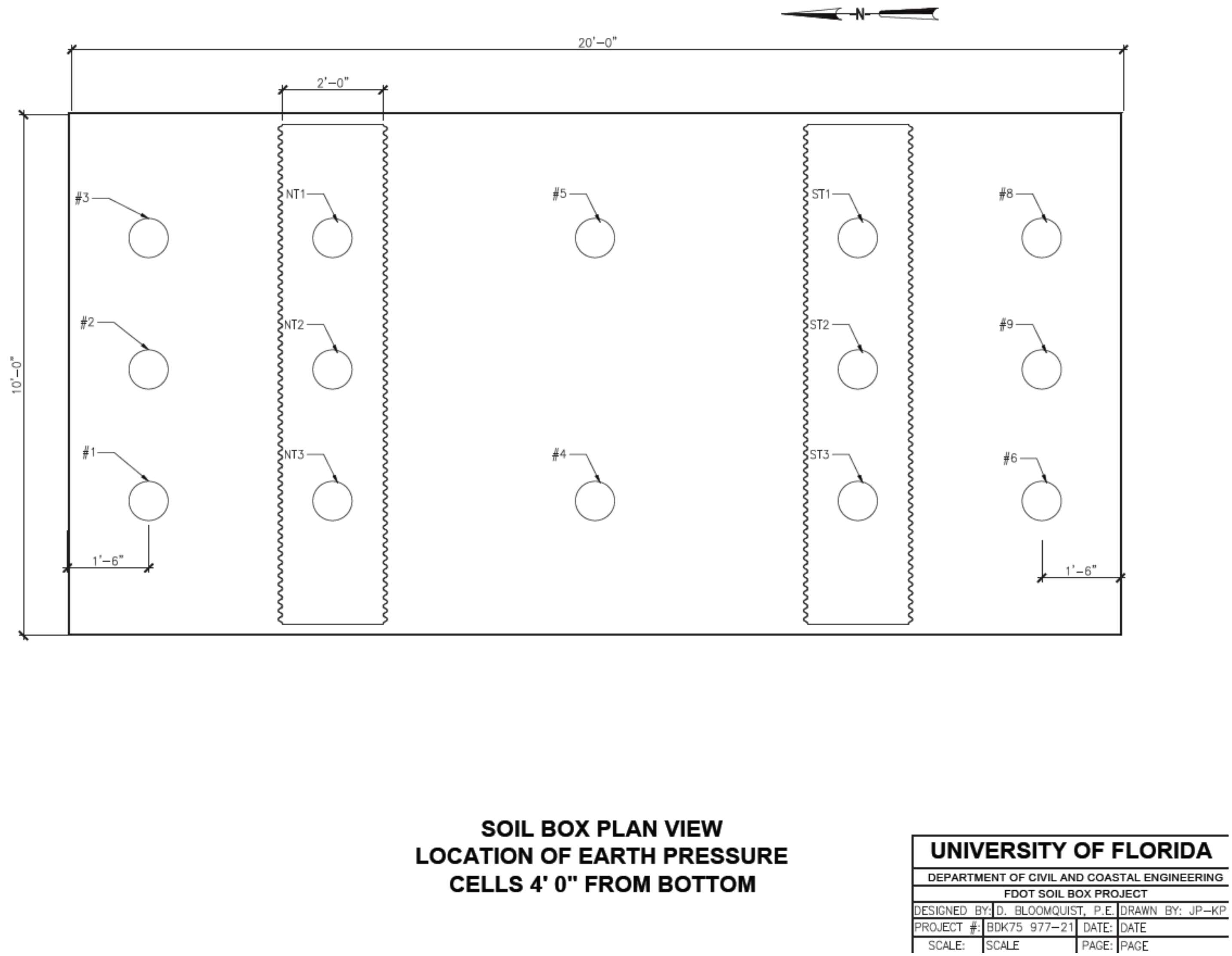


Figure 11-6. Earth pressure cell layout for 24 inch steel pipe test 4'-0" from bottom of Soil Box

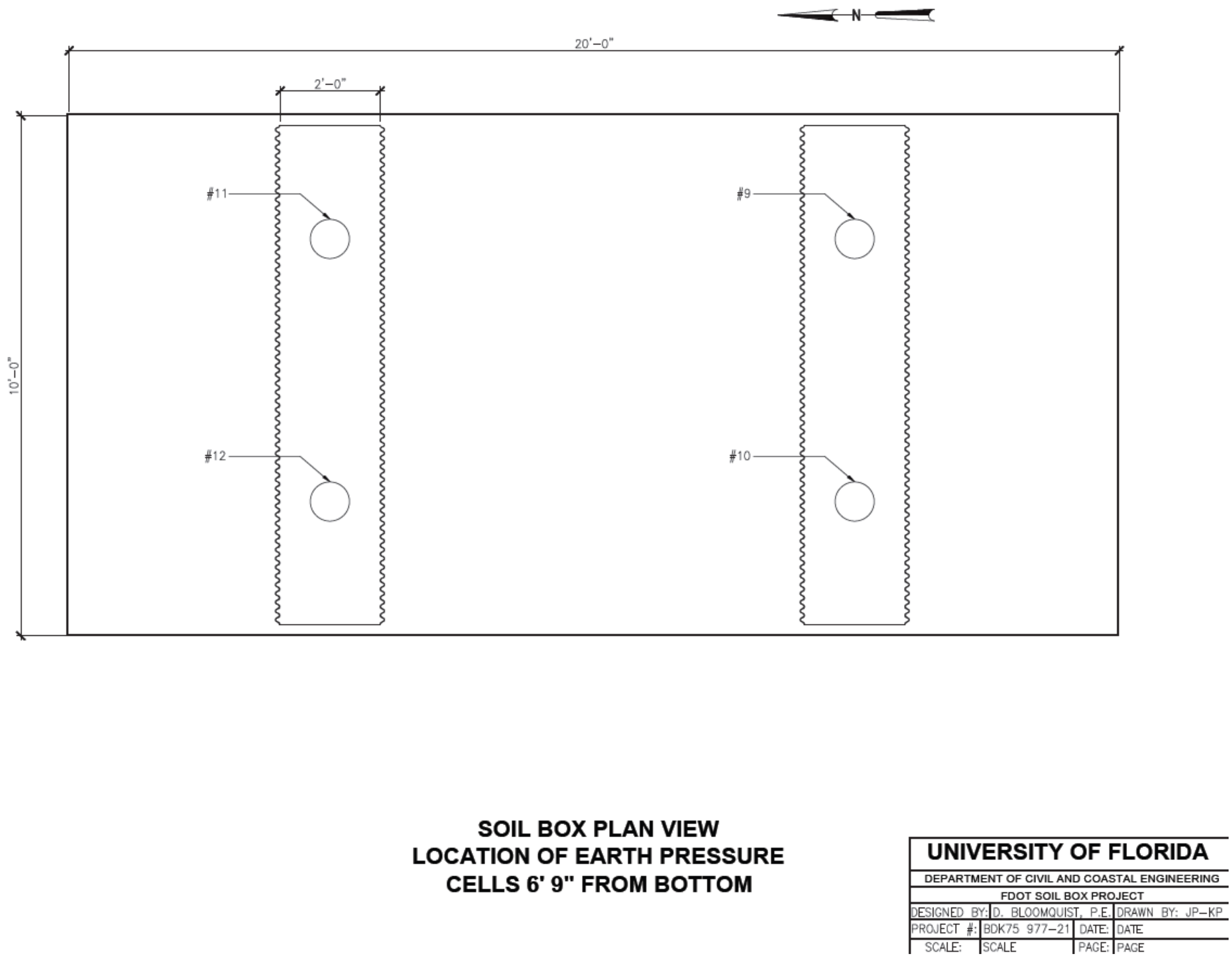


Figure 11-7. Earth pressure cell layout for 24 inch steel pipe test 6’-9” from bottom of Soil Box

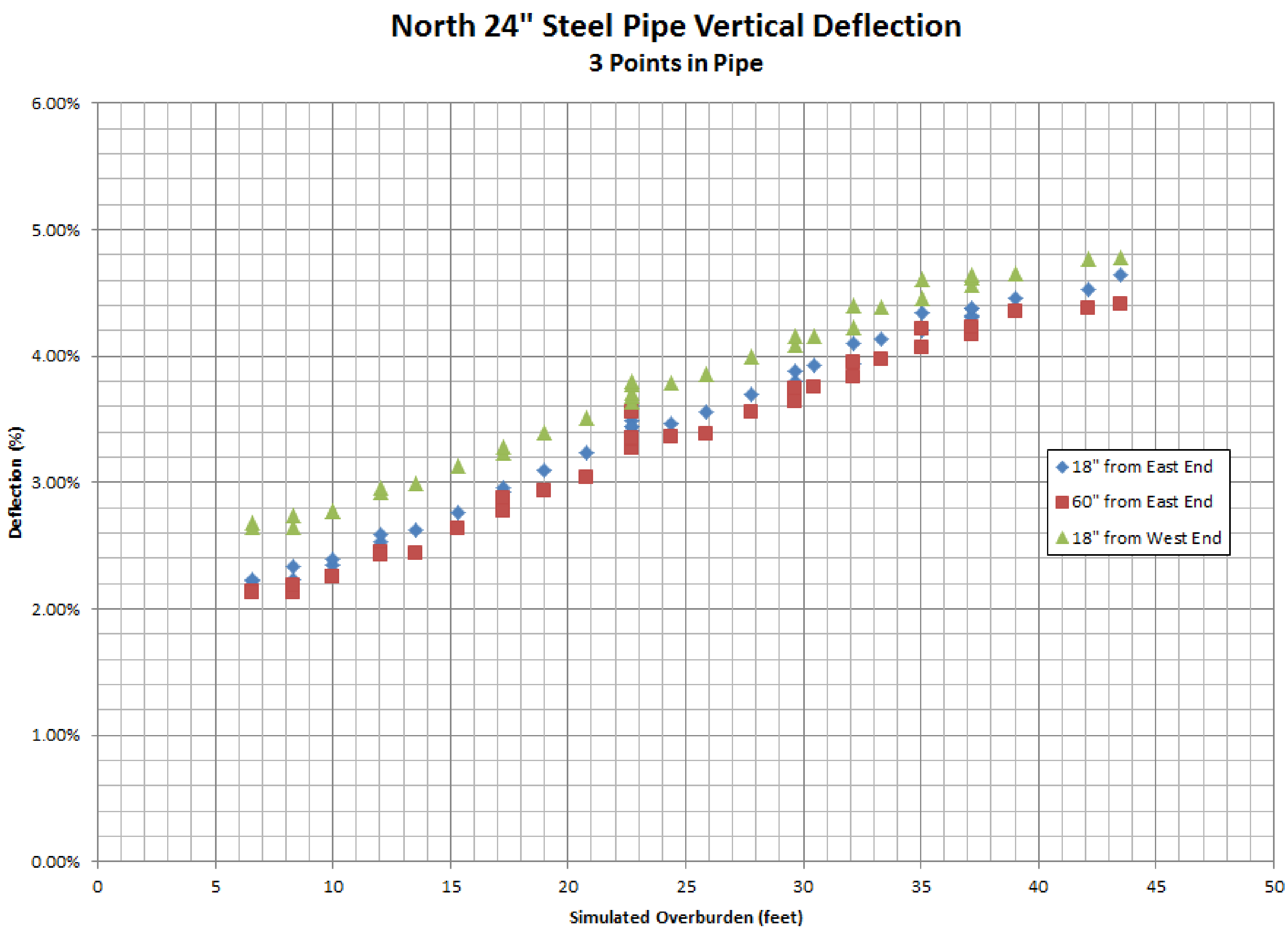


Figure 11-8. Three points in North 24” steel pipe, deflection versus simulated overburden.

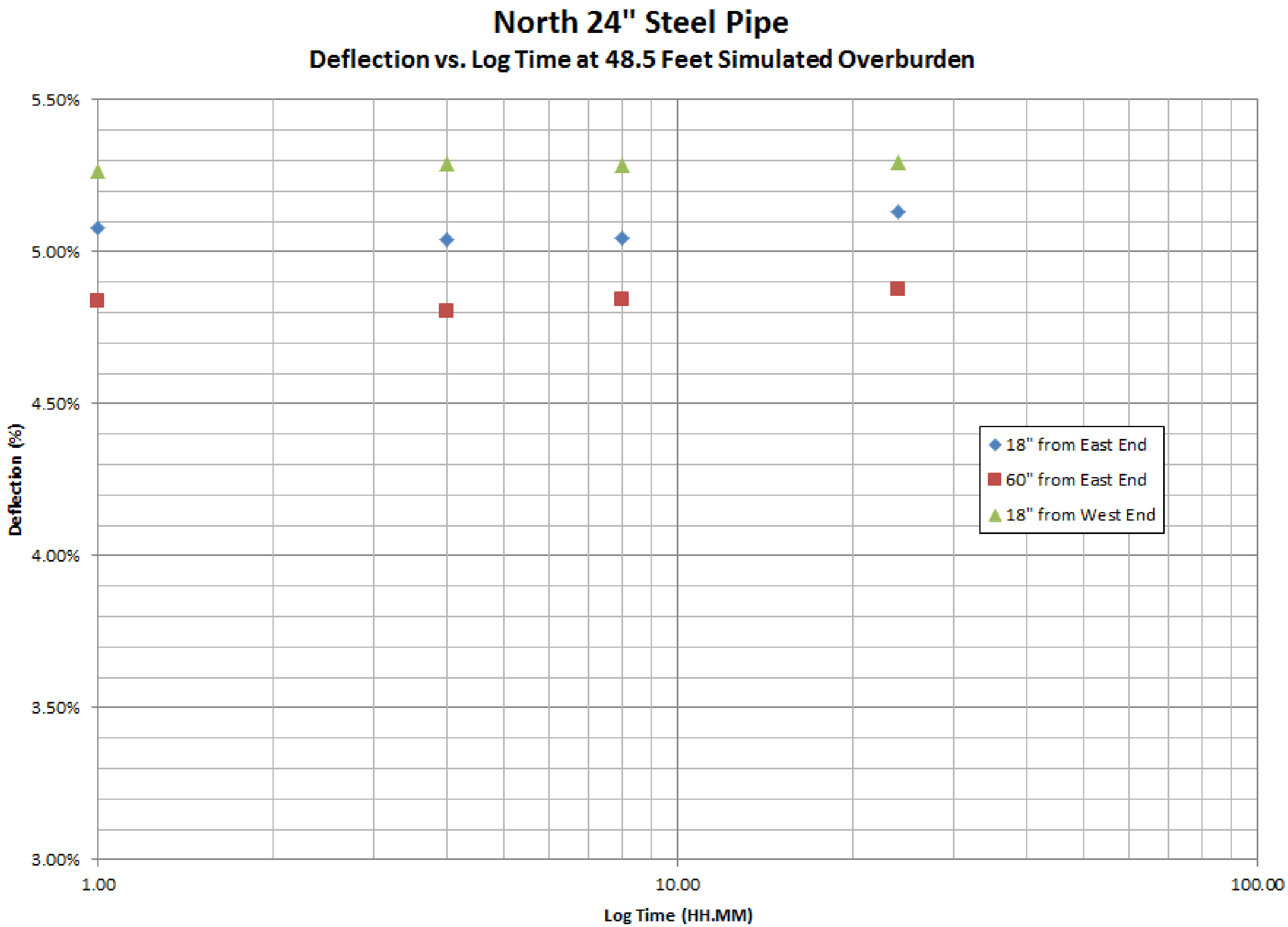


Figure 11-9. Deflection of North 24" steel pipe under 48.5ft of simulated overburden over 24 hours.

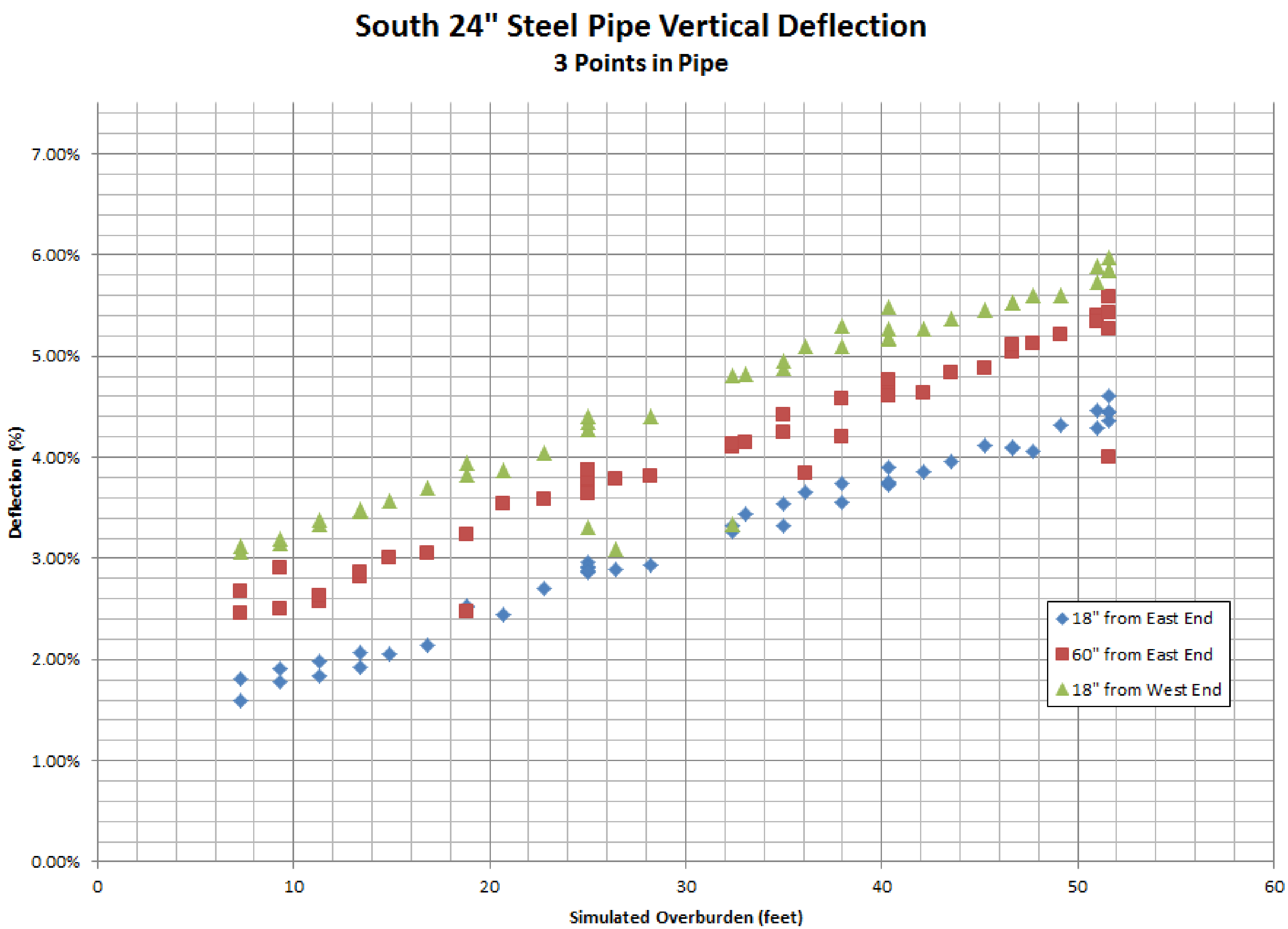


Figure 11-10. Three points in South 24” steel pipe, deflection versus simulated overburden.

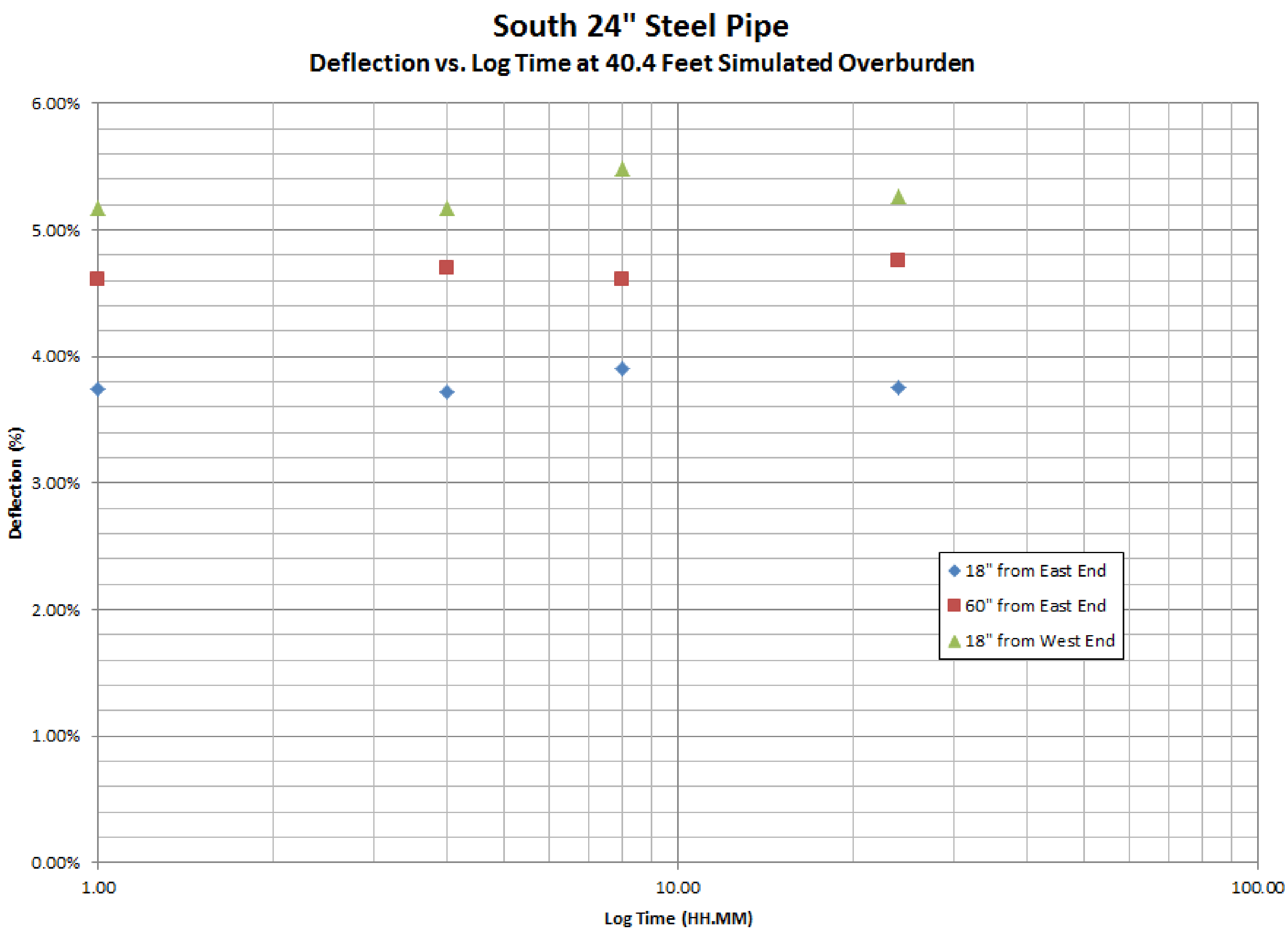


Figure 11-11. Deflection of North 24” steel pipe under 40.4ft of simulated overburden over 24 hours.

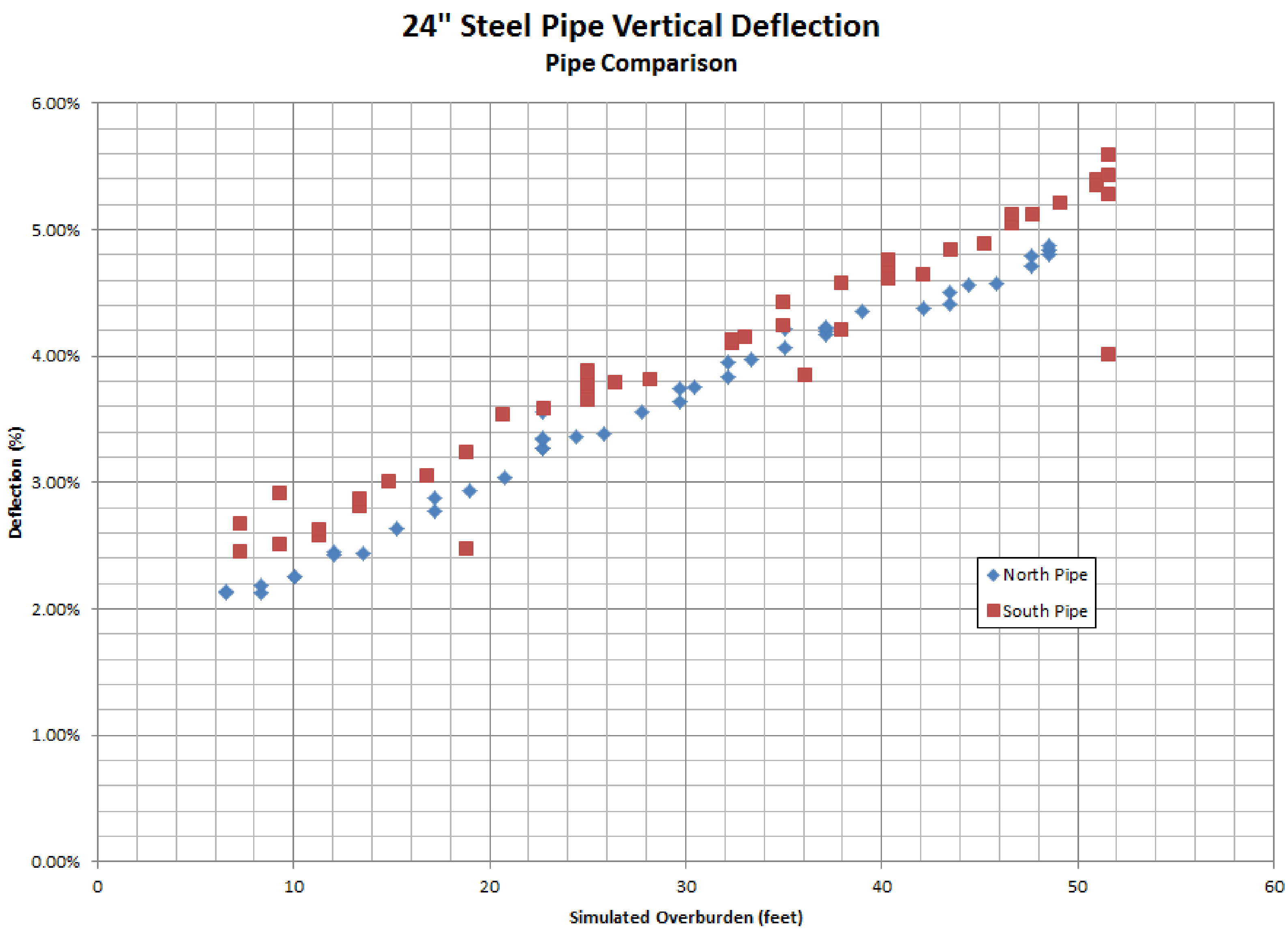


Figure 11-12. Deflection comparison of 24” steel pipes.

Figure 11-13.Photograph of new testing technique showing partially-buried lift bags

Figure 11-14.Photograph of new testing technique showing fully-buried lift bags

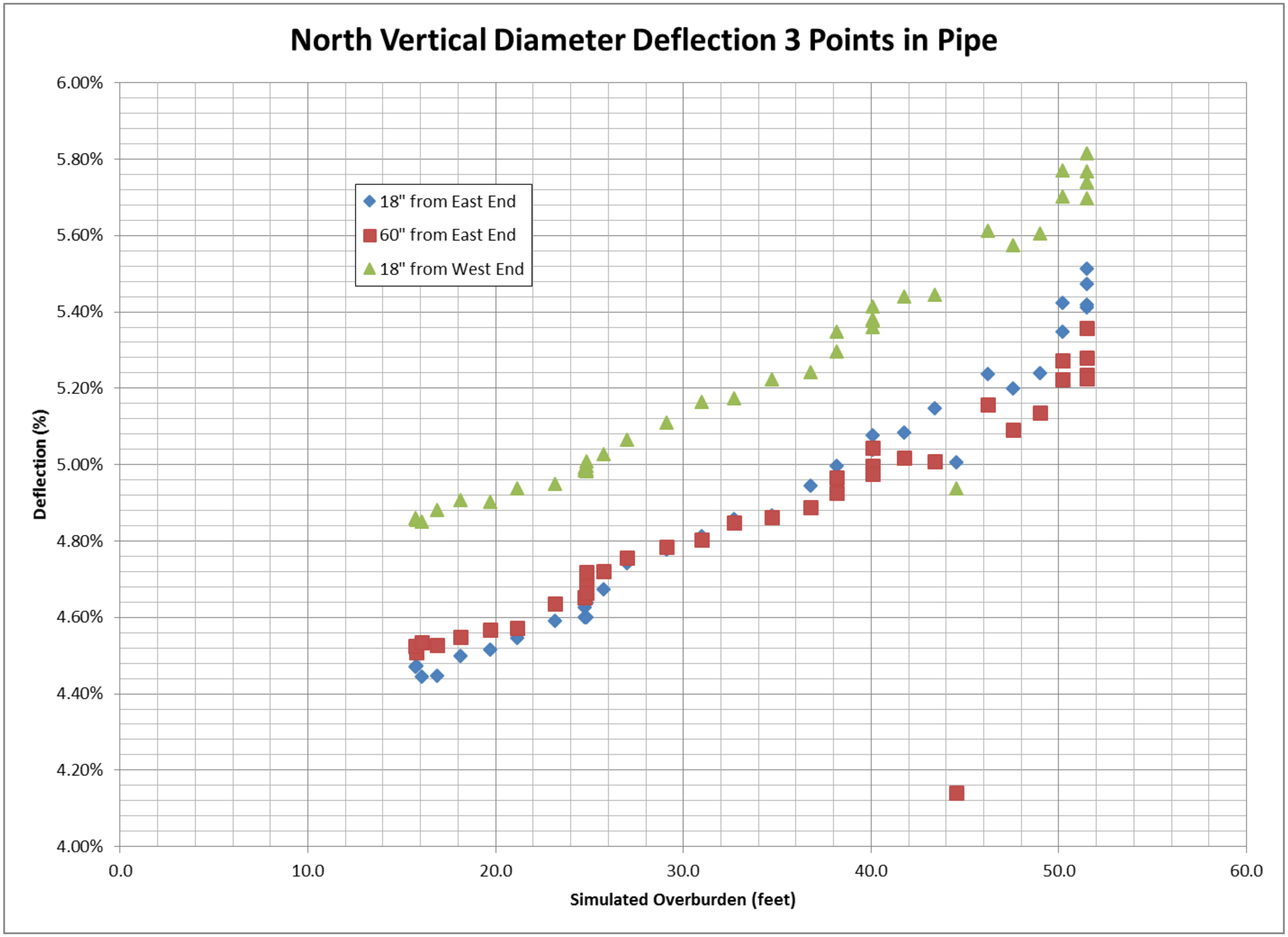


Figure 11-15.North pipe deflection versus simulated overburden using new testing technique

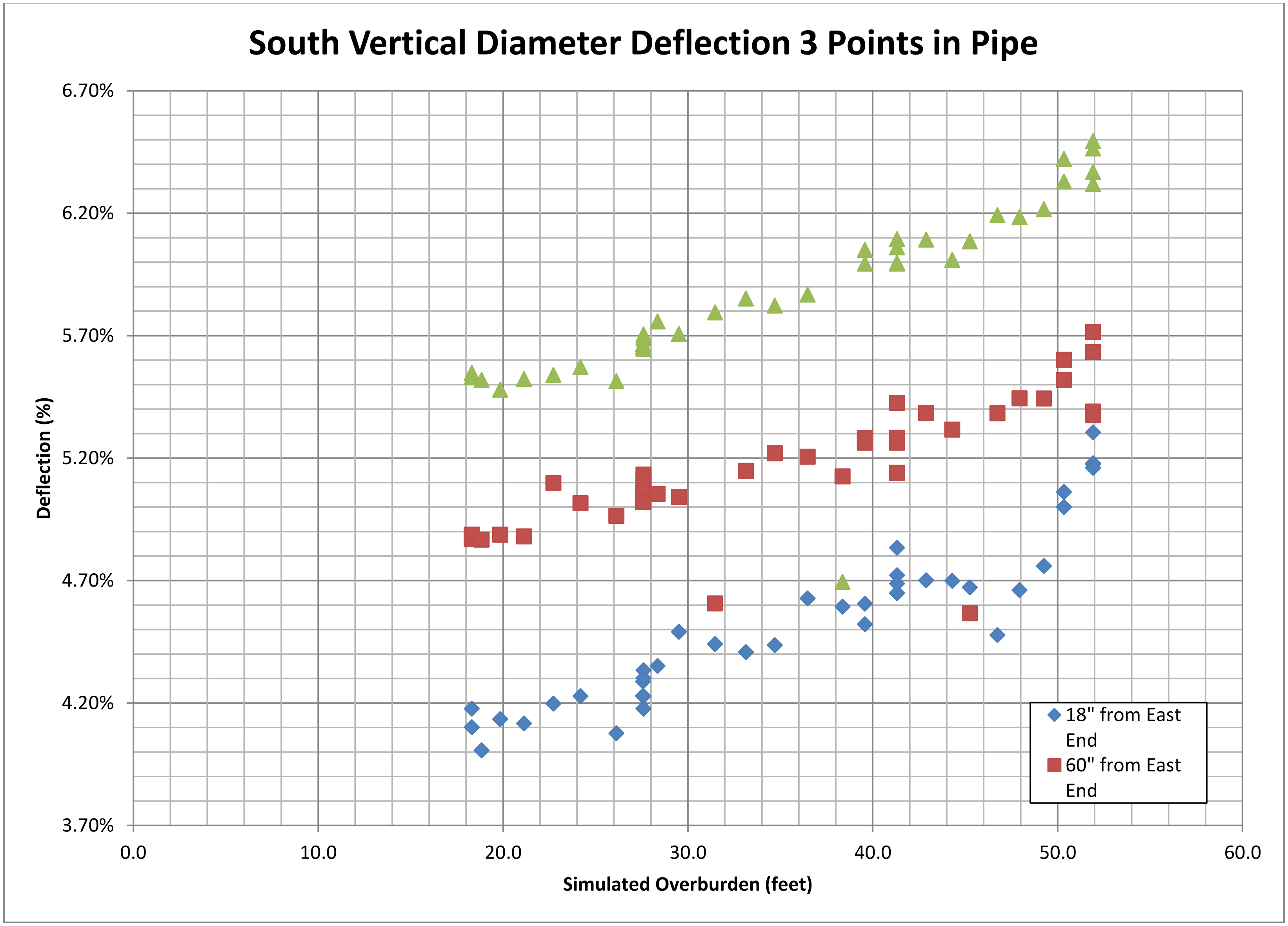


Figure 11-16. South pipe deflection versus simulated overburden using new testing technique.

CHAPTER 12
COMPUTATIONAL MODELING OF SOIL BOX DATA

12.1 Introduction

As mentioned in Chapter 1, results from Soil Box physical tests were to be used as a basis of comparison for finite element (FE) analysis. As per the terms of this project's contract, FE modeling was conducted by Simpson, Gumpertz, and Heger (SGH) of Waltham, MA. A report was prepared by SGH that discusses their model in detail (Appendix A). This chapter discusses highlights from the SGH report.

12.2 FE Model Parameters

As discussed in Appendix A, a two-dimensional model was prepared by SGH using ABAQUS that takes advantage of symmetry (Figure 12-1). This model makes use of a number of assumptions:

1. Although the pipes have a spiraled ridge along their exteriors, the model assumes that a cylinder with equivalent structural material properties (moment of inertia, section modulus, modulus of elasticity, etc.) may be defined to adequately describe the pipe.
2. Due to the cylinder-simplification, three-dimensional effects are assumed to be minimal. Therefore, simple two-dimensional elements may be used to describe the pipe.
3. The soil may be modeled effectively as a linearly-elastic material with Drucker-Parger plasticity and linear hardening.
4. The soil was assumed to have constant density.

Two series of modeled tests were conducted. First, soil was "compacted" to 85% of its standard Proctor density (SPD) and densified to 95% to represent a worst-case series of data that was presumably to correspond with results from the Soil Box tests. Then, soil was compacted to 90% SPD and allowed to densify to 95% to represent a more-typical installation procedure. Additionally, a trench-box installation was simulated.

12.3 SGH Conclusions

Conclusions from the SGH study were as follows:

1. Data failed to match soil box test results. SGH attributed this to testing conditions. Specifically:
    a. Measured soil pressure was known to vary during testing. This is most likely caused by density variations, which would render assumption (4) above invalid.
    b. During two tests, soil escaped from the box. Again, this would appear to render constant-material assumptions invalid.
    c. Target saturation did not appear to be achieved when the sprinkler was used to moisten the soil. Please note – as per discussion in the previous chapters, eventually the sprinkler was replaced with a hose system.
2. Because data failed to match the FE model, SGH used AASHTO pipe deflection equations to validate their model.
3. Results from the FE model appeared to indicate that there is a direct relationship between soil densification and pipe stiffness. Furthermore, this relationship appears to be stronger for lower-stiffness pipe. However, the densification effects should be further studied before any definitive conclusions can be drawn.
4. Creep deflection appears to be minimal.
5. Trench boxes appear to increase pipe deflection, although this parameter is difficult to model.
6. A short-term deflection limit of 3.5% within 30 days of installation relative the initial vertical diameter should provide reasonable assurance that long-term deflections will not exceed 5%.

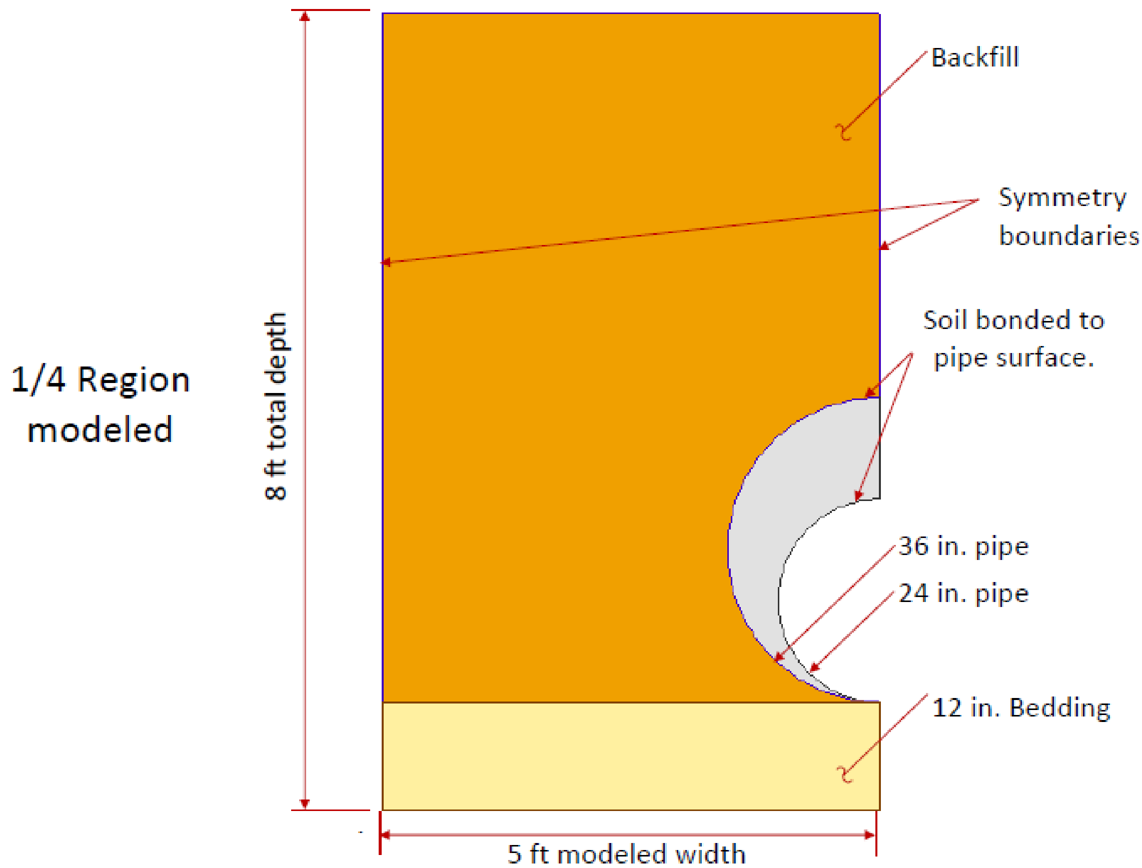


Figure 12-1. SGH FE model (reproduced from Appendix A)

CHAPTER 13
DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

13.1 Discussion of Soil Box Testing

As mentioned in the previous chapter, soil that was used for the FE model was assumed to be uniform because the Soil Box was designed to simulate uniform overburden pressure on buried pipes. As mentioned in Chapter 2, pressure cells were placed throughout the Soil Box to verify/refute the uniform pressure distribution assumption. When UF/UNF investigators realized that SGH modeled results would not match measured data, they hypothesized that the uniform soil assumption may be invalid. Several stress distribution contour plots were developed to demonstrate soil stresses throughout testing for both the aluminum pipe and the HDPE pipe with trench box (Figure 13-1 through Figure 13-13). As shown for these representative test series, measured soil stress variation may become significant.

Investigators attributed this variation to three factors. First, nuclear density data showed that initial densities were not uniform. These non-uniform densities' effects were exaggerated by known, non-uniform moisture content conditions. Conditions were exacerbated even further by non-uniform loading due to differential contact areas between the lift bags and the soil surfaces. Note that after the soil box was filled and the loading mechanism was applied the steel plates tended to settle with the soil. This caused the lift bags to differentially "bulge" depending on the amount by which the plates had settled. Because multiple lift bags were used, each bag "bulged" differently. Therefore, different stresses were induced at the top of the soil matrix – which in turn led to different stresses throughout the soil matrix.

The loading mechanism could easily be improved by utilizing a single lift bag similar to the method employed by Brachman et al. (2000 and 2001). While density variations may be inevitable due to the filling procedure used for the soil box, the differential loading situation would be avoided by applying a true, uniform pressure. To construct such a large lift bag (since none are readily available), the procedure described by Brachman et al. (2000 and 2001) would be duplicated.

13.2 Discussion of FE Modeling

There are several areas where UF/UNF believe computational results could be enhanced. While it is true that there were loading/density variations during testing, investigators at UF/UNF believe that these parameters can be modeled effectively. To do so would require a three-dimensional approach whereby modeled soil density matched variable densities in the Soil Box. Loading would be modeled such that the "match target" became measured simulated overburden pressures instead of pipe deflections.

Similarly, the pipe must be modeled in three dimensions. Modeling a spiraled pipe's interface with soil is extraordinarily difficult because doing so would require aligning the soil grid with the spiral. Because the pipe-spiraling would not meet the soil matrix at right angles, it is unlikely that usable results would be produced. A simpler solution (while still several levels of sophistication higher than the cylinder approximation) would be to develop a three-dimensional equivalent pipe with rings that replaced the spirals. Then, a "squash test" would be performed whereby deflection of the non-spiraled pipe would be compared with deflection of the spiraled pipe under simple loading conditions. If results were similar, the equivalent pipe would be said to be an acceptable representation of the non-spiraled approximation.

UF/UNF investigators tested the feasibility of this three dimensional approach. Three-dimensional grids were developed of a spiraled pipe (Figure 13-14) and a “ringed” spiral equivalent (Figure 13-15) using shell elements in LS-DYNA. Access to LS-DYNA was possible because of the existing UF/UNF collaboration that was developed during FDOT project No. BDK75 977-53 with Argonne National Laboratories' (ANL) Transportation Research and Computing Center (TRACC). A “squash test” was conducted for each pipe (Figure 13-16), and results were compared with one another (Figure 13-17 and Figure 13-18). Maximum deflections appeared to be within 7% of one another while minimum deflections showed an approximately 50% difference between the pipes.

At this point, investigators realized that going further would require significantly more work beyond the scope of this project. While the ability to develop a three-dimensional model in LS-DYNA is encouraging, results are not necessarily useful without pipe deflection verification. In other words, data are needed from a simple “squash-test” loading scenario to verify that the pipe itself is modeled correctly. If one cannot verify the pipe, it would be extraordinarily difficult to verify a pipe-soil interaction problem because the pipe-soil interaction problem is several orders of magnitude more complicated than the simple pipe loading scenario.

In the future, pipe deflection data could be measured by loading the pipes with a steel plate and measuring deflection with string potentiometers. While the maximum deflection data discussed above may be encouraging, it may have very little physical meaning because it is only a comparison of an arbitrary point. A better method of comparison (given sufficient data) would be to compare several string potentiometer reading points with one another.

Once pipe data for a simple case were sufficiently matched, soil would be added and density varied to reflect actual soil conditions as opposed to uniform 85% or 90% SPD conditions. This would improve the chances of achieving matching.

13.3 The Value of a More-Sophisticated FE Model

Development of this model may be worthwhile – especially in the context of trench boxes and short-term deflection guidelines. A three-dimensional approach would allow for modeling a “true” trench box.

More importantly, the 3.5% short-term deflection limit proposed by SGH was not based upon explicit quantitative data. Essentially, FE model that densified from 90% to 95% was assumed to be more typical of regular construction practices than the approximately 85% worst-case scenario that was physically modeled in the Soil Box. The worst-case deflection (for HDPE pipe) from 90% to 95% densification was approximately 3%. Then, SGH realized that creep would be responsible for approximately 0.5% deflection. Meanwhile, based upon some previous research by McGrath, SGH assumed deflection due to installation would only be equal to 2%. The sum of these deflections was approximately 5%.

Then, SGH researched a different material (fiberglass AWWA Manual M45) which requires post-installation deflections of 3.5% for a maximum long-term deflection of 5%. SGH speculated that this 3.5% short-term criteria is due to the sum of 2% deflection due to installation; 0.5% deflection due to creep; and 1% deflection due to densification. An assumption was made that if 3.5% is an acceptable short-term criterion for 5% deflection in fiberglass pipe in the long-term, then it must be a reasonable approximation for other pipes including the steel and HDPE pipes used during this study.

SGH acknowledges difficulties associated with quantifying densification effects – both in the short-term and in the long-term. Development of a more-sophisticated three-dimensional FE model that matches physically-tested results would give investigators more confidence in modeled results. Note that SFH verification was based upon computational analysis only. While it is encouraging that their model appeared to match AASHTO guidelines, it would be beneficial to reconcile the apparent discrepancy between measured and modeled data.

Perhaps more importantly, development of a more-sophisticated three-dimensional model would allow investigators to quantify the time-component associated with pipe deflection. Note that the SGH model only implicitly quantified deflection as a function of time through use of the load-dependent Durcker-Prager, load-depended strain modulus assumption. Development of a sophisticated, three-dimensional model should allow investigators to explicitly quantify time effects. Sophisticated modeling would be conducted such that deflection would be measured as a function of time for several years. During “early” loading, deflection would probably be greatest; and thus, small modeling time steps would be used (on the order of tenths of seconds for the first 24 hours). As time increased to 30 days, time steps of several hours would be used. Beyond 30 days, time steps of approximately a day would be used for approximately a year. Beyond this, time steps would increase toward a full day.

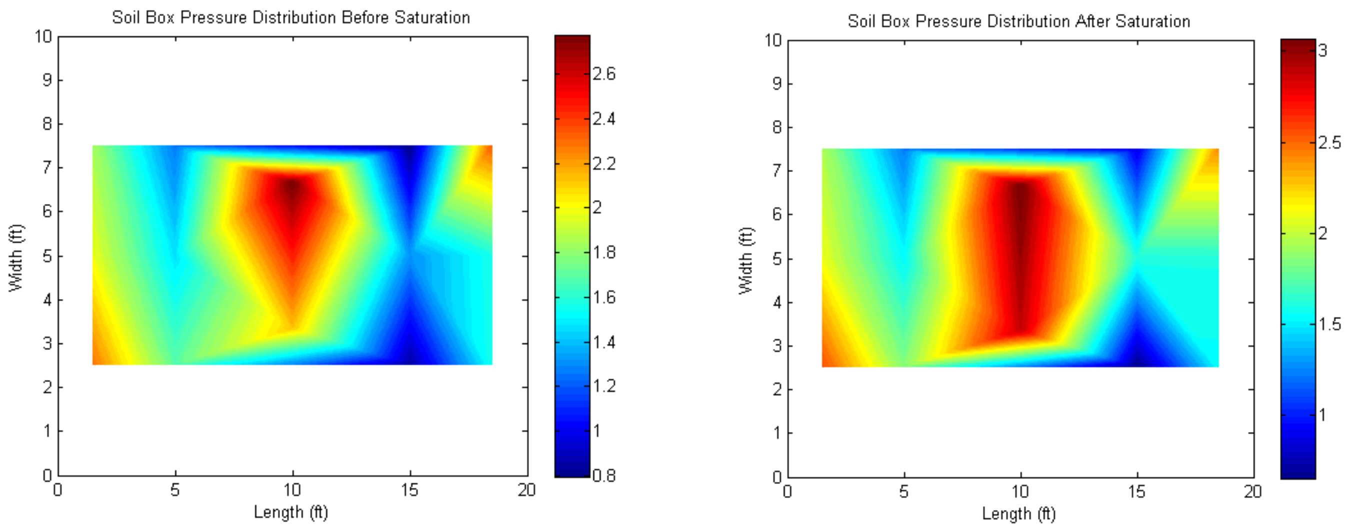
Of course, increasing modeled time steps would reduce modeled accuracy. Therefore, the inherent assumption behind this approach would be that most deflection occurs quickly – within the first week for example. This approach would allow one to quantify densification, and it would explicitly, approximately answer the 30-day early inspection question.

13.4 Summary and Conclusions

The following is a bulleted list of work conducted in this study and associated conclusions:

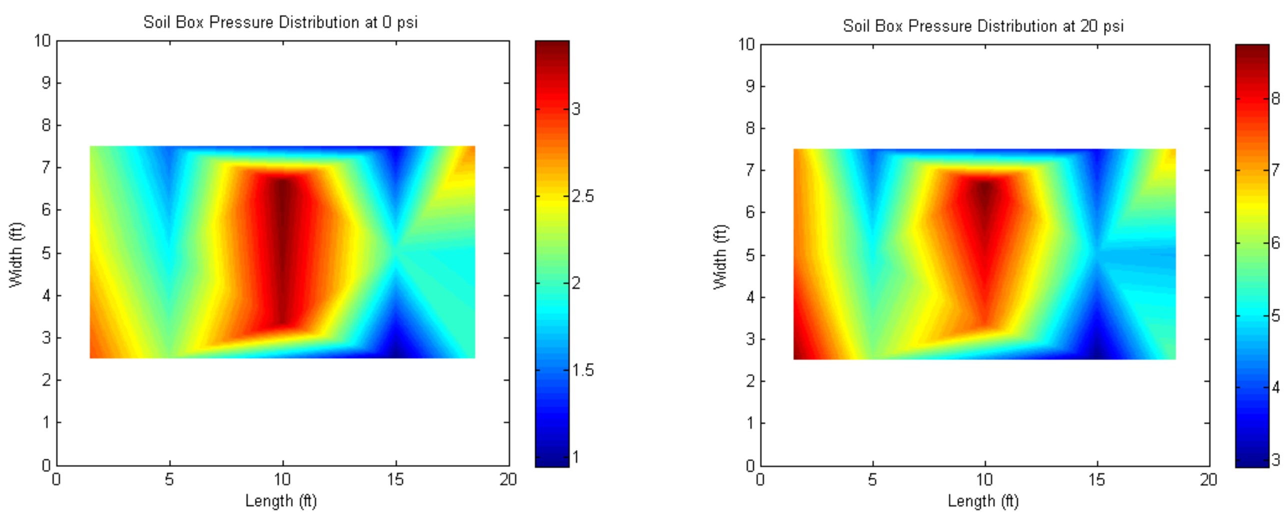
- FDOT's Soil Box was used to test a number of pipes. While simulated overburdens appear to be lower than the box's original design, the box is still capable of producing deflection versus simulated overburden data for pipe deflection tests.
- A new testing technique was developed in an attempt to increase Soil Box overburden stresses whereby the lift bags were buried in sediment. Results appeared to indicate that adding more sediment had more of an effect on overburden pressure than burying the bags.
- Analysis was conducted to determine the Soil Box's effectiveness at simulating uniform overburden pressures on buried pipes. Results appeared to indicate that overburden may be highly variable. This variability was attributed to nonuniform moisture conditions, nonuniform initial soil density, and nonuniform loading contact areas.
- A FE model was prepared by Simpson Gumpertz and Heger. Modeled results failed to match measured data. However, modeled results did appear to match computational guidelines from AASHTO.
- Modeled results appear to show small creep deflection in the buried pipe.
- A 3.5% short-term deflection criterion was recommended by SGH based partially upon modeled results and partially upon research for other pipe-types.
- UF/UNF believe that there may be significant value in developing a more-sophisticated computer model that would allow for more explicit conclusions in terms of identifying pipes that would fail in the long-term.

- Development of a sophisticated computer model hinges first on modeling the pipe itself correctly. If matching could be achieved for a simple loading scenario and a complex grid, it may be possible to add nonuniform soil to the model and achieve matching between modeled data and measured results. If results can be effectively matched, for several loading scenarios or an entire Soil Box data run, it may be possible to extrapolate results to explicitly take time into account throughout the lifetime of a structure.



A B

Figure 13-1. Stress distribution before and after saturation for 36 inch HDPE with trench box and 36 inch aluminum pipes.



A B

Figure 13-2. Stress distribution at 0 psi and 20 psi for 36 inch HDPE with trench box and 36 inch aluminum pipes.

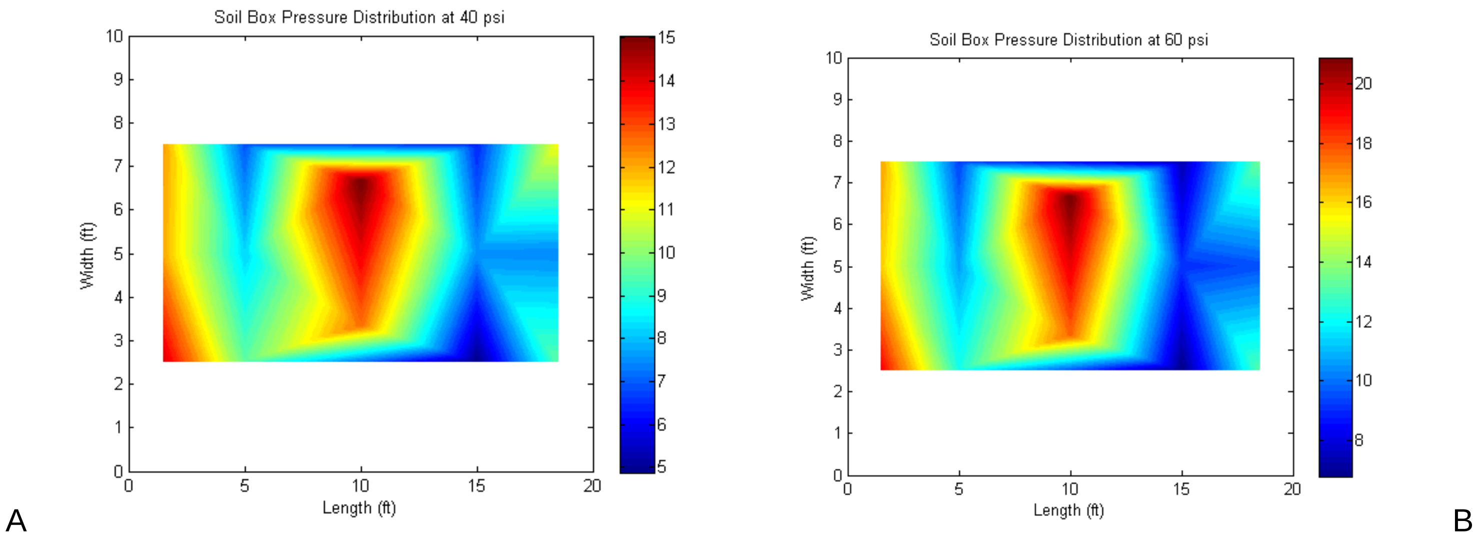


Figure 13-3. Stress distribution 40 psi and 60 psi for 36 inch HDPE with trench box and 36 inch aluminum pipes.

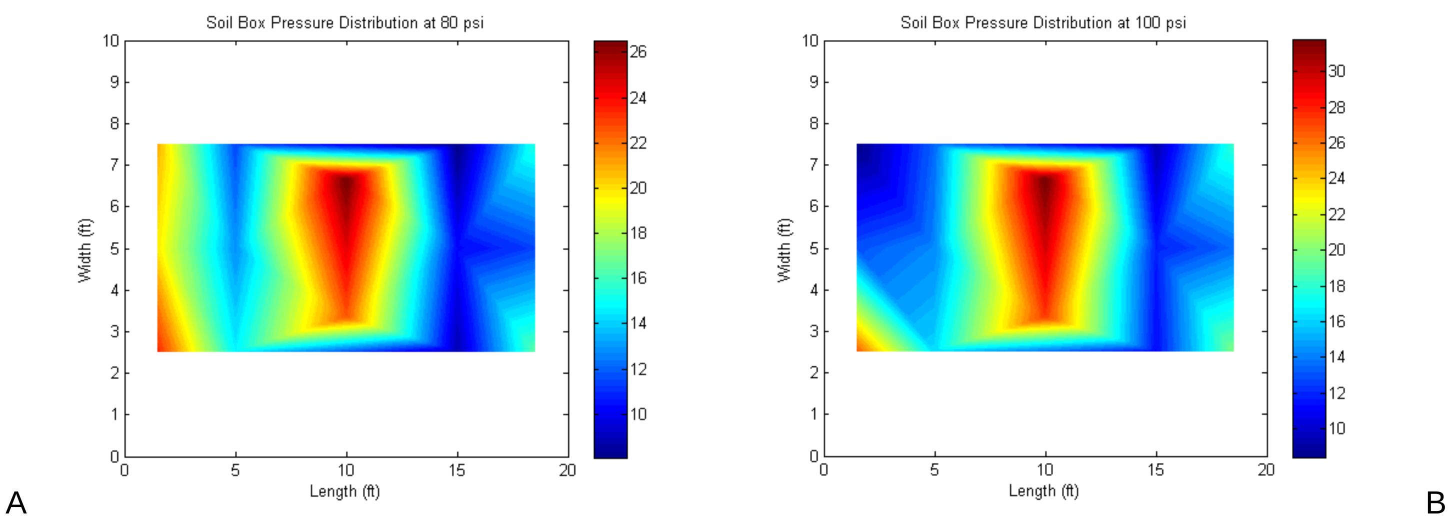
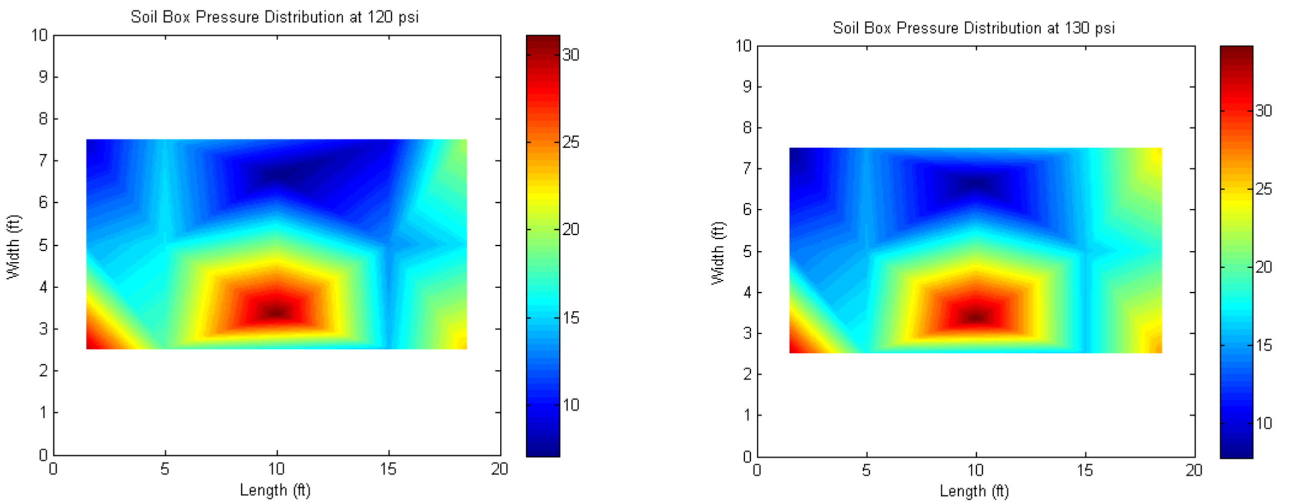


Figure 13-4. Stress distribution at 80 psi and 100 psi for 36 inch HDPE with trench box and 36 inch aluminum pipes.



A B

Figure 13-5. Stress distribution at 120 psi and 130 psi for 36 inch HDPE with trench box and 36 inch aluminum pipes.

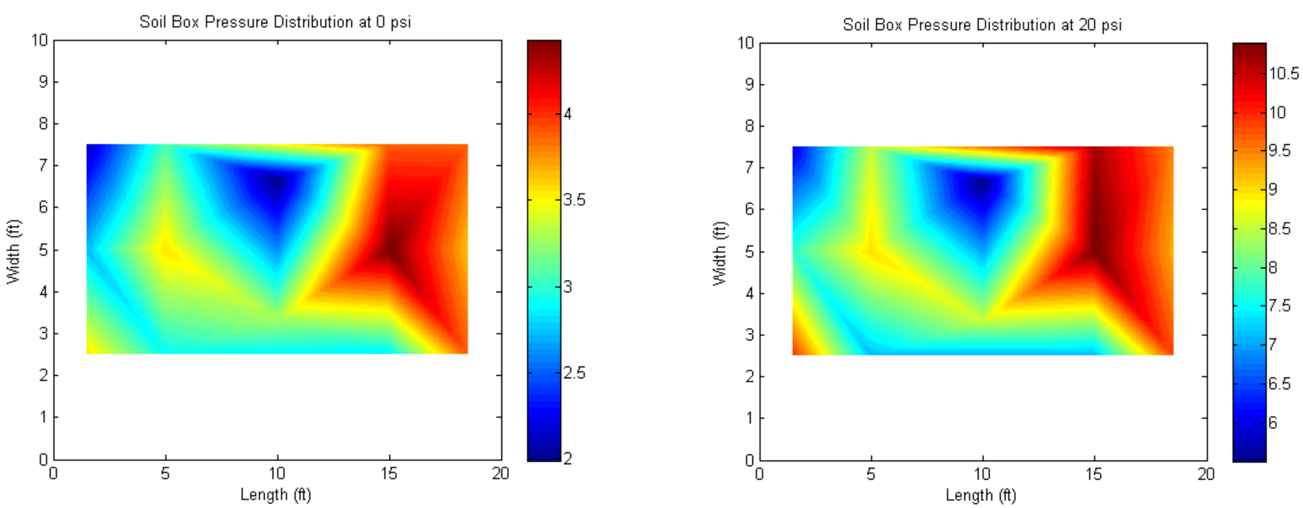


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Figure 13-6. Stress distribution at 0 psi and 20 psi for dual 24 inch HDPE pipes.

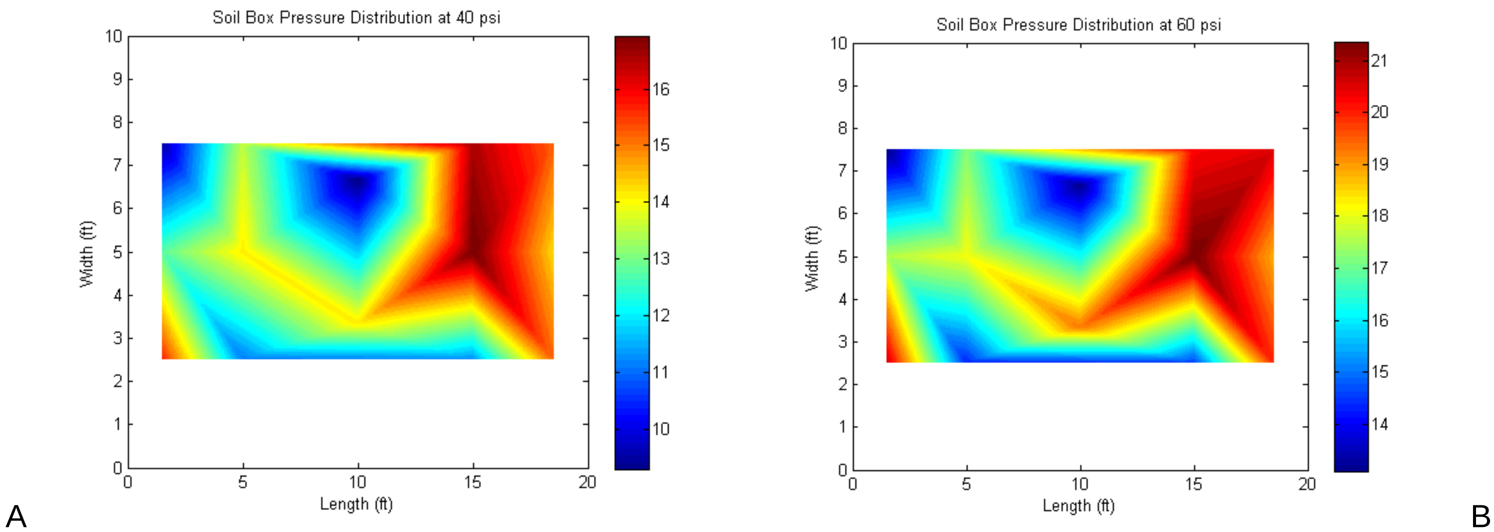


Figure 13-7. Stress distribution at 40 psi and 60 psi for dual 24 inch HDPE pipes.

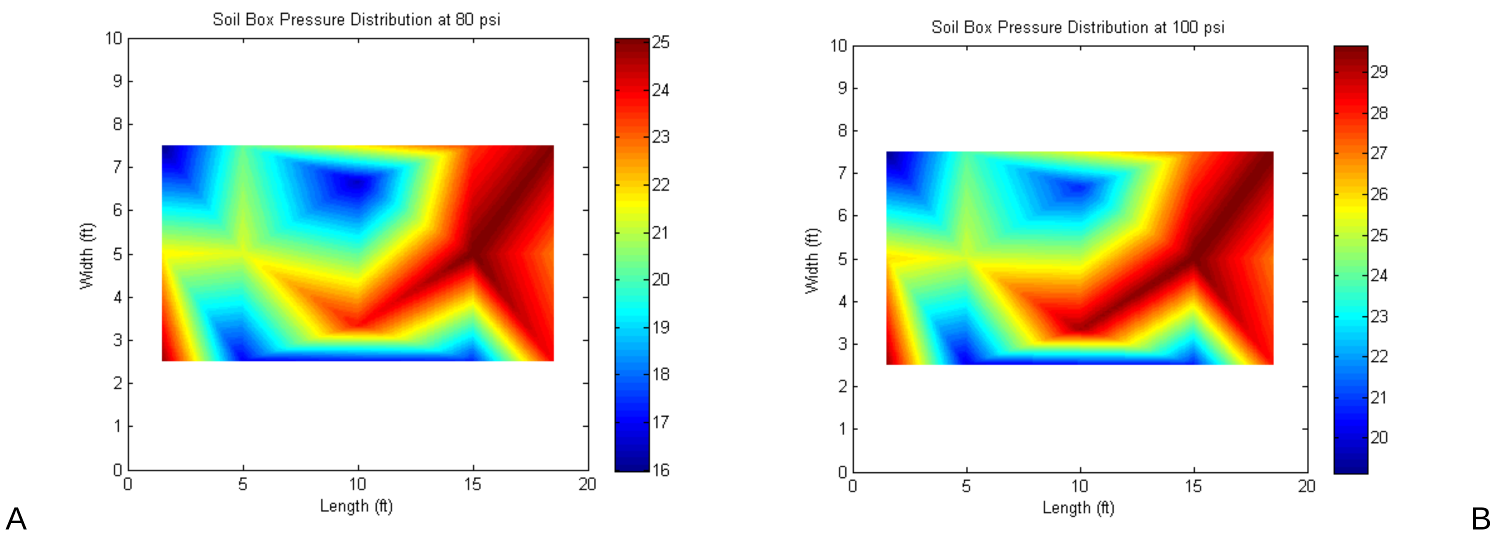
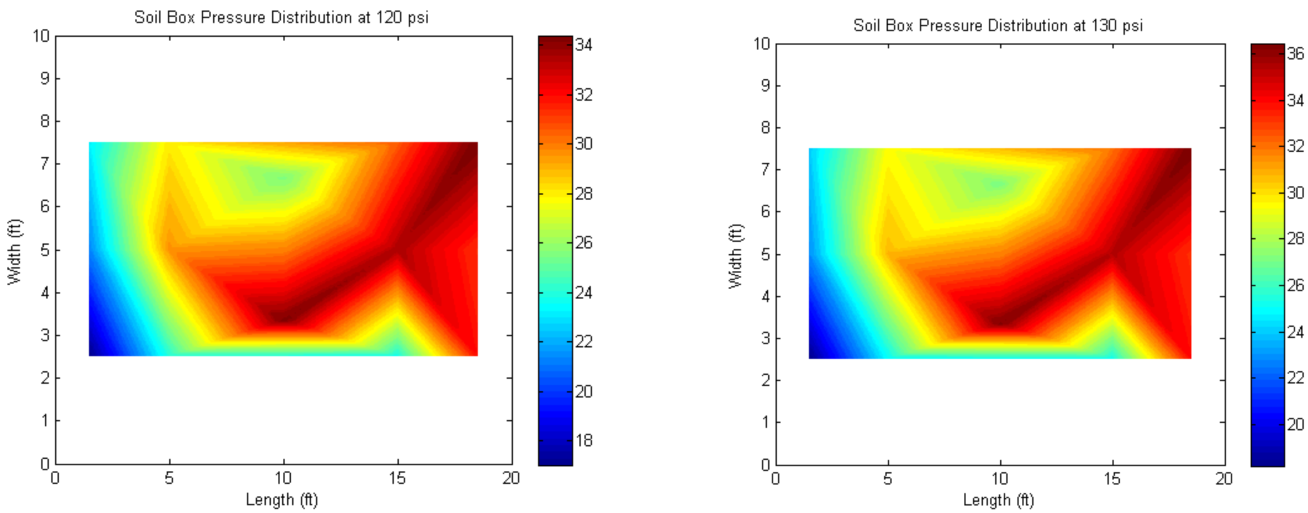


Figure 13-8. Stress distribution at 80 psi and 100 psi for dual 24 inch HDPE pipes.



A B

Figure 13-9. Stress distribution at 120 psi and 130 psi for dual 24 inch HDPE pipes.

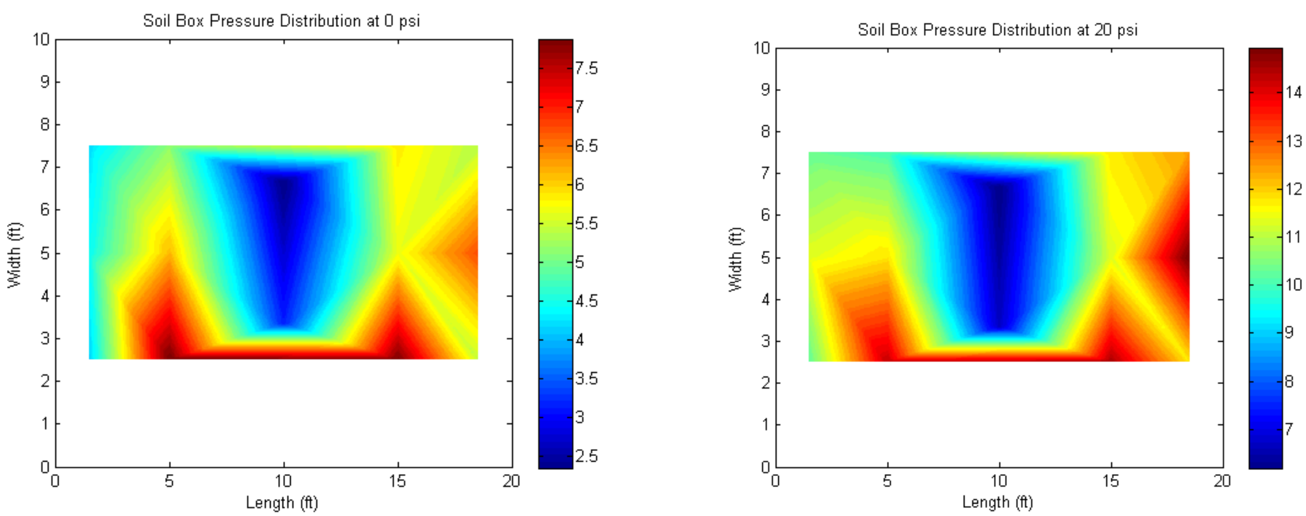


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Figure 13-10. Stress distribution at 0 psi and 20 psi for dual 24 inch steel pipes.

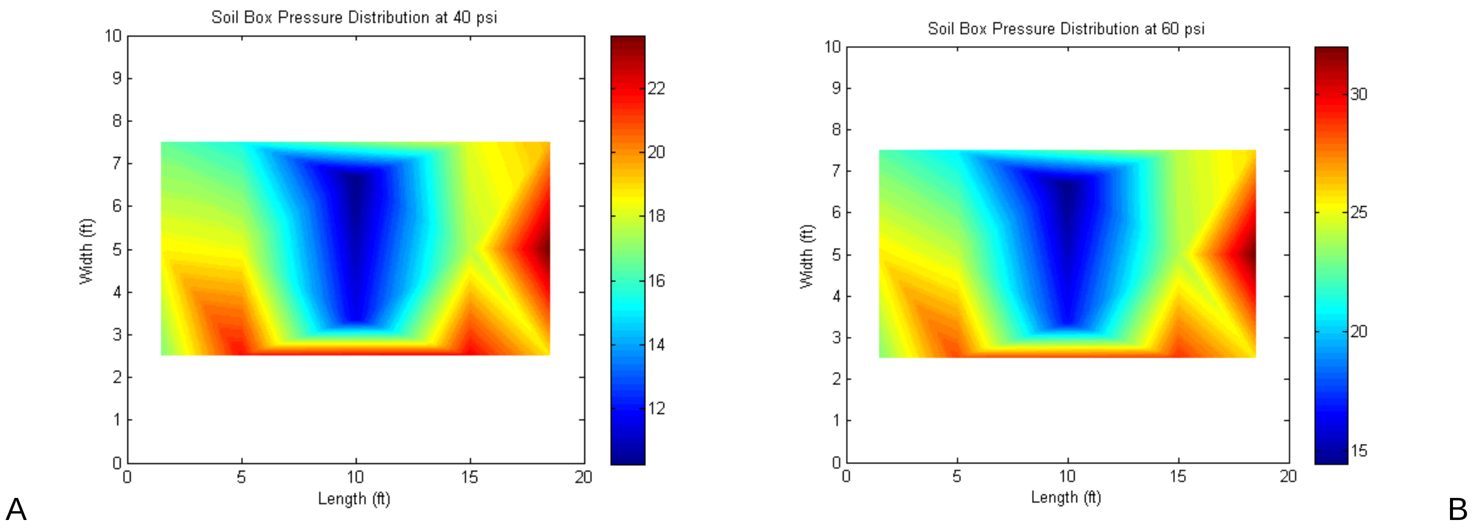


Figure 13-11. Stress distribution at 40 psi and 60 psi for dual 24 inch steel pipes.

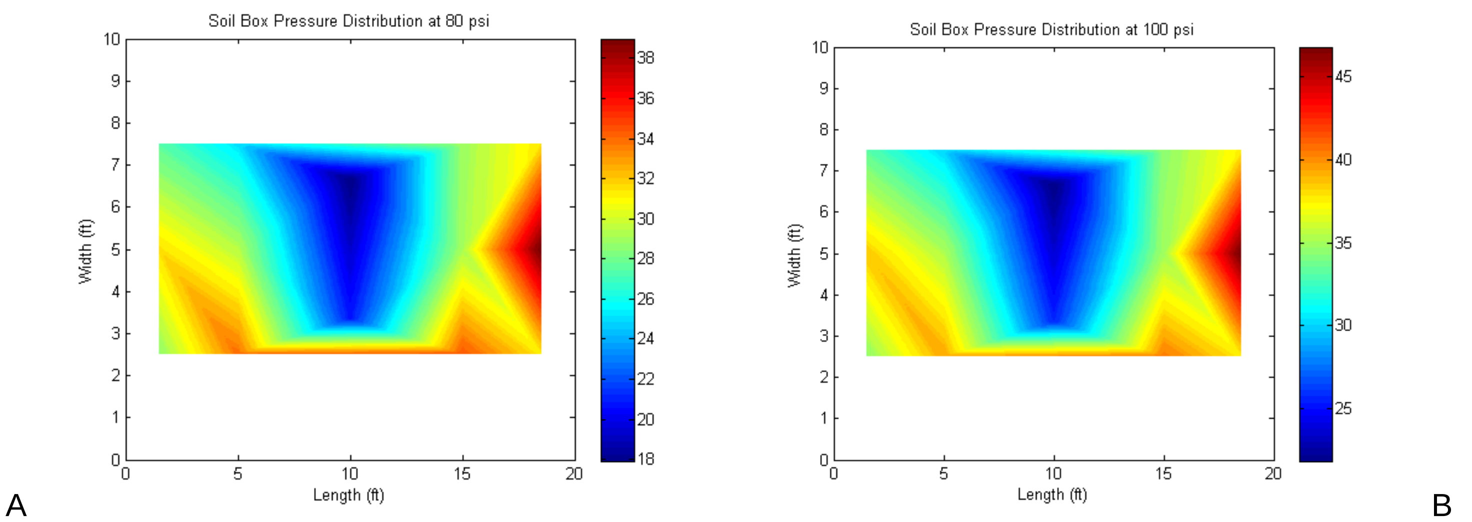
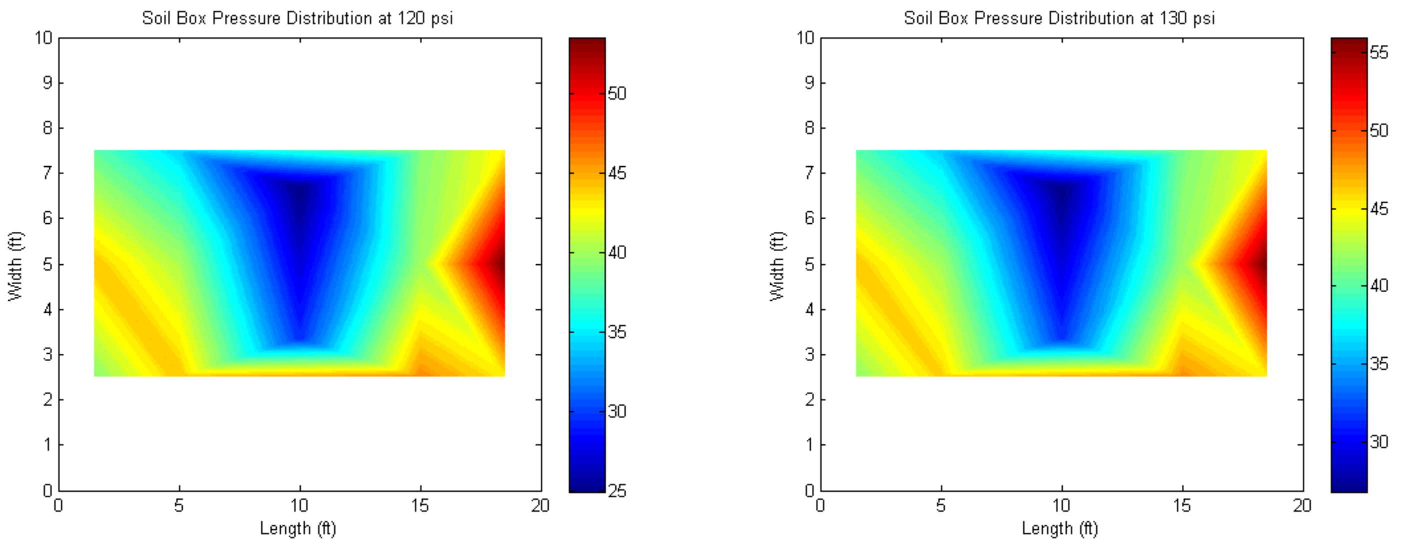


Figure 13-12. Stress distribution at 80 psi and 100 psi for dual 24 inch steel pipes.



A B

Figure 13-13. Stress distribution at 120 psi and 130 psi for dual 24 inch steel pipes.

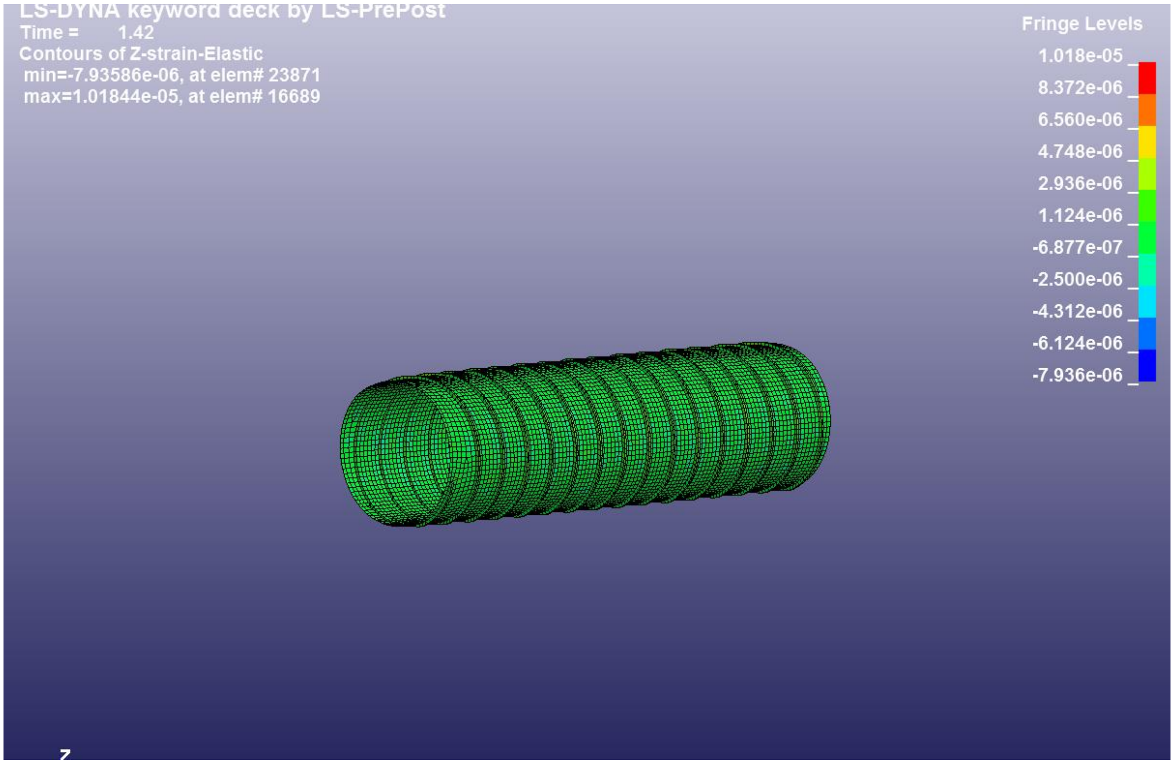


Figure 13-14.Spiraled pipe grid in LS-DYNA

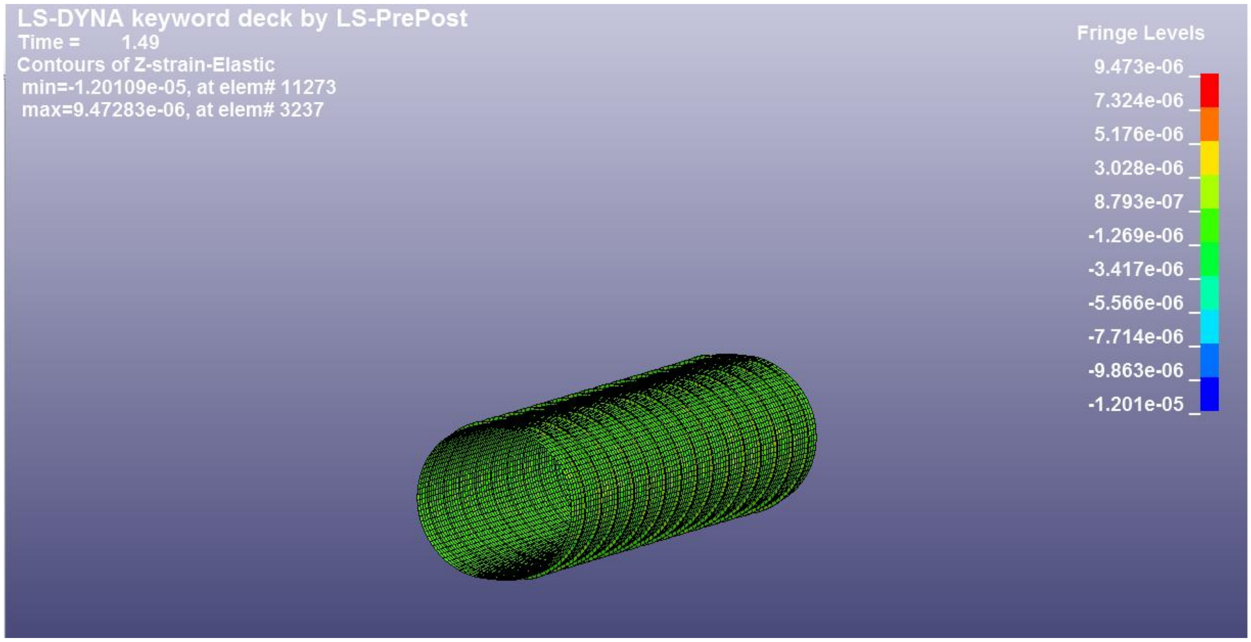


Figure 13-15.Non-spiraled equivalent pipe in LS-DYNA

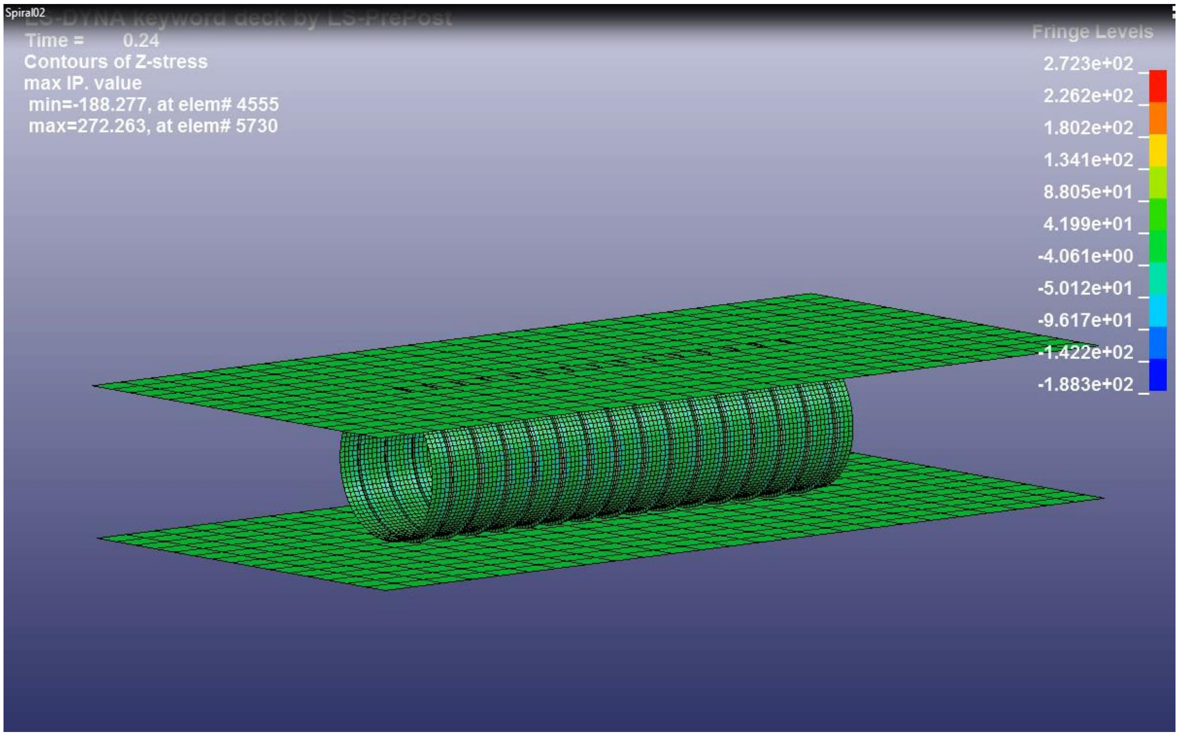


Figure 13-16.Example of “squash test”

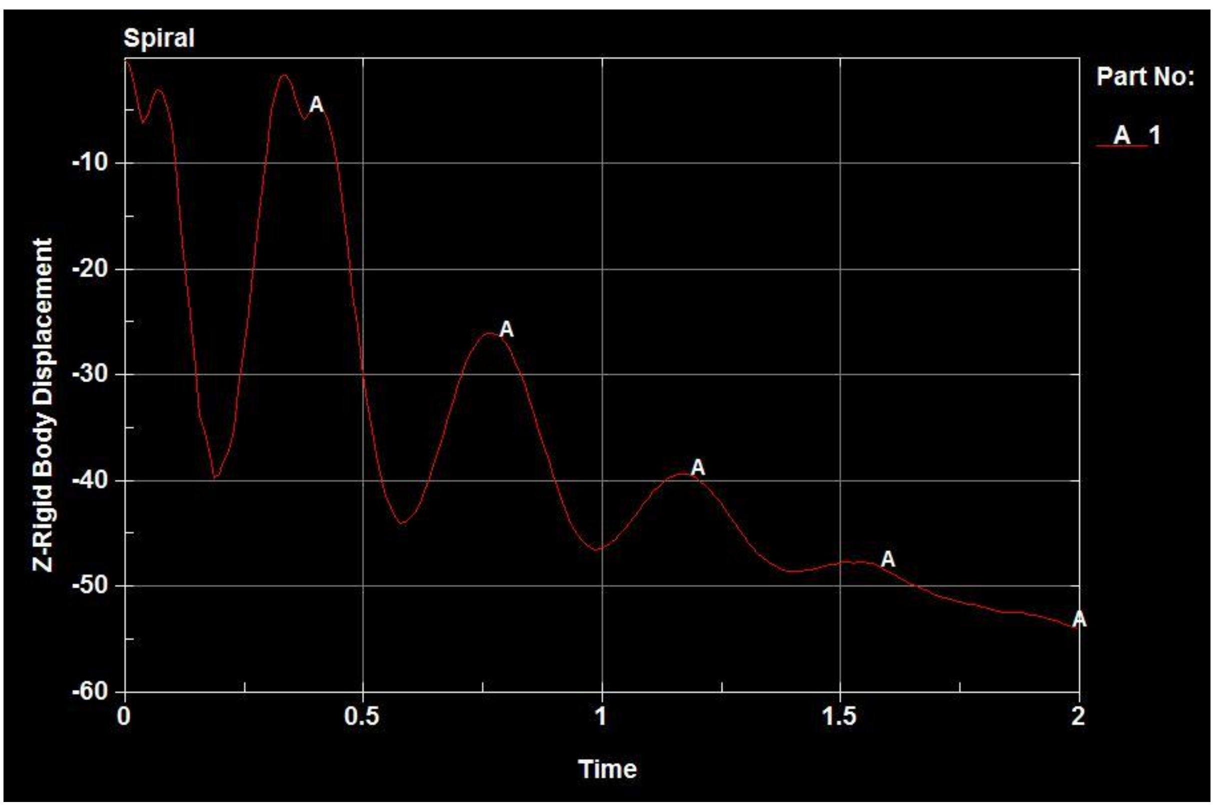


Figure 13-17.Deflection as a function of time for spiraled pipe “squash test”

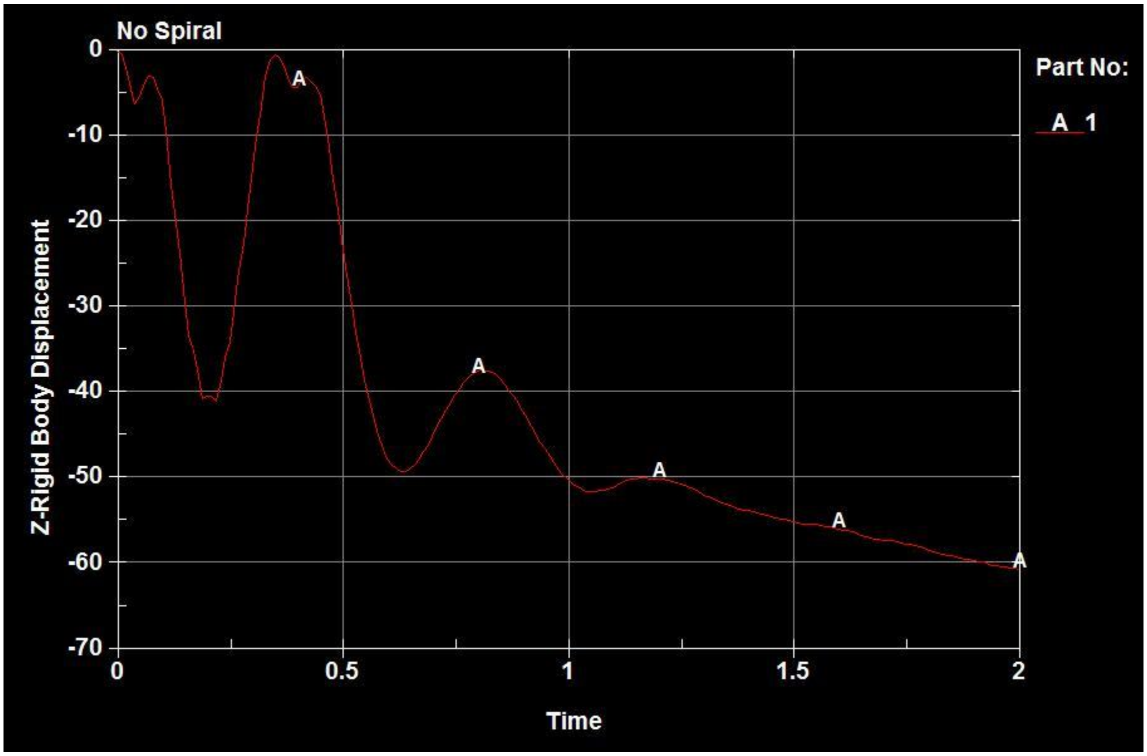


Figure 13-18.Deflection as a function of time for non-spiraled “squash test”

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APPENDIX A
SIMPSON, GUMPERTZ, AND HEGER REPORT

Time-Dependent Load Response of Flexible Pipe Subjected to Sustained Loading

30 September 2013

SGH Project 090928

PHILLIP A. SHARFF
LICENSE
No. 72118
STATE OF FLORIDA
PROFESSIONAL ENGINEER

30 Sept 2013

SIMPSON GUMPERTZ & HEGER

Engineering of Structures
and Building Enclosures

PREPARED FOR:

Florida Department of
Transportation
605 Suwannee Street
Tallahassee, FL 32399-04050

PREPARED BY:

Simpson Gumpertz & Heger Inc.
41 Seyon Street
Building 1, Suite 500
Waltham, MA 02453
Tel: 781.907.9000
Fax: 781.907.9009

SIMPSON GUMPERTZ & HEGER
Engineering of Structures
and Building Enclosures

30 September 2013

Mr. Rick Renna
Florida Department of Transportation
605 Suwannee Street
Tallahassee, FL 32399-04050

Project 090928 – Soil Box Pipe Tests, Analysis of University of Florida Tests, Gainesville, FL

Dear Mr. Renna:

We are pleased to provide you with this final report on our research to estimate time-dependent load response of flexible pipe subjected to sustained loading.

We look forward to discussing our findings with you.

Sincerely yours,

Phillip A. Sharff
Principal
FL License No. 72118

Timothy J. McGrath
Consultant

Bryan P. Strohman
Senior Staff II – Structures

P. Graham Cranston
Staff II – Structures

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TABLES

GLOSSARY

AASHTO	American Association of State Highway and Transportation Officials
AWWA	American Water Works Association
D_L	Deflection Lag Factor
E_i	Initial Elastic Pipe Modulus
E_l	Long-term Elastic Pipe Modulus
EPC	Earth Pressure Cell
E_s	Elastic Soil Modulus
FDOT	Florida Department of Transportation
FE	Finite Element
HDPE	High-Density Polyethylene
K_x	Bedding Coefficient
LRFD	Load Resistance Factor Design
I	Moment of Inertia
P	Confining Pressure Stress
PVC	Polyvinyl Chloride
R	Pipe Wall Centroid Radius
SC	Stiffness Category
SGH	Simpson Gumpertz & Heger Inc.
SPD	Standard Proctor Density
SPIDA	Soil Pipe Interaction Design and Analysis
UF	University of Florida at Gainesville
USCS	Unified Soil Classification System
M_{sb}	Constrained Modulus
β	Friction Angle for Drucker-Prager Soil Model
ϕ	Friction Angle for Mohr-Coulomb Soil Model
ν	Poisson's Ratio
σ_1	First Principal Stress
σ_2	Second Principal Stress
σ_3	Third Principal Stress
σ_y	Vertical Stress
Ψ	Dilation Angle for both the Drucker-Prager and Mohr-Coulomb Soil Models

1. INTRODUCTION

1.1 Background

The Florida Department of Transportation (FDOT) intends to implement an early deflection testing program for flexible buried pipe during construction. The proposed program targets pipe inspection after placement of approximately 3 ft of cover over top of pipe. Based on findings of this initial inspection, FDOT will schedule subsequent inspections for pipes that may be at risk of exceeding FDOT's long-term deflection limit. Implementing this early deflection testing program requires time-dependent deflection data for flexible pipe to allow FDOT to identify specific pipes whose deflections are within the typical 5% specification tolerance at the time of inspection, but may exceed the tolerance at the time of project acceptance or in the long-term. Insufficient time-dependent deflection data exist for flexible pipe, necessitating FDOT to conduct testing and analysis for flexible pipe installed using typical, but imperfect, construction practices for FDOT projects.

In 2009, FDOT engaged the University of Florida (UF) at Gainesville to perform testing of flexible pipe materials under sustained loading using a 20 ft long by 10 ft wide by 8 ft high soil box constructed (FDOT, 2009). UF engaged Simpson Gumpertz & Heger Inc. (SGH) to perform finite element modeling for comparison and calibration with the soil testing results. Soil box testing at UF's Coastal Laboratory began in 2010 and was completed in 2013.

1.2 Objective

The overall project objective was to estimate changes in flexible pipe deflection after installation to project long-term deflections based on tests conducted immediately after installation. SGH's objective was to calibrate finite element models using the soil box test data conditions to provide an analytical basis for predicting time-dependent changes in deflection.

1.3 Scope

UF and SGH completed the following tasks as part of this research[1]:

- Developed general and detailed approaches for soil box testing (UF).
- Conducted soil box testing in accordance with the general approach described in Section 2.2 (UF).

[1] The party responsible for each task is provided in parentheses.

- Reviewed soil box testing results (UF/SGH).
- Developed finite element models and post-processed analysis results (SGH).
- Prepared this report, which outlines the work performed to date and provides recommendations for estimating time-dependent increases in deflection (SGH).

This report provides a synopsis of the UF soil box study and results, details the development and application of our computer models, and presents our findings and recommendations.

2. UF SOIL BOX TESTS

2.1 Soil Box Design

UF constructed a 20 ft long by 10 ft wide by 8 ft high soil box. UF and FDOT designed the soil box to: limit wall deflections to less than 2 mm under 118 psi of vertical earth pressure (approximately equivalent to 147 ft of overburden); and eliminate boundary effects for 36 in. diameter pipes and smaller.

The final soil box design consists of bolted steel top, bottom, and side plates. Steel wide flange sections were welded to the exterior of the steel plates to stiffen the structure (reduce wall deflection). The soil box includes three 18 in. diameter portholes through the two sidewalls to allow inspection and measurements of the test pipes (Pasken, 2011 and Faraone, 2012). We show a photograph of the assembled box in Figure 1.

Figure 1 – Assembled soil box

2.2 Test Procedure

As described in FDOT, 2009, the general testing approach was to install two buried pipes of the same type in the soil box (for redundancy), install backfill, and apply a surface pressure to steel plates using pressurized lifting bags reacting against the top plate of the soil box.

UF performed sieve analyses of the backfill soil and classified the material as poorly graded sand (SP) according to the Unified Soil Classification System (USCS). Table 1 provides the backfill soil gradation.

Table 1 – Backfill Gradation

Sieve Number	Diameter (mm)	% Passing
4	4.75	99.86
10	2	99.45
20	0.85	98.33
30	0.6	96.76
50	0.3	82.61
100	0.15	30.33
200	0.075	4.84
Pan	-	0.00

Pipe materials included high-density polyethylene (HDPE), polyvinyl chloride (PVC), steel, and aluminum. All pipes had corrugated profiles. For two tests, one of the pipes was installed using a trench box that was removed after backfilling leaving a region of disturbed soil. Test pipes were predeflected 4% horizontally intended to simulate poor installation practices and encourage additional pipe deflection. Pipe deflections were recorded by laser scans during various construction phases and loading. Earth pressure cells (EPC) and soil strain gages were installed at specific locations within the soil strata to obtain the stress-strain relationship of the soil during testing. UF performed nuclear density tests during backfill installation.

UF performed testing in accordance with the following general steps:

1. Pipe preparation:
    - Cut pipe specimens to fit in the 10 ft wide box.
    - Using compression turnbuckles at the springline, predeflect each pipe horizontally to 4% increase in diameter.
2. Soil box preparation:
    - Install two layers of 4 mil thick polyethylene Poly-Cover manufactured by Warp Bros. on the soil box walls. Place grease between the two Poly-Cover layers.
    - Install a 12 in. thick layer of compacted soil in the soil box.
3. Install pipe and trench box (if applicable)

4. Backfill installation:

    - Manually place backfill using a shovel in 18 in. thick lifts to approximately 1 ft below the soil box top without deliberate compaction.

    - Remove the trench box from the backfilled soil box, if applicable. Fill the resulting voids at the ground surface with soil.

5. Introduce a flow of water in the backfill using a sprinkler at the ground surface. The sprinkler runs for about 34 hrs. Neither earth pressure cell nor deflection data were collected prior to or during soil saturation. Soil density and strain measurements were not collected after this step was completed.

6. Install 3/4 in. thick steel plates on the backfill surface. Place a pressurized lifting bag on each steel plate. Bolt the soil box top to the side walls.

7. Apply pressure to the lift bags and record earth pressure and pipe deflections.

UF conducted a total of eight tests. Table 2 reports the primary variables of these tests.

Table 2 – Matrix of UF soil box tests

Test No.	Diameter, in.	North Pipe Material	South Pipe Material	Trench Box?	Final Pressure†, psi
1	36	HDPE		No	66.67
2	Soil pressure calibration test – no pipe installed				
3	36	PVC		No	130
4	36	Steel		No	130
5	36	HDPE	Steel	Yes	130
6	36	HDPE	Aluminum	Yes	130
7	24	HDPE		No	130
8	24	Steel		No	130

† The final lift bag pressure is the pressure of the large lift bags

UF post-processed earth pressure cell, pipe deflection, and nuclear density test results from the eight soil box tests. We provide relevant, representative results in the following section.

2.3 Instrumentation

UF made the following measurements during the tests:

- Nuclear density tests of each uncompacted lift while backfilling the soil box.

- Earth pressure measurements from a set of earth pressure cells (EPCs) embedded throughout the backfill. UF installed the EPCs both directly above the pipes and centered between the two pipes at several elevations.

- Vertical deflection measurements from a laser range finder affixed to a trolley inside the pipe.
- Vertical strain measurements using string potentiometers located above the pipe. However, data were not reported in Pasken (2011) nor in Faraone (2012).

Density Test Results

UF informed us that the test procedure was to place backfill without compaction. It is likely that foot traffic in the soil box during backfilling and especially when installing the EPCs compacted the soil to some extent. The uncontrolled compaction would be not produce a uniform density throughout the backfill, particularly in the haunch areas.

The laboratory standard Proctor compaction test by UF provided a maximum dry density of 110.2 pcf, and a maximum wet density at optimum moisture content of 124.0 pcf. Table 3 summarizes the average, maximum, and minimum percent compaction for the backfill in each test. The nuclear density tests were performed at multiple locations and elevations within the backfill, though not in the haunch area.

Table 3 – Backfill compaction from soil box tests (from UF)

Test No.	Diameter, in.	North Pipe Material	South Pipe Material	Compaction, %		
				Average	Maximum	Minimum
1	36	HDPE		87.4	91.3	84.5
2	-	-		-	-	-
3	36	PVC		88.6	91.1	84.3
4	36	Steel		87.9	93.9	84.3
5	36	HDPE	Steel	85.6	89.3	82.2
6	36	HDPE	Aluminum	86.6	91.9	78.1
7	24	HDPE		87.3	89.8	84.6
8	24	Steel		86.2	89.5	82.8

Based on the laboratory compaction curve provided by UF, the average backfill compaction achieved was generally between 85% and 90% at the locations tested. Since UF did not perform density tests at the haunch areas, the compaction there is unknown and based on their description of their backfill procedure the compaction at the haunches is likely significantly lower.

Earth Pressure Data

The earth pressure data vary significantly at a given elevation and applied load. Taking for example pressures at an elevation near the top of the pipe for Test No. 3 (36 in. diameter PVC pipes), the earth pressures varied from 3.6 to 6.8 (-28%/+34% from mean) psi at 10 psi applied pressure, and varied from 18.8 to 30.0 psi (-28%/+17% from mean) at 110 psi applied pressure. These results are representative of the earth pressure cell data variability in the other seven tests. Given the uncertainty in the earth pressure data, our study relies on the pipe deflection alone.

Pipe Deflection Data

Figures 2 and 3 show deflection vs. “simulated overburden” plots for the 36 in. diameter (Test Nos. 1, 3-6) and 24 in. diameter tests (Test Nos. 7 and 8), respectively. UF computed a simulated overburden by dividing the average pressure readings from the earth pressure cell closest to the top of the pipe by the average unit weight of the soil in the soil box. Given that the crown soil pressure over flexible pipe varies widely under the same vertical pressure based on the type of pipe being tested, and the variability in the soil pressure data, the simulated overburden calculation may be unreliable.

The deflections follow a trend that lower pipe stiffness ($EI/0.149\ R^3$) correlates with higher pipe deflection. HDPE pipes have the highest deflections, followed by PVC. Steel pipes have the lowest deflections. There is scatter in the deflection data. For example, the initial deflections of two 24 in. HDPE pipes in Figure 3 differ by about 1.5%.

Figure 2 shows tests of HDPE pipes installed with a trench box. Towards the end of these tests the load-deflection curves turn nearly vertical, suggesting that the pipes have failed. UF observed local buckling in the HDPE pipe wall during this test (Faraone, 2012).

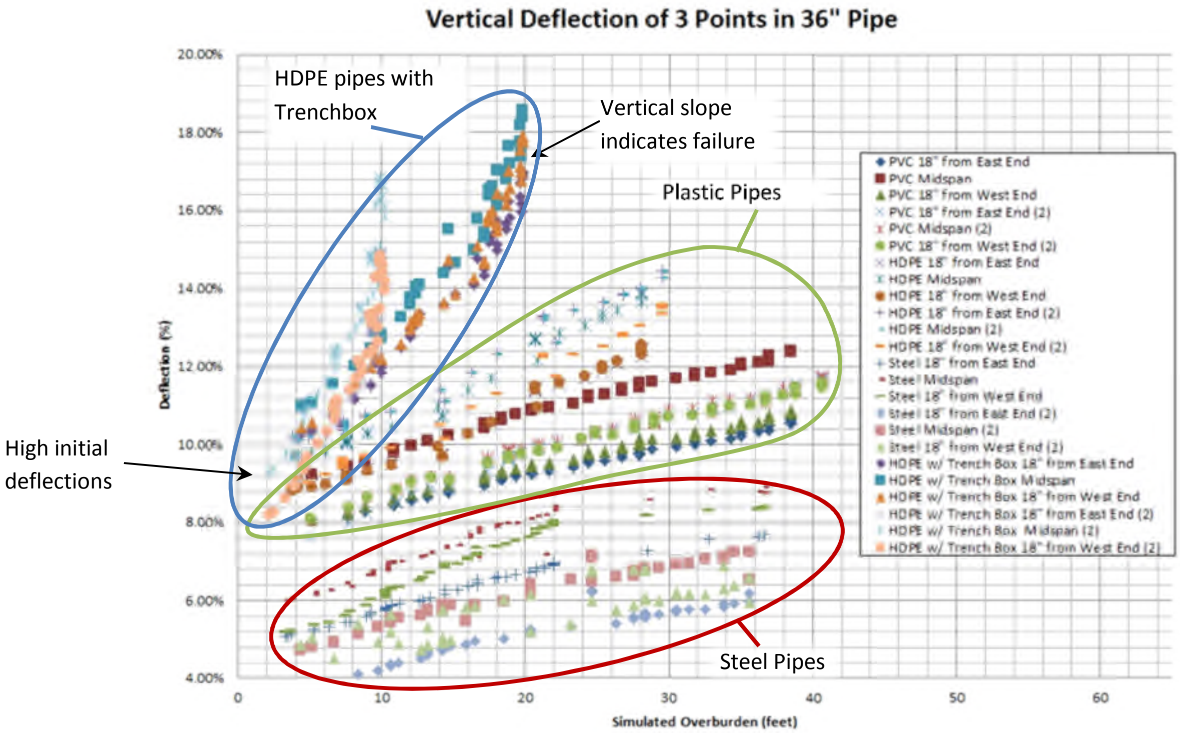

Figure 2 – Deflection vs. simulated overburden from UF tests on 36 in. diameter pipe. Annotations by SGH. East End, Midspan, and West End refer to locations along the length of the pipe. (1) refers to the first of two pipes in a given test and (2) refers to the second pipe in that test.

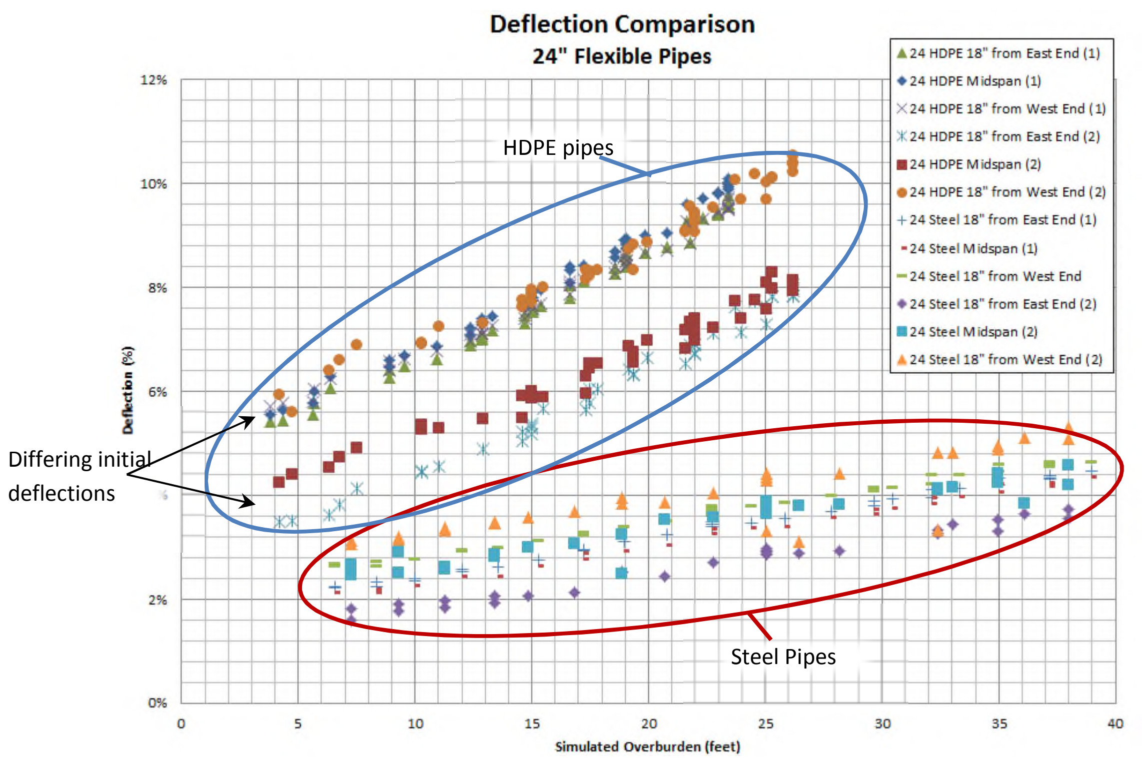

Figure 3 – Deflection vs. simulated overburden from UF tests on 24 in. diameter pipe. Annotations by SGH. East End, Midspan, and West End refer to locations along the length of the pipe. (1) refers to the first of two pipes in a given test and (2) refers to the second pipe in that test.

2.4 Comments on UF Test Results

Figures 2 and 3 show initial deflections (prior to surcharging) in many cases are unrealistically high. For example, tests of 36 in. PVC pipes show the initial deflection at the middle of the pipe to be 9.23% in one case. Hand calculations for the condition of a pipe supporting the entire weight of the soil prism with $\gamma_s = 115\, pcf$ as a line load applied at the crown with no lateral soil support give a deflection of 5.36%. The wide scatter in the deflection data suggests variable control of the installation conditions for each pipe (bedding angle and/or backfill compaction). However, in general, the incremental deflection under surcharge loading is consistent with backfill compacted to 85% standard Proctor density according to hand calculations of deflections following AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications (2012) §12.12.2.2. It appears the introduction of water did not change the stiffness of the backfill and may not have significantly changed the soil density. The lack of deflection data prior to the introduction of water to the backfill prevents clarification on this issue.

UF encountered several issues during testing, documented in Faraone, 2012, including the following:

- During the test of two 36 in. PVC pipes (Test No. 3), soil pressures below the loading plates varied by 30% or more (Faraone, Ch. 2). For subsequent tests, UF modified both the pressure system (by adding regulators for individual lift bags) and the loading plates (by placing chain link fence below the plates to transfer load at the edges).

- During the test of two 36 in. steel pipes (Test No. 4), UF found deflection readings were erratic and applied a flat red paint to the pipe to improve performance of the laser measurement system during later stages of the test (Faraone, Ch. 3). Also during this test, UF noted that the pipe with the higher displacement was subject to the lower pressure according to the EPCs.

- During the wetting phase of the test of a 36 in. steel pipe and a 36 in. HDPE with trench box (Test No. 5), a significant volume of soil escaped from the soil box through a failed membrane seal. UF filled the void and proceeded with the test (Faraone, Ch. 4).

- According to density tests taken at the soil surface (Faraone, Ch. 3), the target saturation of 100% was not achieved in any test. The maximum saturation achieved was 62% after running a sprinkler for 50 hours on the top of the soil box, with only a 3.5% change in the saturation during the final 32.5 hours.

Considering the issues described above, our analyses are based on published values for soil properties and results are compared with deflections calculated according to code equations for model validation. Backfill was compacted to 85% and 90% of maximum standard Proctor density, densified to 95% standard Proctor density, as described in the following chapter.

3. COMPUTER MODELS FOR SOIL BOX TESTS

We investigated the deflections due to applied loads and time-dependent phenomena, using two-dimensional finite element (FE) models of the soil box. The modeling included all pipe diameters and pipe materials tested by UF with loading and boundary conditions similar to those in the tests.

We considered time-dependent soil strain in the backfill and creep in plastic pipes. The time-dependent soil strain, referred to as 'densification', is similar to classic consolidation theory, assuming that the soil strain expected to occur during the wetting phase (Test Step 5, see Section 2.2) would be a result of soil particles rearranging during the wetting and subsequent draining of the soil box.

3.1 Analysis Software and General Approach

We developed 2D plane-strain finite element models of the soil box tests using the commercially available FE program ABAQUS. We modeled the soil using the 4-node 2D plane-strain element CPE4, and the pipe wall using the 2-node 2D linear beam element B21. The model domain comprises one-quarter of the UF soil box, with symmetry planes through the centerline of the pipe and at the mid-point horizontally between pipes, as shown in Figure 4.

For both 24 in. and 36 in. nominal pipe diameters modeled, the pipe was founded on a 12 in. thick compacted bedding layer. Backfill was placed in three lifts: to the springline, to just above the pipe crown, and then to the top of the box. The loading sequence is described in Section 3.3. Soil lifts were added using the *MODEL, CHANGE method. The soil in the haunch areas was the same type and compaction level as elsewhere in the backfill.

The pipes were bonded to the soil. The soil material model, described in Section 3.2, included Drucker-Prager plasticity. Any slipping between or separation of the soil and the pipe was captured in the plastic deformation of the soil.

A second mesh of linear-elastic material was included over the soil zone, sharing the same nodes as the soil elements. This material had a very low elastic modulus, and serves two purposes:

- Improve the numerical conditioning of the problem during large plastic deformation, e.g., shearing/slipping planes and areas of tensile stress in the soil, and

- Adjust the position of the nodes for the backfill and cover lifts before those lifts are added to the model to smooth the mesh and reduce the tendency for distorted elements along the boundary between lifts.

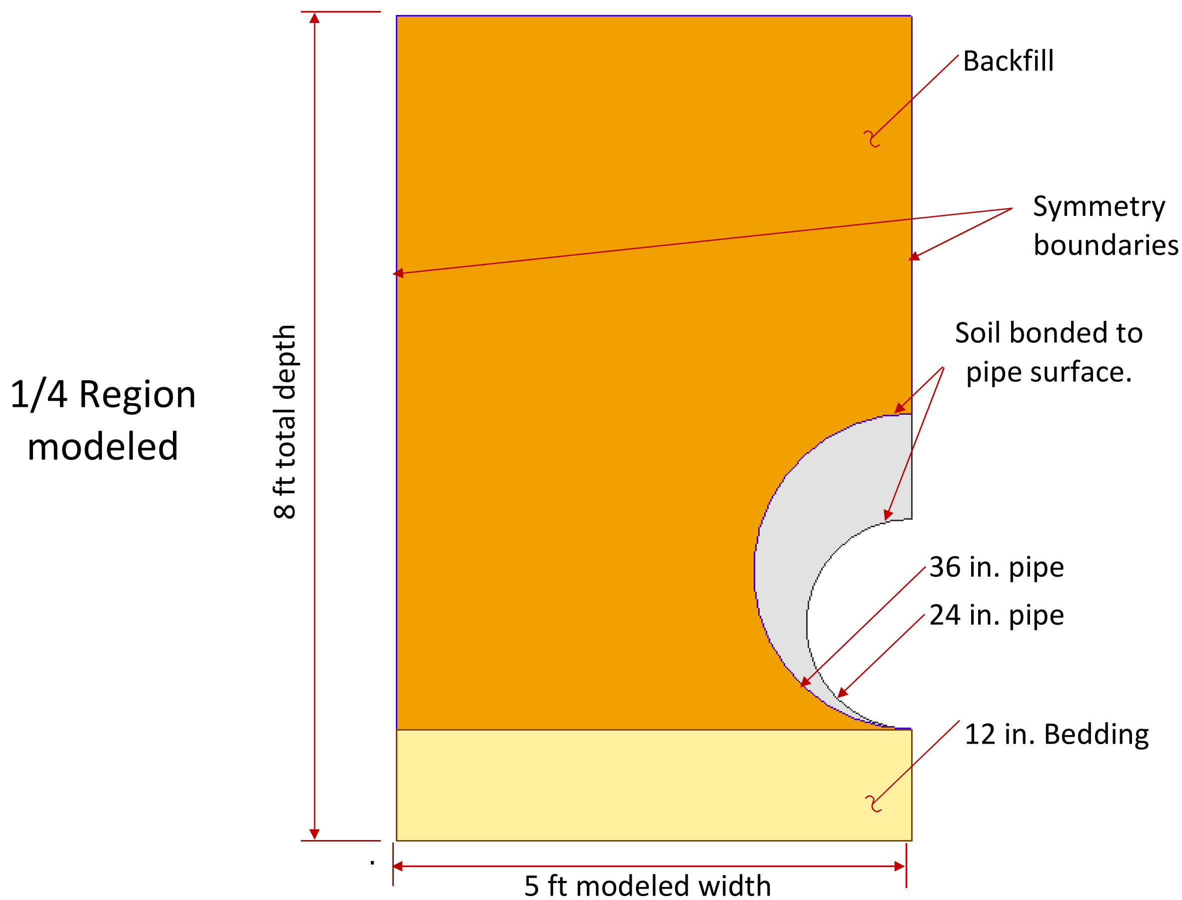

Figure 4 – Overview of FE model, including location and size of pipes tested, bedding depth, and cover used in analysis.

Table 4 shows the material and section properties for the eight pipes analyzed. The cross-section properties for PVC and HDPE pipes were obtained from manufacturers' cut sheets; the cross-section properties for steel and aluminum pipes were obtained from AASHTO LRFD Bridge Design Specifications (2012), Tables A12-2 and A12-5, respectively. Centroid radius (R) is the radial distance from the pipe axis to the centroid of the pipe wall cross-section. Area and moment of inertia (I) are values for the pipe wall per unit length along the pipe axis. For plastic pipes, both the initial, E_i, and long-term, E_l, elastic moduli were taken from AASHTO Table 12.12.3.3-1.

Table 4 – Pipe section properties

Diameter	Material	SECTION GEOMETRY			MATERIAL PROPERTIES			Calculated Pipe Stiffness, lbf/in.†
		Centroid Radius, in.	Area, in.²/in.	I, in.⁴/in.	E_i, ksi	E_l, Ksi	Unit Weight, pcf	
36 in.	PVC	18.38	0.395	0.146	400	140	81.1	63.24
	HDPE	19.07	0.403	0.319	110	22	59.3	33.96
	Steel	18.26	0.042	0.003	29,000		495.0	90.33
	Alum.	18.27	0.035	0.003	10,000		169.3	28.18
24 in.	PVC	12.15	0.270	0.042	400	140	81.1	63.06
	HDPE	12.72	0.330	0.133	110	22	59.3	47.84
	Steel	12.26	0.042	0.003	29,000		495.0	298.50
	Alum.	12.27	0.035	0.003	10,000		169.3	93.01

Notes:
† – Pipe stiffness was calculated as $EI/(0.149\ R^3)$. Note that calculated pipe stiffness usually varies from tested pipe stiffness; however, for the purposes of this study the variation is not significant.

3.2 Soil Models – Behavior, Approach to Densification

Approach

We developed soil material definitions using data from AWWA[2] Manual of Practice M45 (2005), Selig (1990), and SPIDA[3] (Heger et. al, 1985). The backfill utilized in the soil box tests (poorly graded sand (SP)) is one of several soil types (SW, SP, GW, and GP) classified in AWWA M45 as Stiffness Category (SC) 2, defined as clean, coarse grained soils with less than 12% passing the No. 200 sieve. Our soil definitions are based on properties of SC2 soils and are referred to as "SW" soils for purposes of finite element analyses. The soil material was linear-elastic with Drucker-Prager plasticity and linear hardening, and included stress-dependent elastic modulus and Poisson's ratio.

Elasticity

The elastic stiffness of soil depends on the confining pressure. This behavior is included in design standards such as AWWA M45 (2005) and the AASHTO LRFD Bridge Design Specifications, 6th Ed. (2012). AWWA M45 Table 5-4 gives the values for constrained modulus M_{sb} of backfill soils as a function of vertical stress, shown in Table 5, based on the work of McGrath (1998). The same values are also given in AASHTO Table 12.12.3.5-1. The soil classification in AWWA M45 is based on Stiffness Category and percent Standard Proctor

[2] American Water Works Association.

[3] Soil Pipe Interaction Design and Analysis.

Density (SPD) Compaction. USCS group SW soils are consistent with Stiffness Category SC2 per M45, Table 6-1).

Table 5 – Soil constrained modulus as a function of depth for backfill soils (AWWA M45)

Vertical Stress Level, psi	Constrained Modulus, psi		
	SW95	SW90	SW85
1	2,000	1,275	470
5	2,600	1,500	520
10	3,000	1,625	570
20	4,350	1,800	650
40	4,250	2,100	825
60	5,000	2,500	1,000

Similar to the soil stiffness, Poisson's ratio also depends on confining pressure. Selig (1990) published values of Poisson's ratio, ν, recommended for design of buried pipes as a function of vertical stress, shown in Table 6.

Table 6 – Soil Poisson's ratio as a function of vertical stress (Selig (1990))

Vertical Stress Level, psi	Poisson's Ratio	
	SW95	SW85
1	0.40	0.26
5	0.29	0.21
10	0.24	0.19
20	0.23	0.19
40	0.25	0.23
60	0.29	0.28

We defined an elastic modulus and Poisson's ratio for each soil based on the values in Table 5 and Table 6 that depend on the confining pressure stress $P = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$ evaluated at each point in the material. For Poisson's ratio for SW90 soil, we interpolated between the values given for SW85 and SW95. The values of constrained modulus and Poisson's ratio are so-called 'secant' parameters, in that they represent the average stiffness and volume change from zero to a given stress level, rather than the instantaneous stiffness or volume change at that stress level.

The elastic modulus E_s, Poisson's ratio ν, and constrained modulus M_{sb} at a given vertical stress level are related by the following equation:

$$E_s = \frac{M_{sb}[1+\nu][1-2\nu]}{[1-\nu]} \qquad \text{Eq. 1}$$

For the specific case of a block of material constrained in two transverse directions and compressed vertically, the confining pressure stress is expressed as a function of the vertical stress as:

$$P = \frac{\sigma_y}{3}\left[\frac{2\nu}{1-\nu} + 1\right] \qquad \text{Eq. 2}$$

where ν is the Poisson's ratio that also depends on the vertical stress, σ_y.

To define the pressure-dependent elastic modulus and Poisson's ratio in the analyses, we tabulated values of constrained modulus and Poisson's ratio as functions of *pressure* stress based on Eq. 1 and Eq. 2. Custom-written subroutines look up the pressure stress at each material integration point and store the values as field variables at each increment. The values used for element calculations are then interpolated from the tabulated values (input) based on the pressure stresses from the previous increment (state). We developed the pressure-dependent soil models for SW85, SW90, and SW95 soils.

Figure 5 compares the elastic modulus, constrained modulus, and Poisson's ratio that form the basis of the material definition for SW90 soil with those same parameters calculated from three-element tests under a 60 psi vertical surcharge load and constrained laterally in both directions. The calculated moduli and Poisson's ratio show excellent agreement with the input material parameters, validating the material model.

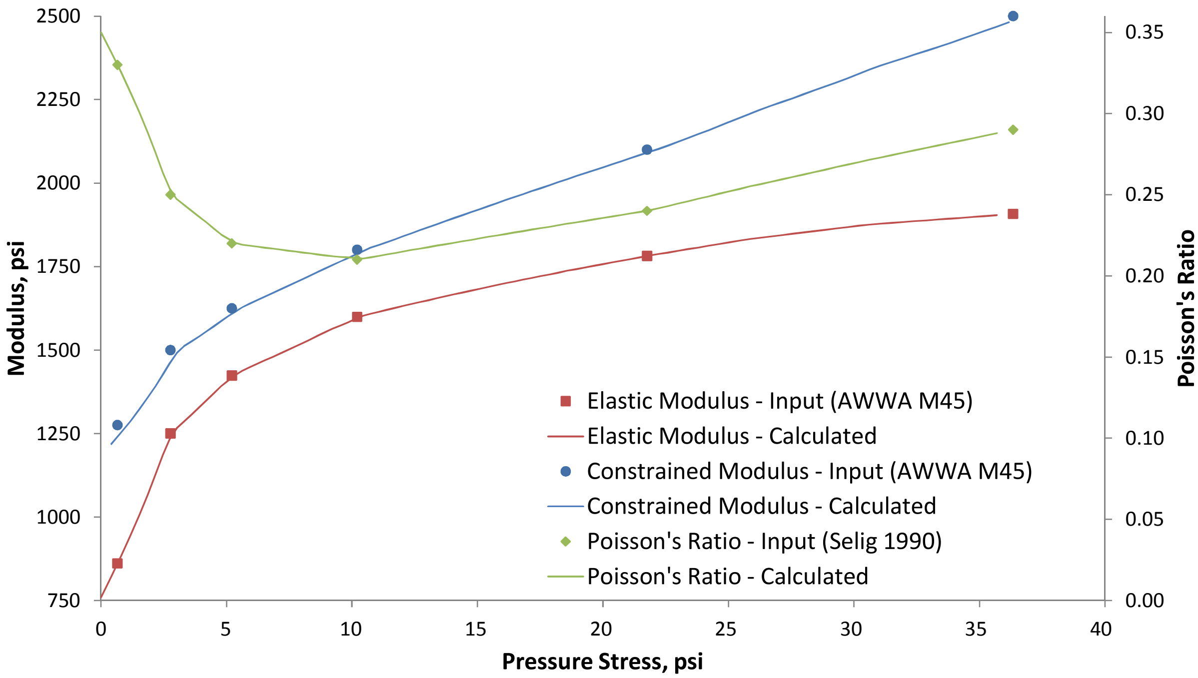

Figure 5 – Comparison of input soil parameters and those calculated from three-element test of pressure-dependent soil material definition

Plasticity

Drucker-Prager plasticity parameters were calculated following the procedure in ABAQUS §23.3 from the Mohr-Coulomb plastic parameters and unit weight from SPIDA (Heger et. al, 1985). A nominal cohesion of 0.1 psi was included for numerical stability. Dilation angle ψ is taken as $\psi = \phi - 30°$, based on Bolton (1986), where ϕ is the friction angle for the Mohr-Coulomb soil model. We present the soil parameters in Table 7.

Table 7 – Soil material plastic parameters

Designation	Mohr-Coulomb Parameters			Drucker-Prager Parameters			Unit Weight, pcf
	Friction Angle, ϕ, deg.	Dilation Angle, ψ, deg.	Cohesion, psi	Friction Angle, β, deg.	Dilation Angle, ψ, deg.	Yield Stress, psi	
SW95	48	18	0.10	51.05	18	0.111	141
SW90	42	12	0.10	48.51	12	0.126	134
SW85	38	8	0.10	46.20	8	0.257	126

Densification

In the UF soil box tests, the effect of soil wetting on the pipe deflection and earth pressure was not measured or evaluated. We therefore approximated the time-dependent soil strains by densifying the soil in our FE model. The densification was intended to simulate the soil strain that occurred during Step 5 of the test procedure, soil wetting (see Section 2.2). We thus simulated the densification by imposing a vertical strain in the soil. For all analyses (regardless of initial compaction at 85% or 90% SPD), we increased the soil density to a level consistent with SW95 after the backfill was placed but before the surcharge was applied.

Densification from SW90 to SW95 is equivalent to a unit weight change from 134 to 141 pcf (using SPIDA unit weights), or a vertical strain of 5.1%. Densification from SW85 to SW95 is equivalent to a unit weight change from 126 to 141 pcf (using SPIDA unit weights), or a vertical strain of 11.2%. We selected these densification changes as approximate indications of the effects of time-dependent consolidation of soil. The actual magnitude of time-dependent effects is influenced by many factors, including soil type, backfill depth, and moisture conditions. Quantifying such effects requires controlled density testing over extended time periods which was not included in the scope of the soil box study. We therefore selected these density changes to represent a likely range of field conditions.

3.3 Loading Sequence and Analysis Matrix

The loading sequence includes up to nine steps grouped into five stages, as follows:

Stage 1 - Predeflection:

1. Predeflect pipe horizontally by 4%, consistent with UF testing

Stage 2 - Backfilling:

2. Apply self-weight to the bedding lift
3. Apply self-weight to the predeflected pipe
4. Add first backfill lift to springline and apply self-weight to the new lift
5. Add second backfill lift to top of pipe and apply self-weight to the new lift
6. Add third backfill lift to the top of the soil box and apply self-weight to the new lift

All backfill lifts in the model for a particular run were assigned the same soil type, either SW85 or SW90 based on an assumed compaction level for that run of either 85% SPD or 90% SPD, respectively.

Stage 3 - Densification:

7. Densify all soil to SW95 (see Section 3.2)

Stage 4 - Surcharge:

8. Apply 60 psi surcharge as a uniform pressure on the backfill (Runs 1-21, see Table 8)

Stage 5 - Pipe creep:

9. Reduce pipe elastic modulus to long-term (50 yr.) modulus (Runs 17-20 and 22-25, see Table 8)

We completed analyses including combinations of pipe diameter, material, and densification magnitude. For plastic pipes, we also analyzed the effect of creep by reducing the modulus of the pipe to 50-yr values from AASHTO Table 12.12.3.3-1, with and without applying the 60 psi surcharge. Also, we analyzed a trench box installation for the 36 in. diameter PVC pipe only, discussed further in the results section below. The analyses completed are identified in Table 8, and the most comparable UF test number is also shown:

Table 8 – Matrix of FE analyses completed

Run ID	UF Test No.	Diameter		Pipe Material				Soil Material		Pipe Creep	Trench Box	60 psi Surch.
		24 in.	36 in.	PVC	HDPE	Steel	Alum.	SW85	SW90			
1	—	×		×					×			x
2	3		×	×					×			x
3	7	×			×				×			x
4	1		×		×				×			×
5	8	×				×			×			×
6	4		×			×			×			×
7	—	×					×		×			×
8	6[†]		×				×		×			×
9	—	×		×				×				×
10	3		×	×				×				×
11	7	×			×			×				×
12	1		×		×			×				×
13	8	×				×		×				×
14	4		×			×		×				×

Run ID	UF Test No.	Diameter		Pipe Material				Soil Material		Pipe Creep	Trench Box	60 psi Surch.
		24 in.	36 in.	PVC	HDPE	Steel	Alum.	SW85	SW90			
15	—	×					×	×				×
16	6[†]		×				×	×				×
17	—	×		×					×	×		×
18	3		×	×					×	×		×
19	7	×			×				×	×		×
20	1		×		×				×	×		×
21	—		×	×					×		×	×
22	—	×		×					×	×		
23	3		×	×					×	×		
24	7	×			×				×	×		
25	1		×		×				×	×		

[†]The other pipe during this test was installed with a trench box which may affect the soil box test results.

3.4 Results and Discussion

The pipe deflection calculated for a representative analysis is shown in Figure 6, separated into the four stages of the loading sequence. The results show the initial imposed 4% deflection. During backfill placement very little deflection occurs because the pipe is held at a minimum of 4% deflection, and therefore not until the final backfill lift is applied does the weight of soil increase the pipe deflection beyond the initial 4%. The deflection due to densification from SW90 to SW95 is on the same order as the deflection for the 60 psi surcharge; during both stages the deflection is approximately 1.5-2%. Extensive plastic deflection occurred in the soil during densification for all runs.

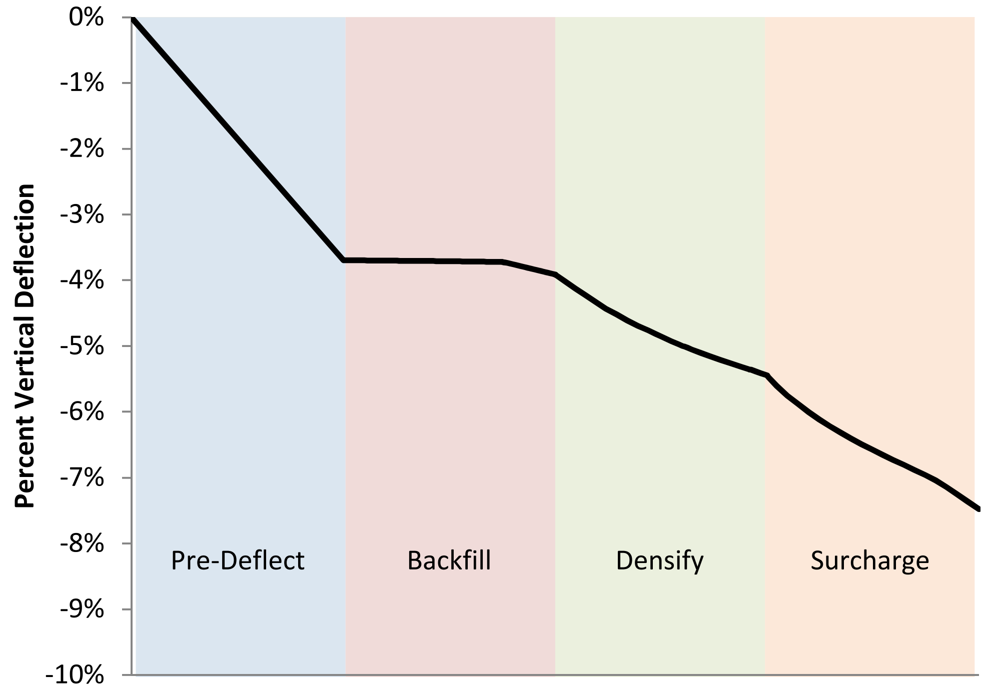

Figure 6 – Typical vertical deflection over the four stages of the analysis

Verification

To validate the model, we compared the portion of the deflection that occurs from the 60 psi surcharge with upper and lower bound deflections calculated according to AASHTO §12.12.2.2 pipe deflection equation (Equation 12.12.2.2-2), shown in Figure 7. The upper and lower bound deflections reflect values of 0.11 and 0.083 for the Spangler bedding coefficient K_x, representing a poor (line support) and a good (uniform support) condition, respectively. Deflection lag factor D_L was taken as unity so as not to include any effect of soil consolidation when comparing with deflections calculated from surcharge loads only. The soil-constrained modulus for SW95 was used since the surcharge was applied after densifying the soil to this level. We plotted the pipe deflections against the pipe stiffness (Figure 7), expressed as the calculated value of $EI/(0.149\ R^3)$ where E is the initial elastic pipe modulus, I is the pipe moment of inertia, and R is the pipe wall centroid radius.

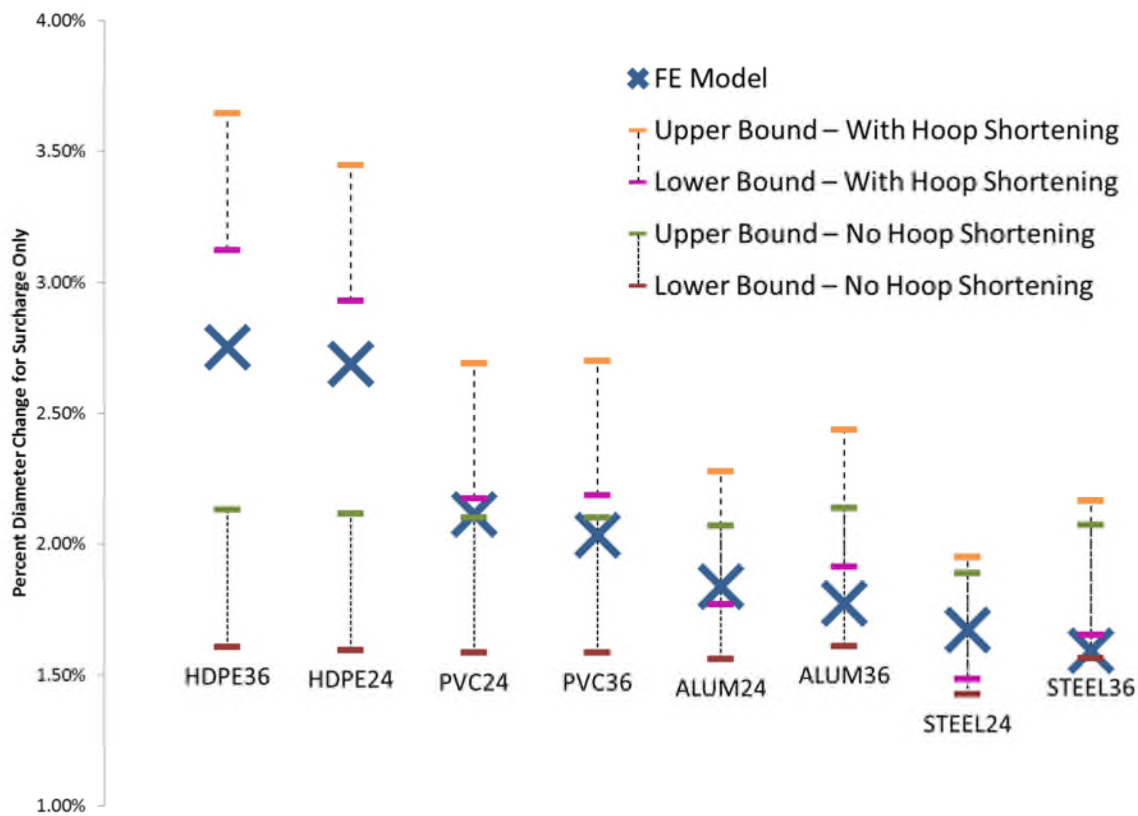

Figure 7 – Pipe deflection from FE model due to 60 psi surcharge plotted against pipe stiffness, and comparison with upper and lower bound hand calculated deflections for analysis runs 1–8

Deflections are calculated with and without including the effect of elastic hoop shortening, which AASHTO approximates by dividing the axial thrust at the pipe springline by the axial stiffness of the pipe wall to determine hoop strain, and multiplying by the pipe diameter to arrive at vertical diameter change. This approach assumes the axial thrust in the pipe wall is uniform around the circumference and will overestimate the diameter change from hoop shortening where the thrust is a maximum at the springline and varies around the circumference of the pipe.

Deflections generally show good agreement with hand calculations. Steel and aluminum pipes have deflections within the hand-calculated bounds, and the hoop shortening has a small effect. PVC and HDPE pipes have deflections between the hand-calculated values with and without hoop shortening. The effect of hoop shortening in plastic pipes is significant. The agreement between the hand-calculated and model deflections gives confidence in the model, and particularly in the soil material model.

Densification

We completed two sets of analyses to study the diameter change that may occur as the backfill consolidates with time and/or due to soil wetting: Runs 1 through 8 used initial material properties consistent with SW90 backfill and Runs 9 through 16 used initial material properties consistent with SW85 backfill. After the backfill was placed but before the surcharge was applied, we densified the soil to a level consistent with SW95 for both sets, as described in Section 3.2. The deflection during densification varied by pipe material and diameter, as shown in Figure 8.

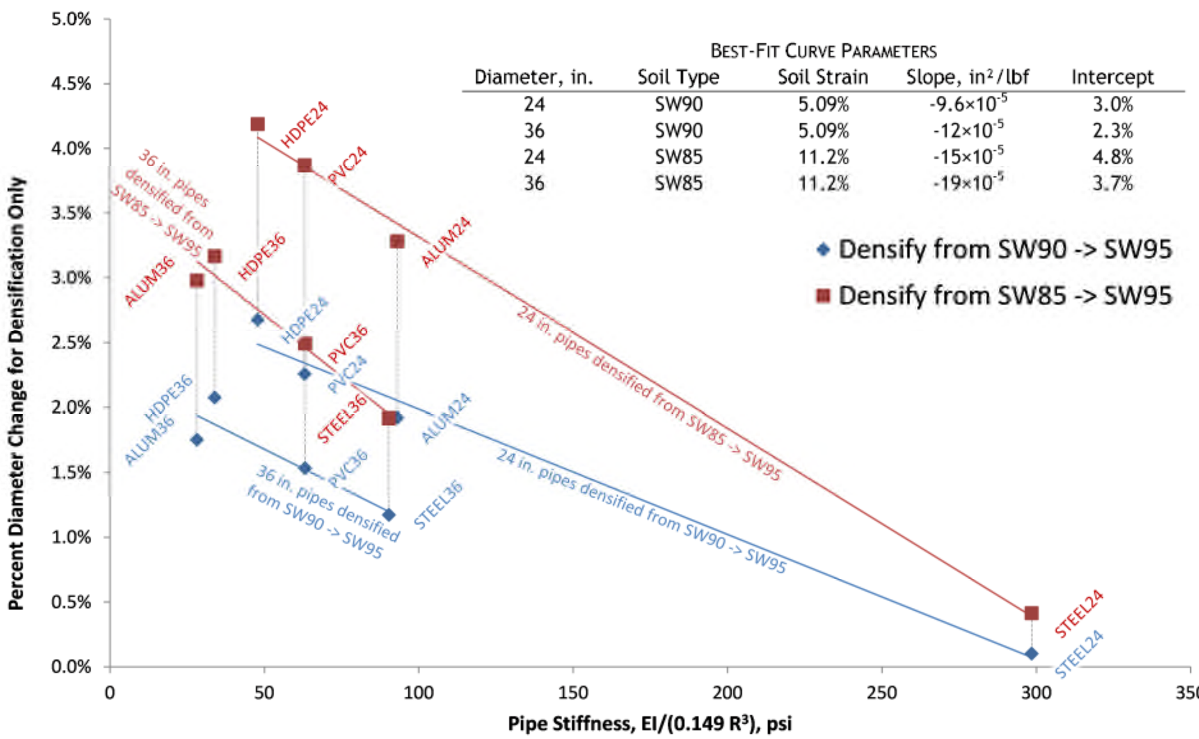

Figure 8 – Pipe deflection during densification to SW95 for Runs 1-16

The plot shows that although the imposed soil strain is the same, the stiffer pipes experience lower deflections than the more flexible ones as a result of soil densification. The stiffer pipes push back against the soil with more force when deformed and cause high shear stresses between the soil directly above the pipe and the soil next to the pipe. Shearing planes initiate in the soil mass as the pipe resists the imposed soil strain, and the pipe overall has lower deflection.

The data show a roughly linear relationship between pipe stiffness and deflection during densification within a given pipe diameter. Slopes are similar and range between -9.6×10^{-5} in.^{2}/lbf and -19×10^{-5} in.^{2}/lbf. The blue and red curves show the regression lines for data from the four runs with the same diameter and soil. Vertical black lines link matching runs with the same pipe stiffness for SW85 and SW90 soils. With additional analyses for multiple pipe diameters and verification with testing it may be possible to derive a relationship between pipe stiffness, level of densification, and pipe diameter to calculate the deflection due to post-installation consolidation, as modeled in this study. The slope and intercept for a given pipe diameter and extent of densification are shown on the figure.

Behavior of Pipe-Soil System from Surcharge Load

Following densification, a 60 psi surcharge was applied. The pipe deflection relative to the condition at the start of surcharge application is shown on Figure 9. The plot shows the mean plus and minus one standard deviation for the two sets of eight runs each starting from SW85 and SW90 soils.

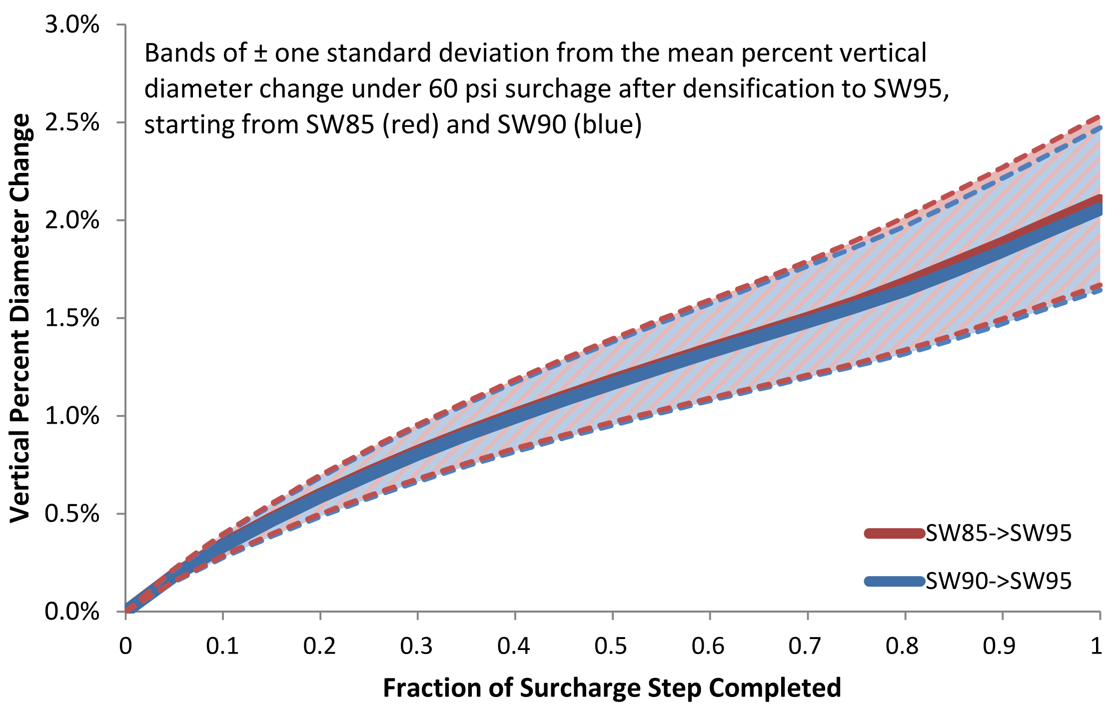

Figure 9 – Pipe deflection during surcharge only, calculated relative to the deformed shape after densification, Runs 1-16

The data show little difference in the pipe deflection when starting from either SW85 or SW90 soils, indicating that after densification the two material models are comparable, as they should be. Some differences may arise for example in the specific pressure distribution in the soil, upon which the soil modulus depends, leading to different effective soil stiffness and therefore somewhat different deflections.

Creep in Pipe Wall

Analysis Runs 17–20 evaluated the effect of creep after densification from SW90 to SW95 and surcharge for the PVC and HDPE pipes with 24 and 36 in. diameters. Creep was modeled by reducing the elastic modulus of the pipe wall from short-term to 50-year properties published in AASHTO Table 12.12.3.3-1. Accordingly, the 50-year modulus for PVC is taken as 35% of the short-term modulus and the 50-year modulus for HDPE is taken as 20% of the short-term modulus.

Table 9 – Deflection from 60 psi surcharge and creep

Material	Diameter in.	Vertical %-Deflection			Ratio of Long-term to Short-term:	
		Surcharge	Creep	Total	Deflection (Total/Surcharge)	Modulus (E_l/E_i)
PVC	24	2.1%	0.7%	2.8%	1.32	0.35
	36	2.0%	0.7%	2.7%	1.33	
HDPE	24	2.7%	1.1%	3.8%	1.41	0.20
	36	2.8%	1.1%	3.9%	1.41	

Table 9 shows the pipe deflections from surcharge and from creep. The models showed that for PVC pipes the additional deflection from creep was approximately 33% of the deflection from the 60 psi surcharge which was calculated from the configuration after densification. For HDPE pipes the additional deflection from creep was approximately 41% of the deflection from the 60 psi surcharge. As expected, the deflections from creep were higher for HDPE than for PVC, but load transfer from the pipe to the soil during creep results in a deflection increase less than the ratio of short-term to 50-yr moduli.

Additional analyses (Runs 22–25) where pipes were softened to long-term elastic moduli after backfill to 5 ft and densification only, but no surcharge found that creep in pipes is responsible for an additional 0.29% and 0.25% vertical deflection in 24 and 36 in. diameter PVC pipes, and an additional 0.53% and 0.45% vertical deflection in 24 and 36 in. diameter HDPE pipes, respectively. The 5 ft of cover is typical of pipe installations in the State of Florida and these values are more representative of creep deflections to be expected over a 50 yr service life.

Summary of Combined Deflections

The deflections from our analyses for the typical installation under 5 ft of cover are shown in Table 10 for each stage of the loading considered:

Table 10 – Percent deflections for typical installation with 5 ft of cover

	PVC		HDPE		Steel		Aluminum	
	24 in.	36 in.	24 in.	36 in.	24 in.	36 in.	24 in.	36 in.
Backfill	4.0	3.9	4.0	4.0	3.8	3.8	3.9	3.9
Densify†	2.3	1.5	2.7	2.1	0.1	1.2	1.9	1.8
Creep	0.3	0.2	0.5	0.4	—	—	—	—
Total	6.6	5.6	7.2	6.5	3.9	5.0	5.8	5.7

†Densify from SW90 to SW95

Table 11 below shows the vertical deflection at each loading stage, as well as the total deflection for all runs that did not include the effects of a trench box (Runs 1–20). The table is sorted within each group of runs from highest to lowest total percent deflection.

Table 11 – Incremental and total deflection for analysis runs 1–25 (omit run 21 with trench box)

	Pipe Parameters				Incremental Vertical %-Deflection						Total Vertical %-Deflection		
	Run ID	Material	Diameter, in.	Pipe Stiffness $EI/0.149\,R^3$, psi	Predeflection	Backfill	Densify SW90->SW95	Densify SW85->SW95	Surcharge	Creep in Pipe Wall	Densify SW90->SW95	Densify SW85->SW95	Densify SW90->SW95 and Creep in Pipe Wall
Start from SW90	3	HDPE	24	48	3.6	0.4	2.7	—	2.7	—	9.4	—	—
	4	HDPE	36	34	3.6	0.4	2.1	—	2.8	—	8.8	—	—
	1	PVC	24	63	3.7	0.3	2.3	—	2.1	—	8.3	—	—
	7	Alum.	24	93	3.7	0.2	1.9	—	1.8	—	7.7	—	—
	2	PVC	36	63	3.7	0.2	1.5	—	2.0	—	7.5	—	—
	8	Alum.	36	28	3.7	0.2	1.8	—	1.8	—	7.5	—	—
	6	Steel	36	90	3.7	0.1	1.2	—	1.6	—	6.6	—	—
	5	Steel	24	298	3.7	0.1	0.1	—	1.7	—	5.6	—	—
Start from SW85	11	HDPE	24	48	3.6	0.7	—	4.2	2.7	—	—	11.3	—
	12	HDPE	36	34	3.6	0.7	—	3.2	2.8	—	—	10.3	—
	9	PVC	24	63	3.7	0.4	—	3.9	2.2	—	—	10.1	—
	15	Alum.	24	93	3.7	0.2	—	3.3	1.9	—	—	9.1	—
	16	Alum.	36	28	3.7	0.5	—	3.0	1.9	—	—	9.1	—
	10	PVC	36	63	3.7	0.3	—	2.5	2.1	—	—	8.6	—
	14	Steel	36	90	3.7	0.2	—	1.9	1.6	—	—	7.5	—
	13	Steel	24	298	3.7	0.1	—	0.4	1.6	—	—	5.8	—

	Pipe Parameters				Incremental Vertical %-Deflection						Total Vertical %-Deflection		
Creep	19	HDPE	24	48	3.6	0.4	2.7	—	2.7	1.1	—	—	10.5
	20	HDPE	36	34	3.6	0.4	2.1	—	2.8	1.1	—	—	10.0
	17	PVC	24	63	3.7	0.3	2.3	—	2.1	0.7	—	—	9.0
	18	PVC	36	63	3.7	0.2	1.5	—	2.0	0.7	—	—	8.2
Creep no surch.	24	HDPE	24	48	3.6	0.4	2.7	—	—	0.5	—	—	7.3
	25	HDPE	36	34	3.6	0.4	2.1	—	—	0.4	—	—	6.5
	22	PVC	24	63	3.7	0.3	2.3	—	—	0.3	—	—	6.5
	23	PVC	36	63	3.7	0.2	1.5	—	—	0.2	—	—	5.7

Trench Box

Analysis Run 21 evaluated the potential impact of a trench box installation where the trench box is removed by pulling it vertically out of the soil after backfilling is completed, leaving disturbed regions of low-density soil. This approach was developed to simulate the UF testing. To represent the low compaction in the backfill after pulling the trench box, we defined a column of soil with very low unit weight centered on the void left by removing the trench box. The trench box walls in the UF tests were 8 in. thick.

We assumed that the void left by the 8 in. thick trench box walls would be filled by the adjacent soil, leaving a region of low-density soil wider than the trench box walls. We calculated the width of the column of soft soil so that the weight of soil occupying the soil column when the trench box walls were removed had a compaction equivalent to SW60 – resulting in a width of approximately 25 in. We selected a compaction equivalent to SW60 for the disturbed region, considering the minimum compaction possible for SW material placed without compaction. The soft soil column was densified by the same amount as the rest of the backfill. That is, a vertical strain of 5.1% was imposed on all soil.

Comparison of the trench box analysis to the equivalent analysis without a trench box (Run 2) showed pipe deflections during densification were somewhat lower, and deflections from the 60 psi surcharge and total deflections were notably higher, as shown in Table 12. The total deflection listed includes backfill, trench box removal, densification, and surcharge, but does not include the initial 4% predeflection.

Table 12 – Pipe deflection with and without the effects of a trench box removed after backfilling

Stage	Vertical %-Deflection		Ratio of Run 21 to Run 2
	No trench box (Run 2)	Trench box (Run 21)	
Densification	1.5%	1.2%	0.80
Surcharge	2.0%	3.5%	1.71
Total	3.8%	4.8%	1.27

The lower deflections during densification are likely due to deformation of the soft soil column representing the trench box removal. The soft soil column deforms to accommodate the stiffer soil as it moves around the pipe, rather than a more uniformly stiff soil deforming and imposing deflections on the pipe. The higher deflection from surcharge loads occurs because again the soft soil column reduces the stiffness of the soil mass and therefore reduces the support it provides the pipe under the surcharge load, leading to higher deflections. The total deflection shown omits the initial 4% deflection imposed before the pipe was placed and backfilled; in all, deflections were 1.27 times higher for the trench box condition analyzed. The results of the trench box analysis demonstrates analytically what is well known in practice – that improper use of trench boxes has a significant negative impact on pipe performance and could increase the post construction deflection change beyond that predicted here.

4. EVALUATION OF POTENTIAL INCREASE IN DEFLECTION AFTER CONSTRUCTION

FDOT wishes to estimate potential increases in vertical pipe deflection after the completion of construction. The testing and analyses presented were conducted to provide input to this decision. We present here our evaluation of such deflection changes for an installation with a final depth of fill of approximately 5 ft, a typical Florida condition.

Construction

The testing and thus the analyses assumed a predeflection of about 4% for all pipes to consider the possible effects of high initial deflection during the backfill process. This predeflection would not be present in an actual installation. An investigation by McGrath, et al., 1999, into actual deflections during backfill placement and compaction demonstrated that with proper construction practices, the vertical diameter should increase during compaction of backfill at the sides of the pipe and then decrease slightly as the first few feet of backfill are placed above the pipe. The magnitude of upward deflection during compaction of sidefill varies with the type of backfill and amount of compactive energy applied, but could vary from 0% to 1.5% for pipes in the size range considered in this study. Under conditions of no sidefill compaction the net downward deflection at about three feet of fill was always less than 2%. However, experience with contractor field installations does not always produce the same results as research projects do. The reasons why are not clear, except that the economic pressures on contractors to place pipe and compact backfill quickly leads them to attempt various methods to speed the process, many of which are not consistent with good practice. Allowing for this reality suggests that pipe could be deflected during backfill placement and compaction up to 2% vertically downward in typical Florida installations of about 5 ft depth provided there is some attempt to follow the FDOT installation guidelines.

The laboratory testing and analysis conducted in this study both show that improper use of trench boxes can result in unacceptably high deflections. Any construction using trench boxes should be monitored for proper use. Whenever possible, trench boxes should be kept above the top of the pipe zone to avoid disturbing sidefill during box removal. See ASTM D2321 for recommendations on proper use of trench boxes.

Increases in Deflection after Construction

After construction is complete, deflections may continue to increase due to changes in the soil mass around the pipe, lumped into the process called densification in this report, and due to creep in the pipe material, which can be significant in plastic pipes.

Densification – This study investigated the effect of densification by introducing a vertical soil strain consistent with increasing the backfill density from 85% or 90% to 95% of maximum standard Proctor density. Since FDOT specifies 95% compaction for backfill around pipes, the condition of 85% initial compaction would represent a poor construction while the condition of 90% might represent a more typical condition of modest compliance.

When this densification takes place is not clear, but based on undocumented observations and Spangler's (1941) initial study of flexible pipe deflection, influxes of water due to rain or flooding likely contribute significantly.

If a practical field assumption is to assume that a backfill compaction of 90% of maximum standard Proctor density should be the minimum acceptable condition, then the findings presented in Table 11 indicate the likely increase in deflection due to time related changes in the soil around the pipe. This table indicates that deflection increase due to such effects would vary from about 1% or less for steel pipe, about 2% for aluminum pipe, and up to about 3% for HDPE pipe. Densification from 85% to 95% compaction would result in substantially larger deflection increases, about 2% or less for steel pipe, about 3% for aluminum pipe, and up to 4% or more for HDPE pipes.

Creep – Creep was investigated in this study for the thermoplastic pipes which show significant drops in elastic modulus under continuous loading – 65% for PVC and 80% for HDPE. The analyses (Table 10) suggest that under about 5 ft of fill deflections could increase up to 0.5% over time. As expected, HDPE showed the largest increase since the elastic modulus under sustained loading decreases the most. However, this increase is largely due to increase in hoop thrust and not bending (see Figure 7 for the significant effect of hoop shortening in HDPE relative to the other pipe materials under high surcharge load). Under modest earth loads, hoop thrust is not a significant contributor to limiting deflection or strain conditions in the pipe.

Summary

This analysis suggests the following scenario for flexible pipe deflection during and after backfilling:

- Initial at completion of backfill to 5 ft: 2% max.
- Increase due to densification (90%-95%): up to 3% for HDPE and 1% for steel
- Increase due to densification (85%-95%): 4%+ for HDPE and 2% for steel
- Increase due to material creep: up to 0.5% for HDPE and PVC

Thus, long-term deflections should not exceed 5% if the initial backfill is compacted to at least 90% of maximum standard Proctor density, a condition which does not meet FDOT standards but should result in good long-term pipe performance – i.e., a realistic consideration of construction variability.

The FDOT goal is to measure deflections fairly shortly after installation is completed. The difficulty of setting a deflection limit at this time is the uncertainty of how much of the densification process has occurred. While we like to think it rains all the time in Florida or that all buried pipes are installed in wet conditions, such a generalization cannot be made for all installations. Looking to other materials, fiberglass pipe (AWWA Manual M45) requires pipe deflection to be less than 3.5% after installation to limit long-term deflection to 5%. Using the above summary, this suggests up to 2% at installation, 0.5% creep and perhaps 1% due to partial completion of the densification process. The actual contributions of each deflection component would of course vary for each type of pipe and within each type of pipe depending on material and pipe stiffness. Smaller diameter pipes tend to be stiffer and would deflect less.

Setting a deflection limit of 3.5% for measurement of deflections within 30 days of installation for flexible pipe seems to be a reasonable limit. This deflection limit should identify installations where there have been serious variations from FDOT construction specifications and long-term deflection may result in performance issues. While a 3.5% limit may permit some installations to eventually exceed the long-term limit of 5%, such installations are not likely to result in long-term performance issues, which should not occur until deflections reach 7.5% or more according to current AASHTO design methods.

The best control of construction practices during installation and backfilling of buried pipe is full-time monitoring. This is expensive and ultimately may not be cost effective. The next best control is post-construction inspection including deflection measurement. With the risk of inspection and possible repairs, contractors have a great incentive to improve practices during construction to minimize deflection failures. A proper pipe installation in accordance with FDOT guidelines should not fail the suggested 3.5% limit 30 days after construction. The proposed limit accepts some variability from specified practices, which is realistic, but should identify those installations where deviations have been significant.

5. CONCLUSIONS

FDOT initiated a study to investigate changes in vertical pipe deflection during and after the installation of flexible buried pipe, including HDPE, PVC, steel, and aluminum. The study included laboratory testing at the University of Florida and analytical modeling by SGH. This report presents the results of the SGH analysis. We conclude the following:

- Pipe deflections from the FE model agree well with hand-calculated deflections.
- As installed backfill soils densify due to moisture and time-dependent consolidation, pipe deflections increase. The increase is higher for low stiffness pipe, especially plastic pipe.
- The actual magnitude of time and moisture related soil consolidation is difficult to predict and would require further study that is beyond the scope of this project.
- Creep related deformation of plastic pipe adds to the overall time-dependent deflection, but the increase is minimal.
- The use of a trench box in the embedment zone adds to pipe deflection; although difficult to model, the effect can be significant.

A limit on short-term deflections (within 30 days of installation) of 3.5% relative to the initial inside vertical diameter will provide FDOT with reasonable assurance that long-term deflections will not exceed 5%.

6. RECOMMENDATIONS

Based on this limited study of deflection that occurs during and after backfill, we recommend that FDOT limit the short-term vertical deflection (within 30 days of installation) to 3.5% of the inside pipe diameter prior to installation.

Actual pipe performance during and after installation will vary with backfill type, pipe type, and most especially, construction practices. Thus, this recommendation should be considered an approximation that will identify installations requiring additional monitoring over time.

We recommend that FDOT apply the proposed limit on short-term (within 30 days of installation) deflections on a temporary basis until a more refined value can be supported by actual field data.

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