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ABSTRACT

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The objective of this research work was to study the performance of flowable fill as a backfill material around buried plastic corrugated pipes. The flowable fill materials used in this study contained varying proportions of fly ash, bottom ash, river sand, waste foundry sand, and cement. The relationships between flowability and compressive strength were investigated for different mixtures by using ASTM test methods in order to design suitable mixtures that meet WVDOT materials specifications. The second phase of the research was to design and construct a laboratory-scale pipe testing apparatus. The final phase was to find the pipe-soil interactions under several variable laboratory conditions using the constructed pipe testing apparatus and different backfill materials. These variables included: trench width, pipe diameter, in-situ soil strength, backfill strength, and surcharge loading. The results of these experiments show that the fly ash based flowable fill materials can be successfully used as a backfill around buried pipes even with narrow trench widths.

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Acquiring a graduate degree has been one of the most rewarding experiences of my life. After thousands of hours of work and many long nights trying to accomplish tomorrow's deadline, the end is now in sight. It is tremendously satisfying to work hard at a goal for several years. It is even more satisfying to see all of those goals come to maturity.

Along my journey I have acquired an Associate's Degree, a Bachelor's Degree, and a family. My wife, Catherine, and my son, Lucas, give meaning and purpose to my work. Catherine is my support, my encouragement, and my sanity through all times be they good or bad. My family gives me life-sustaining love and I thank them for all of their support.

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CHAPTER 1
GENERAL INTRODUCTION

1.1 Controlled Low-Strength Material

Controlled low strength material or CLSM, is a common term for the material also known as flowable fill or K-Krete (Bhat and Lovell 1996). K-Krete Inc. was a company formed in the early 70’s as a joint venture between a power plant trying to find more use for its fly ash and a cement company trying to find more use for their cement trucks. K-Krete Inc. specialized in the manufacturing of flowable fills and patented their procedures. By 1974 K-Krete Inc. was nation-wide and in 1977 it was sold (Hitch 1998). Through the 70’s many other companies began to produce similar products but there was very little control of these products.

In the early 80’s some attempts were made to standardize the technologies and testing procedures for flowable fills. Flowable fills do not fit in perfectly with concrete testing standards and they do not fit in perfectly with soil testing standards either. One of the first attempts to establish national standards to control these mixtures was with the formulation of ACI committee 229 on Controlled Low Strength Materials in 1982 (Brewer 1990). The American Concrete Institute has helped to standardize the use of CLSM and since the inception of committee 229 many new ASTM standards have been written that deal solely with CLSM. The following five ASTM standards deal solely with CLSM (ASTM 2000)

- D 4832: ASTM Test Method for Preparation and Testing of Controlled Low Strength Materials (CLSM) Test Cylinders.
- D 5791: ASTM Practice for Sampling Freshly Mixed Controlled Low Strength Material.
- D 6023: ASTM Test Method for Unit Weight, Yield and Air Content (Gravimetric) of Controlled Low Strength Material.
- D 6024: ASTM Test Method for Ball Drop on Controlled Low Strength Material to Determine Suitability for Load Application.
- D 6103: ASTM Test Method for Flow Consistency of Controlled Low Strength Material

CLSM's are characterized by their ability to flow under their own weight, and harden to a strength that is equal to or greater than that of compacted soil. These mixtures typically contain fly ash, cement, and an aggregate. However, many companies and State specifications have provisions for mixtures that do not use any fly ash. These are usually sand-cement mixtures that have faster curing times and are commonly referred to as quick-set flowable fill (Landwermyer and Rice 1999). The ASTM definition of CLSM's does not specify any required ingredients other than water. ASTM defines a CLSM as "A mixture of soil, cementitious materials, water, and sometimes admixtures, that hardens into a material with a higher strength than the soil but less than 1200 psi" (ASTM 2000). Since one of the objectives of the present research project was to establish mix designs that would use the maximum amount of waste material, no mixtures were prepared that did not contain fly ash.

Fly ash represents 50% of the by-product waste stream from coal burning power plants, but only 25% of it is utilized. The other 50% of the coal power waste is bottom ash, which is more heavily utilized and is considered an acceptable material for the aggregate in CLSM's (Monson 1996). Environmentally speaking, CLSM's that contain waste materials are a very beneficial product because it reduces the need for landfilling the waste and reduces the transportation costs of moving the waste to the landfill. Other waste materials that have been included in flowable fill include: crushed glass, chipped tires, crushed concrete aggregates, AMD sludge, and waste foundry sand (Hook 1998). These materials have been used for backfill and pipe trenches for a couple of decades, but since there is a wide variety of mix designs and applications there is still a great need for research (Brinkley 1998).

CLSM's have gained the most popularity in filling trenches, but have also been used for structural fill, thermal fill, slope stabilization and highway subgrades. In trenching, using a flowable mixture that can fill all of the voids from the top of the trench can greatly reduce the number of workers in overhead trenches. These materials harden in less time than it would take to fill the trench in layers and compact each layer, so there is less investment in labor and less inconvenience to travelers (Ramme 1999). Digging much narrower trenches can even further reduce the labor costs, because the trenches do not need to accommodate compaction equipment (Hegerty and Eaton 1998). In addition, these materials exhibit very little settlement, which is a strong benefit when compared to compacted fills that can have destructive settlement problems (Bhat 1998). Flowable fills also exhibit uniform density and strength properties from the top of the trench to the bottom. With all of these considerations in mind, flowable fills are often the most economical way to fill an excavated trench. One study showed that strong CLSM's will reduce the stresses and deflections experienced in pipes, especially those buried at shallow depths (McGrath and Hoopes 1998). The potential savings in labor, the increased safety of the workers, the utilization of waste materials, and the quicker construction times that present less inconvenience to the general public are making this material a popular choice with transportation departments around the country.

These benefits have led many states to consider using flowable fill in many more fill situations than before. The West Virginia Department of Transportation has specifications for three types of CLSM's. Class A and class B mixtures are the typical lower strength materials. Class A has a minimum 28-day compressive strength of 50 psi (345 Pa) and a maximum 28-day compressive strength of 300 psi (2,070 Pa). Class B has a minimum compressive strength of 50 psi (345 Pa) and no upper limit on the compressive strength. Class A and B mixtures would be used in a utility trench that might expect future excavation. The lower strength material will facilitate easy excavation if the utility needs future repairs. Class C CLSM must have a 28-day unconfined compressive strength in excess of 1,000 psi (6,900 Pa). This implies that the upper limit for Class B mixtures is 1,000 psi (6,900 Pa) because if it exceeded this number it would be a Class C mixture. Class C mixtures are likely to be in excess of the ASTM definition that requires a CLSM to be under 1,200 psi (8,270 Pa), but they are very useful in applications that will not need to be excavated in the future. Results from a previous study (Mullarky 1998) showed that the percentage of fly ash and the air content would affect the long-term strength gain of CLSM mixtures. This report indicated that mixtures with high fly ash contents are likely to continue gaining strength well after 28 days.

Current construction methods practiced by the West Virginia Department of Transportation (WVDOT) are described in their construction specification book (WVDOT 2000). It appears that the WVDOT does not use any pipes smaller than 18 inches (45.7 cm) in any new construction (WVDOT 2000). A trench must have at least 1-

diameter on each side of the pipe for an 18 inch (45.7 cm) pipe and the specification continues to only require 18 inches (45.7 cm) on each side of the pipe for pipes up to 54 inches (137 cm) in diameter. All pipes 60 inches (152 cm) and larger in diameter require at least one diameter on each side. So the specifications primarily require a trench to be three times the diameter but pipes between 18 inches (45.7 cm) and 54 inches (137 cm) do not appear to have this requirement. A 54 inch (137 cm) pipe can be laid with a trench width-to-diameter ratio of only 1.67. Of course, this only applies to new construction. Maintenance or rehabilitation projects on existing pipe have different standards. The construction specification also requires that all pipes be trenched. This means that when a fill is being constructed, the soil must first be built up and then trenched to lay the pipe. This insures that there will be enough surrounding soil pressure to protect the pipe.

Flowable fill has some drawbacks that can be easily avoided. If the mixture does not contain enough water all of the voids may not get filled. Moreover, if too much cement is added then future excavations will be very difficult. Pipe floating needs to be prevented by bracing. The mixture can flow into the pipe if any of the joints are not properly seated. In addition to these human errors, there are complaints about the high variability of fly ash sources (ACI 1999). This variability causes the need for constant testing. Subsidence is listed by ACI as a potential problem. Subsidence is caused by a reduction in volume due to the loss of water either to the surrounding soil or as bleed water. An advantage to CLSM is that the subsidence happens early. Typical subsidence is about ¼ inch per foot (2 cm per meter) of depth for high water content mixtures. Some mixtures will also be prone to segregation. It is typically up to the contractor to make sure that the mixtures do not segregate. Another drawback is that the strength tests are usually conducted no less than seven days after the pouring. If a problem in the mixture is not detected until seven days after the construction it will be very hard to correct the mistake. In the case of compacted fill a nuclear density gage can be used to determine whether or not the required densities have been met before proceeding any further (Webb et al. 1998).

Meeting the requirements of the CLSM is solely up to the contractor. Most contractors work from experience to estimate the strength they will get from different combinations of ingredients. ACI committee 229 on Controlled Low Strength Materials does not give any mix design information but does say that any samples should be tested to see what effects the ingredients have (ACI 1999). Many transportation departments have very strict mix design criteria and a lot of research has been done on mix designs. The chemical properties of the aggregates can have a significant affect on the strength while the gradation of the aggregate will affect both flowability and strength. It has also been shown (Brewer 1993) that the initial water content will affect the strength. The strength of a mixture can be adjusted by adjusting the water content. More water can be added to reduce the strength or the mixture can be a little dry to increas the strength

Because of the popularity of flowable fill materials with transportation departments it has also been considered for many other applications as well. For instance, since the material can easily be pumped into tunnels and cavities, it has been used for filling abandoned tunnels and sewers. Flowable fill has also been used for backfilling abandoned mines (Gray 1998). Flowable fill is useful when space is limited, and has been used for fill around embutments for bridges both in construction and maintenance (Hook 1998). CLSM can be used in subgrades and was found useful in approach embankments for bridges (Snethen and Benson 1998). CLSM has been used to fill the voids on The Boston Harbor Tunnel Project (The Big Dig) (Sullivan 1999). There are innumerable instances when limited space or accessibility can make flowable fill the best option.

1.2 BURIED PIPES

This research deals with the interaction between flowable fill and buried pipe, and some background information on the design and testing of buried flexible pipes is given in this section. One of the first recorded uses of flowable fill was the use of soil-cement slurry for pipe bedding in the early 1960's. The project was a 296 mile (410 km) long pipe on the Canadian River Project in Texas (Hitch 1998). The use of soil cement slurries by the Bureau of Reclamation has been sporadic even though this early project worked quite well. However, the work done in Texas led to the first draft of ASTM Test Method for Preparation and Testing of Soil-Cement Slurry Test Cylinders (D 4832). In 1997 a slightly altered form of this test method was officially adopted and the title "Soil Cement Slurry" was replaced by "Controlled Low Strength Material." From the beginning flowable fill has been recognized for its beneficial use with buried pipes.

This research deals with buried flexible pipes, which have very different properties and design procedures than stiff pipes. Flexible pipes deflect under the vertical loading which causes a decrease in the vertical stress directly above the pipe and an increase in the horizontal pressures (Journal 1991). This vertical stress decrease, known as arching, is caused because the deflection in the pipe allows the soil mass above the pipe to slip downwards. As the fill material slips, the friction forces between the in-situ soil and the fill increase and reduce some of the vertical stress on the pipe. The horizontal stress increases because the pipe is pushing outward towards the soil. A proper flexible pipe design should take into consideration strength characteristics of the in-situ soil and fill material, and the bending characteristics of the pipe (Daniels 1990).

The equation used for computing horizontal pipe deflections differs slightly in different texts, but they all have the following form (Bulson 1985).

$$\Delta X = \frac{loading\ \ parameter}{ring\ \ stiffness\ factor + \ soil\ \ stiffness\ \ factor} \qquad \text{(Eq. 1.1)}$$

This is known as Spangler 's Iowa equation (Bulson 1985). In addition to this equation, it is known from the fundamental static equations that govern deflections in a ring that the magnitude of the horizontal deflections is 91.3% of the vertical deflections (Bulson 1985). However, this ratio between vertical and horizontal displacements was derived from a simple example using an unsupported ring with diametrically opposed forces. In a buried pipe, there is support around the whole structure that will vary in the vertical and horizontal directions and because of variability in the compacted fill, the real deflected shape of a buried pipe will not be represented by such simple static relationships.

Soil stiffness factors are highly variable and will not only depend on the type of soil but also on the level of compaction and the quality of compaction. The highest values for a densely compacted sand or gravel will be about 100 times higher than the value for a fine grained soil of medium plasticity (Howard 1996). One of the benefits to using flowable fill is the uniform nature of the fill. It is not as variable as compacted fills because it’s strength is controlled, it is more homogeneous than many fills, and it will leave no voids under or around the buried pipe.

The ring stiffness factor is related to the bending characteristics of the pipe. Many commercial flexible pipes are available in a wide range of strengths. The selection of a pipe will depend on the anticipated loading conditions and the soil stiffness. It can be seen from Equation 1.1 that if the loading conditions and the soil stiffness are known, a value for the maximum allowable deflection can be assumed to solve for the required pipe stiffness. A contractor then only needs to pick a pipe that meets the calculated criteria. By using the Iowa equation, it has been shown that the deflections of a pipe will be reduced by ½ if you use CLSM rather than conventional backfill (Brewer 1990). In this reference it has been assumed that all material, fill, and soil properties are the same. It has been shown that CLSM backfill can affect every stage of flexible pipe design, from vertical load estimation to selection of the pipe size and trench width requirements (Brewer 1993).

Another phenomena found in HDPE flexible pipes is stress relaxation. If a certain load is applied to a pipe to force it to 5 % deflection instantaneously, and the load is measured with time, over time the load required to hold the pipe at 5 % deflection will reduce. The pipe gets set at its new shape and has a reduction in internal stresses (Daniels 1990). Stress relaxation is only found in pipes that are held at a certain deformation for a long time. All of the tests conducted in this research deal with instantaneous loading so the details of this theory are not discussed in this report.

Finite Element Analysis Method (FEM) is also a valuable tool for measuring buried pipe deflections (Zaman and Laguros 1990). The Iowa equation is good for simple geometries but it does not give a complete picture of the pipe-soil interactions. FEM has the power to find the deflections around the whole perimeter of the pipe, as well as the interactions between the soil interfaces. Using FEM the entire geometry is discretized into interconnected elements. By knowing some of the values in the system and knowing the boundary conditions in the system, the unknown displacements and stresses can be determined. Several programs exist for designing pipes using finite element method. Culvert Analysis and Design (CANDE) is a commonly used program for such applications (Katona et al. 1976)

In the following figure (Figure 1.1), a schematic of a buried pipe is shown. The geometry for a typical trench filled with compacted earth, and the geometry that might be used when using CLSM as the backfill are both shown in Figure 1.1. Trenches filled with

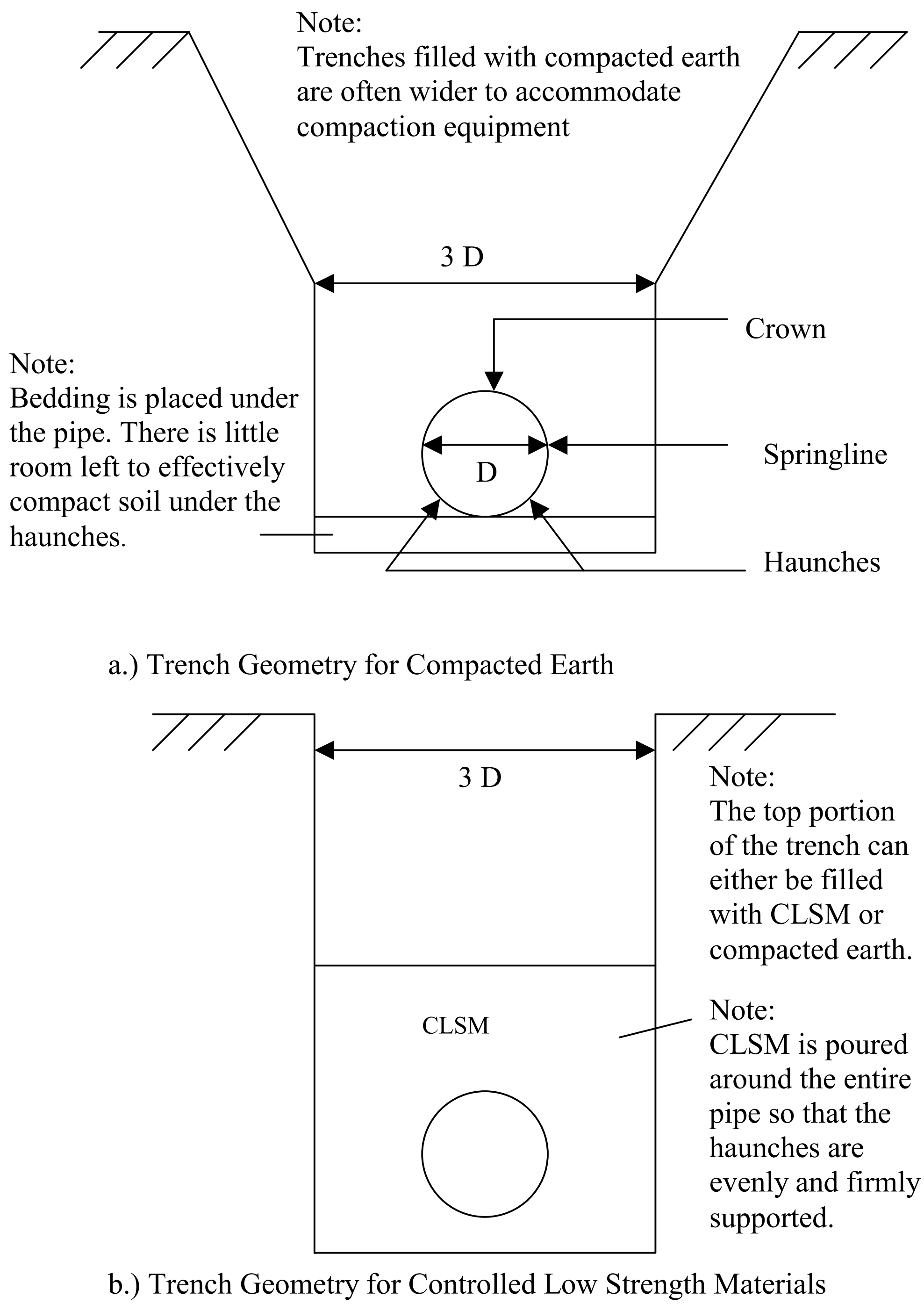


Figure 1.1: Schematic Diagram of a Buried Pipe

CLSM can be narrower because CLSM offers better support under the haunches of the pipe (Brewer 1993).

1.3 Research Objectives

Specific objectives of the research are listed below.

1.) Determine strength and flowability information for mixtures that will incorporate at least two different aggregate types. The percentage of aggregate should be varied such that data can be useful to relate the strength and flowability to the percentage of aggregates in the mixture.
2.) Develop a mix design procedure based on this information.
3.) Choose two mixtures for pipe testing. One high strength mixture meeting the requirements for a WVDOT class C CLSM mixture and one low strength mixture meeting the requirements for a WVDOT class B mixture.
4.) Design and construct a testing apparatus for laboratory testing of 6 inch (15 cm) and 8 inch (20 cm) corrugated plastic pipes. The device should be sufficiently instrumented to measure the external applied loads, deflections, and soil stresses.
5.) Conduct laboratory pipe tests with the constructed apparatus to find the relationships between trench width, pipe size, deflections and soil stresses for different backfill strength and in-situ soil strength.
6.) Analysis of laboratory data and preparation of the final report.

The details of these objectives are presented in the following chapters of this report.

CHAPTER 2
MIX DESIGN PROCEDURE

Information on all of the materials used in this research is presented in this chapter. The research that was performed in order to develop a mix design procedure is also discussed. This research includes flowability testing and compressive strength testing. All of these tests were conducted according to ASTM standards. Tables and graphs are presented to support the information in the body of this chapter.

2.1 MATERIAL INFORMATION

Controlled low strength materials consist of aggregates and cement and they may contain some fly ash and some admixtures to improve flowability or set time. Fly ash can vary widely from one geographical region to another so it is important to characterize the fly ash. The West Virginia Department of Transportation (WVDOT) gives the following specifications for an approved class C fly ash:

1) Amount retained on # 325 sieve must be less than 34%
2) Loss on ignition must be less than 12%
3) (SiO_2 + AlO_3 + Fe_2O_3) must be greater than 50%

The fly ash used for the experiments outlined in this report was obtained from Morgantown Energy Associates (MEA) in Morgantown, West Virginia. Based on unpublished data (personal communication with MEA 2002), it was found that the average loss on ignition is 2 % for the fly ash and 1 % for the bottom ash. This reference also reported the percentage of (SiO_2 + AlO_3 + Fe_2O_3) to be 63 % and the Calcium Oxide (CaO) content was reported as 18.2% to 19.0%. A sieve analysis showed that 85 % passed the #325 sieve. No gradation curve can be formed for the fly ash because it did not settle independently in a hydrometer analysis. In addition to this information, the dry unit weight, specific gravity, and water content were also measured. They were found as follows:

Dry unit weight, γ_{dry} = 42.9 lb/ft^3 (6.7 kN/m^3)

Specific gravity, G = 2.78

Water content, w = 0.13 %

These numbers are all in the expected ranges. The water content is so low because the material is poured directly into a large steel drum from the furnace, which is at a temperature of 400^0C. The containers are then sealed and stored in a dry location until the fly ash is used. An attempt was made to measure the liquid and plastic limits but this material is non-plastic and the results were not conclusive. Neither ASTM nor AASHTO have recommendations for these tests.

Other research has shown that fluidized bed fly ash is higher in sulfur than a typical fly ash (Ziemkiewicz and Black 2000). MEA is authorized to burn lower quality coal than most other plants in West Virginia because they use fluidized bed combustion (Howard 1983). There are only two fluidized bed combustion plants in West Virginia. The other constituent in fly ash that is of concern is Calcium Oxide (CaO). This is the main cementing ingredient in Portland cement (Hibbeler 1997). Fly ash naturally contains some CaO and the percentage of this compound will greatly affect the pozzolanic behavior of the ash. As stated earlier the CaO content in fly ash coming from MEA varies between 18.2% and 19.0%. ASTM reports an expected CaO content as 24 % for a class C fly ash so the values found in the MEA ash is not far below the expected values (Ziemkiewicz and Black 2000).

Several aggregates were chosen for testing. The first is the bottom ash from MEA, the second is Ohio River Sand commercially available in Morgantown, West Virginia, and the third is foundry sand obtained from a West Virginia foundry.

The dry unit weights of the three aggregates are as follows:

Bottom Ash = 82.8 lb/ft^3 (13.0 kN/m^3)

River Sand = 99.2 lb/ft^3 (15.6 kN/m^3)

Foundry Sand = 94.4 lb/ft^3 (14.8 kN/m^3)

The grain size distributions of each material were also measured and the gradation curves of all three materials are shown in Figure 2.1 with respect to the WVDOT aggregate specifications. The bottom ash has an additional regulation that it has a loss on ignition no greater than 12%. The bottom ash used in this study has a loss on ignition of 1 % (personal communication with MEA 2002). The cement used in testing was normal type I Portland Cement.

Morgantown Energy Associates (MEA) does not separate their bottom ash from their fly ash under normal operating procedures. Both materials are dumped onto the same conveyor and then stored in silos in the mixed condition for disposal. This company was extremely helpful by filling the barrels of fly ash and bottom ash directly from the supply lines leaving the furnace. The bottom ash had no measurable water content because it was put into a container directly after leaving the furnace. The bottom ash also contains CaO so it also contributes to the pozzolanic activity of the CLSM mixture.

The Ohio River sand used in CLSM mixtures is also used for the loose in-situ soil in the pipe testing apparatus. It should be noted that only the portion of sand passing the 3/8 inch (9.5 mm) sieve was used in CLSM mixtures. This sieve does not affect the gradation of the sand. It serves to remove any leaves, litter, or gravel that inadvertently fall into the sand.

The foundry sand is a fine uniformly graded material. The foundry sand was only used for flowability and strength testing because it was in limited supply. There was not enough material for use in the pipe testing. Additional research on flowable fill mixtures with foundry sands can be found elsewhere. Information on designs utilizing different proportions of waste foundry sands can be found in the literature (Bhat and Lovell 1996; Bhat and Lovell 1998). Other literature is also available that describes adding other waste materials, such as phosphogypsum and acid mine drainage (AMD) sludge, to CLSM mixtures (Landham et al. 1996; Monson 1997).

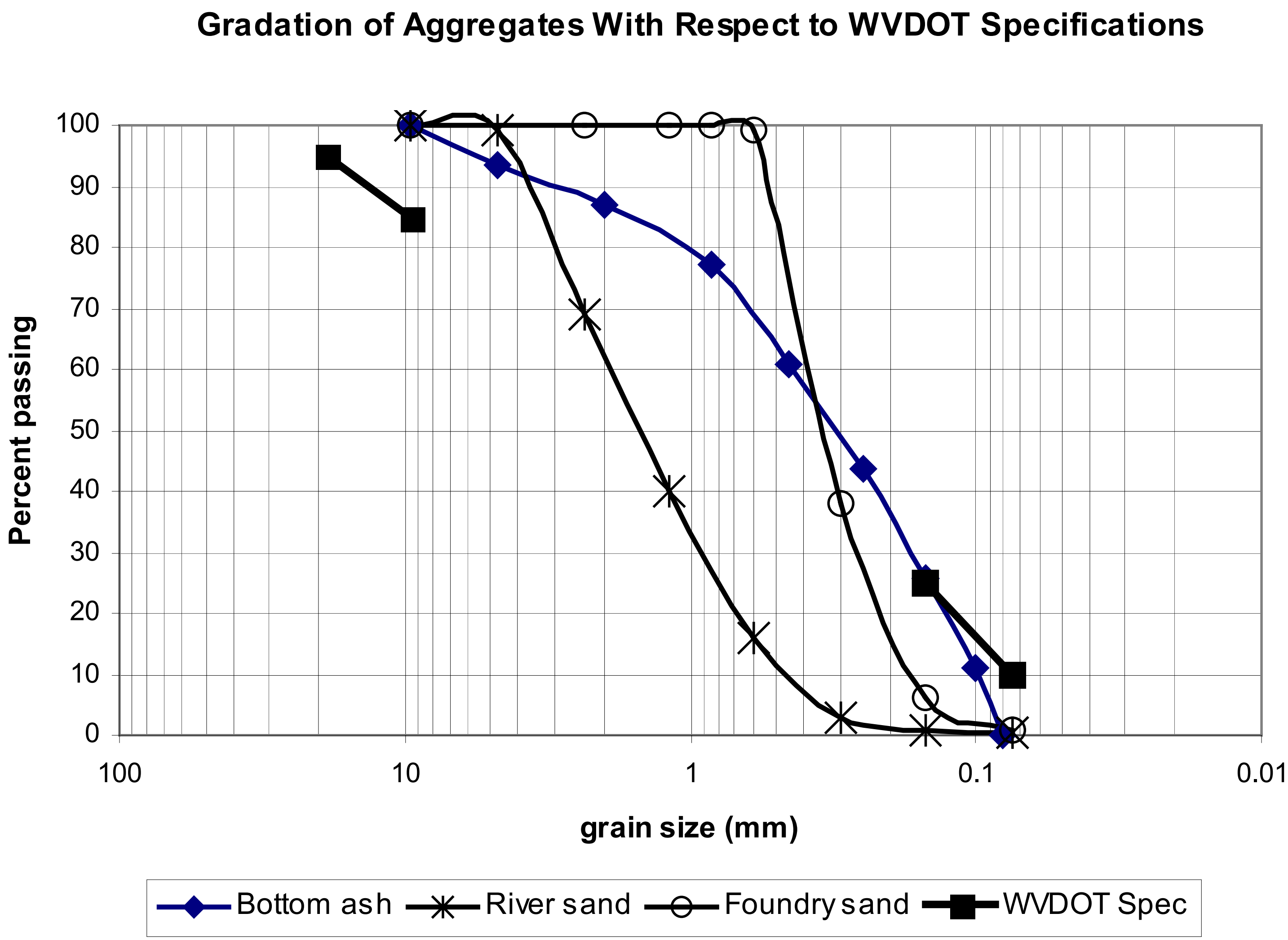


Figure 2.1 Gradation Curves For Aggregates in Flowable Fill

2.2 Flowability Testing

Details of flowability testing of CLSM are given in ASTM specification D6103 (ASTM 2000). This test was adopted as an official ASTM standard in 1997. It gained provisional status in 1995. Prior to the adoption of this standard, many companies judged flowability by eye and relied on the experience of their workers to produce a quality material. Some companies attempted to apply slump tests to flowable fills but for very flowable mixes the slump test does not give meaningful results.

The flowability test consists of a non-porous surface and a 3 inch (7.6 cm) diameter by 6 inch (15.2 cm) high cylinder. The cylinder is filled with the CLSM and lifted in a non-twisting uniform motion so that the CLSM will spill onto the non-porous surface. The diameter of the spill is measured in at least two perpendicular directions and their average value is recorded as the spread. ASTM recommends that the spread be between 8 inch (20 cm) and 12 inch (30 cm) for an adequate CLSM mixture. WVDOT only requires that the material have a spread of 6 inch (15 cm), but it gives the additional requirement that the material must be able to fill all voids without vibration or roding. ASTM also requires that the cylinder be lifted in a straight non-twisting and non-jarring motion. To ensure the accuracy of these tests, a testing frame was built. The frame provides a rigid and level glass spill surface with two perpendicular axis for measuring the spill diameter. The constructed frame also has a handle for lifting that insures a strait and non-jarring lift. The test stand is illustrated in Figure 2.2.

Table 2.1 shows all of the materials that were tested for flowability. All of the values in this table show the percentage by weight of the various materials. This set of experiments generated data on the flow characteristics of each aggregate as well as the flow characteristics of any admixtures. The tests clearly showed that replacing up to 3 % of the fly ash with cement had almost no effect on the flow characteristics. The testing also showed that bentonite decreased the flowability of the material so it was removed from further consideration as an admixture.

Each mix was placed in a bowl and water was gradually added. Four samples were used to find the water content at four different diameters of spread for each mix. It was attempted to find some water contents that would correspond to flow below the ASTM minimum requirement of 8 inch (20 cm) and some water contents that would

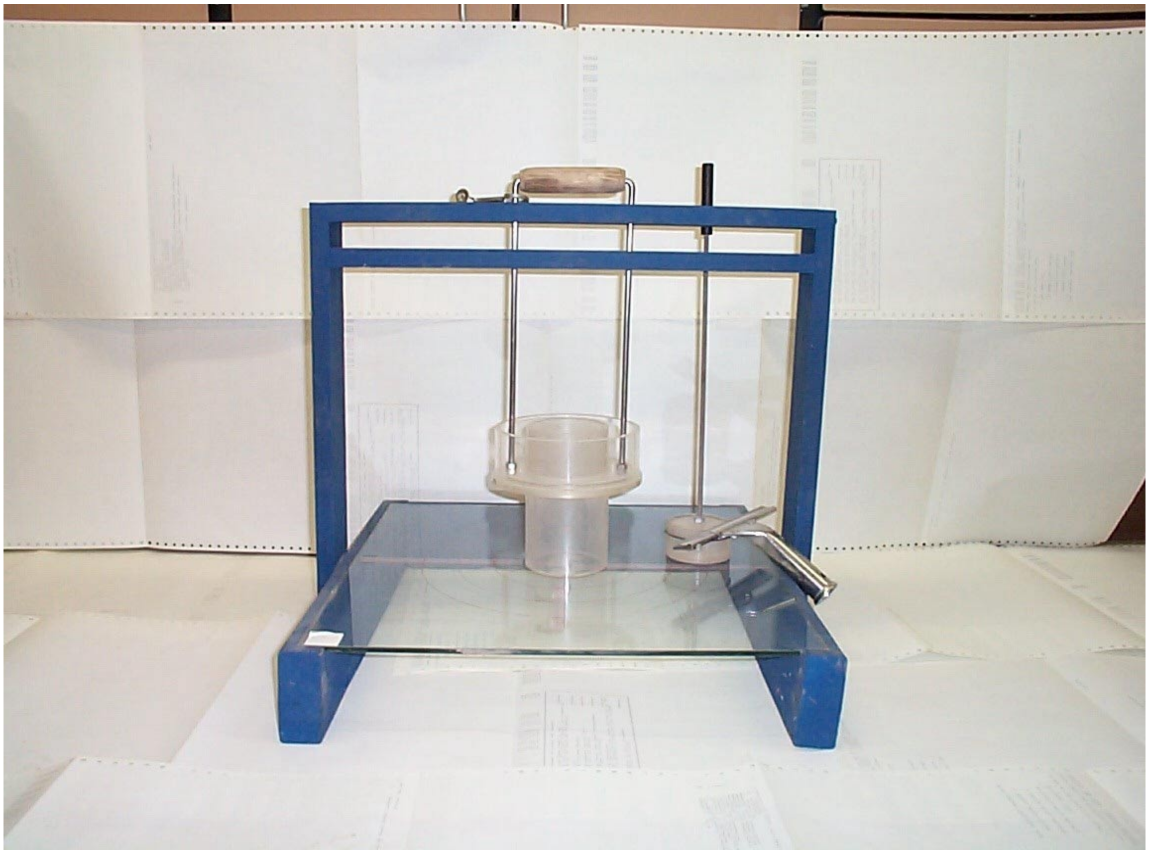


Figure 2.2: Flowability Testing Stand

Table 2.1: Mixtures Tested for Flowability

		Percentage of Dry Weight					
	Mix	Fly ash	Bottom ash	river sand	foundry sand	Bentonite	cement
1	M	100					0
2	M1C	99					1
3	M3C	97					3
4	M6C	94					6
5	M5BA	95	5				0
6	M10BA	90	10				0
7	M25BA	75	25				0
8	M50BA	50	50				0
9	M75BA	25	75				0
10	M90BA	10	90				0
11	M2BA3C	95	2				3
12	M7BA3C	90	7				3
13	M22BA3C	75	22				3
14	M47BA3C	50	47				3
15	M72BA3C	25	72				3
16	M87BA3C	10	87				3
17	M5RS	95		5			0
18	M10RS	90		10			0
19	M25RS	75		25			0
20	M50RS	50		50			0
21	M75RS	25		75			0
22	M90RS	10		90			0
23	M2RS3C	95		2			3
24	M7RS3C	90		7			3
25	M22RS3C	75		22			3
26	M47RS3C	50		47			3
27	M72RS3C	25		72			3
28	M87RS3C	10		87			3
29	M5FS	95			5		0
30	M10FS	90			10		0
31	M25FS	75			25		0
32	M50FS	50			50		0
33	M75FS	25			75		0
34	M90FS	10			90		0
35	M2FS3C	95			2		3
36	M7FS3C	90			7		3
37	M22FS3C	75			22		3
38	M47FS3C	50			47		3
39	M72FS3C	25			72		3
40	M87FS3C	10			87		3
41	M1B	99				1	0
42	M3B	97				3	0
43	M6B	94				6	0
44	M10B	90				10	0

correspond to flow above this value. The mixture was visually judged as water was added. A flow test was conducted when the mixture looked like it would begin to flow. More water would then be added and the consecutive flow tests were conducted. This is similar to the rules followed when performing liquid limit tests. The mixture started off dry and water was gradually added as consecutive tests were completed so that there was never a need for redrying the mixture.

This information yields graphs that can be used to find the water needed for any given spread. Figure 2.3 shows a sample of one of these graphs. In addition, curves can be generated to show the relationship between aggregate content and water content for constant spread as shown in Figure 2.4. Figure 2.4 shows the water content versus percentage of aggregate for all three aggregate mixtures with and without cement for a spread of 9 inch (23 cm). This spread was chosen because it is between 8 inches (20 cm) and 12 inches (30 cm), as recommended by ASTM (ASTM 2000).

Figure 2.3, Figure 2.4 and Table 2.1 also show the naming convention that was adopted for this research. The first letter reflects where the fly ash came from. All of the mixtures start with the letter M to reflect that they came from MEA. This was used in case it was decided to use fly ash from another location, but in the course of this research only MEA fly ash was used so all mixes will start with the letter M. The next group of numbers and letters describe the percentage of aggregate by dry weight, and the third number and letter combination describe the amount of cement or additives by weight. The labels used are BA for bottom ash, RS for river sand, FS for foundry sand, and C for cement. For instance, the mixture M75BA3C would be 75 % bottom ash, 3 % cement and the remaining 22 % would be MEA fly ash. The mixture M50RS would contain 50 % river sand and the remaining 50% would be MEA fly ash. This naming convention allows one to easily know what the mixture contains without referring back to this section.

The flow testing clearly shows a trend in that more aggregates will decrease the water demand. However, if the mixture has too much aggregate it will not flow under its own weight. The water will immediately segregate and the remaining pile will not spread to 9 inches (23 cm). The bottom ash had less of a problem with water segregation than

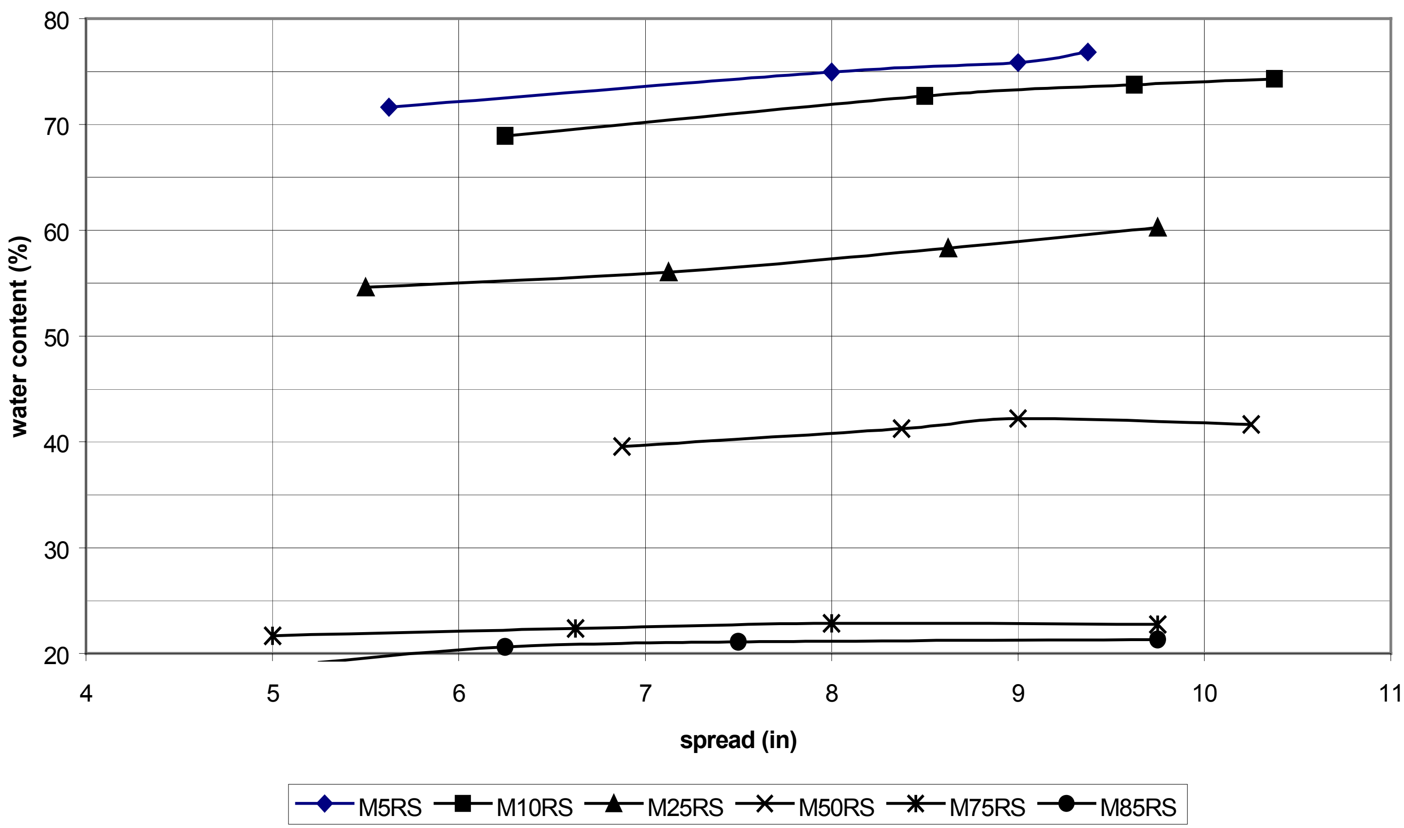


Figure 2.3: Water Content versus Spread for River Sand Mixtures

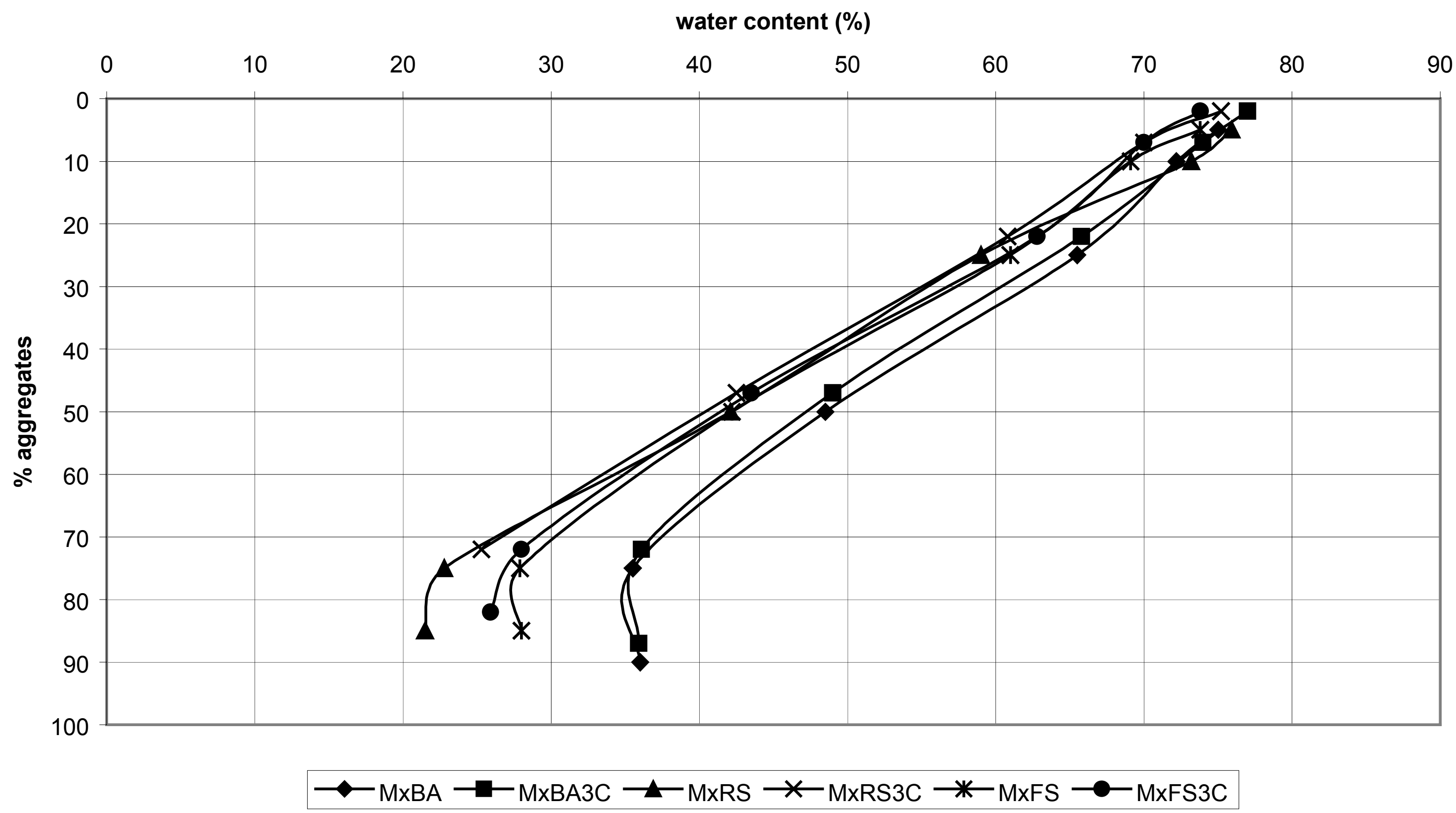


Figure 2.4: Water Content versus Percentage of Aggregate for 9 inch (23 cm) Spread

did the two sands. This is why it was possible to test bottom ash at 90% aggregates while the two sands could only be tested at 85% aggregates. The flow tests also show a trend for the river sand to require less water than the foundry sand. The foundry sand required less water than the bottom ash. This is because of the particle size distribution of the three aggregates. The river sand is less uniform than the foundries sand so the variety of particle sizes takes up more space and leave less room for the water. The foundry sand is very uniform so it will have more interparticle friction at high aggregate contents, and will need slightly more water to separate the particles and encourage flow. The bottom ash has the highest demand because of the structure of the bottom ash particles. Bottom ash particles are more porous, like a sponge. This porous nature makes the bottom ash mixtures require more water than the sands (Vipulanandan et al. 1998).

2.3 Strength Testing

Strength testing for controlled low strength materials is usually done at 28-days. This was the procedure used in earlier days of CLSM when it was lumped in with concrete testing procedures. Concrete mixes have been shown to gain most of their strength at 28-days, but this correlation is not necessarily the same for CLSM. Some quick set CLSM claims to have its full strength in just a couple days, while some other CLSM may have strength gains well past 28-days (Mullarky 1998). Many mixes will meet state requirements for maximum strength at 28-days, but the strength exceeds that requirement several years later (Brinkley 1998). Regardless of the complete suitability of the test standards, the ASTM recommended procedures were followed in this research program.

27 different mixes were originally cast for unconfined compression tests. These tests were done in accordance with ASTM D 4832. Some tests were repeated to insure the accuracy of the information obtained. When it became apparent that none of the original mixtures would qualify for a WVDOT class C CLSM (ultimate 28-day compressive strength above 1,000 psi (6,895 Pa)) 15 additional high strength tests were conducted. All the mixtures are shown in Tables 2.2 and 2.3. Ten cylinders (3 inch (7.6 cm) diameter and 6 inch (15.2 cm) in height) were cast for each of the 42 mixes. Two cylinders were used for 7, 14, and 28-day strengths and the reported strength for each of

Table 2.2 Low Compressive Strength Mixtures.

Mixture	Fly Ash	Bottom Ash	River Sand	Foundry Sand	Cement
M90BA	10 %	90 %			
M83BA	17 %	83 %			
M75BA	25 %	75 %			
M50BA	50 %	50 %			
M25BA	75 %	25 %			
M10BA	90 %	10 %			
M85RS	15 %		85 %		
M80RS	20 %		80 %		
M75RS	25 %		75 %		
M50RS	50 %		50 %		
M25RS	75 %		25 %		
M10RS	90 %		10 %		
M85FS	15 %			85 %	
M80FS	20 %			80 %	
M75FS	25 %			75 %	
M50FS	50 %			50 %	
M25FS	75 %			25 %	
M10FS	90 %			10 %	
M90BA1C	9 %	90 %			1 %
M83BA1C	16 %	83 %			1 %
M75BA1C	24 %	75 %			1 %
M85RS1C	14 %		85 %		1 %
M80RS1C	19 %		80 %		1 %
M75RS1C	24 %		75 %		1 %
M85FS1C	14 %			85 %	1 %
M80FS1C	19 %			80 %	1 %
M75FS1C	24 %			75 %	1 %

Table 2.3 High Compressive Strength Mixtures.

Mixture	Fly Ash	Bottom Ash	River Sand	Foundry Sand	Cement
M40BA2C	58 %	40 %			2 %
M50BA2C	48 %	50 %			2 %
M60BA2C	38 %	60 %			2 %
M70BA2C	28 %	70 %			2 %
M80BA2C	18 %	80 %			2 %
M40BA3C	57 %	40 %			3 %
M50BA3C	47 %	50 %			3 %
M60BA3C	37 %	60 %			3 %
M70BA3C	27 %	70 %			3 %
M80BA3C	17 %	80 %			3 %
M40BA4C	56 %	40 %			4 %
M50BA4C	46 %	50 %			4 %
M60BA4C	36 %	60 %			4 %
M70BA4C	26 %	70 %			4 %
M80BA4C	16 %	80 %			4 %

these tests is the average of these two compressive strengths. More cylinders were made than required so that defective cylinders, such as the ones that break during the process of removal from the plastic cylindrical molds, do not have to be used.

The cylinders were prepared by measuring all of the dry ingredients into a container and gradually adding the required amount of water to achieve 9 inch (23 cm) spread. The mixture would be mixed with a trowel until it was completely mixed and then the flowability would be measured to verify that it had 9 inch (23 cm) spread. Pouring the mixture with a large ladle, with no vibration or tamping all ten cylinders were filled. The plastic molds were all coated with a thin layer of oil on the inside to prevent the mixture from sticking to the walls during pouring and to make the removal at 7 days easier. The cylinders were stored in a controlled 100 % humidity room and all cylinders were stripped of their plastic molds after 7 days of curing.

One problem that has been noted with fly ash cylinders is that the bottom of the cylinder will have crumbled edges. This makes the area of contact reduced in the compression testing. The thin coat of oil helps to reduce this damage but some edge crumbling still occurs. To overcome this, a piece of Celotex, which is a rigid foam used for insulating houses, was placed on each side of the cylinders before crushing. This foam effectively fills in the parts that have crumbled by deforming to the exact shape of the fly ash cylinder. The foam completely deforms at 35 psi (240 Pa) so it will not effect the ultimate strength of the samples. This capping method is in accordance with ASTM C 1231, which recommends using elastomeric pads for capping partially crumbled cylinders (ASTM 2000). The only drawback to using the foam is that a stress-strain curve cannot be accurately constructed, but by measuring the samples after they had reached their ultimate stress it was found that the samples undergo an average of about 2 % strain before failure.

All tests were done on a Wykeham Farrance compression test machine with calibrated load rings to measure the load. Two load rings were used to accommodate the wide range of ultimate loads. The cylinders were removed from the humidity room eight hours before testing, then the average height and width were measured. The cylinders were placed in the compression machine with the Celotex on top and bottom. The load was increased up to 35 psi (240 Pa) to seat the cylinder into the Celotex pads. This seating load was then released and measures were taken to insure that the cylinder was straight. The strain rate was set at 2 % per minute and the cylinder was loaded until failure. The compressive strength is the maximum load divided by the cross sectional area. All of the values recorded can be found in appendix B.

The compression testing results show different trends for the mixes. The bottom ash was clearly the strongest mixture. It was stated earlier that a higher strength material can sometimes be desired and it appears that the bottom ash mixtures are the only mixtures that will exceed the WVDOT class C limit of 1,000 psi (6,895 Pa) without adding large amounts of cement. The bottom ash had the widest distribution of particles and it is believed that this helped it to form a better matrix and therefore had the highest strengths. Both of the sands were uniformly graded and had similar performances. One interesting feature was that the bottom ash formed a bell shape curve when 28-day strength was compared to the percentage aggregate. This supports the idea that the larger particle distribution forms a good matrix. Just like in concrete mixtures, a strong composite needs both small and large aggregates as well as the finer binding agents. This combination of sizes allows the particles to nestle among each other and form a stronger composite. Moreover, the bottom ash contains some CaO that contributes to the pozzolanic activity of the mixes.

Both of the sands make good candidates for WVDOT Class A or Class B CLSM mixtures since their strengths did not exceed 300 psi (2,070 Pa) in any of the cases. Both of the sands had strengths that were in the 200 psi (1,380 Pa) range for all mixtures with one exception. The strength of the two mixtures containing 85% sand dropped significantly to the 100 psi (690 Pa) range. The sand mixtures also formed bell shaped curves that peaked between 50% and 75% aggregates; however, these results are less significant because of the small range of values for compressive strengths in these mixtures.

The mixtures containing only bottom ash and fly ash showed the highest strengths in the range of 50 % to 75 % of bottom ash. Only one original sample (M75BA1C) exceeded 1,000 psi (6,890 Pa) compressive strength but not all four samples exceeded this limit. For this reason, higher strength testing was conducted on bottom ash mixtures with varying bottom ash contents of 40 % to 80 %. These tests were performed on mixtures with up to 4 % cement so that the effects of varying cement content could be analyzed. These mixtures resulted in some very high strength samples. The final two tests, 70 % and 80 % bottom ash with 4 % cement, had to be tested on a different compression machine because their strengths exceeded the mechanical limits of the load rings for the Wykeham Farrance compression machine. Figure 2.8 shows the influence of cement content on strength values of bottom ash mixtures. The peak strength occurred when the bottom ash content was about 70% for all of the tests.

There is however some discrepancies in the data. The line representing 3% cement was consistently slightly below the line representing 2% cement. This was caused by an experimental error. The batch with 3% cement appeared to have air-entrained cement rather than normal type I cement. The data still shows the expected shape of the curve, however the values are slightly reduced. In addition, the data point that corresponds to M70BA4C was the last sample to be tested on the Wykeham Farrance compression machine. It was stated that these samples were too strong to be accurately tested on this machine so there were no reliable results for M70BA4C. The data point shown in Figure 2.8 was found by a best-fit method. The samples with 60%, 70%, and 80% bottom ash and 4 % cement were tested again at 35 days to validate the data that was obtained on the Wykeham Farrance machine at 28 days. Comparing the four graphs made from the strength testing at 7, 14, 28 and 35 days to each other, the value for the 28 day strength of M70BA4C was estimated. It should be noted that the point shown on the graph represents this estimated number.

The samples that were chosen for repeat tests had points that did not fall in the expected range. They were repeated under the same conditions as the original samples. The new values were computed by averaging all four values, the two original values and the two new values. The graphs presented in the body of this report represent the corrected data. All of the original data is presented in the appendices. Figures 2.5, 2.6, and 2.7 show the strength characteristics of the river sand, foundry sand, and bottom ash mixtures that did not contain cement, respectively.

Compressive strength tests were also conducted on saturated CLSM cylinders. These tests were conducted to find the influence of water saturation on the final compressive strength. The samples were prepared by the same methods discussed before.

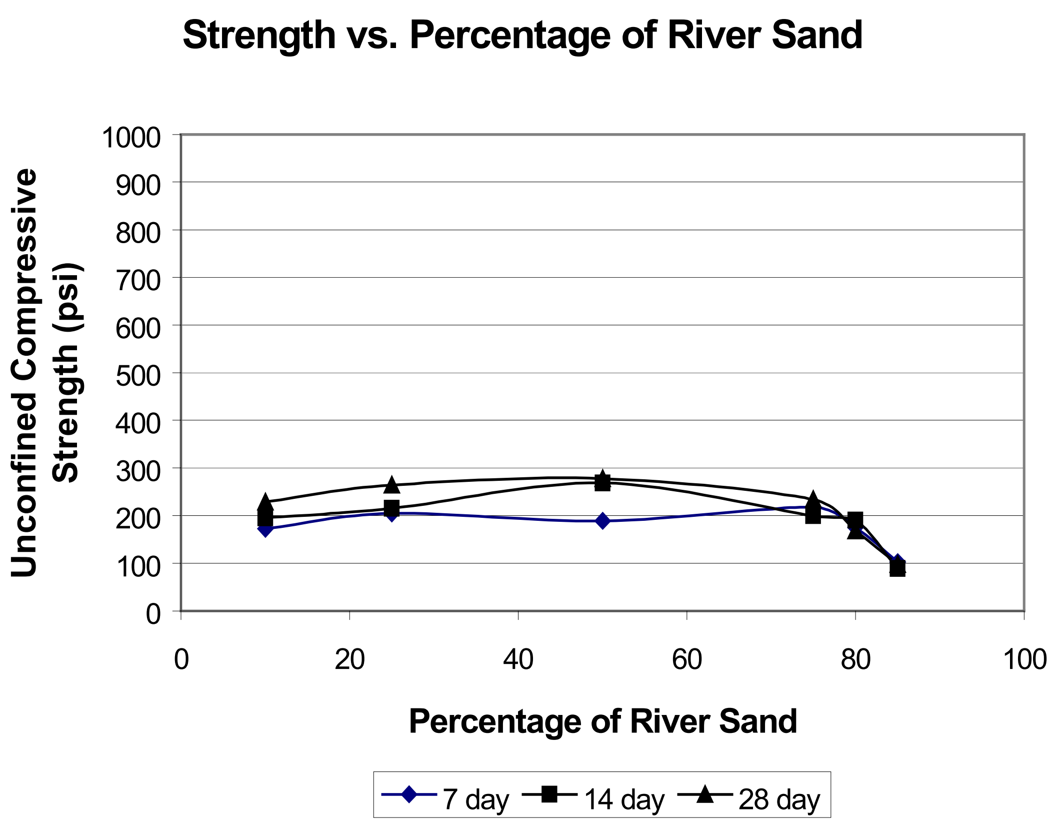


Figure 2.5: Strength Curves for River Sand Mixtures

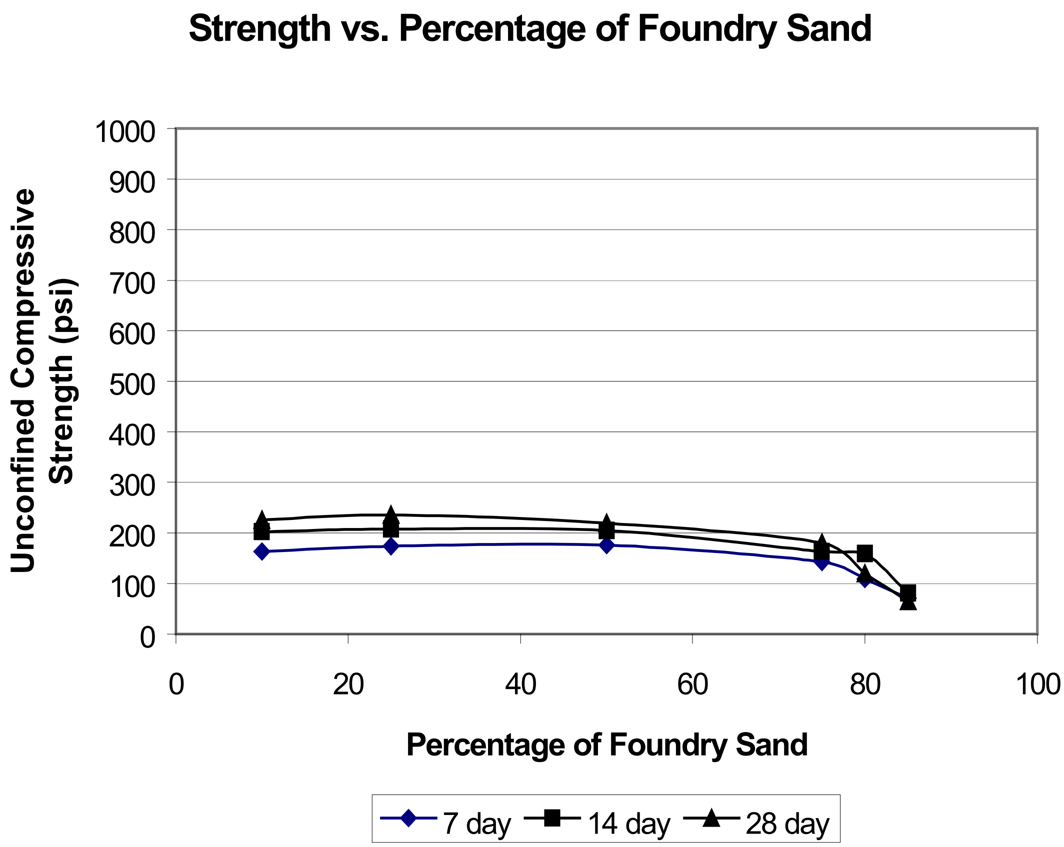


Figure 2.6: Strength Curves for Foundry Sand Mixtures

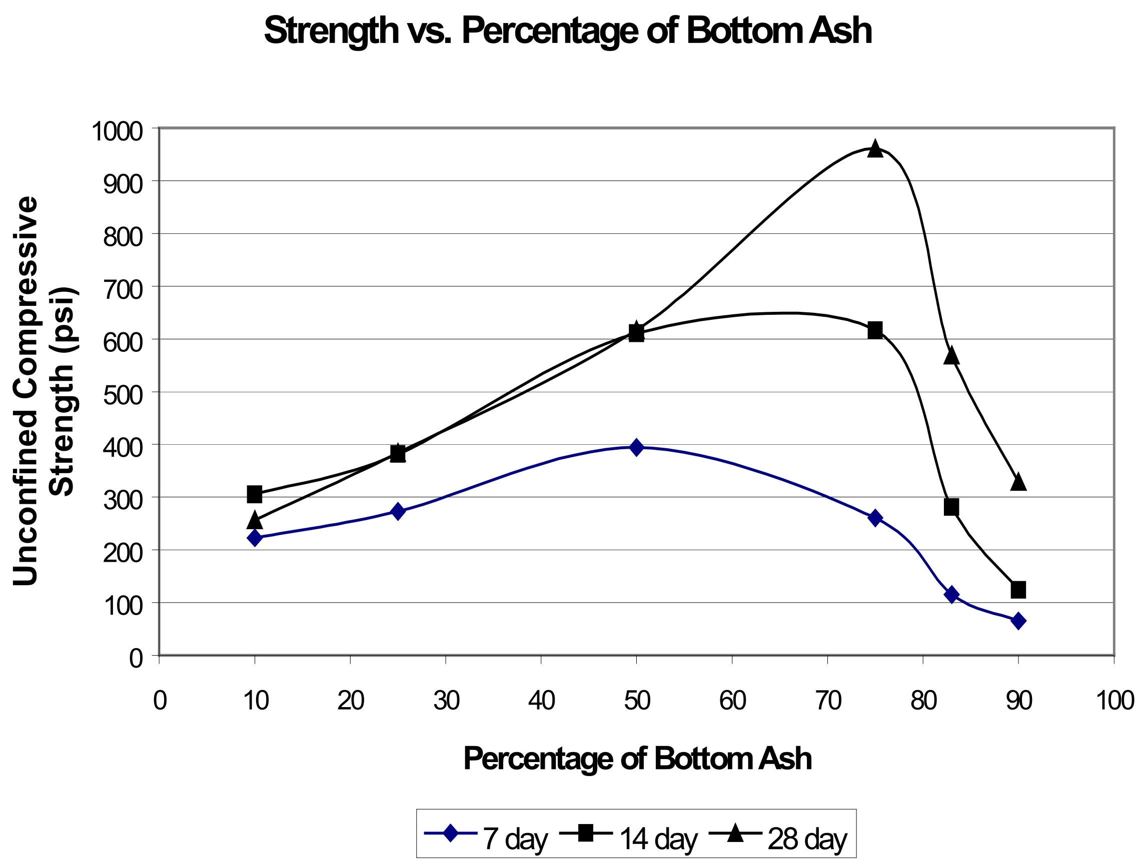


Figure 2.7: Strength Curves for Bottom Ash Mixtures

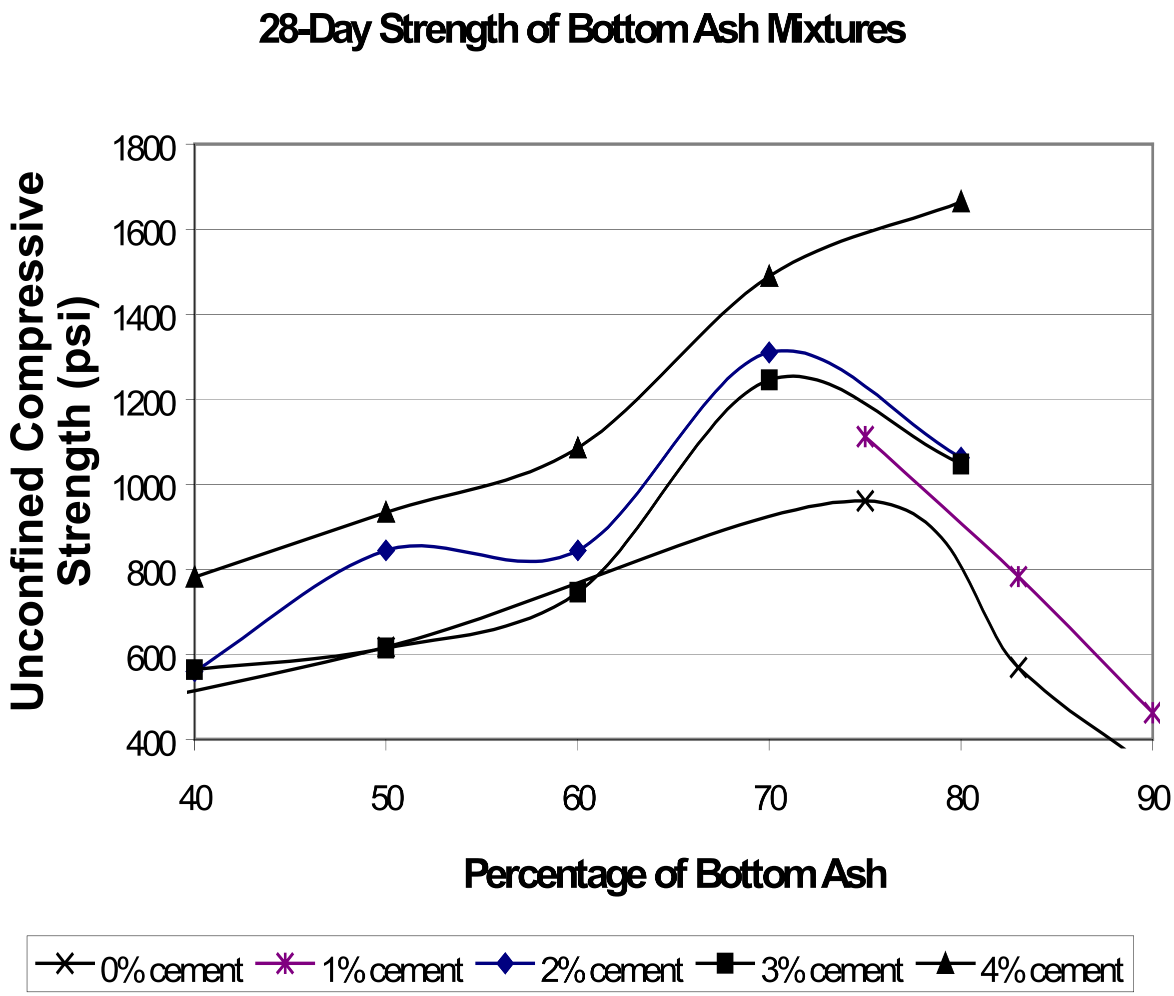


Figure 2.8: 28-Day Strength Curves for Bottom Ash and Cement Mixtures

However, after 7 days when the samples were removed from the plastic molds, the samples were submerged in water. The samples remained submerged in water for 21 days. The 28-day compressive strength was recorded and compared to the regular, unsaturated, value. These tests showed that the compressive strength decreased as much as 20% due to saturation. The mixtures containing Foundry sand had the most reduction in strength, particularly samples with a high percentage of foundry sand in the mixture. The bottom ash mixtures had the least reduction in strength. Some of the bottom ash mixtures only had strength reductions of 12%. A limited number of tests were conducted to investigate the influence of water saturation. These tests show that the compressive strength of CLSM mixtures reduced by 12% to 20% when the mixtures were cured in saturated conditions.

2.4 PENETRATION RESISTANCE

In addition to the compressive strength tests, penetration resistance tests were also performed. The penetration resistance is a test recommended by ASTM but is not mentioned in the WVDOT specification book. These tests were performed in accordance with ASTM C 403 which requires that the material be put into a container that is at least 6 inches square by 9 inches deep (15 cm square by 23 cm deep). The material was kept for 14 days in a humidity room before conducting any tests. This research also included tests performed during the first 24 hours to get a better understanding of how the material hardens. The tests were conducted with a Humboldt spring-loaded penetrometer with a maximum reading of 110 lb (490 N) and head sizes varying from 1 in^2 to 1/40 in^2 (6.5 cm^2 to 0.004 cm^2). This means that the penetrometer can determine a maximum penetration resistance of 4,400 psi (30.3 kPa). Most of the samples exceeded the maximum penetration resistance so it is difficult to draw any conclusions on their hardening characteristics from these tests.

All of the samples that were tested for compressive strength were also tested for penetration resistance. Figure 2.9 shows all of the results measured at 24 hours for the penetration resistance testing. The penetration resistance tests show that the bottom ash tends to have the highest 24 hr strength, while the foundry sand tends to have the lowest 24 hr strength. This is related to the gradation of each material. The bottom ash has a better distribution of aggregate sizes than do the sands. This allows the material to have a better matrix and increased strength. The bottom ash also has a higher percentage of large aggregates that would assist in the removal of bleed water. The river sand has some variation in size so it too can form a quality matrix and the presence of large particles helps the bleed water leave the sample. The foundry sand had consistently lower strength because of the uniform particle size. It does not form a good matrix and there are no large particles to allow the escape of the bleed water. The data collected during the penetration resistance tests can be found at the end of appendix B.

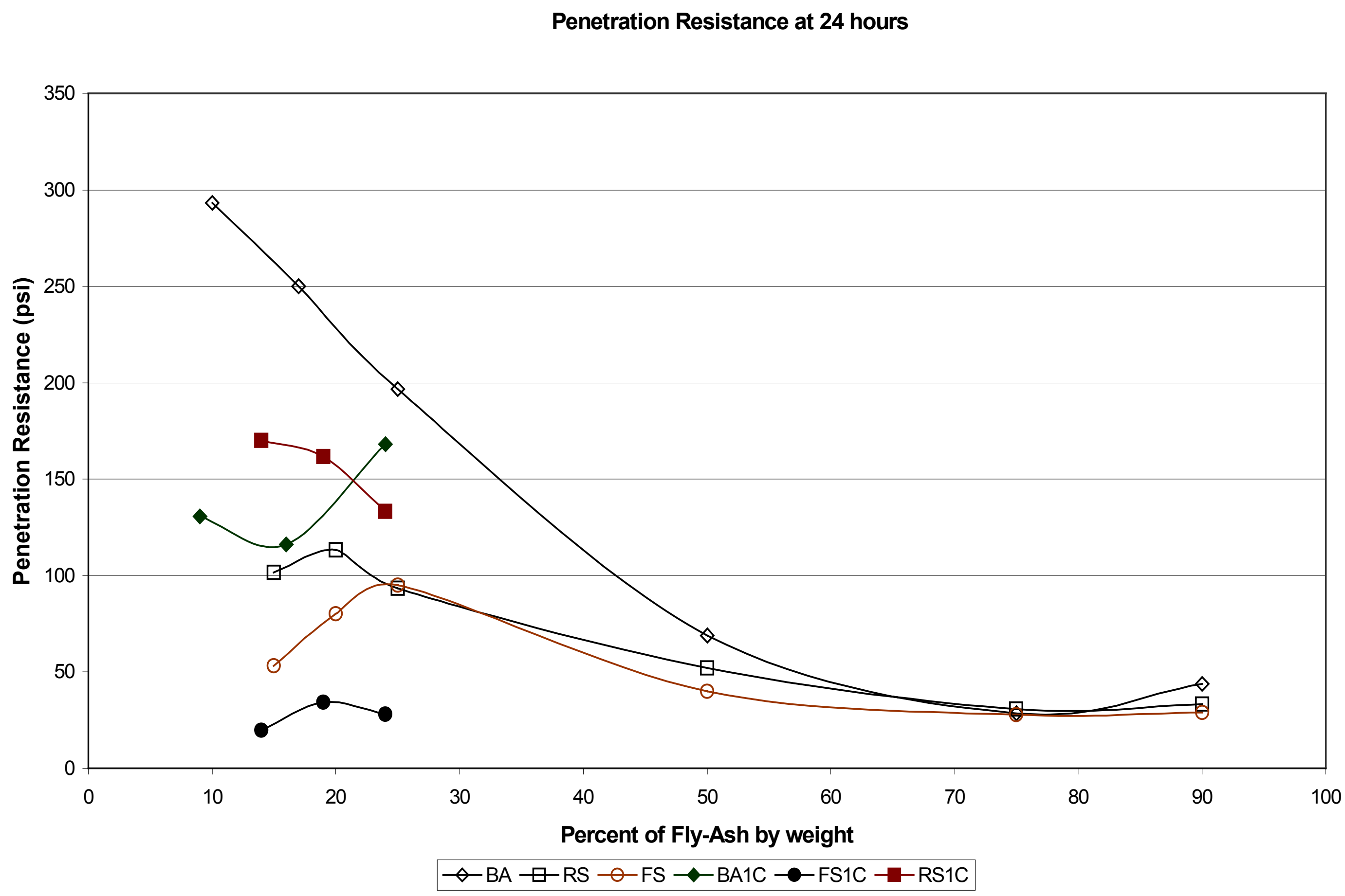


Figure 2.9: Penetration Resistance Testing Results

CHAPTER 3

DESIGN OF THE PIPE TESTING APPARATUS

In this chapter, the design of the pipe testing apparatus is discussed. This discussion includes the specific goals of the testing apparatus, the structural design, and the data acquisition system. Some figures and calculations are presented in the body of this chapter. Appendix D contains all the figures and calculations that were performed during the structural analysis of the pipe testing apparatus.

3.1 SPECIFIC GOALS OF THE PIPE TESTING APPARATUS

The pipe testing apparatus should have the capability to test several different sizes of pipe in a controlled environment. The device should be able to test trench widths up to 3 times the diameter of the pipe. The testing device should be able to apply a uniform surcharge load and measure the deflections in the pipe and the stresses in the soil mass.

To accommodate available deflectometers, the minimum diameter of pipe needed to be 6 inches (15 cm). Therefore, the objective was to test flexible pipes with diameters of 6 inches (15 cm) and 8 inches (20 cm). To test a trench width of three times the 8 inch (20 cm) diameter pipe the box needed to be at least 24 inches (60 cm) wide. The box also needed to be at least 18 inches (46 cm) deep to provide 2 inches (5 cm) bedding, and one pipe diameter cover. The size of the test box was chosen as 40 inch long by 25 inch wide by 20 inch deep (102 cm (L) x 64 cm (W) x 51 cm (D)).

The next step was to devise a method for application of uniform surcharge loading on the buried pipe. Many companies make inflatable jacks that are shaped like large pillows and can be used with air pressure or water pressure. As an alternative to this, motorcycle inner tubes were considered. Motorcycle inner tubes are designed for pressures of at least 60 psi (412 Pa) and come in a variety of sizes. Inner tubes for a 16 inch (41 cm) diameter 6 inch (15 cm) wide tire were chosen. When this tire tube is folded flat, it has the shape of a rectangle 6 inches (15 cm) wide and 25 inches (64 cm) long with the stem sticking straight up. For a 16 inch (41 cm) diameter tube, 25 inches (64 cm) is one half of the circumference. Eight of these tubes folded flat and laid side by side occupies a space that is 40 inch (102 cm) by 25 inch (64 cm). The final height of the test box was chosen as 20 inches (51 cm) to allow some room for the inflation of the inner tubes.

The last stipulation for the design of the test box was that the interior should be smooth to prevent any frictional losses and the deflections of the test box should be kept to a minimum level. The interior of the box consisted of smooth 1/8 inch (3.2 mm) steel plates welded together in the corners. A reaction frame was installed to apply direct loading over the center of the test box. The following sections provide details of the design of the test box.

3.2 STRUCTURAL DESIGN OF THE PIPE TESTING APPARATUS

The goal in the design of the testing apparatus was to hold a sufficient volume of soil to bury and surround corrugated pipe for testing the soil-pipe interactions under surcharge loading. The surface loading applied to the soil acts as a simulation of surcharge stress. In addition, a cross beam was designed for the application of a direct loading that simulates a wheel load or other point loads that might be applied above the pipe. The testing apparatus designed is a steel box with the inside dimensions of 40 inch (L) x 25 inch (W) x 20 inch (D) (102 cm (L) x 64 cm (W) x 51 cm (D)). The box is fully reinforced so that the deflections of the box are negligible in relation to the deflections of the pipe. Although the box was only expected to reach 32 psi (220 Pa) during the coarse of this research, it was designed for 70 psi (480 Pa) for potential future use.

3.2.1 REQUIREMENTS FOR OPERATING PRESSURE OF 70 psi (480 Pa)

(i) Design of the Reinforcing Bars

For ease of construction, the box is made from 1/8” steel plates with three steel angles welded to each side to prevent deflections. Each angle can be modeled as a fixed-fixed beam with a continuous load across it. The equation for the maximum deflection in a fixed-fixed beam with continuous loading is given as (PCI 1999):

$$y = 5(W)(L^4)/(384EI) \qquad \text{(Eq. 3.1)}$$

In this equation y is the maximum deflection, W is the load per length, L is the length of the member, E is Young’s modulus, and I is the moment of inertia. For a minimum allowable deflection of 0.01 inch (0.25 mm), the minimum required moment of inertia for the supporting members can be calculated as:

$$I = (8.98x10^{-5})(W)(L^4) \qquad \text{(Eq. 3.2)}$$

In this equation a value of 29,000 ksi (200 MPa) for Young's Modulus was assumed. Using this equation, the required moment of inertia could be calculated for each side of the pipe testing apparatus. Based on the calculations for I, the following support members were chosen. The lid and the 25 inch (64 cm) side were supported by ¼ inch (6.4 mm) thick steel angles with legs that measure 3 inches (7.6 cm) each. The lid was supported by 8 of the 3 inch (7.6 cm) steel angles and the 25 inch (64 cm) sides were supported with 3 of the 3 inch (7.6 cm) steel angles on each side. The 40 inch (102 cm) side was supported with ¼ inch thick steel angles with legs that measure 6 inches (15.2 cm) and 3.5 inches (8.9 cm). The 40 inch (102 cm) side used 3 of the 6 inch (15.2 cm) steel angles on each side. The bottom of the box was supported with some 3 inch (7.6 cm) angles and some 6 inch (15.2 cm) angles. The structural support for the bottom of the box is in excess of the calculated requirements. The feet on the box, which are 6 inch (15.2 cm) steel angles provide this excess reinforcement. The deflections that will result from these supports can be found by substituting the known moments of inertia in Equation 3.1. The maximum possible deflections at an operating pressure of 70 psi (480 Pa) can be calculated as 0.0091 inch (0.23 mm) on the 40 inch (102 cm) long side and 0.0098 inch (0.25 mm) on the 25 inch (64 cm) wide side. The supporting calculations can be found in appendix A. Figure 3.1 is a schematic of the pipe testing apparatus.

(ii) Design of the Lid Fasteners

The lid is held down by 20 steel bolts of ½ inch (12.7 mm) diameter with a manufacturer specified yield stress of 70,000 psi (480 kPa). It is a common conservative practice to use a design stress that is (2/3) of the yield stress (Kassimali 1995).
The equation for the average stress in a bolt is given as (Hibbeler 1997):

$$\sigma = (P)(FS)/(A)(N) \qquad \text{(Eq. 3.3)}$$

Where σ is the average stress, P is the load, FS is the factor of safety, A is the cross sectional area of each bolt, and N is the number of bolts. The factor of safety against the bolts failing at 70 psi (480 Pa) can be calculated as:

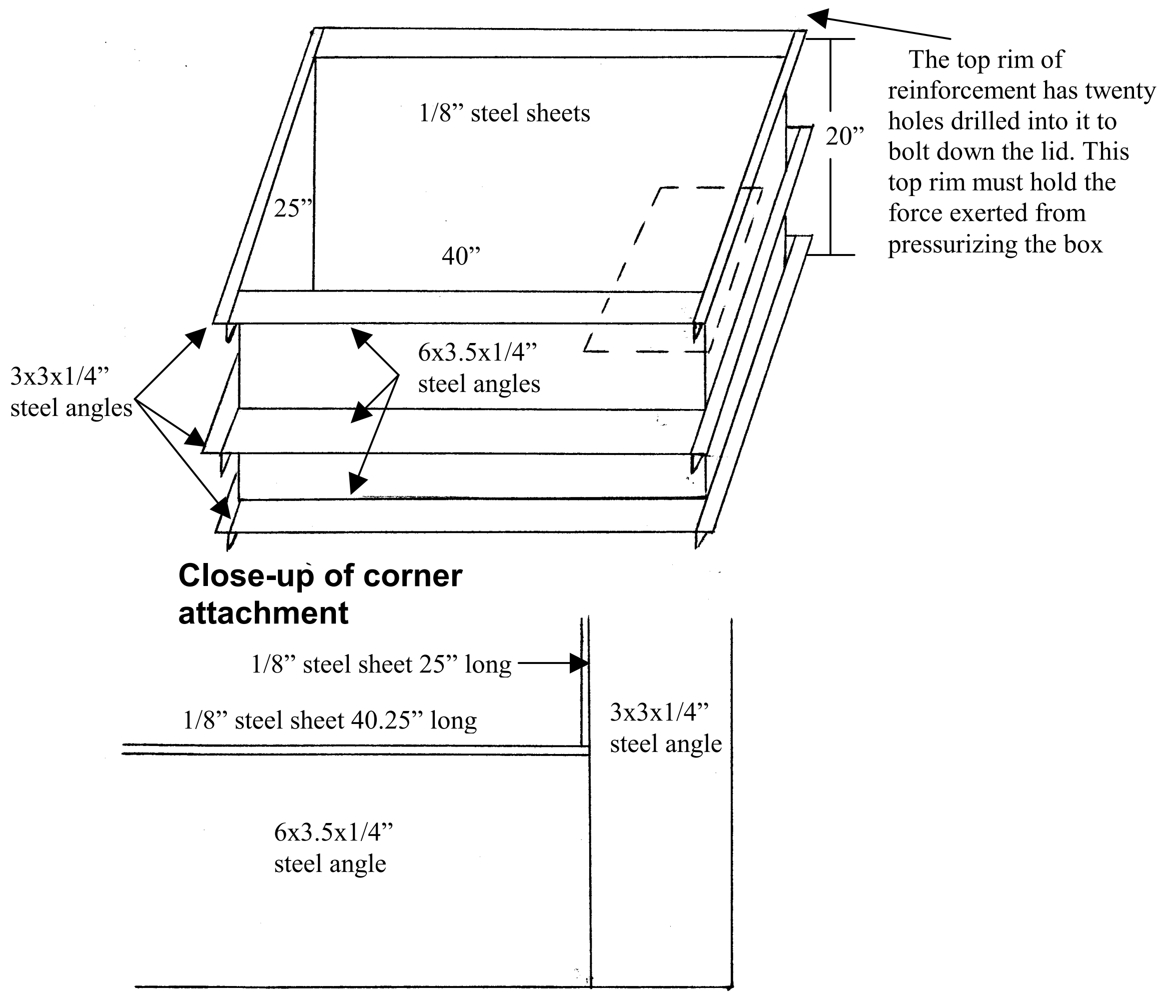


Figure 3.1: Schematic of the Testing Apparatus without the Reaction Frame

$FS = \sigma_{design}(A)(N) / (70\ psi)\ (40in x 25in)$ (Eq. 3.4)

The factor of safety against failure for the lid bolts is computed as 2.62. The bolts should also be replaced on a regular schedule to insure that they will not fail due to fatigue.

(iii) Weld Strength

Figure 3.2 illustrates several portions of the box that are potentially subject to failure of the welds. All the interior seams are welded together with continuous welds. These welds could potentially fail. In addition, the top rim of steel angles serves two purposes. The first is as reinforcement against deflections and the second is as the anchor point for the lid. The lid is bolted to the top rim during operation. Separation of the welds holding down the top rim is a potential failure mode. The free body diagram for the top rim is included in Figure 3.2. The equation for weld strength is given as (Lincoln 1973):

$\sigma_{weld} = 0.3\ \sigma_{tension}\ 0.707\omega$ (Eq. 3.5)

This specifies the strength of weld per unit length of weld, where ω is the leg width of the members being welded, and $\sigma_{tension}$ is the tensile strength of the weld material. The following values of weld strength were calculated using equation 3.5.

$\sigma_{weld} = 3180$ lb/in for ¼ inch (6.4 mm) welds

$\sigma_{weld} = 1590$ lb/in for $^1/_8$ inch (3.2 mm) welds

The four sides and the bottom of the box have continuous $^1/_8$ inch (3.2 mm) welds. Figure 3.2 shows the geometry of each potential failure. The force (P) acting on the 40 inch (102 cm) side, 25 inch (64 cm) side, and bottom are: 56 K (249 kN), 35 K (156 kN), and 70 K (311 kN), respectively. The total weld length (L) for the 40 inch (102 cm) side, 25 inch (64 cm) side, and bottom are 80 inch (203 cm), 65 inch (165 cm), and 130 inch (330 cm), respectively. These values are illustrated in Figure 3.2. Using the following equation, the factor of safety against weld failure can be calculated (Lincoln 1973). The summation sign is used because the 40 inch (102 cm) side and the 25 inch (64 cm) side both have an additional 24 inches (61 cm) of ¼ inch (6.4 mm) weld from the reinforcing bars that prevents their separation.

$FS = \Sigma\sigma_{weld}*L/P$ (Eq. 3.6)

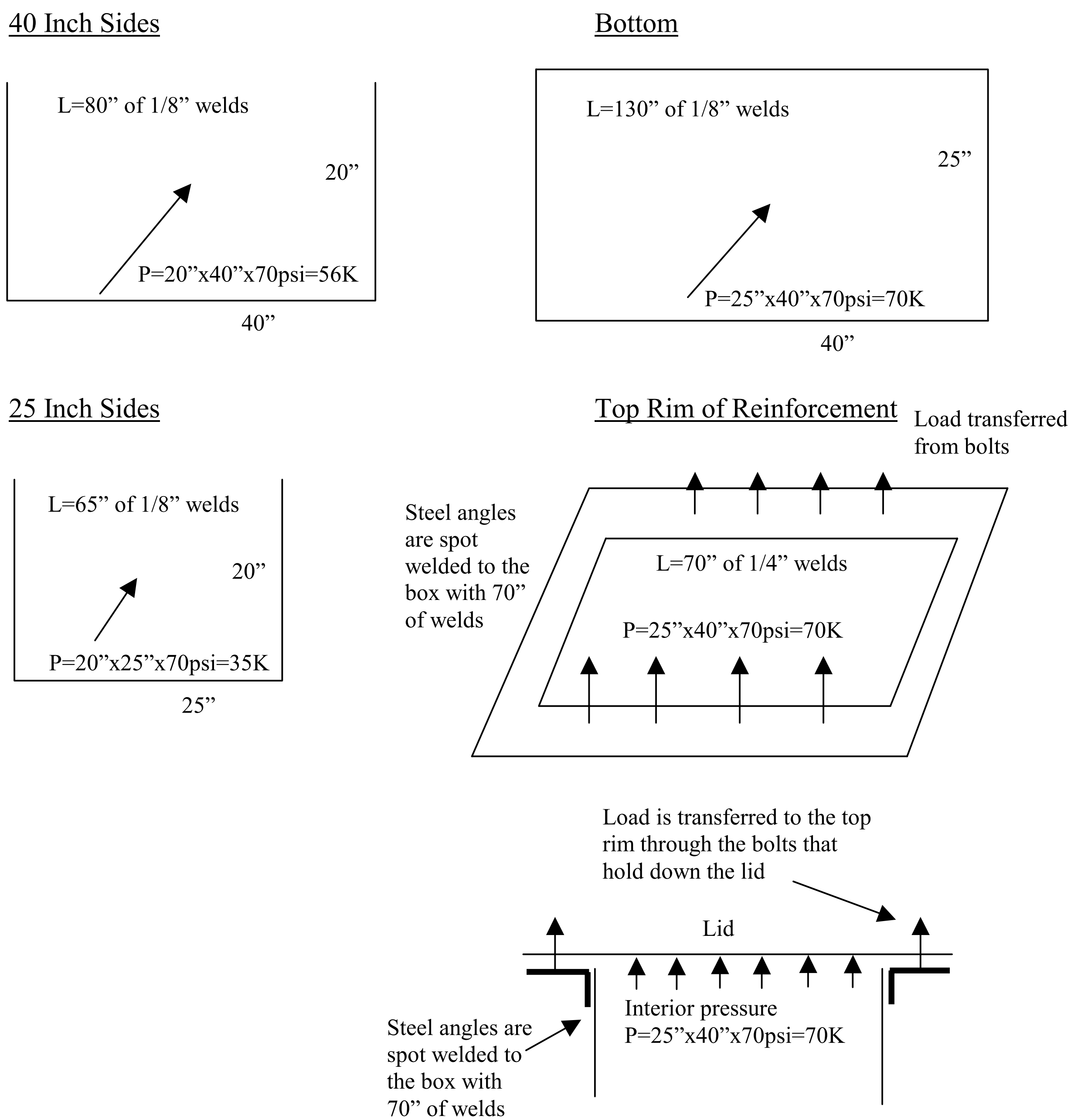


Figure 3.2: Geometry of Welded Areas

This makes the minimum factor of safety against failure of the welds to be 2.95, which occurs along the bottom plate. The factor of safety for the top rim being pulled from the box can be found using the same equation. This top rim has 70 inches (178 cm) of ¼ inch (6.4 cm) weld which results in a factor of safety of 3.18 when the box is pressurized to 70 psi (480 Pa).

3.2.2 REQUIREMENTS FOR OPERATING PRESSURE OF 30 psi (207 Pa)

It was stated that the actual maximum operational pressure in the experimental procedures is 30 psi (207 Pa). This means that the actual operational deflections and factors of safety are much more conservative than those shown in section 3.2.1. The operational factors of safety and deflections for the experimental procedure are:

y (40 inch side) = 0.004 inch (0.10 mm)

y (25 inch side) = 0.004 inch (0.10 mm)

FS (lid bolts) = 5.72

FS (welds on 25 inch (64 cm) side) = 11.22

FS (welds on 40 inch (102 cm) side) = 7.94

FS (welds on the bottom) = 6.45

FS (separation of top rim) = 6.96

3.2.3 DESIGN OF THE REACTION FRAME

The reaction frame has four major reactions that need to be accounted for: the vertical deflection of the loading beam, the force in all of the bolts, the shear forces in the steel around the bolts, and the possible moment that can result from an off-centered placement of the load. These are considered in the following section. Figure 3.3 shows the geometry of the reaction frame.

(i) Vertical Deflections of the Loading Beam

The deflection of the loading beam can be found by modeling it as a pin-pin beam with a center load. The maximum deflection is given as (Kassimali 1995):

$$y = (P)(L^3) / 48EI \qquad \text{(Eq. 3.7)}$$

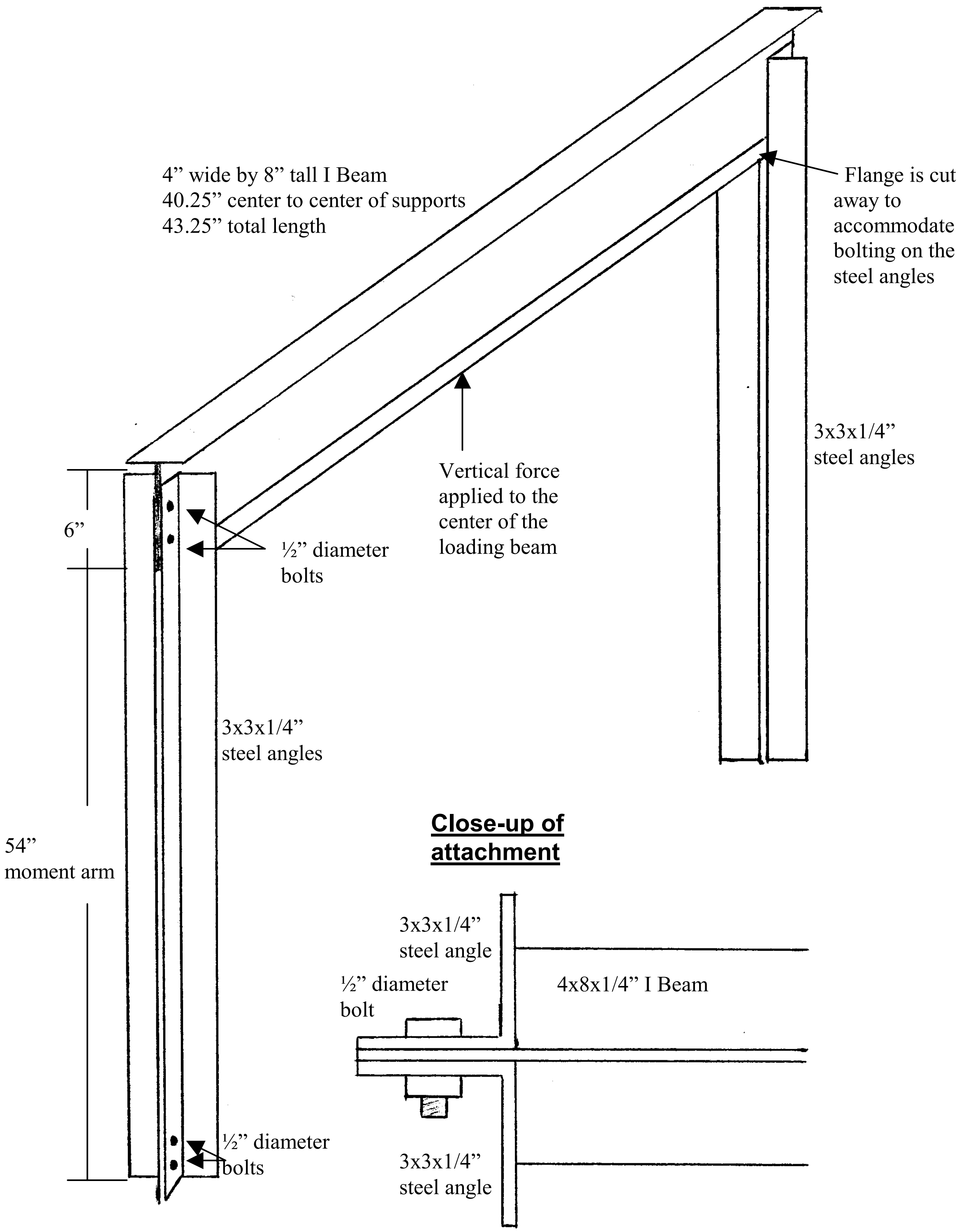


Figure 3.3 Schematic of the Reaction Frame

In this equation, P is the point load and the other variables have been previously defined. For a maximum deflection of 0.05 inch (1.3 mm) and a maximum load of 10,000 lb (44,500 N), the 40.25 inch (102.2 cm) long beam will need a minimum moment of inertia of 23.9 in^4 (994 cm^4). An I-beam 4 inches (10 cm) wide by 8 inches (20 cm) tall with a moment of inertia of 38.8in^4 (1614 cm^4) was ussed. Substituting the moment of inertia back into Equation 3.7 will result in a vertical deflection of 0.031 inch (0.79 mm). This level of deflection is justifiable because no deflection measurements will be taken near the loading beam.

(ii) Bolts on the Reaction Frame

The bolts used were ½ inch (12.7 mm) bolts with 70,000 psi (483 kPa) yield strength. As shown in Figure 3.3, there are four bolts at top of the vertical legs and four bolts at bottom of the vertical legs. Each bolt must hold 2,500 lb (11,120 N) in a double shear configuration. The following equations can be used to calculate the factor of safety against bolt failure (Hibbeler 1997):

$$\sigma_{shear} = 0.5\sigma_{design} \qquad \text{(Eq. 3.8)}$$

$$\sigma = P/2A \qquad \text{(Eq. 3.9)}$$

$$FS = \sigma_{shear} / \sigma \qquad \text{(Eq. 3.10)}$$

As done before, σ_{design} equal to (2/3) of the yield strength was used. Equation 3.10 results in a factor of safety of 3.67 for the bolts on the reaction frame.

(iii) Shear Forces in the Reaction Frame

The shear forces in the steel members supporting the bolts can be calculated by considering the possible failure surface and the resulting area that will be under shear. The load beam must carry the whole 2,500 lb (11,120 N) from each bolt and the potential failure surfaces would be small rectangular blocks above each bolt. This block has 2 sides with a height of 2 inch (5.1 cm) (the bolt spacing) and a width of ¼ inch (6.4 mm) (the thickness of the steel members). Figures that show the geometry of these potential shear failures are in appendix D. Equation 3.9 shows the force divided by two times the area because the bolts are in double shear. The shear in the web is not in double shear so Equation 3.11 is used to calculate the shear stress in the web. The area of the potential failure surface is calculated by using Equation 3.12. Equations 3.8 and 3.10 can be used to find the design shear stress and the factor of safety. The yield strength of the steel I-beam is 60,000 psi (414 kPa). This value is less than the yield strength of the bolts.

$$\sigma = P/A \quad \text{(Eq. 3.11)}$$

$$A = H(1/4)(2) \quad \text{(Eq. 3.12)}$$

Solving Equations 3.8, 3.10, 3.11, and 3.12 for the I-beam leads to a factor of safety against failure of the web due to shear equal to 8.

(iv) Horizontal Deflections of the Reaction Frame

To prevent the load bar from twisting it needs to be considered like a cantilever beam with some percentage of the force causing it to rotate. As seen in figure 3.3, four 3 inch x 3 inch (7.6 cm x 7.6 cm) steel angles were used for the vertical support. There are two steel angles on each side of the load bar and these angles must resist the cantilever action. If the maximum expected deflection is 5 degrees, the force acting perpendicular to the load bar (P) is given as:

$$P = (\sin\theta)(10{,}000 \text{ lb}) = 872 \text{ lb } (3880 \text{ N})$$

The equation for deflection of a cantilever beam is given as (Kassimali 1995):

$$y = (P)(L^3) / 3EI \quad \text{(Eq. 3.13)}$$

Given that the total moment of inertia of the four 3 inch (7.6 cm) steel angles is 10.36 in^4 (431 cm^4) and the length of the cantilever section is 54 inch (137 cm) (the height of the loading beam as shown in figure 3.3), the maximum horizontal deflection of the load bar would be 0.15 inch (0.38 cm). All measures would be taken to prevent this bending from occurring, this is only the worst case scenario.

3.2.4 SUMMARY

In summary, the following can be said about the design of the pipe testing apparatus. Taking all possible failure modes into consideration, the testing apparatus can sustain a pressure of 70 psi (480 Pa) or a load of 10,000 lb (44,500 N) with a minimum factor of safety against failure of 2.62 which occurs in the bolts holding down the lid. However, this is a feature that is easily modified so it can be strengthened if any future tests need to use higher pressures. This factor of safety was found by using conservative measures, and was still well above typical acceptable ranges for steel design. Moreover, the bolts will be replaced several times to eliminate any affects of fatigue. The only other factor of safety below 3 is the weld strength of the bottom plate; however, there are a lot of frictional forces in and around the box that were not considered that actually help increase the safety of this potential failure mode. All other components of the box have factors of safety, higher than 3. Table 3.1 shows a summary of all of the deflection values and factors of safety that were calculated.

3.3 DATA COLLECTION

To collect the data from inside the pressurized box, a data collection system was developed. A Humboldt data logger that is capable of recording up to 8 signals simultaneously was used. Digital deflectometers that are compatible with the Humboldt data logger were used. The deflectometers have a range of 1 inch (2.5 cm). The required range was determined to be 10 % of an 8 inch (20 cm) pipe or 0.8 inch (2.0 cm). Three soil pressure gauges were used for measuring stresses in the soil mass. A load cell was used to measure the external load.

The instrumentation included three deflectometers placed inside the pipe in a vertical orientation. The deflectometers were placed with one at the center of the pipe and the other two placed at 6 inches from each side as shown in Figure 3.4. One soil pressure cell was placed at the center of the in-situ soil, 2 inches (5 cm) from the bottom of the test box. The other two cells were placed under the centerline of the pipe. Since the cells are 6 inches (15 cm) in diameter, their centers are both 3 inches (7.6 cm) away from the center of the pipe. The placement of these cells is also indicated in Figure 3.4. All of the soil pressure cells were calibrated in the pipe testing apparatus. The soil pressure cells were shown to be accurate within 1 psi (6.9 Pa) of the surcharge loading for pressure ranges between 5 psi and 50 psi (35 Pa to 345 Pa). The soil pressure cells are less accurate for pressures below 5 psi (35 Pa).

A small slit was cut in the lid so that the instrumentation wires could pass to the outside of the test box. There was also a circular hole cut in the lid over the center of the test box. This hole allowed direct loading to be applied through a load plate at the same time as the surcharge loading was being applied. This simultaneous load configuration

Table 3.1: Summary of Design Results

Factors of Safety and Deflections When Pressurizing	**Operating Pressure**	
	32 psi	**70 psi**
Deflection of box on 40" side	0.004"	0.0091"
Deflection of box on 25" side	0.004"	0.0098"
Factor of safety (lid bolts)	5.72	2.62
Factor of safety (welds on 25" side)	11.22	5.13
Factor of safety (welds on 40" side)	7.94	3.63
Factor of safety (welds on bottom)	6.45	2.95
Factor of safety (failure of top rim)	6.96	3.18
Factors of Safety and Deflections For the Reaction Frame at 10,000 lb		
Vertical deflection of the load bar	0.012"	
Horizontal deflection of the load bar	0.15"	
Factor of safety (bolts)	3.67	
Factor of safety (web)	8	

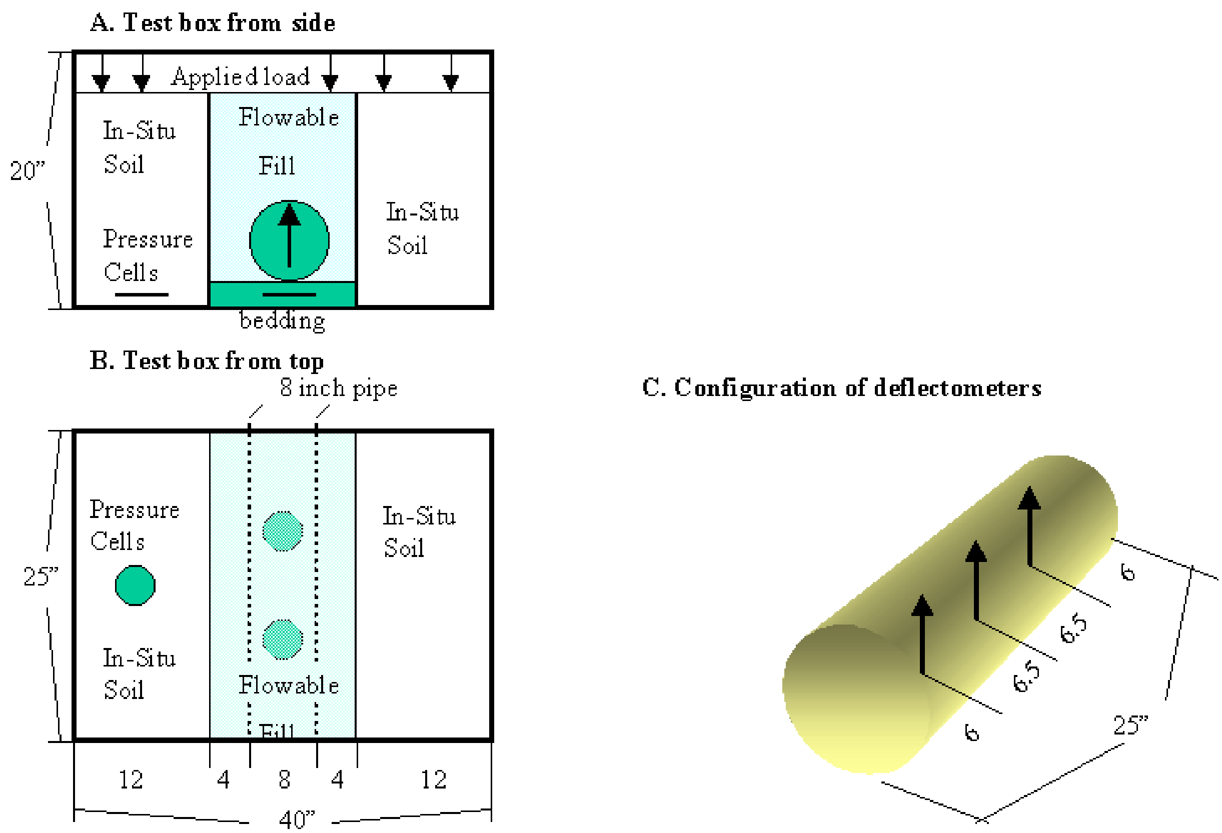


Figure 3.4: Configuration of Instrumentation

was not used in this research project. The direct loading experiments were always conducted without any applied surcharge loading.

3.4 PIPE TESTING APPARATUS

A final pipe testing apparatus that met all the requirements for this research project was designed and built. The pipe testing apparatus can be safely pressurized to 70 psi (483 Pa) or a 5 ton (44.5 kN) direct load can be applied directly to the backfill. The entire structure was designed to minimize weight without compromising any strength. The structure is light enough that it can be transported on a dolly. It is rigid enough that no discernable deflections have been noticed while conducting experiments. Figure 3.5 shows a picture of the pipe testing apparatus.

Figure 3.5: Completed Pipe Testing Apparatus

CHAPTER 4
EXPERIMENTAL PROCEDURES

In this chapter, the experimental procedures are discussed. First, background information on the two types of in-situ soil used in this research is presented. The procedures for placing the in-situ soil in the pipe testing apparatus are also discussed. The operational procedures for conducting the pipe testing are outlined, as well as the pipe preparation procedures. Specific information about the properties of the pipes is found in sections 5.1 and 6.1.

4.1 IN-SITU SOIL PREPERATION

Two soils were used as in-situ material. This is the soil that is used to fill the portion of the box not being filled with the flowable fill. The width of this in-situ material would vary anywhere from 15.5 (39 cm) on each side for a 9 inch (22 cm) trench width to 8 inch (20 cm) on each side for a 24 inch (61 cm) trench width. The two soils used were a compacted cohesive soil and a loose non-cohesive soil. Both soils were held in place with soil dividers until after the CLSM was poured in place. The soil dividers can be seen in Figure 4.1. The inner surface of the entire test box was oiled and lined with a plastic sheet before placing any soil in it. This greatly reduces the friction between the soils and the test box. The plastic sheet was replaced when it was damaged or torn so that the testing set up would be consistent for all pipe tests.

The loose non-cohesive soil was the same river sand used for the CLSM mixtures described in section 2.3. The sand was not sieved and no effort was made to compact it. Once the soil dividers were in place, the soil was simply poured on both sides until the sides were filled. A soil pressure cell was always placed on the left side 2 inches (5 cm) from the bottom of the box. The geometry is symmetric so there is no difference between placing the soil pressure cell on the left or the right sides.

The same sand was also used for the bedding material. The bedding was prepared by pouring 2 inches (5 cm) of sand into the trench, placing both of the soil pressure cells, and filling in another 2 inches (5 cm) of loose sand. The sand was then tamped down to a 3 inch (7.5 cm) thickness.

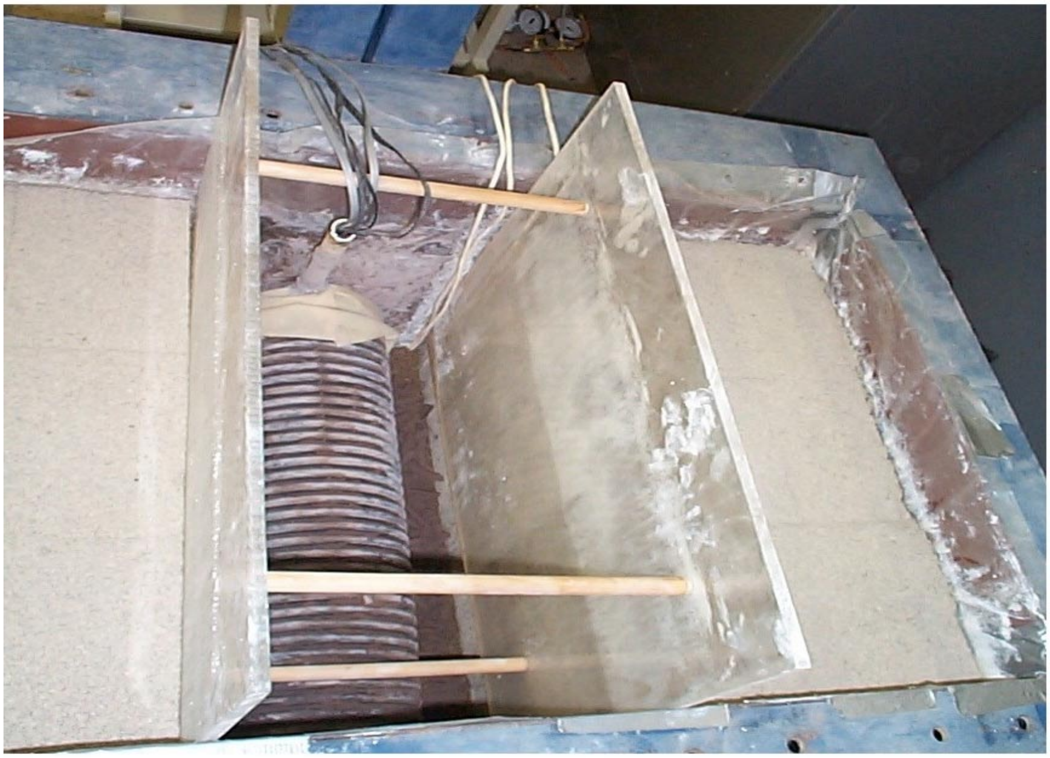


Figure 4.1: Prepared Trench for the Pipe

The cohesive soil was a mixture of 10% Kaolin and 90% river sand on a dry weight basis. The mixture was always kept at moisture contents between 9 % and 12%. This mixture was placed in three 6 inch (15 cm) lifts. The soil would be scooped into each side to a depth of about 8 inches (20 cm). The soil was then smoothed out with a spoon to give a level surface at a depth of 7 inches (18 cm). The soil was then compacted by using an 8 inch x 8 inch (20 cm x 20 cm) plate. The plate was moved around to cover the entire surface and at each location the plate was struck three times with a standard compaction hammer. Both sides of the trench were prepared before compaction to make sure that the trench dividers did not shift from the compaction effort. The instrumented pipe was placed in the trench before compaction began. By having the pipe in place before filling the sides, the trench dividers could be left in place. If the pipe was not already in the trench, one would not be able to place the pipe into the trench because the trench shoring would be in the way. Figure 4.1 shows the pipe placed in the trench and the compacted soil around the trench with the soil dividers still in place. The configuration shown in Figure 4.1 is the final stage before the trench was filled except for shoring the pipe against flotation.

Figure 4.1 also shows some of the preparation at the ends of the pipe. The pipes used were corrugated polyethylene (PE). The pipes had no liner, so there were corrugations on the inside of the pipe as well. The pipe sections were cut to the proper length with a utility knife. The open ends of the pipe were sealed with plastic membranes to keep the flowable fill from filling the pipe. The instrumentation wires also had to come out of the pipe. The instrumentation wires were enclosed in a short piece of PVC tubing to protect them from potential damage. The 25 inch (64 cm) section of pipe was instrumented and sealed before mixing the flowable fill so that the pipe instrumentation could be tested before it was too late to correct any errors. The only other preparation was to shore the pipe against flotation. Propping two wooden dowel rods between the pipe and the load beam sufficiently prevented flotation during the placement of flowable fill.

Once the box reached the stage in Figure 4.1, the flowable fill was prepared. The dry ingredients and the water were gradually added to a mortar mixer until all of the material was being mixed. A flowability test was performed when the mixture was fully mixed to make sure that the flowable fill would have 9 inch (23 cm) spread. The mixture was poured into the trench with a large bowl, one bowl at a time. When the trench was full, the soil dividers were removed and then cleaned. After 10 to 20 minutes the material usually had set enough for the flotation shoring to be removed. The entire box was then covered with a plastic sheet and left alone for 48 hours.

4.2 PREPARATION OF THE TEST BOX

After 48 hours of letting the flowable fill cure, the box was prepared for surcharge loading. The surcharge loading was applied with eight motorcycle inner tubes as described in chapter 3. The inner tubes were each 6 inches (15 cm) wide with a 16 inch (41 cm) diameter. This size inner tube has a circumference of 50 inches (128 cm). If the tube is folded flat in half it will cover a rectangular area of 25 inch by 6 inch (64 cm by 15 cm) and the tube stem will be sticking straight up. Placing eight of these tubes side by side covered the entire 40 inch by 25 inch (102 cm by 64 cm) loading area with no voids. These tubes were confined to a height of only 2 inches (5 cm) so they do not move or shift when being pressurized. Several preliminary tests were done with the inner tubes to insure that the pressure distribution would be even. For this preliminary testing the test box was entirely filled with the cohesive soil and then sealed for pressurizing. After pressurizing the box, the soil surface could be examined to see the patterns that the inner tubes left on the soil surface. These tests showed that the soil surface deformed evenly across the whole loading area and there were no ridges or voids formed from the seams between two adjacent inner tubes.

When the inner tubes were in place, the lid was fastened. All eight of the inner tube stems were protruding from the lid. The tubes were then hooked up to pressure regulators. The three inner tubes on the left and the three inner tubes on the right were put together as two different groups and were hooked up to two pressure regulators. The two inner tubes in the center were hooked up to the third pressure regulator. This set up allows variable pressure to be applied across the loading surface, including a more intense load directly over the centerline of the pipe. However, a variable pressure configuration was not used in this research. The pressure gauges used were accurate to ½ psi (3.4 Pa) for pressure ranges of 0 to 100 psi (690 Pa). There were also three additional pressure gauges at the inner tube stems that were accurate to 1 psi (7 Pa) for pressure ranges of 10 psi (70 Pa) to 60 psi (410 Pa). These less accurate gauges were used only for verification purposes. They would be able to warn if any of the inner tube sets was not receiving the correct pressure.

4.3 OPERATIONAL PROCEDURES

Once the test box was prepared for testing, an initial set of readings was taken. These readings include the three initial deflectometer readings and the three initial soil stress readings. The test box was then pressurized to 5 psi (34 Pa) and another set of readings was taken when the readings became constant. The time required for the readings to become constant varied with different tests. For the high strength flowable fill tests there was almost no movement at 5 psi (34 Pa) so the system stabilized quickly. For the low strength flowable fill with narrow trench widths the waiting time for stabilization was several minutes. After these readings were taken, the pressure was increased 1 psi (7 Pa) at a time up to 10 psi (69 Pa) and another set of readings was taken when the readings stabilized. The readings were taken at 5 psi (34 Pa) increments up to 30 psi (207 Pa). Many of the experiments took as much as 30 minutes to stabilize under an applied surcharge load of 30 psi (207 Pa). However, during this 30-minute stabilizing period, the total measured deflection only increased by about 1 to 2 one-hundredths of an inch (0.25 to 0.5 mm). Since the total deflections at 30 psi (207 Pa) were often in the range of ½ inch (1.3 cm), this means that more than 95 % of the total deflection would occur as soon as the pressure was applied. The pressure was then gradually released and a reading was taken at 0 psi (0 Pa) after the readings had stabilized. This reading was commonly taken after ½ hour. The last set of readings was taken after 24 hours. This 24-hour reading serves two purposes. It marks the starting point for the direct loading and it gives an indication of how much the system will rebound after removal of surcharge loading.

The direct loading was more of a laboratory measure than a practical field simulation. It was assumed to simulate the influence of a wheel load. A steel plate with dimensions of 12 inch x 12 inch x ½ inch thick (31 cm x 31 cm x 1.3 cm thick) was placed directly above the centerline of the pipe. The pipe only had about 7 inches (18 cm) of cover in the laboratory. In the field the pipe would have at least two feet (61 cm) of cover. Therefore, this loading does not reflect any practical field loading scenarios for the

WVDOT. However, many people place flexible pipes at the head of their driveways that are not more than 6 inches (15 cm) deep. Wheel loads on this type of pipe would be similar to the loading configuration in this test. This test also helps get a better understanding of the pipe-soil interactions so its inclusion is beneficial.

The apparatus was designed to use direct loading in conjunction with the surcharge loading. However, in this research work direct loading was used without any applied surcharge load. The load plate was placed and then an eight-ton jack was used to apply the load on the plate. The eight-ton jack was hand operated. A load cell was placed directly above the jack and a small load plate with a rotator cusp was placed on the top end of the jack to distribute the load to the load cell. The load cell has a maximum capacity of 10,000 lb (44.5 kN) and measures the load with a minimum accuracy of 10 lb (44 N). The test was performed by slowly jacking the jack until the load cell read a load of 1,000 lb (4,450 N). This load was maintained for at least 20 seconds and then slowly increased up to 2,000 lb (8,900 N). This process was continued in 1,000 lb (4,450 N) increments until it was no longer possible to maintain a constant load. Since the flowable fill was deflecting under the jack, the load was constantly decreasing as the settlements took place. For smaller loads it was easy to add more pressure to the jack to keep the load range within ±20 lb (90 N). For loads higher than 5,000 lb (22.2 kPa), it becomes difficult to maintain the load near the target load. Most of the fills took between 5,000 lb and 7,000 lb (22 kPa and 31 kPa) loads. The data collection was automated so the load, the soil stresses, and the pipe deflections were all logged simultaneously while the experiment was conducted. After the maximum load was reached, the pressure in the jack was released and the system was allowed to rebound. When the readings stabilized the data collection system was terminated. This portion of the experiment usually took about 20 minutes for completion.

The flowable fill was then removed and discarded. The in-situ soil was removed and kept for the next experiment. The in-situ soil was always kept covered between experiments to keep it from drying out. It was stated earlier that the cohesive in-situ soil was kept at a water content of 9 % to 12 %. Water content measurements were regularly taken, and water was periodically added to insure that all the tests were conducted with a soil that had a similar water content.

CHAPTER 5
EXPERIMENTAL RESULTS

One of the primary objectives of this research was to find the influence of the trench width ratio on the overall performance of buried pipe. Independent and dependent variables used in this research are identified in this chapter. These variables will each be examined and the relationship of these variables to the trench width ratio will be considered. Experimental data from the laboratory buried pipe testing are presented, as well as a discussion on the significance of this data and the trends that the data illustrate.

5.1 EXPERIMENTAL VARIABLES

The control variables in this experiment were, pipe diameter (d), trench width ratio (n_r), in-situ soil strength, CLSM strength, and loading. The dependent variables were the pipe deflections and the soil stresses. Since all tests were done with the same type of pipe, pipe stiffness was a constant for any given pipe diameter. The stiffness values of 6 inch (15.2 cm) and 8 inch (20.3 cm) pipes were 49 psi (337.9 Pa) and 35 psi (241.3 Pa), respectively. Both of these values were measured in the laboratory using ASTM Test Method for Pipe Stiffness (D 2412). This ASTM method also shows how to find the modulus of elasticity (E) from the information obtained in the pipe stiffness test. The calculated values for the modulus of elasticity at 5% deflections for the 6 inch (15.2 cm) and 8 inch (20.3 cm) pipes were 94 ksi (648.1 kPa) and 67.2 ksi (463.3 kPa), respectively. These calculated pipe stiffness values were very close to the values reported by the pipe manufacturers. When doing calculations in this research, the only pipe stiffness and pipe modulus of elasticity values used are the values measured in the laboratory, not the values supplied by the manufacturers.

The trench width was varied by at least three values for every test configuration. The experimental program included trench width ratios of 1.5, 2.0, and 2.5 except for two tests for which a trench width ratio of 3.0 was used. Having at least three points on a graph gives a better feel for the relationship between the variables of interest. The graphs relating trench width ratio and pipe deflections are some of the most relevant results of this report. All of the graphs and data are presented in more detail in the following sections. These figures show a tendency for larger trench widths to reduce the deflections in the pipe.

The in-situ soil was varied by two types, a low strength soil and a high strength soil. Since only two types of in-situ soil were used, it would not be possible to make predictions to other soil strengths with any level of confidence. The CLSM results also only compare two different strengths. The two CLSM mixes tested in this study were a WVDOT class A mixture with a compressive strength of 280 psi (1,931 Pa) and a WVDOT class C mixture with a compressive strength of 1,150 psi (7,929 Pa). The strength of WVDOT class A mixtures only range from 50 psi (345 Pa) to 300 psi (2,069 Pa) and the strength of WVDOT class C mixtures should only range from 1,000 psi (6,895 Pa) to 1,200 psi (8,274 Pa). There is not a wide range of permitted CLSM strength values that can be tested when using these two mixtures. While the two strength values that were tested provide information beneficial to comparing a low strength CLSM to a high strength CLSM, a few intermediate WVDOT class B mixtures should be tested in order to determine a relationship between CLSM strength and the other variables.

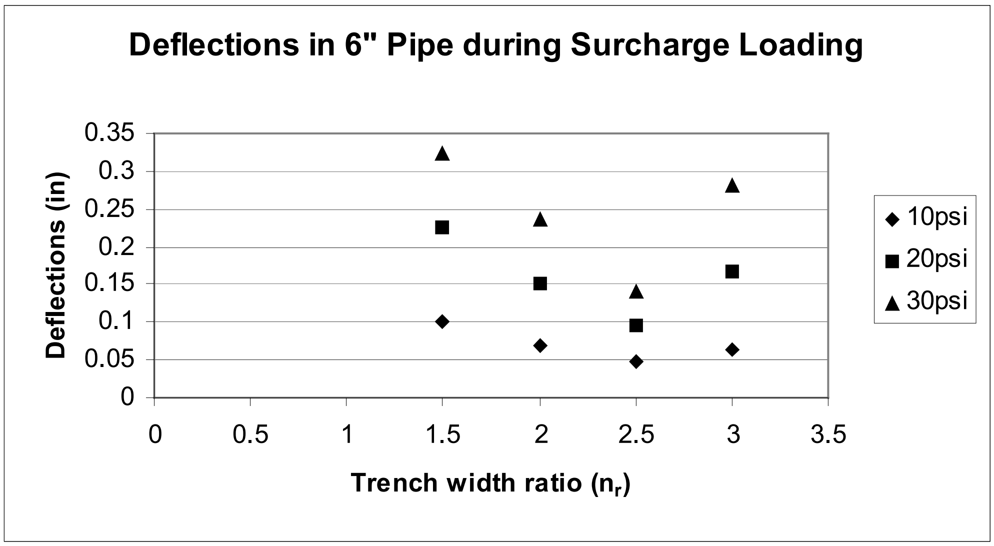
The pipe diameters also only varied by two values. A comparison of 6 inch (15.2 cm) and 8 inch (20.3 cm) pipe measurements is valuable for relating the information to other pipe diameters. However, before any conclusive statements could be made about the relationships between pipe diameter and the other variables, more pipe diameters should be tested. It appears that WVDOT does not use any pipe smaller than 18 inch (46 cm) in new construction and there is no official upper limit on permissible pipe sizes (WVDOT 2000). Time constraints limited the scope of this research from covering more than two pipe diameters, but future research should incorporate more pipe diameters so that this valuable data can be collected.

5.2 PIPE DEFLECTIONS DUE TO SURCHARGE LOADING

All of the data from the pipe testing can be found in appendix C. The following figures (Figures 5.1 through 5.4) show families of curves for surcharge loading from 10 psi (69 Pa) to 30 psi (207 Pa). In these figures, deflections are compared to the trench

Deflections Under Uniform Surcharge Loading recorded in inches

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	0.1017	0.4229	0.2259	0.727	0.3244	
2	0.07	0.0747	0.1523	0.151	0.2362	0.2168
2.5	0.0473	0.056	0.0967	0.131	0.141	0.2275
3	0.064		0.167		0.2801	



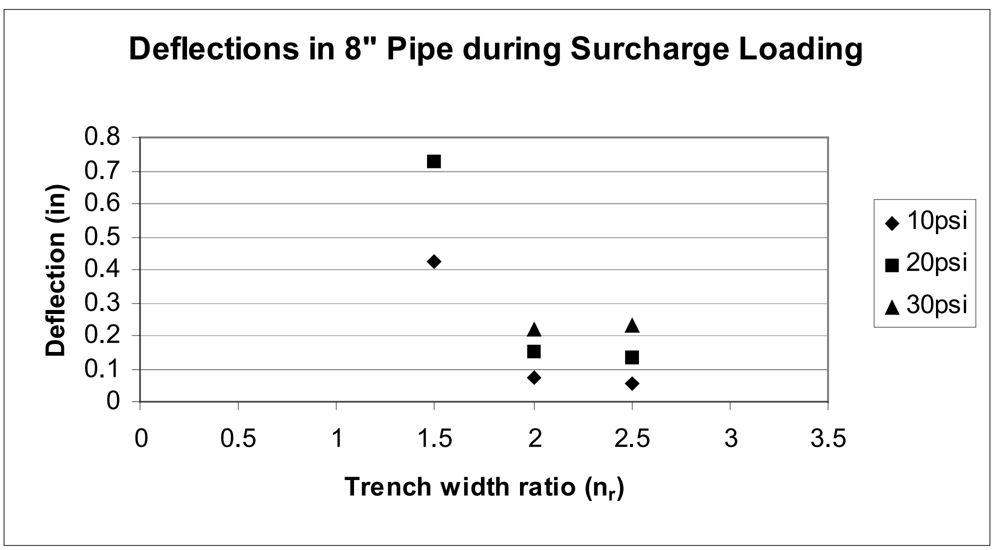
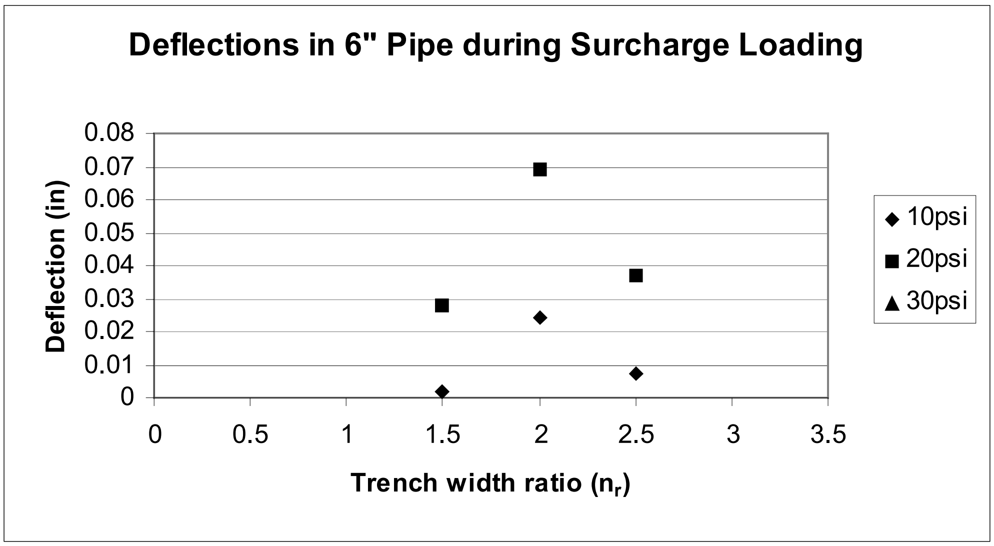


Figure 5.1: Deflections of Pipes in Cohesive In-Situ Soil With Low Srength Backfill

Deflections Under Uniform Surcharge Loading recorded in inches

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	0.0016	0.1808	0.028	0.3696		0.482
2	0.0245	0.0323	0.069	0.0973		0.162
2.5	0.0071	0.0099	0.037	0.0454		0.0856
3						



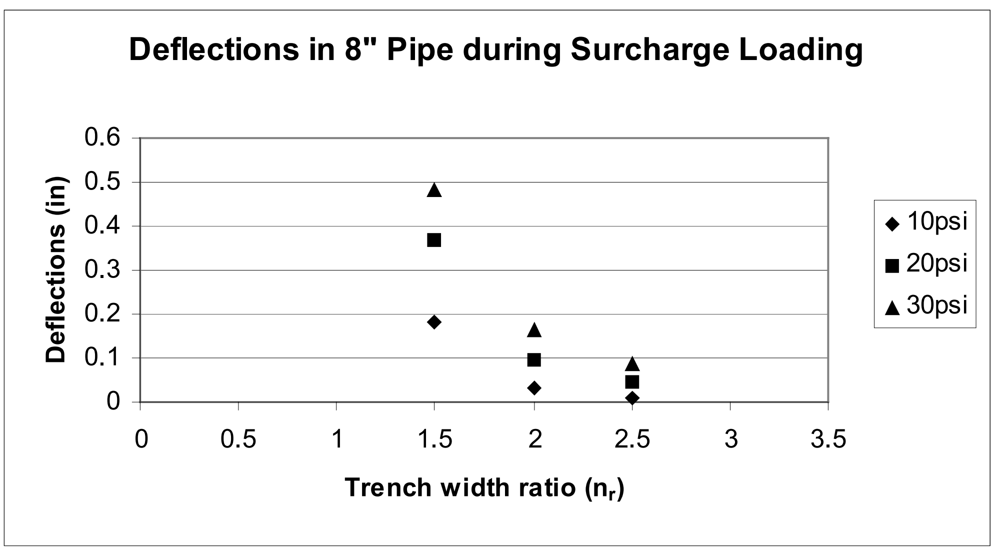
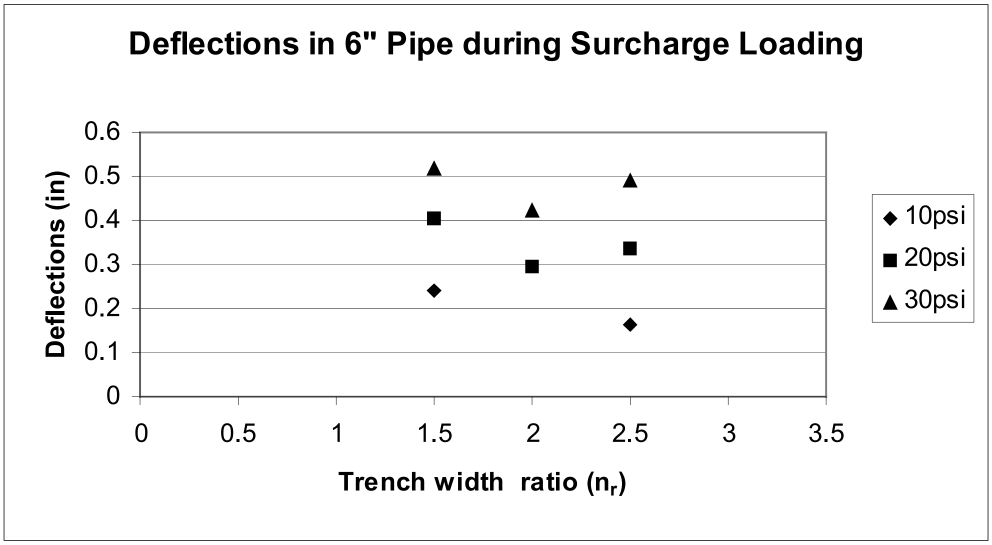


Figure 5.2: Deflections of Pipe in Cohesive In-Situ Soil With High Strength Backfill

Deflections Under Uniform Surcharge Loading recorded in inches

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	0.2398	0.212	0.4024	0.416	0.5172	0.596
2		0.243	0.2966	0.455	0.4247	0.6294
2.5	0.1647	0.0792	0.3354	0.2561	0.4908	0.4085
3		0.121		0.325		0.499



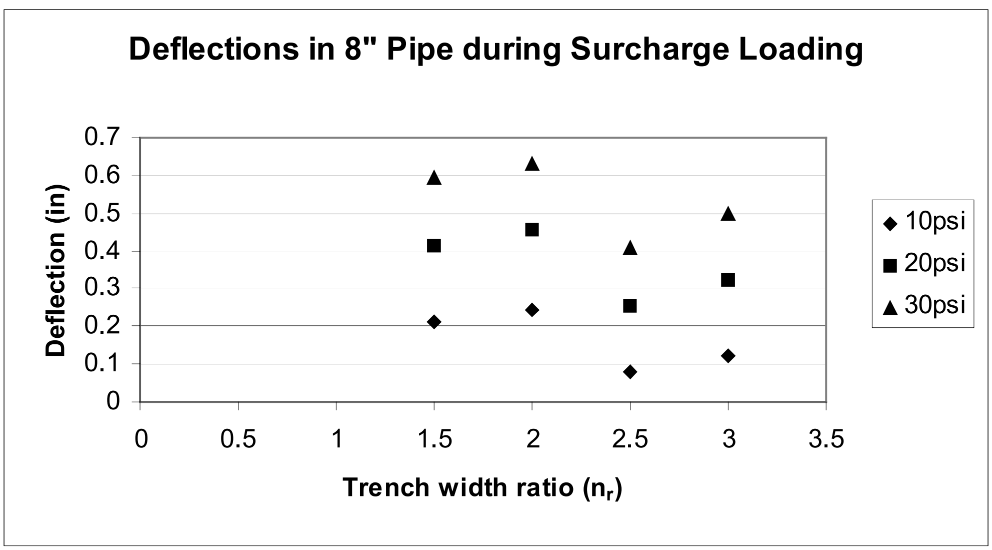
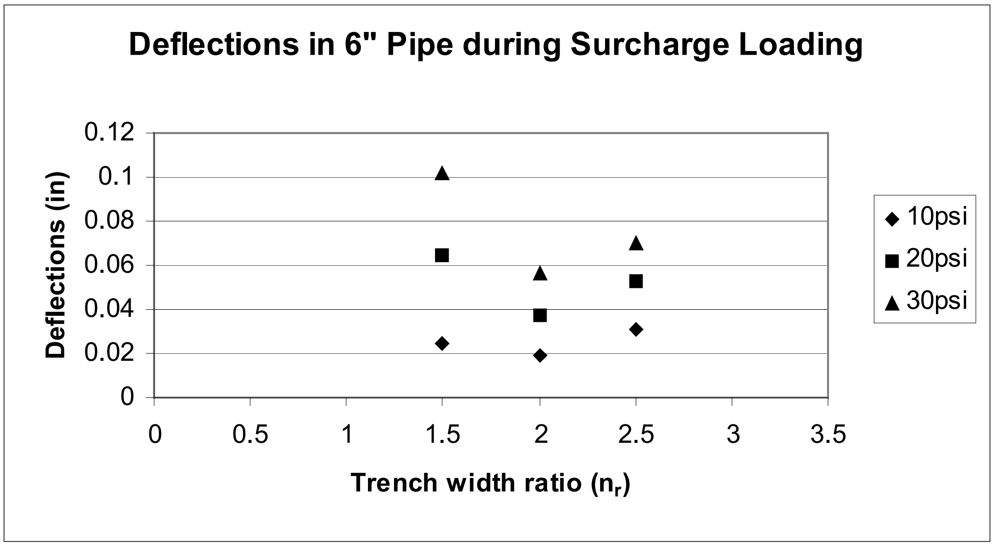


Figure 5.3: Deflections of Pipe in Loose In-Situ Soil With Low Strength Backfill

Deflections Under Uniform Surcharge Loading recorded in inches

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	0.0245	0.029	0.0649	0.127	0.102	0.1327
2	0.0194	0.0359	0.0377	0.0922	0.0566	0.1425
2.5	0.031	0.0269	0.0527	0.0671	0.0699	0.106
3						



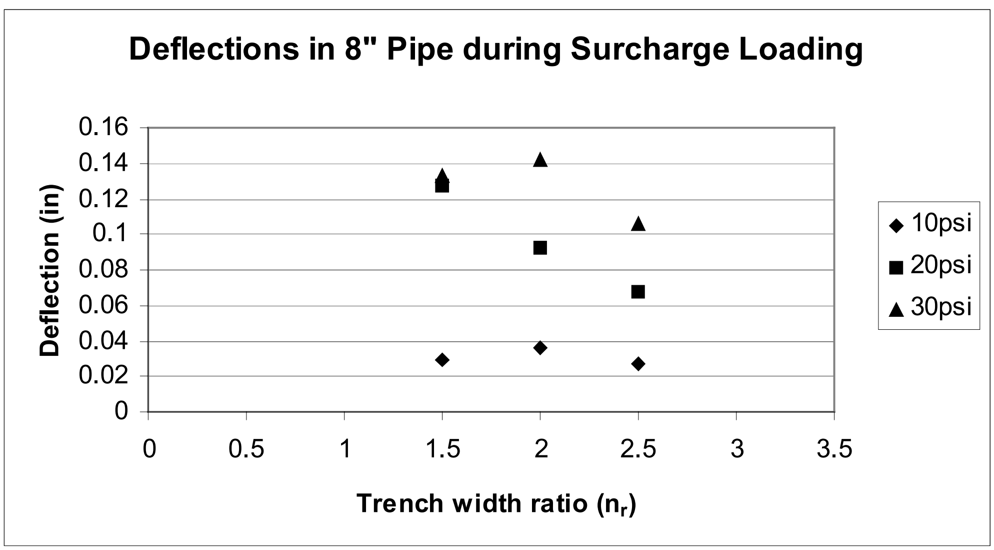


Figure 5.4: Deflections of Pipe in Loose In-Situ Soil with High Strength Backfill

width ratio (n_r). The trench width ratio is defined as the trench width divided by the pipe diameter.

From the preceding graphs and tables several conclusions can be drawn about the relationships between the variables. Figures 5.1 through 5.4 show a definite trend for the pipe deflections to decrease with an increase in trench width. This is as expected, because a larger trench width provides more structural integrity than does a smaller trench width. When using the Spangler's Iowa equation (Howard 1996) to predict deflections, a wider trench width will result in less deflection. This equation and its use are presented in more detail in the following chapter. One interesting feature of these graphs is that the slopes of the trend lines are very similar for all of the different configurations. The average slope of all of the trend lines at 20 psi (138 Pa) in Figures 5.1 through 5.4 is 0.062. The range of values for the slope is from 0.04 to 0.095. With more testing of different pipe diameters a reliable correlation between the trench width and the pipe deflections can be obtained.

These figures also show the performance of the pipes with respect to the 5% deflection limit. Every test conducted with low strength CLSM at trench width ratios of 1.5 had deflections greater than the 5% limit at 30 psi (207 Pa) surcharge loading. However, 30 psi (207 Pa) is more surcharge than most pipes will ever carry in service so this information does not exclude trench width ratios of 1.5 from service. At surcharge pressures of 20 psi (138 Pa) only 3 out of 8 mixtures showed 5 % deflection at a trench width ratio of 1.5 and only 1 out of 8 mixtures showed 5 % deflections at a trench width ratio of 2.0. Both tests that combined the low strength CLSM with the low strength in-situ soil had deflections greater than the 5% limit at every trench width ratio when the surcharge pressure was increased to 30 psi (207 Pa).

Most pipes will not even carry 20 psi (138 Pa) of surcharge in service (Watkins and Anderson 2000). At a surcharge loading of 20 psi (138 Pa) there was only one experimental set up (Figure 5.1, 8 inch pipe, n_r of 1.5) where a pipe had a deflection that was greater than 7 %. There is a current effort to increase the allowable flexible pipe deflections to 7 % for design purposes (Gabriel 1998). If the limit was at 7 % then the performance of all of the mixtures at any trench width can be considered as very good.

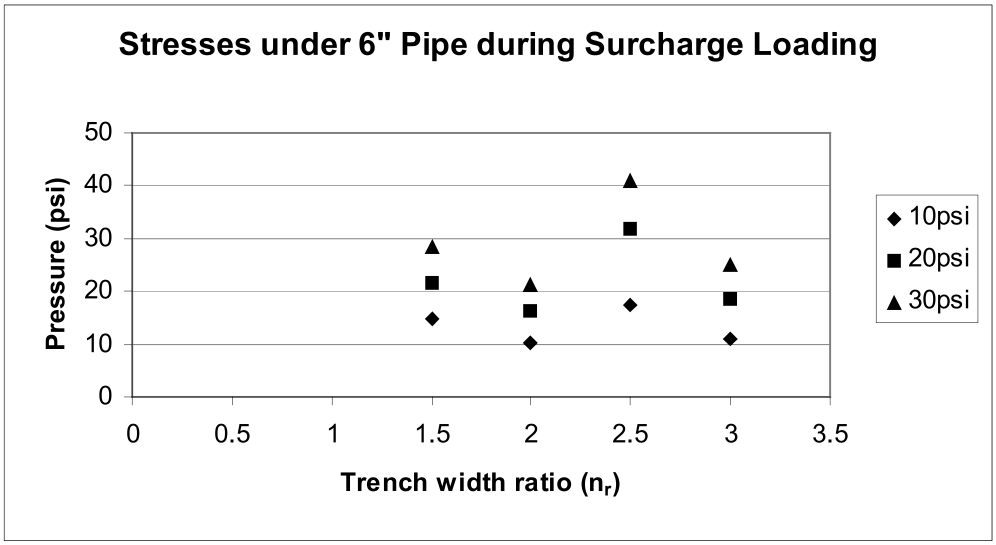
5.3 SOIL STRESSES DUE TO SURCHARGE LOADING

Appendix C also contains the data for soil stresses due to surcharge loading. The following figures (Figures 5.5 through 5.8) also show families of curves for surcharge loading between 10 psi (69 Pa) and 30 psi (207 Pa) but these figures show the soil stresses under the centerline of the pipe compared with the trench width ratio. The centerline soil stress decreases with an increase in trench width ratio. The theory of arching indicates that when a material in a trench deflects the frictional forces on the interface between the in-situ soil and the backfilled material will carry some of the surface load (Watkins and Anderson 2000). This reduction in stress can actually cause negative pressures near the crown of the pipe and lead to tension cracks. However, in the case of flowable fill the backfilled material is much stronger and deflects less than the in-situ soil. This leads to a phenomenon that is opposite that of arching. In this case, the in-situ soil is deflecting and the CLSM is taking on additional loads due to the friction caused by the moving mass of in-situ soil. The loads in the CLSM are being amplified rather than reduced.

In a finite area, such as the area of the pipe testing apparatus used in this research, as the trench width is increased the area of in-situ soil that is under pressurized surface loading must decrease. Since the pressurized area of the in-situ soil is decreasing so is the frictional forces that the in-situ soil is applying to the CLSM. Arching theory is applied when there are two soil masses that settle at different rates. In the case of CLSM, the load distributions are opposite of what is expected from conventional arching theory. The in-situ soil is applying additional load to the backfill rather than relieving some of the load. This could have implications for a fairly weak CLSM in an in-situ soil mass that will have significant settlements. A fairly weak CLSM with a small trench width might be carrying much more load than is expected because of the interparticle friction of the settling in-situ soil. This could lead to high deflections or even pipe failure if the CLSM is not designed to carry these additional loads.

Soil Stresses Under Uniform Surcharge Loading recorded in psi

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	14.65	10.78	21.78	16.27	28.36	
2	10.22	14.42	16.16	19.82	21.21	24.35
2.5	17.28	10.21	31.87	15.98	40.94	20.89
3	11.07		18.62		24.85	



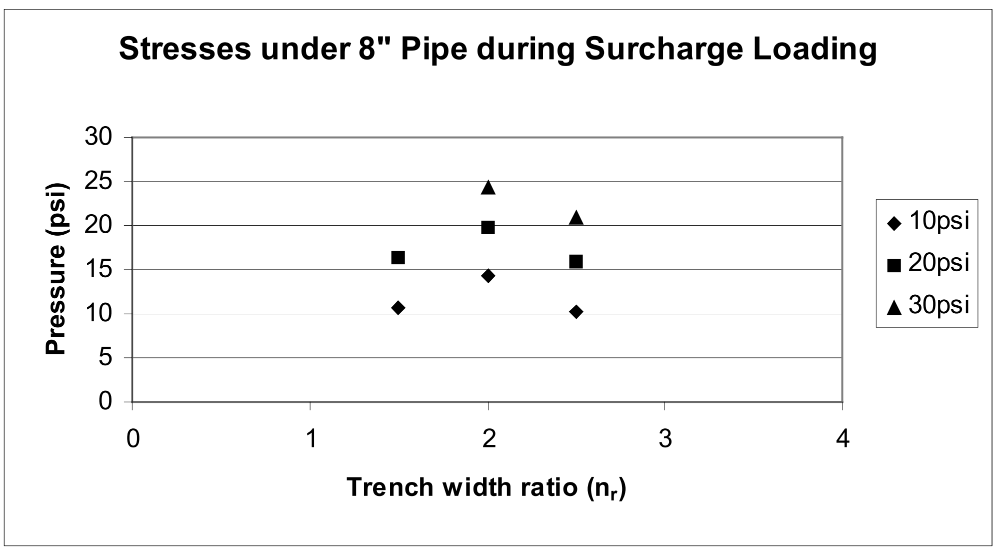
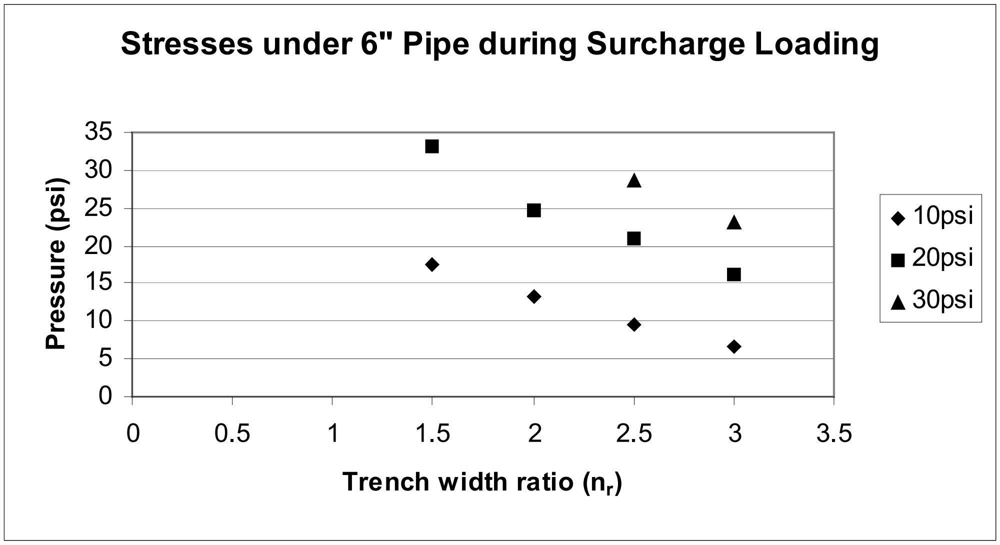


Figure 5.5: Soil Stresses in Cohesive In-Situ Soil With Low Srength Backfill

Soil Stresses Under Uniform Surcharge Loading recorded in psi

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	17.39	15.66	33.12	23.79		29.4
2	13.15	7.85	24.73	14.67		19.6
2.5	9.6	4.54	20.87	8.55	28.72	10.61
3	6.64		16.06		23.03	



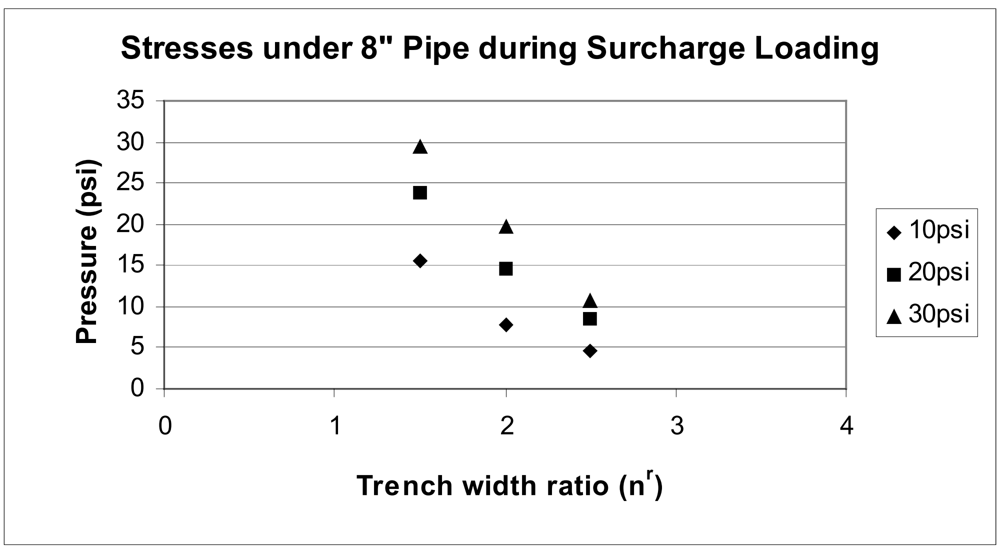
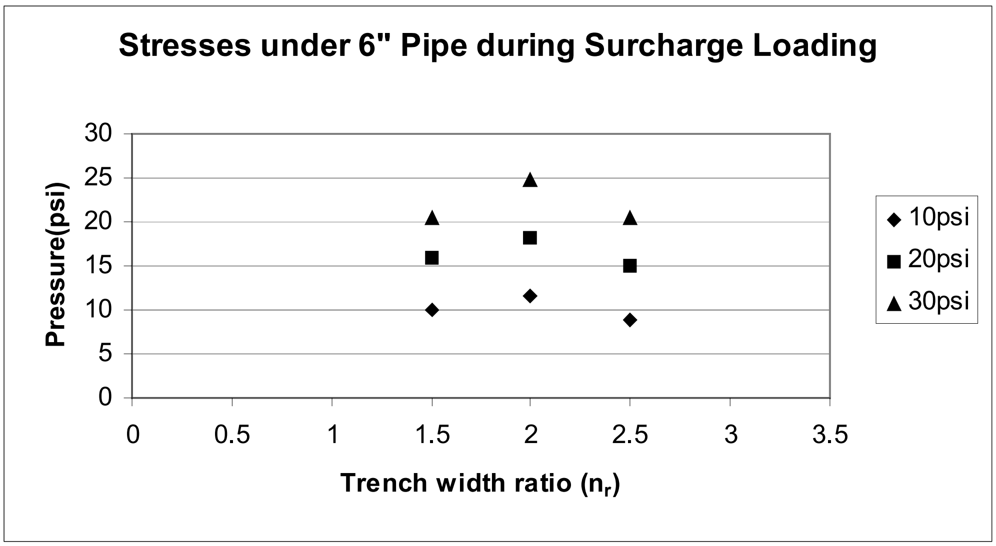


Figure 5.6: Soil Stresses in Cohesive In-Situ Soil With High Strength Backfill

Soil Stresses Under Uniform Surcharge Loading recorded in psi

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	10.05	10.2	15.92	16.29	20.5	21.71
2	11.65	10.09	18.29	15.45	24.78	19.59
2.5	8.93	7.73	14.95	13.26	20.45	17.26
3		7.55		12.13		16.97



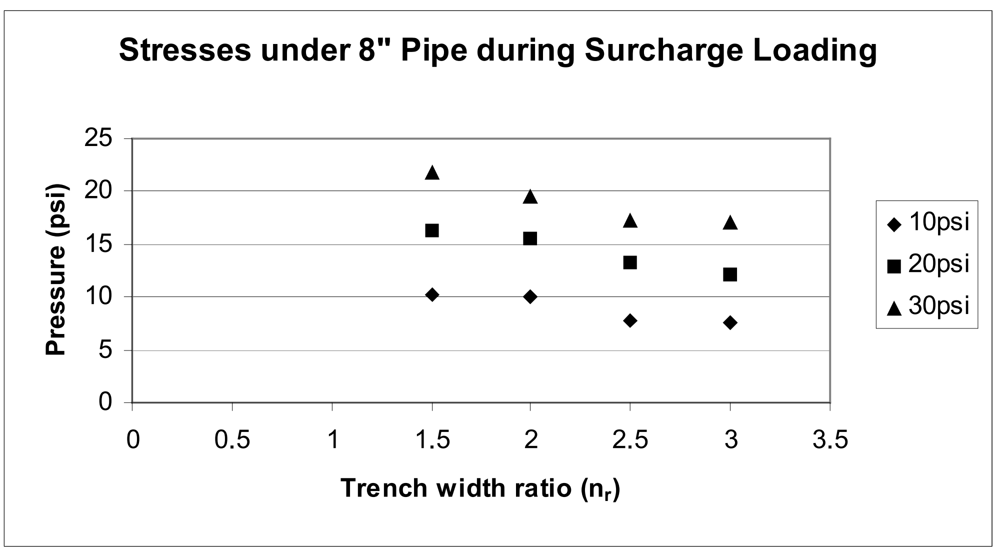
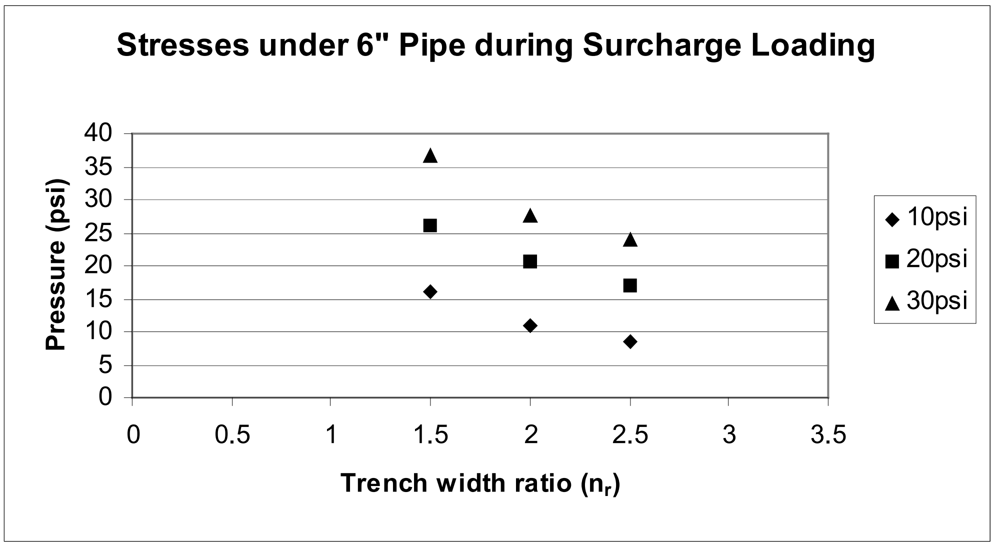


Figure 5.7: Soil Stresses in Loose In-Situ Soil With Low Strength Backfill

Soil Stresses Under Uniform Surcharge Loading recorded in psi

	10psi (69 Pa)		20psi (138 Pa)		30psi (207 Pa)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	16.09	9.18	26.07	15.94	36.57	21.65
2	11.04	15.18	20.5	26.83	27.6	32.17
2.5	8.43	9.83	16.91	18.24	24.06	23.49
3						



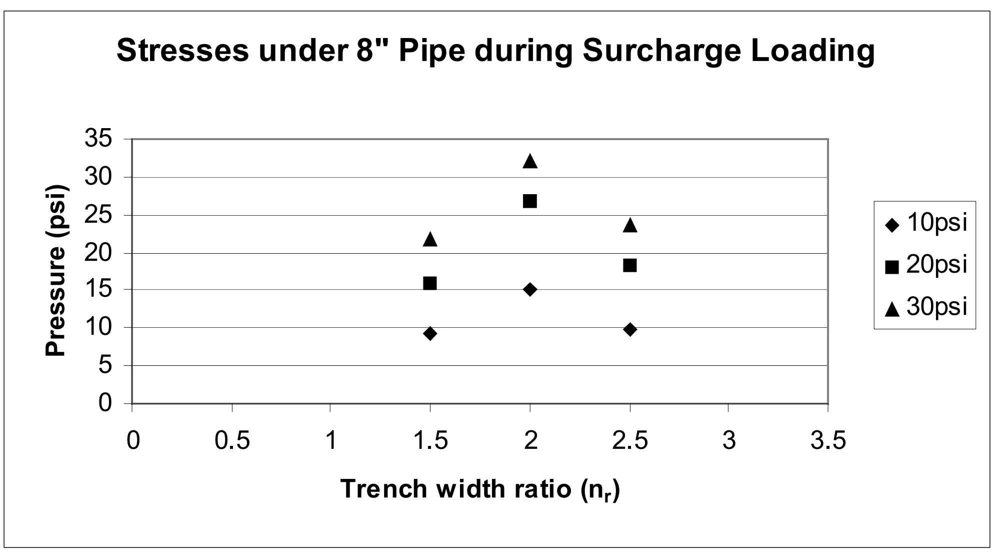


Figure 5.8: Soil Stresses in Loose In-Situ Soil With High Strength Backfill

5.4 SOIL STRESSES DUE TO DIRECT LOADING

Figures 5.9 through 5.12 show the measured centerline soil stresses while direct loading. There are eight graphs with families of curves for loads of 1,000 lb (4,448 N), 2,000 lb (8,896 N), and 3,000 lb (13,344 N) that show the centerline soil stresses versus the trench width ratio. However, this information is not as valuable because of the complex interactions that have occurred before the direct loading took place. Since the direct loading always took place the day after the pressure testing, the centerline stresses and deflections that happen during direct loading depend heavily on the deflections and rebounding that happened during and after the pressure test. As expected, the centerline soil stresses increase as the applied load increases, but these figures primarily show that there is no statistically significant correlation between direct loading, trench widths, and soil stresses. However, one interesting feature is made clear if Figures 5.5 through 5.8 are compared to Figures 5.9 through 5.12. Comparing these figures shows that the measured stresses during direct loading tend to mirror the measured stresses during the surcharge loading. This supports the fact that measurements taken during direct loading depend heavily on the interactions that occurred during the surcharge loading.

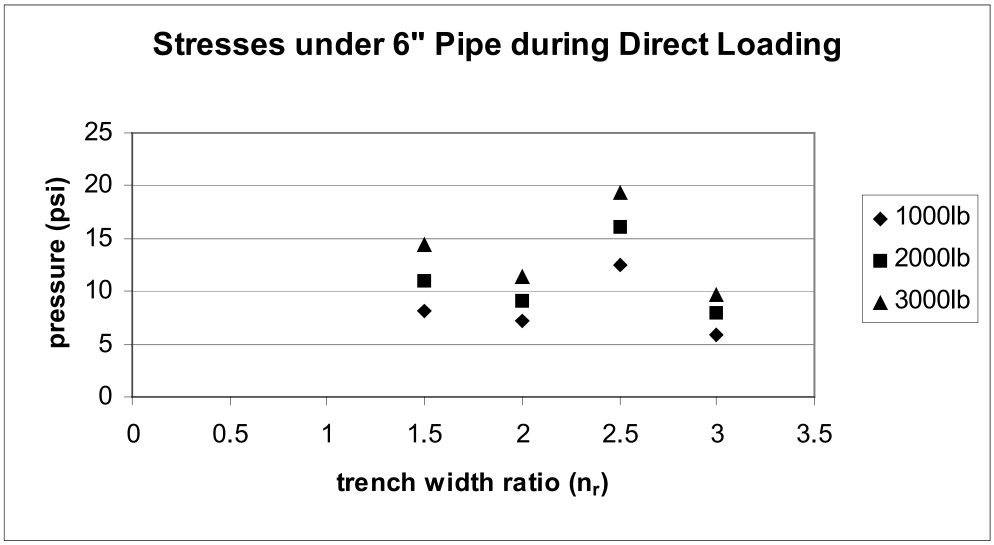
5.5 SUMMARY OF RESULTS

Table 5.1a shows the maximum and minimum deflections for all experimental configurations while surcharge loading. Table 5.1b shows the maximum and minimum soil stresses for all experimental configurations while surcharge loading. Except for one case, the n_r value corresponding to the maximum deflection was always smaller than the n_r value corresponding to the minimum deflection. The one exception occurred with a 6 inch (15.3 cm) pipe with high strength CLSM and cohesive soil, the strongest scenario. In this particular set up the difference between the maximum and the minimum deflections was only a couple hundredths of an inch. Furthermore, the measured deflections were about one-tenth of the measured deflections for tests that used low strength CLSM. This reinforces the statement that larger trench widths have smaller deflections

Except for one case, the n_r value corresponding to the maximum soil stress is always smaller than the n_r value corresponding to the minimum soil stress, as seen in Table 5.1b. This exception occurred when there was a high strength CLSM and loose fill.

Soil Stresses Under Direct Loading recorded in psi

	1000 lb (4448 N)		2000 lb (8896 N)		3000 lb (13,344 N)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	8.15	5.57	10.95	6.53	14.46	6.92
2	7.14	8.99	9.05	10.15	11.44	10.26
2.5	12.46	7.19	16.03	8.97	19.29	10.88
3	5.85		7.87		9.59	



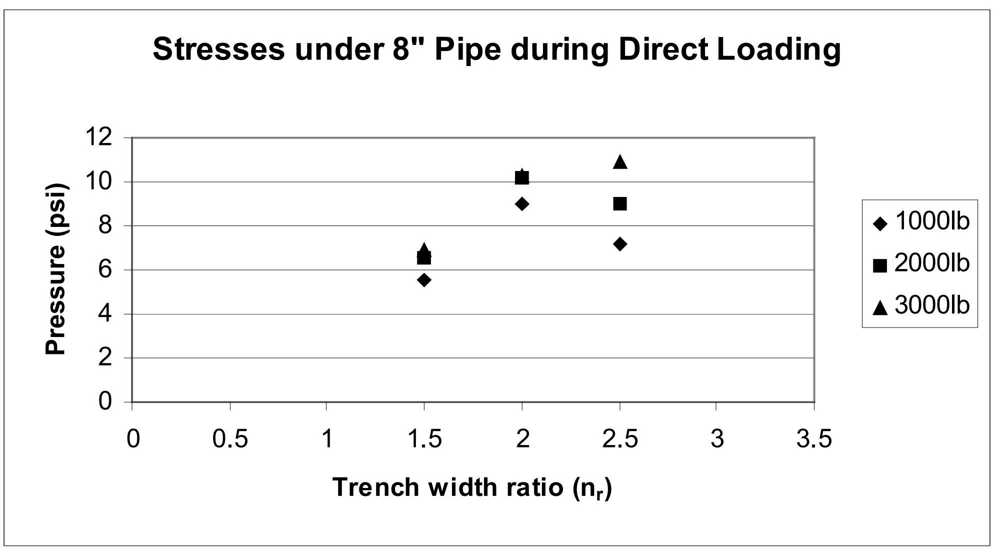
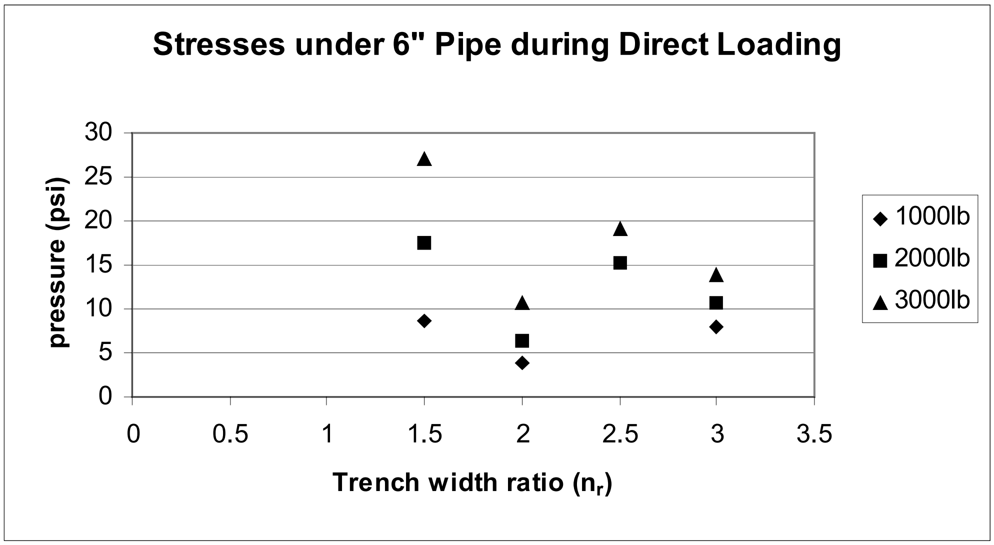


Figure 5.9: Soil Stresses in Cohesive In-Situ Soil With Low Srength Backfill During Direct Loading

Soil Stresses Under Direct Loading recorded in psi

	1000 lb (4448 N)		2000 lb (8896 N)		3000 lb (13,344 N)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	8.63	8.94	17.45	11.67	27.05	13.53
2	3.86	6.53	6.33	8.32	10.69	10.83
2.5		4.13	15.24	6.13	19.04	8.25
3	8		10.67		13.91	



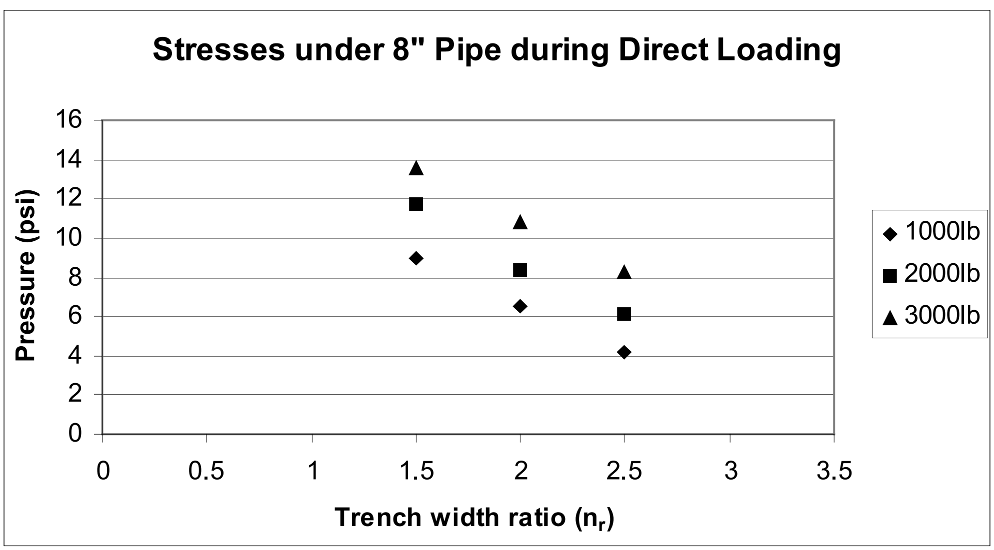
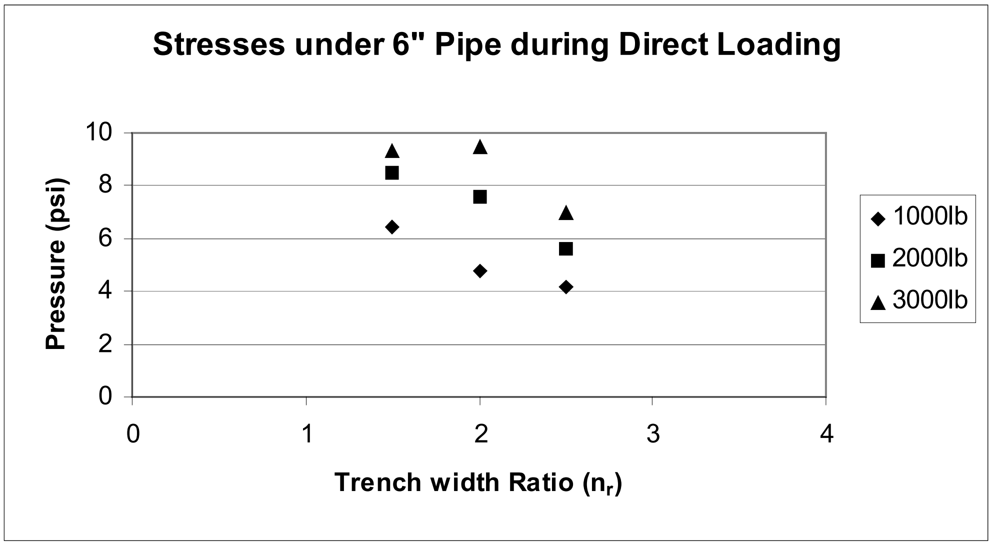


Figure 5.10: Soil Stresses in Cohesive In-Situ Soil With High Strength Backfill During Direct Loading

Soil Stresses Under Direct Loading recorded in psi

	1000 lb (4448 N)		2000 lb (8896 N)		3000 lb (13,344 N)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	6.46	3.79	8.45	5.15	9.35	6.82
2	4.78	5.3	7.6	6.41	9.49	6.86
2.5	4.17	2.91	5.6	3.62	6.96	3.87
3		2.8		3.6		4.21



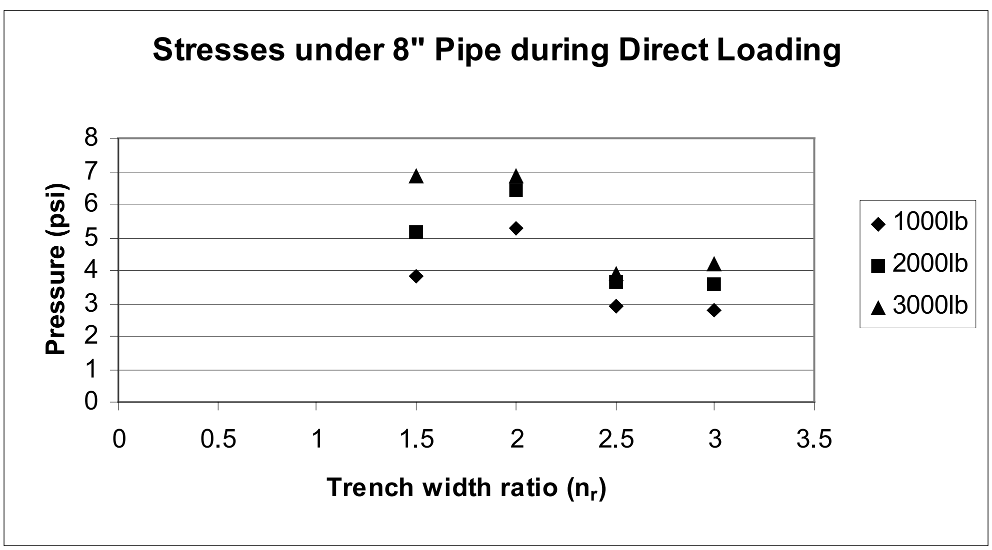
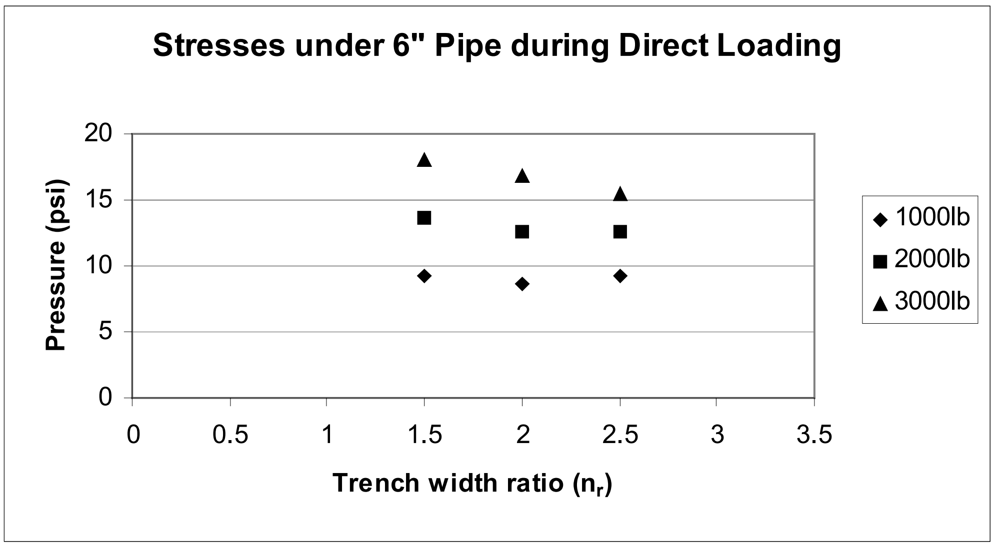


Figure 5.11: Soil Stresses in Loose In-Situ Soil With Low Strength Backfill During Direct Loading

Soil Stresses Under Direct Loading recorded in psi

	1000 lb (4448 N)		2000 lb (8896 N)		3000 lb (13,344 N)	
trench width ratio n_r	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe	6 inch pipe	8 inch pipe
1.5	9.24	8.3	13.7	11.1	18	13.07
2	8.61	11.85	12.65	14.99	16.88	15.4
2.5	9.3	8	12.51	10.74	15.5	13.59
3						



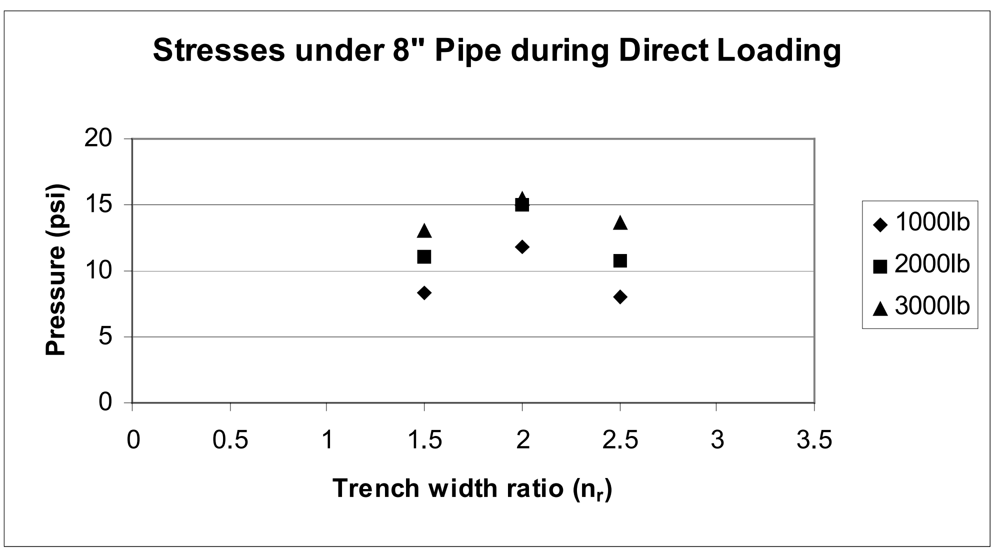


Figure 5.12: Soil Stresses in Loose In-Situ Soil With High Strength Backfill During Direct Loading

Table 5.1 Summary of Surcharge Loading Maximums and Minimums

a.) Maximum and Minimum Pipe Deflections

Pipe Diameter Inch (cm)	CLSM Strength	In-Situ Soil Type	Max Deflection (in)	Min Deflection (in)
6 (15)	High	Cohesive	0.069 n_r=2	0.028 n_r=1.5
6 (15)	High	Loose	0.102 n_r=1.5	0.057 n_r=2
6 (15)	Low	Cohesive	0.324 n_r=1.5	0.141 n_r=2.5
6 (15)	Low	Loose	0.517 n_r=1.5	0.425 n_r=2
8 (20)	High	Cohesive	0.482 n_r=1.5	0.162 n_r=2
8 (20)	High	Loose	0.143 n_r=2	0.106 n_r=2.5
8 (20)	Low	Cohesive	Failure n_r=1.5	0.217 n_r=2
8 (20)	Low	Loose	0.629 n_r=2	0.409 n_r=2.5

b.) Maximum and Minimum Soil Stresses

Pipe Diameter Inch (cm)	CLSM Strength	In-Situ Soil Type	Max Soil Stress (psi)	Min Soil Stress (psi)
6 (15)	High	Cohesive	29.60 n_r=1.5	19.60 n_r=2
6 (15)	High	Loose	32.17 n_r=2	21.65 n_r=1.5
6 (15)	Low	Cohesive	24.35 n_r=2	20.89 n_r=2.5
6 (15)	Low	Loose	21.71 n_r=1.5	16.97 n_r=3
8 (20)	High	Cohesive	33.12 n_r=1.5	16.02 n_r=3
8 (20)	High	Loose	36.57 n_r=1.5	24.06 n_r=2.5
8 (20)	Low	Cohesive	40.94 n_r=2	21.21 n_r=2.5
8 (20)	Low	Loose	24.80 n_r=2	20.50 n_r=2.5

This set up would maximize the reverse arching that was discussed in the previous sections because there is a high amount of differential settlement between the weak in-situ soil and the strong CLSM. Tables 5.1a and 5.1b also show that while most of the maximum deflections occurred at a trench width ratio of 1.5, only half of the maximum centerline stresses occurred at a trench width ratio of 1.5. This shows that the maximum deflection and maximum soil stress do not happen under the same conditions. While the smallest trench widths allow the most deflections, the reverse arching is not as pronounced because there is not as much differential settlement between the two soil masses. The centerline soil stress is not necessarily maximized when deflections are maximized. There is not as much difference between the settlements of the in-situ soil and the CLSM at the smallest trench width so the frictional forces between them is not as great.

All of the deflection and centerline soil stress maximums that did not occur at a trench width ratio of 1.5 occurred at trench width ratios of 2.0. As seen in Tables 5.1a and 5.1b, there was never a case where either the deflections or centerline stresses were maximum at a trench width ratio of 2.5 or 3.0. This is a strong basis to support the fact that both stresses and deflections should decrease as the trench width ratio is increased. While this research shows that the largest trench width ratio consistently performed better than the smaller trench width ratios, all of the trench width ratios adequately supported the pipe.

Table 5.1a shows that only one pipe test out the twenty-six pipe tests conducted had a failure (also shown in figure Figure 5.1). This failed specimen had a large crack form over the centerline of the pipe and two transverse cracks that propagated outward from the springline of the pipe. This effectively broke the hardened column of CLSM into three separate pieces. Most of the low strength CLSM mixtures had small micro cracks form over the centerline of the pipe, but no other visible cracking occurred. Figure 5.13 shows a mixture with micro cracks that did not have a failure.

Figure 5.13: CLSM with Micro Cracks Over the Centerline

CHAPTER 6
ANALYSIS OF RESULTS

This chapter involves a numerical analysis and a statistical analysis of the results from pipe testing. Spangler's Iowa equation (Howard 1996) is used to predict pipe deflections and those predictions are compared to the measured pipe deflections. In addition, the measured pipe deflections are used in the Iowa equation to calculate the stiffness of the CLSM. Lastly, the predicted and measured deflections are compared graphically and the correlation of these numbers is discussed.

6.1 Numerical Analysis

The form of the Iowa equation that was used in the analysis of the results can be found in Equation 6.1 (Howard 1996). This equation finds the vertical deflection as a percentage of the pipe diameter. To use this equation, one of three rules must be satisfied. The first rule is that the in-situ soil should be as strong as the backfill material or stronger than the backfill material. This rule is certainly not satisfied. The second rule is that the trench width should be at least three times the pipe diameter. Only two of the twenty-six experiments satisfied this rule. The third rule states that if the first two rules are not satisfied then a composite soil stiffness needs to be calculated. Equation 6.2 is the equation used for finding the composite soil stiffness.

$$\Delta Y = \frac{T_f (0.07)\gamma h}{EI/r^3 + 0.061 F_d E'} \qquad \text{(Eq. 6.1)}$$

Where,

ΔY = percent vertical deflection

T_f = time lag factor

0.07 = combination of conversion factors including a bedding factor of 0.1

γ = backfill unit weight, lb/ft^3

h = depth of cover, ft

EI/r^3 = pipe stiffness factor, psi

0.061 = constant developed for the Iowa equation

F_d = design factor, dimensionless

E' = Modulus of soil reaction, psi

The terms T_f and F_d are dimensionless. The time lag factor (T_f) varies from 1.5 to 2.5 and is used to account for the plastic stress relaxation that is common in plastic pipes (Gabriel 1998). Since no long-term tests were done in this research, T_f is not needed. The minimum recommended value of 1.5 was used in all of the calculations. The design Factor (F_d) varies from 0.5 to 1 and it can account for how effectively the backfill material is placed. Since CLSM flows into all the void spaces and is much more homogeneous than compacted fill, the F_d term is always assumed to be 1 for CLSM.

There are two other dimensionless constants, the first one listed (0.07) comes from many conversion factors. Adjusting any of the conversion factors can change this number. One the conversion factors that is commonly adjusted is the bedding factor. A bedding factor value of 0.1 was used, which represents the best type of bedding. The bedding factor accounts for how well the compacted fill supports the haunches of the pipe. Since flowable fill flows around the pipe and fills all voids under the haunches of the pipe, flowable fill always uses a bedding factor of 0.1. The last unitless constant (0.061) was developed by Spangler to improve the accuracy of this empirical equation. This last constant cannot be changed. All of the unitless constants are derived from using the Iowa equation with English units. To use the equation correctly all of the variables must use the units listed.

The term (γh) is the total vertical stress above the pipe. Since an additional surface loading was applied in the experiments, the applied surface loads were added to the vertical stress. The term EI/r^3 defines the strength of the pipe. Earlier in this report the modulus of elasticity was reported as 94 ksi (648 kPa) and 67.2 ksi (463 kPa) for 6 inch (15.2 cm) and 8 inch (20.3 cm) pipes, respectively. The moment of inertia was supplied by the pipe manufacturers as 0.002 in^4 (0.083 cm^4) for the 6 inch (15.2 cm) pipe and 0.005 in^4 (0.208 cm^4) for the 8 inch (20.3 cm) pipe. The following pipe stiffness factors (EI/r^3) were calculated: 6.96 lb/in for the 6 inch (15.2 cm) pipe, and 5.25 lb/in for the 8 inch (20.3 cm) pipe. The units on these two pipe stiffness values must be in pounds per inch to work in Equation 6.1.

The last term, The modulus of soil reaction (E'), has to be a composite value between the CLSM stiffness and the in-situ soil stiffness because the trench widths are less than three diameters for most cases, and the in-situ soil is always weaker than the backfill material. These were the three rules used for applying Spangler's Iowa equation that were discussed earlier in this chapter. The composite modulus of soil reaction can be calculated using Equation 6.2 as (Howard 1996):

$$E'_{com} = SE' \qquad \text{(Eq. 6.2)}$$

Where,

E'_{com} = the composite modulus of soil reaction (psi)

S = a reduction factor (unitless)

E' = CLSM modulus of soil reaction (psi)

6.2 PREDICTED DEFLECTIONS

Table 6.1 shows the values for E'_{com} that were calculated. The S values were obtained from available literature (Howard 1996). The recommended values for CLSM modulus of soil reaction are 3,000 psi and 25,000 psi for low and high strength CLSM, respectively (Howard 1996). The in-situ soil strength was assumed as 1,000 psi for the loose sand and 2,000 psi for the compacted cohesive soil (Howard 1996). The calculated composite modulus of soil reaction values are used to calculate the predicted deflections.

Table 6.2 shows a comparison between the predicted pipe deflections and the measured pipe deflections. This table shows a close correlation between the measured deflections and the predicted deflections. Figure 6.1 and Figure 6.2 graphically show the correlation between these variables. These figures both show that the predicted deflections are much closer to the measured deflections at smaller values, but as the values increase the measured deflections become much larger than the predicted deflections. This is expected because the Iowa equation has been shown to be less effective when the deflections exceed 5% (Watkins and Anderson 2000). The two figures show that this relationship can be closely approximated with a parabolic curve. The correlation between the measured displacements and the predicted displacements can be considered as excellent.

Table 6.1 E'com Values For the Different Geometries

E' high strength CLSM = 25,000 psi (assumed)

E' low strength CLSM = 3,000 psi (assumed)

E'_n loose sand = 1,000 psi

E'_n compacted cohesive soil = 2,000 psi

CLSM Stiffness	soil stiffness	E'_n/E'	trench width ratio (n_r)	S	E'_{com} (psi)
high	high	0.08	1.5	0.15	3,750
high	high	0.08	2	0.3	7,500
high	high	0.08	2.5	0.6	15,000
high	low	0.04	1.5	0.1	2,500
high	low	0.04	2	0.23	5,750
high	low	0.04	2.5	0.55	13,750
low	high	0.66	1.5	0.75	2,250
low	high	0.66	2	0.87	2,610
low	high	0.66	2.5	0.91	2,730
low	low	0.33	1.5	0.53	1,590
low	low	0.33	2	0.63	1,890
low	low	0.33	2.5	0.82	2,460

Table 6.2 Predicted and Measured Deflections at 20 psi

CLSM Stiffness	soil stiffness	trench width ratio	8 inch (20.3 cm) pipe		6 inch (15.2 cm) pipe	
			Predicted deflections (in)-[cm]	Measured deflections (in)-[cm]	Predicted deflections (in)-[cm]	Measured deflections (in)-[cm]
high	high	1.5	0.105 [0.267]	0.369 [0.939]	0.078 [0.199]	0.028 [0.071]
high	high	2	0.053 [0.135]	0.097 [0.247]	0.040 [0.101]	0.069 [0.175]
high	high	2.5	0.027 [0.068]	0.045 [0.115]	0.020 [0.051]	0.037 [0.094]
high	low	1.5	0.156 [0.396]	0.064 [0.165]	0.116 [0.294]	0.127 [0.323]
high	low	2	0.069 [0.176]	0.037 [0.096]	0.052 [0.131]	0.092 [0.234]
high	low	2.5	0.029 [0.074]	0.052 [0.134]	0.022 [0.055]	0.067 [0.170]
low	high	1.5	0.173 [0.439]	0.727 [[1.847]	0.128 [0.325]	0.225 [0.574]
low	high	2	0.150 [0.380]	0.151 [0.384]	0.111 [0.282]	0.152 [0.387]
low	high	2.5	0.143 [0.364]	0.131 [0.333]	0.106 [0.270]	0.096 [0.246]
low	low	1.5	0.241 [0.611]	0.416 [1.057]	0.178 [0.451]	0.402 [1.022]
low	low	2	0.204 [0.519]	0.455 [1.156]	0.151 [0.384]	0.296 [0.753]
low	low	2.5	0.158 [0.403]	0.256 [0.650]	0.118 [0.300]	0.335 [0.852]

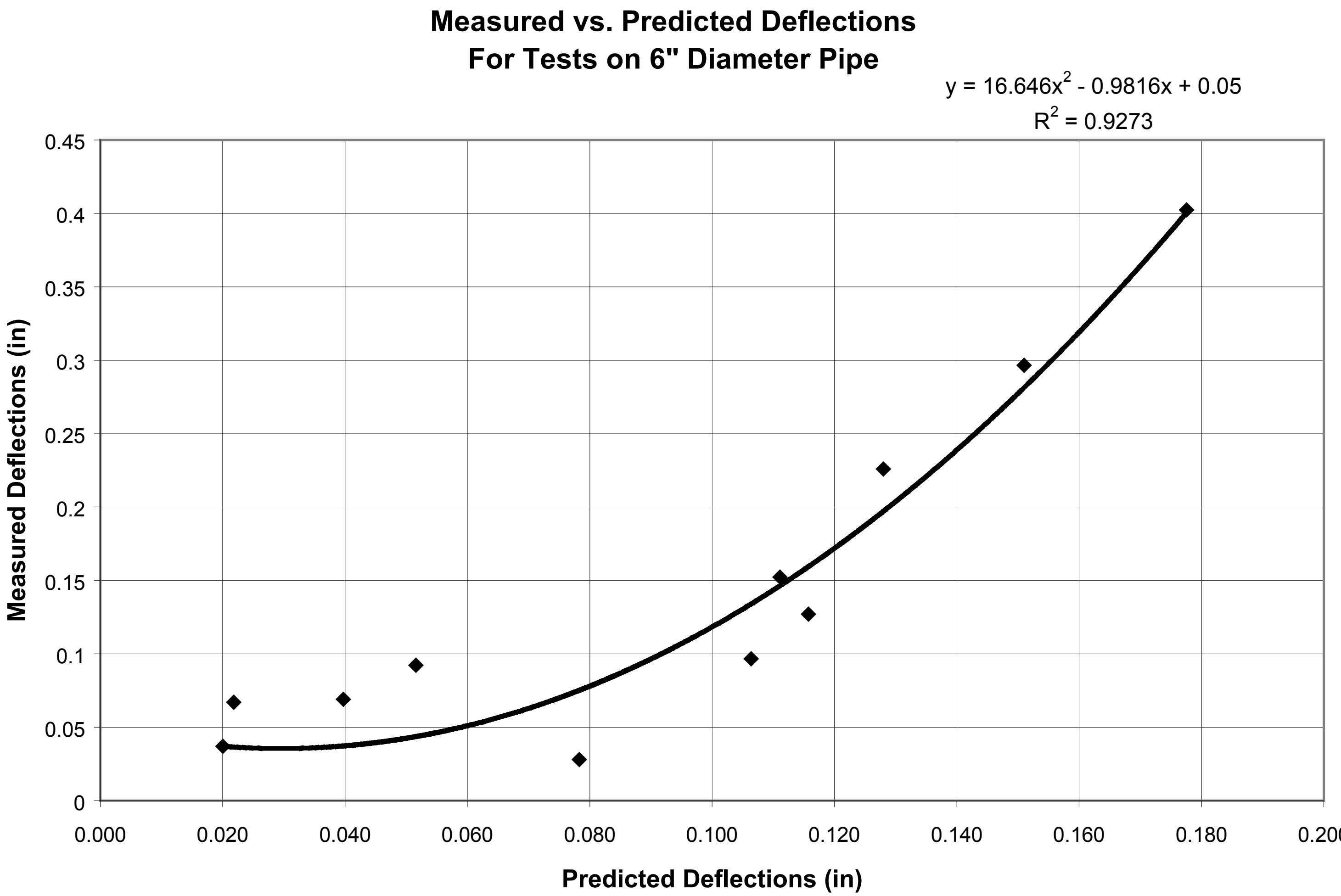


Figure 6.1: Measured versus Predicted Deflections for Tests on 6 inch Pipe

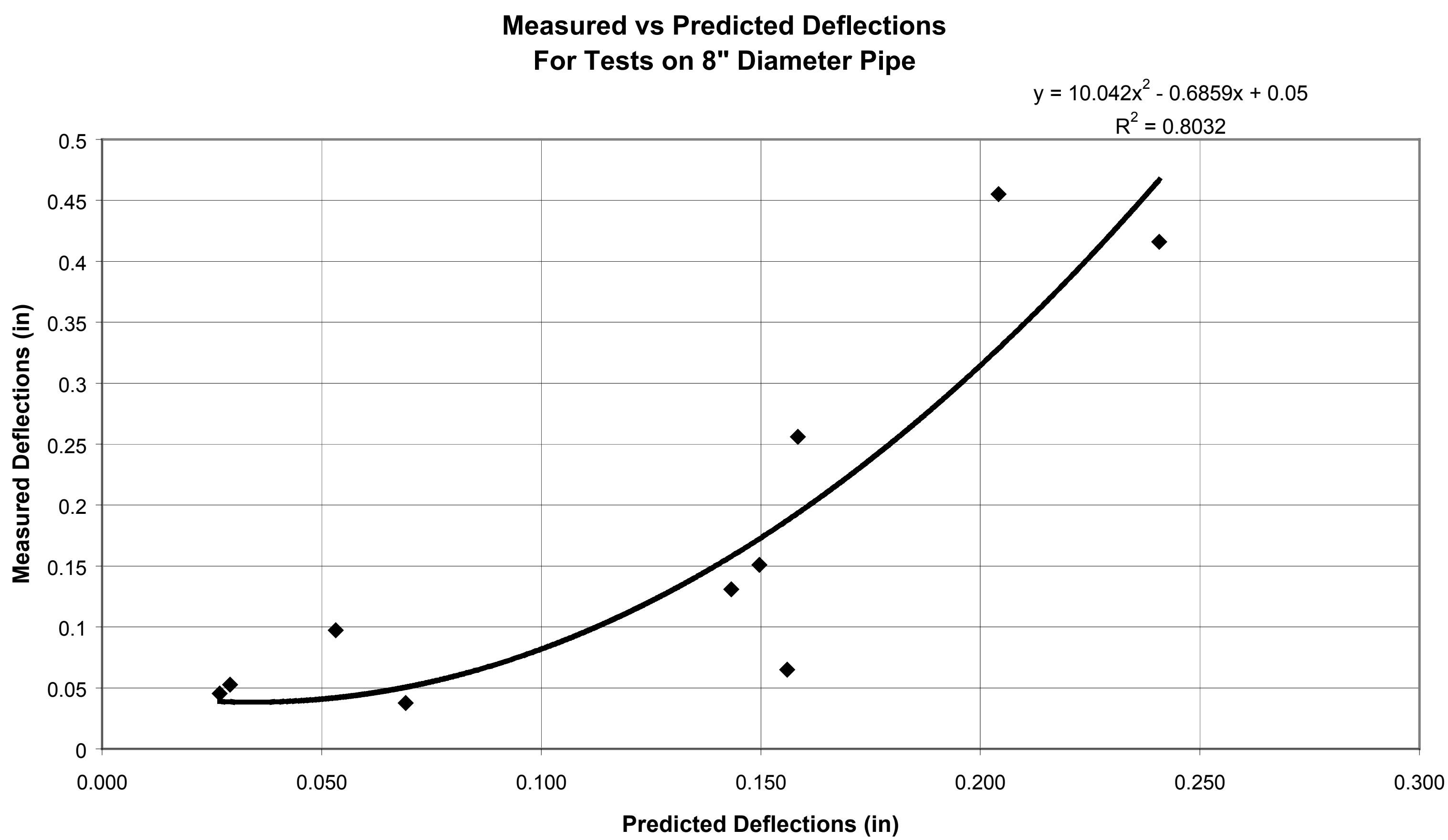


Figure 6.2: Measured versus Predicted Deflections for Tests on 8 inch Pipe

6.3 CALCULATED MODULUS OF SOIL REACTION

The high strength CLSM mixtures tend to have measured deflections that were greater than the predicted values. This is mainly due to the assumed values for the modulus of soil reaction. However, observations in this research can be used to estimate the actual E′ values of the two flowable fill mixtures by back calculating in the Iowa equation. The results of these calculations are found in Table 6.3. These calculations show that the low strength CLSM had an average modulus of soil reaction of 3,000 psi (20,685 Pa), which was the same as the recommended value. The range of back-calculated values fall between 1,000 psi (6895 Pa) and 7,500 psi (51,713 Pa).

The high strength CLSM had slightly higher deflections than the expected values. The back calculated modulus of soil reaction was determined to be 19,200 psi (132,384 Pa). This value should be below the recommended value of 25,000 psi (172,375 Pa) since the deflections were greater than expected. The range of back-calculated values for E' fall between 7,000 psi (48,265 Pa) and 65,000 psi (448,175 Pa) for all of the high strength CLSM mixtures that did not have failure. The calculated values of E' would be useful in setting up a computer simulation of the pipe testing program.

Table 6.3 Back Calculated Values for E'

CLSM stiffness	soil stiffness	trench width ratio	8 inch (20.3 cm) pipe			6 inch (15.2 cm) pipe		
			measured deflection (in)	calculated E'com (psi)	CLSM E' (psi)	measured deflections (in)	calculated E'com (psi)	CLSM E' (psi)
high	high	1.5	0.370	1,005	1,000	0.028	10,693	66,000
high	high	2	0.097	4,060	10,000	0.069	4,271	10,000
high	high	2.5	0.045	8,801	13,000	0.037	8,064	13,000
high	low	1.5	0.065	6,130	40,000	0.127	2,268	15,000
high	low	2	0.038	10,616	33,000	0.0922	3,167	10,000
high	low	2.5	0.053	7,570	12,500	0.0671	4,395	7,000
low	high	1.5	0.727	468	Xxfailure	0.2259	1,225	1,200
low	high	2	0.151	2,585	5,000	0.1523	1,872	1,800
low	high	2.5	0.131	2,993	7,500	0.0967	3,015	7,500
low	low	1.5	0.416	883	1,000	0.4024	637	1,000
low	low	2	0.455	800	1,000	0.2966	906	1,000
low	low	2.5	0.256	1,489	5,000	0.3354	788	1,000

Average E' value for high Strength CLSM = 19,200 psi (132,384 Pa)

Average E' value for low strength CLSM = 3,000 psi (20,685 Pa)

CHAPTER 7
CONCLUSIONS

7.1 CONCLUSIONS

This report has covered three major sections; mix design of flowable fill, design of a test apparatus, and testing of buried pipes. This research has led to the construction and operation of a laboratory scale pipe testing device that was used for testing the effects of flowable fill around buried pipe.

The section on mix design presents relationships between the percentage of aggregates and the strength and flowability of different mixes. Based on the results, it is possible to find the optimum aggregate content for maximizing strength for different materials. All of the mix designs also showed the point where mixes have segregation problems. Being able to control the strength of CLSM mixes and prevent segregation are two of the most primary concerns in flowable fill mix design. All of these tests show that mixtures that are above 90 % aggregates will have problems with segregation, and the strength is maximized with 50 % to 80 % aggregates. This data provides useful information on CLSM mixes that can be used for transportation projects.

The mix design also showed the relationship of water content and flowabilty. Mixes with high percentage of aggregates require less water to achieve adequate flowability up to the point when segregation becomes a problem. At very high aggregate contents, the water simply separates immediately and the mixture exhibits no flow. Since fly ash is very hydrophylic, higher fly ash content in mixtures will require more water to achieve flowability. Lastly, these tests found that a minimum flow of 6 inches (15 cm) could be problematic for adequately filling all of the voids, so all pipe tests were conducted with mixtures that had 9 inches (23 cm) of flow.

One of the objectives of this research was to find mix designs that could maximize the amount of waste material used. All of the fly ash and bottom ash used in this research came from a fluidized bed combustion power plant that burns lower grade high sulfur content coal. The high alkalinity of this ash helps contribute to the pozzolanic activity of the CLSM. Many suitable mix designs were found that would not require any cement and could use as much as 75 % fly ash. The results from the study on mix design shows that mixes containing 100 % waste material in the form of fly ash and bottom ash would not form any mixes that would be currently accepted by the WVDOT as class A or class C mixtures. Since both of these materials are pozzolanic the mixtures have strengths that are in excess of the low strength CLSM, but too small to meet the requirements for high strength CLSM. However, by adding just a little bit of cement mixes composed of primarily bottom ash and fly ash could make suitable high strength CLSM. The high strength CLSM used in the pipe testing in this report used 97 % waste materials and only needed 3 % cement. Cement is an expensive ingredient, so using pozzolonic materials to reduce the required amount of cement is highly desired. Moreover, it was found that CLSM samples cured in water submersion had strength reductions of 12 % to 20 %.

The second major section of the report dealt with the design of a testing apparatus. Once the mix design was completed, a laboratory scale device that could test these mixtures was needed. The device that was built did not show any signs of significant deflections or stresses and should be able to carry significantly higher loads than those applied in the course of this research. Both the weight and the cost were successfully minimized. The cost was minimized by using motorcycle inner tubes rather than expensive pressure jacks that would not have given any improvement to the operation of the device. Using deep L-beam reinforcement over thin 1/8 inch (3.2 mm) sheet metal minimized the weight. The depth of these L-beam flanges provides significant resistance to bending without adding too much weight.

The last section was the laboratory scale pipe testing by using the designed mixtures and the test apparatus. These tests show that both the centerline soil stresses and the deflections are reduced when the trench width ratio is increased. Some attempts have been made to quantify these relationships but there is an inadequate number of tests for the results to be statistically significant. These tests have provided a strong foundation for further research to fully investigate the relationship of these variables.

High strength CLSM led to less deflection than low strength CLSM. High strength cohesive soil led to less deflection than did loose cohesionless soil. Smaller pipes deflect less than larger pipes, and larger loads produce greater deflections and pressures than smaller loads. These results are consistent with expected trends.

One interesting result was shown by the high centerline soil stress induced from having a high strength CLSM in conjunction with a low strength in-situ soil. As explained by arching theory, differential settlements in two soil masses will cause the transfer of loads from one soil mass to the other in the form of interparticle friction. This relationship, as well as all the other interparticle soil forces and soil-to-pipe reactions can be made into statistically significant relationships with the inclusion of additional testing.

The final portion of the research was the numerical analysis. The numerical analysis using Spangler's Iowa formula was completed. This analysis showed that the deflections can be reasonably predicted as long as there is accurate information on the soil stiffness. The analysis showed that the high strength CLSM had an average modulus of soil reaction of 19,200 psi (132 kPa) with vales that ranged from 1,000 psi (6,900 Pa) to 66,000 psi (448 kPa). The low strength CLSM had an average modulus of soil reaction of 3,000 psi (20,700 Pa) with values that ranged from 1,000 psi (6,900 Pa) to 7,500 psi (51,700 Pa). Design manuals (Howard 1996) often suggest using 3,000 psi (20,700 Pa) for low strength CLSM and 25,000 (172 kPa) for high strength CLSM. In addition, the correlations between the measured displacements and the predicted displacements can be considered as excellent. As with the other tests, this information could be improved if more tests were conducted.

The results of the pipe testing showed a good performance for all trench width ratios. At a trench width ratio of 1.5, only one out of eight tests had greater than 7 % deflection when 20 psi (138 Pa) surcharge loading was applied. The results of this research show narrow trench widths can be successfully used in many cases if the in-situ soil strength and the CLSM strength are appropriately accounted for.

7.2 RECOMMENDATIONS FOR FURTHER RESEARCH

As stated earlier, more pipe testing should be done to find statistically significant relationships between all the control variables. The following is a list of specific experimental sequences that would generate useful results.

- Conduct tests on 10 inch (25 cm) and 12 inch (30 cm) pipe diameters. This will give a full range of pipe diameters for comparative purposes.
- Conduct tests using an even lower strength CLSM and an intermediate WVDOT Class B mixture. The low strength CLSM that was used in this research falls near the top of the required low strength range. These additional

tests will provide a fuller range of strength values for the CLSM mixtures as well as help to establish the relationships between CLSM strength, in-situ soil strength, and centerline soil stress beneath the pipe.

- Conduct tests with various depths of cover. All of these tests used the same depth of cover, but this could easily be an additional variable for further research.
- Conduct long term loading tests. In all of the tests conducted in this research pressure was immediately removed. Applying a set pressure for several days or weeks could lead to valuable information on stress relaxation in the pipe and creep behavior of the CLSM.

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VITA

I was born and raised in California. When my Grandfather died, I moved to Garrett County Maryland to be the caretaker of a beautiful piece of family property. I graduated with Honors from Garrett Community College, receiving an Associate's Degree in Math and Natural Science in 1996. Garrett community college is also where I met the woman who would eventually become my wife. After graduating I spent one year as a volunteer for the American Conservation Corps.

I transferred to West Virginia University the following year. While studying for my Bachelor's Degree in Civil and Environmental Engineering I married my loving companion. I received my Bachelor's Degree in 1999, graduating with high honors. Since graduating my wife and I were blessed with a beautiful baby boy. I have been working diligently to receive my Master's of Science Degree while learning to be a father.

Appendix A
Data Collected From Flowability Testing

Flowability Data:

Mix: M: 100% fly ash

Container	5	6	7	2
Mass c. (g)	32.37	32.83	28.73	25.91
mass full (g)	107.6	131.57	131.57	130.73
mass dry (g)	74.8	88.14	85.93	82.84
W	77.30379	78.52106	79.79021	84.12085
Spread (inch)	6.5	7.5	9	9.5

Mix: M1C: 99% fly ash, 1% cement

Container	8	10	11	13
Mass c. (g)	28.66	27.8	27.49	28.04
Mass full (g)	123.82	118.5	131.38	126.13
Mass dry (g)	83.18	78.9	85.13	82.33
W	74.54145	77.49511	80.23942	80.67784
Spread (inch)	6	7.625	9.375	9.875

Mix: M3C: 97% fly ash, 3% c

Container	1	3	31	20
Mass c. (g)	30.84	36.11	36.32	36.6
Mass full (g)	117.03	145.35	148.14	155.82
Mass dry (g)	80.02	97.87	98.7	102.74
W	75.25417	76.87824	79.25617	80.25401
Spread (inch)	6	7.25	9.25	10.375

Mix: M6C: 94% fly ash, 6% cement

Container	24	29	33	38
Mass c. (g)	36.42	36.53	36.59	36.47
Mass full (g)	127.76	142.16	131.48	137.38
Mass dry (g)	89.05	96.28	89.99	92.8
W	73.55121	76.78661	77.69663	79.14078
Spread (inch)	6.125	8	8.75	9.625

Mix: M5BA: 95% fly ash, 5% bottom ash

Container	7	17	10	18
Mass c. (g)	31.14	27.64	27.61	26
Mass full (g)	108.6	111.8	134.42	121.45
Mass dry (g)	77.6	77.36	89.15	80.27
W	66.72406	69.2679	73.56191	75.87986
Spread (inch)	4.875	6.125	8.125	9.625

Mix: M10BA: 90% fly ash, 10% bottom ash

Container	1	8	5	11
Mass c. (g)	33.58	27.74	30.61	27.68
Mass full (g)	121.71	118.21	109.22	125.18
Mass dry (g)	86.59	80.57	76.15	83.73
W	66.25165	71.2474	72.61748	73.95183
Spread (inch)	4.875	7.5	9.25	9.875

Mix: M25BA: 75% fly ash, 25% bottom ash

Container	24	31	30	33
Mass c. (g)	36.62	36.82	36.45	36.69
Mass full (g)	137.91	158	140.38	158.42
Mass dry (g)	99.34	110.49	99.19	110.17
W	61.49554	64.49029	65.6519	65.66413
Spread (inch)	6.875	8	9.5	10.375

Mix: M50BA: 50% fly ash, 50% bottom ash

Container	26	23	29	38
Mass c. (g)	36.48	36.28	36.76	36.59
Mass full (g)	160.94	142.82	141.71	174.15
Mass dry (g)	121.68	107.93	107.37	128.78
W	46.07981	48.69505	48.63334	49.21358
Spread (inch)	6.75	8.375	9	10

Mix: M75BA: 25% fly ash, 75% bottom ash

Container	18	13	14	9
Mass c. (g)	29.56	30.95	27.86	31.15
Mass full (g)	137.96	137.95	155.8	139.29
Mass dry (g)	110.81	109.39	122.32	110.89
W	33.41538	36.40999	35.44357	35.61575
Spread (inch)	4.875	8	8	9

Mix: M90BA: 10% fly ash, 90% bottom ash

Container	19	37	34	35
Mass c. (g)	29.38	36.61	36.43	36.04
Mass full (g)	162.41	197.13	222.05	206.13
Mass dry (g)	127.99	155.6	172.63	160.83
W	34.90518	34.90209	36.28488	36.30099
Spread (inch)	5.625	6	9.375	9.625

Mix: M2BA3C: 95% fly ash, 2% bottom ash, 3% cement

Container	14	5	18	7
Mass c. (g)	29.56	33.21	29.96	31.24
mass full (g)	105.71	115.58	101.93	116.04
mass dry (g)	73.79	80.35	70.45	78.94
W	72.16821	74.73483	77.74759	77.77778
Spread (inch)	5.75	7.75	9.5	10

Mix: M7BA3C: 90% fly ash, 7% bottom ash, 3% cement

Container	29	30	33	31
Mass c. (g)		36.48	36.81	36.79
Mass full (g)		144.35	120.78	157.29
Mass dry (g)		98.89	84.86	105.5
W		72.84089	74.75546	75.37476
Spread (inch)		8	9.5	9.75

Mix: M22BA3C: 75% fly ash, 22% bottom ash, 3% cement

Container	13	1	9	18
Mass c. (g)	30.87	30.84	32.94	27.74
Mass full (g)	112.11	121.12	133.27	128.39
Mass dry (g)	80.98	86.36	93.7	88.11
W	62.12333	62.60807	65.12508	66.72188
Spread (inch)	5.625	7.25	8.5	10.125

Mix: M47BA3C: 50% fly ash, 47% bottom ash, 3% cement

Container	23	26	38	24
Mass c. (g)	36.22	36.6	36.63	36.68
Mass full (g)	134.7	130.8	122.77	168.92
Mass dry (g)	104.52	101.49	95.61	125.16
W	44.18741	45.16875	46.04951	49.4575
Spread (inch)	4.5	6.75	7.75	9.25

Mix: M72BA3C: 25% fly ash, 72% bottom ash, 3% cement

Container	17	10	8	11
Mass c. (g)	25.95	26.03	26	27.73
Mass full (g)	126.02	131.08	134.32	144.88
Mass dry (g)	100.66	104.43	105.83	113.55
W	33.94459	33.99235	35.68834	36.50664
Spread (inch)	6	7.5	8.25	9.75

Mix: M87BA3C: 10% fly ash, 87% bottom ash, 3% cement

Container	18	5	9	2
Mass c. (g)	29.46	32.25	30.67	30.47
Mass full (g)	141.76	152.07	136.44	141.38
Mass dry (g)	113.03	121.22	108.04	111.8
W	34.37837	34.67461	36.70673	36.37034
Spread (inch)	4.5	7.875	9.625	9.625

Mix: M5RS: 95% fly ash, 5% river sand

Container	TPC	21	7	29
Mass c. (g)	29.33	35.97	28.5	36.72
Mass full (g)	101.36	104.87	111.89	168.53
Mass dry (g)	71.3	75.35	75.92	111.25
W	71.62259	74.96191	75.85407	76.85496
Spread (inch)	5.625	8	9	9.375

Mix: M10RS: 90% fly ash, 10% river sand

Container	33	31	35	30
Mass c. (g)	36.84	36.85	36.11	36.52
Mass full (g)	116.83	117.19	118.63	156.17
Mass dry (g)	84.2	83.37	83.6	105.17
W	68.8978	72.69991	73.7629	74.28988
Spread (inch)	6.25	8.5	9.625	10.375

Mix: M25RS: 75% fly ash, 25% river sand

Container	5	21	7	2
Mass c. (g)	32.02	30.95	27.79	33.33
Mass full (g)	110.73	112.37	122.06	122.99
Mass dry (g)	82.93	83.13	87.34	89.28
W	54.60617	56.0368	58.30395	60.25022
Spread (inch)	5.5	7.125	8.625	9.75

Mix: M50RS: 50% fly ash, 50% river sand

Container	30	38	24	33
Mass c. (g)	36.59	36.65	36.71	36.86
Mass full (g)	167.56	156.35	162.12	182.77
Mass dry (g)	130.43	121.4	124.9	139.86
W	39.56735	41.23894	42.20433	41.66019
Spread (inch)	6.875	8.375	9	10.25

Mix: M75RS: 25% fly ash, 75% river sand

Container	11	8	18	17
Mass c. (g)	26	26.02	28.66	27.7
Mass full (g)	121.89	150.74	156.05	150.03
Mass dry (g)	104.8	127.95	132.37	127.37
W	21.68782	22.35848	22.8329	22.73503
Spread (inch)	5	6.625	8	9.75

Mix: M90RS: 10% fly, 90% river sand

Container	10	3	9	13
Mass c. (g)	26.04	33.92	29.7	
Mass full (g)	126.91	163.47	112.65	
Mass dry (g)	108.23	139.75	96.44	
W	22.72783	22.4133	24.28828	
Spread (inch)	7	6.625	6.375	

This mix does not produce adequate flow

Mix: M85RS: 15% fly ash, 85% river sand

Container	30	34	38	33
Mass c. (g)	36.2	36.5	36.63	36.89
Mass full (g)	206.11	155.33	218.71	260.46
Mass dry (g)	178.76	135.01	186.97	221.21
W	19.1849	20.62735	21.11215	21.29449
Spread (inch)	5.25	6.25	7.5	9.75

Mix: M2RS3C: 95% fly ash, 2% river sand, 3% cement

Container	26	34	23	31
Mass c. (g)	36.57	36.5	36.38	36.84
Mass full (g)	124.41	135.8	147.01	174.05
Mass dry (g)	88.6	94.22	99.86	114.3
W	68.82568	72.03742	74.27536	77.13659
Spread (inch)	4.875	7	8.375	10

Mix: M7RS3C: 90% fly ash, 7% river sand, 3% cement

Container	7	19	14	18
Mass c. (g)	31.25	32.22	30.46	28.68
Mass full (g)	120.63	128.35	132.69	128.32
Mass dry (g)	85.68	89.49	90.26	86.4
W	64.21091	67.85402	70.95318	72.62647
Spread (inch)	5	6.125	8.375	9.25

Mix: M22RS3C: 75% fly ash, 22% river sand, 3% cement

Container	1	5	18	tpc
Mass c. (g)	30.84	30.69	26.77	28.84
Mass full (g)	133.19	143.51	137.33	132.16
Mass dry (g)	96.3	101.99	96.1	92.02
W	56.35503	58.23282	59.46921	63.53276
Spread (inch)	4.875	7.25	8.375	10.125

Mix: M47RS3C: 50% fly ash, 47% river sand, 3% cement

Container	9	21	10	8
Mass c. (g)	29.7	30.77	27.79	31.2
Mass full (g)	123.91	115.48	131.58	137.21
Mass dry (g)	97.87	91.26	100.9	105.04
W	38.19862	40.03968	41.96416	43.56717
Spread (inch)	4.5	6.25	8.5	9.75

Mix: M72RS3C: 25% fly ash, 72% river sand, 3% cement

Container	24	33	38	30
Mass c. (g)	36.68	36.84	36.66	36.57
Mass full (g)	122.08	150.92	131.78	197.93
Mass dry (g)	106.56	129.01	112.65	166.26
W	22.2095	23.77129	25.17437	24.41977
Spread (inch)	4.75	7.5	9	10

Mix: M87RS3C: 10% fly ash, 87% river sand, 3% cement

Container	5	7	2	11
Mass c. (g)	28.38	31.49	27.89	26.83
Mass full (g)				
Mass dry (g)	This mix does not produce adequate flow			
W				
Spread (inch)	8.75	8.625		

Mix: M5FS: 95% fly ash, 5% foundry sand

Container	5	7	2	11
Mass c. (g)	28.29	31.47	27.87	28.81
Mass full (g)	95.67	120.57	109.79	122.22
Mass dry (g)	68.75	83.66	74.87	82.16
W	66.53485	70.72236	74.29787	75.08903
Spread (inch)	4.75	6.625	9.5	10.25

Mix: M10FS: 90% fly ash, 10% foundry sand

Container	18	1	32	8
Mass c. (g)	29.91	30.93	31.2	27.82
Mass full (g)	108.91	123.81	132.42	137.52
Mass dry (g)	78.41	86.17	91.19	92.11
W	62.8866	68.13903	68.72812	70.63307
Spread (inch)	4.25	7.625	8.75	9.625

Mix: M25FS: 75% fly ash, 25% foundry sand

Container	10	7	18	5
Mass c. (g)	28.81	31.18	29.17	30.71
Mass full (g)	128.4	125.62	142.85	134.9
Mass dry (g)	92.19	90.62	100.24	95.04
W	57.13159	58.88291	59.95497	61.96176
Spread (inch)	5.5	7	8.125	9.75

Mix: M50FS: 50% fly ash, 50% foundry sand

Container	19	14	9	21
Mass c. (g)	31.02	29.69	27.96	30.85
Mass full (g)	128.94	147.51	146.23	132.65
Mass dry (g)	100.95	112.92	111.64	101.73
W	40.02574	41.55953	41.33604	43.62302
Spread (inch)	5	7	8.125	10

Mix: M75FS: 25% fly ash, 75% foundry sand

Container	26	23	24	31
Mass c. (g)	36.57	36.32	36.66	36.9
Mass full (g)	193.39	218.31	222.14	242.78
Mass dry (g)	160.96	180.15	182.23	197.81
W	26.07123	26.53132	27.41636	27.9473
Spread (inch)	4.25	7	8	9.375

Mix: M90FS: 10% fly ash, 90% foundry sand

Container	30	34	38	33
Mass c. (g)	36.53	36.48	36.62	36.88
Mass full (g)				
Mass dry (g)	This mix does not produce adequate flow			
W				
Spread (inch)	5.125			

Mix: M85FS: 15% fly ash, 85% foundry sand

Container	14	18	17	9
Mass c. (g)	29.36	26.04	25.93	33.69
Mass full (g)	136.59	131.29	158.64	135.46
Mass dry (g)	114.39	108.1	130.26	112.67
W	26.10843	28.25981	27.20215	28.85541
Spread (inch)	5	8.5	9.625	10.625

Mix: M2FS3C: 95% fly ash, 2% foundry sand, 3% cement

Container	8	32	14	7
Mass c. (g)	28.89	27.78	31.96	28.86
Mass full (g)	121.09	125.5	131.6	127.43
Mass dry (g)	82.95	84.32	89.15	84.73
W	70.55124	72.83339	74.22626	76.42742
Spread (inch)	6	7.75	9.625	11

Mix: M7FS3C: 90% fly ash, 7% foundry sand, 3% cement

Container	7	1	5	21
Mass c. (g)	31.27	30.87	28.21	30.75
Mass full (g)	126.31	125.01	137.64	139.46
Mass dry (g)	87.76	86.63	92.75	94.62
W	68.24217	68.8307	69.55377	70.2051
Spread (inch)	7	7.5	8.5	9.25

Mix: M22FS3C: 75% fly ash, 22% foundry sand, 3% cement

Container	5	18	9	11
Mass c. (g)	30.77	27.12	33.11	26.17
Mass full (g)	141.26	120.12	147.75	130
Mass dry (g)	100.31	85.1	103.57	89.09
W	58.88697	60.40014	62.70224	65.01907
Spread (inch)	6.25	7.25	8.875	10.875

Mix: M47FS3C: 50% fly ash, 47% foundry sand, 3% cement

Container	18	14	17	9
Mass c. (g)	28.95	29.12	25.96	31.08
Mass full (g)	143.78	145.91	139.71	136.39
Mass dry (g)	110.02	110.85	105.34	103.26
W	41.64302	42.89734	43.29806	45.89914
Spread (inch)	7	8.375	9	12

Mix: M72FS3C: 25% fly ash, 72% foundry sand, 3% cement

Container	14	18	10
Mass c. (g)	30.84	28.42	31.55
Mass full (g)	119.5	130.89	149.33
Mass dry (g)	101.12	108.67	123.15
W	26.15253	27.68847	28.58079
Spread (inch)	6.25	8.375	10.25

Mix: M87FS3C: 10% fly ash, 87% foundry sand, 3% cement

Container	4	29	35
Mass c. (g)	36.72	36.81	36.03
Mass full (g)	198.57	223.77	233.06
Mass dry (g)	166.11	185.58	192.48
W	25.08695	25.6705	25.938
Spread (inch)	8.25	8.625	10.25

Appendix B
Data Collected From Strength Testing

Bottom ash 7 day strength

Mix: M90BA 7 day

Height (in)	5.701		Height (in)	5.538	**AVG.**
Diameter (in)	3.018		Diameter (in)	3.015	
Area (in^2)	7.1536		Area (in^2)	7.1394	
Load number (lb/2.75)	184		Load number (lb/2.75)	153	
Compressive strength (psi)	70.733		Compressive strength (psi)	58.933	**64.833**

Mix: M83BA 7 day

Height (in)	5.635		Height (in)	5.561	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.023	
Area (in^2)	7.1441		Area (in^2)	7.1773	
Load number (lb/2.75)	348		Load number (lb/2.75)	250	
Compressive strength (psi)	133.95		Compressive strength (psi)	95.786	**114.87**

Mix: M75BA 7 day

Height (in)	5.645		Height (in)	5.785	**AVG.**
Diameter (in)	3.015		Diameter (in)	3.031	
Area (in^2)	7.1394		Area (in^2)	7.2154	
Load number (lb/2.75)	454		Load number (lb/2.75)	516	
Compressive strength (psi)	174.87		Compressive strength (psi)	196.66	**185.77**

Mix: M50BA 7 day

Height (in)	5.793		Height (in)	5.73	**AVG.**
Diameter (in)	3.03		Diameter (in)	3.042	
Area (in^2)	7.2106		Area (in^2)	7.2678	
Load number (lb/6.62)	237		Load number (lb/6.62)	251	
Compressive strength (psi)	217.58		Compressive strength (psi)	228.62	**223.11**

Mix: M25BA 7 day

Height (in)	5.641		Height (in)	5.688	**AVG.**
Diameter (in)	3.031		Diameter (in)	3.025	
Area (in^2)	7.2154		Area (in^2)	7.1868	
Load number (lb/6.62)	319		Load number (lb/6.62)	275	
Compressive strength (psi)	292.67		Compressive strength (psi)	253.30	**272.99**

Mix: M10BA 7 day

Height (in)	5.672		Height (in)	5.628	**AVG.**
Diameter (in)	3.022		Diameter (in)	3.029	
Area (in^2)	7.1726		Area (in^2)	7.2059	
Load number (lb/6.62)	275		Load number (lb/6.62)	208	
Compressive strength (psi)	253.81		Compressive strength (psi)	191.08	**222.45**

Bottom Ash 14 day Strength

Mix: M90BA 14 day

Height (in)	5.622	Height (in)	5.774	**AVG.**
Diameter (in)	3.038	Diameter (in)	3.018	
Area (in^2)	7.2487	Area (in^2)	7.1536	
Load number (lb/2.75)	333	Load number (lb/2.75)	318	
Compressive strength (psi)	126.33	Compressive strength (psi)	122.24	**124.29**

Mix: M83BA 14 day

Height (in)	5.615	Height (in)	5.7	**AVG.**
Diameter (in)	3.038	Diameter (in)	3.018	
Area (in^2)	7.2487	Area (in^2)	7.1536	
Load number (lb/2.75)	681	Load number (lb/2.75)	789	
Compressive strength (psi)	258.35	Compressive strength (psi)	303.30	**280.83**

Mix: M75BA 14 day

Height (in)	5.738	Height (in)	5.635	**AVG.**
Diameter (in)	3.041	Diameter (in)	3.033	
Area (in^2)	7.2631	Area (in^2)	7.2249	
Load number (lb/3)	1030	Load number (lb/3)	1124	
Compressive strength (psi)	389.98	Compressive strength (psi)	427.82	**408.9**

Mix: M50BA 14 day

Height (in)	5.73	Height (in)	5.703	**AVG.**
Diameter (in)	3.033	Diameter (in)	3.028	
Area (in^2)	7.2249	Area (in^2)	7.2011	
Load number (lb/6.62)	711	Load number (lb/6.62)	675	
Compressive strength (psi)	651.46	Compressive strength (psi)	620.52	**636**

Mix: M25BA 14 day

Height (in)	5.72	Height (in)	5.69	**AVG.**
Diameter (in)	3.02	Diameter (in)	3.038	
Area (in^2)	7.1631	Area (in^2)	7.2487	
Load number (lb/6.62)	373	Load number (lb/6.62)	459	
Compressive strength (psi)	344.71	Compressive strength (psi)	419.18	**381.95**

sample: M10BA 14 day

Height (in)	5.612	Height (in)	5.602	**AVG.**
Diameter (in)	3.02	Diameter (in)	3.012	
Area (in^2)	7.163	Area (in^2)	7.1252	
Load number (lb/6.62)	257	Load number (lb/6.62)	401	
Compressive strength (psi)	237.513	Compressive strength (psi)	372.56	**305.04**

Bottom Ash 28 day Strength

Mix: M90BA 28 day

Height (in)	5.678	Height (in)	5.776	**AVG.**
Diameter (in)	3.015	Diameter (in)	3.025	
Area (in^2)	7.1394	Area (in^2)	7.1868	
Load number (lb/6.62)	330	Load number (lb/6.62)	382	
Compressive strength (psi)	305.99	Compressive strength (psi)	351.86	**328.93**

Mix: M83BA 28 day

Height (in)	5.72	Height (in)	5.629	**AVG.**
Diameter (in)	3.031	Diameter (in)	3.042	
Area (in^2)	7.2154	Area (in^2)	7.2678	
Load number (lb/6.62)	729	Load number (lb/6.62)	515	
Compressive strength (psi)	668.84	Compressive strength (psi)	469.09	**568.97**

Mix: M75BA 28 day

Height (in)	5.622	Height (in)	5.671	**AVG.**
Diameter (in)	3.02	Diameter (in)	3.023	
Area (in^2)	7.1631	Area (in^2)	7.1773	
Load number (lb/6.62)	997	Load number (lb/6.62)	895	
Compressive strength (psi)	921.40	Compressive strength (psi)	825.49	**873.45**

Mix: M50BA 28 day

Height (in)	5.721	Height (in)	5.742	**AVG.**
Diameter (in)	3.034	Diameter (in)	3.034	
Area (in^2)	7.2297	Area (in^2)	7.2297	
Load number (lb/6.62)	624	Load number (lb/6.62)	535	
Compressive strength (psi)	571.37	Compressive strength (psi)	489.88	**530.63**

Mix: M25BA 28 day

Height (in)	5.681	Height (in)	5.701	**AVG.**
Diameter (in)	3.033	Diameter (in)	3.032	
Area (in^2)	7.2249	Area (in^2)	7.2201	
Load number (lb/6.62)	426	Load number (lb/6.62)	412	
Compressive strength (psi)	390.33	Compressive strength (psi)	377.75	**384.04**

Mix: M10BA 28 day

Height (in)	5.675	Height (in)	5.5	**AVG.**
Diameter (in)	3.025	Diameter (in)	3.02	
Area (in^2)	7.1868	Area (in^2)	7.1631	
Load number (lb/6.62)	264	Load number (lb/6.62)	291	
Compressive strength (psi)	243.17	Compressive strength (psi)	268.93	**256.06**

<u>River sand 7 day Strength</u>

Mix: M85RS 7 day

Height (in)	5.724		Height (in)	5.723	**AVG.**
Diameter (in)	3.005		Diameter (in)	3.006	
Area (in^2)	7.0921		Area (in^2)	7.0968	
Load number (lb/2.75)	293		Load number (lb/2.75)	236	
Compressive strength (psi)	113.61		Compressive strength (psi)	91.448	**102.53**

Mix: M80RS 7 day

Height (in)	5.772		Height (in)	5.728	**AVG.**
Diameter (in)	3.012		Diameter (in)	3.019	
Area (in^2)	7.1252		Area (in^2)	7.1584	
Load number (lb/2.75)	440		Load number (lb/2.75)	468	
Compressive strength (psi)	169.81		Compressive strength (psi)	179.78	**174.8**

Mix: M75RS 7 day

Height (in)	5.714		Height (in)	5.64	**AVG.**
Diameter (in)	3.022		Diameter (in)	3.025	
Area (in^2)	7.1726		Area (in^2)	7.1868	
Load number (lb/2.75)	757		Load number (lb/2.75)	740	
Compressive strength (psi)	290.235		Compressive strength (psi)	283.15	**286.69**

Mix: M50RS 7 day

Height (in)	5.651		Height (in)	5.583	**AVG.**
Diameter (in)	3.034		Diameter (in)	3.028	
Area (in^2)	7.2297		Area (in^2)	7.2011	
Load number (lb/6.62)	214		Load number (lb/6.62)	179	
Compressive strength (psi)	195.95		Compressive strength (psi)	164.55	**180.25**

Mix: M25RS 7 day

Height (in)	5.689		Height (in)	5.695	**AVG.**
Diameter (in)	3.032		Diameter (in)	3.028	
Area (in^2)	7.2201		Area (in^2)	7.2011	
Load number (lb/6.62)	240		Load number (lb/6.62)	205	
Compressive strength (psi)	220.04		Compressive strength (psi)	188.45	**204.25**

Mix: M10RS 7 day

Height (in)	5.63		Height (in)	5.668	**AVG.**
Diameter (in)	3.029		Diameter (in)	3.023	
Area (in^2)	7.2059		Area (in^2)	7.1773	
Load number (lb/6.62)	**183**		Load number (lb/6.62)	191	
Compressive strength (psi)	168.12		Compressive strength (psi)	176.16	**172.14**

River Sand 14 day Strength

Mix: M85RS 14 day

Height (in)	5.692		Height (in)	5.585	**AVG.**
Diameter (in)	3.011		Diameter (in)	3.02	
Area (in^2)	7.1205		Area (in^2)	7.1631	
Load number (lb/2.75)	230		Load number (lb/2.75)	231	
Compressive strength (psi)	88.827		Compressive strength (psi)	88.611	**88.755**

Mix: M80RS 14 day

Height (in)	5.755		Height (in)	5.744	**AVG.**
Diameter (in)	3.02		Diameter (in)	3.009	
Area (in^2)	7.1631		Area (in^2)	7.1110	
Load number (lb/2.75)	547		Load number (lb/2.75)	441	
Compressive strength (psi)	209.99		Compressive strength (psi)	170.54	**190.27**

Mix: M75RS 14 day

Height (in)	5.78		Height (in)	5.72	**AVG.**
Diameter (in)	3.012		Diameter (in)	3.017	
Area (in^2)	7.1252		Area (in^2)	7.1489	
Load number (lb/2.75)	479		Load number (lb/2.75)	609	
Compressive strength (psi)	184.87		Compressive strength (psi)	234.26	**209.57**

Mix: M50RS 14 day

Height (in)	5.778		Height (in)	5.637	**AVG.**
Diameter (in)	3.036		Diameter (in)	3.027	
Area (in^2)	7.2392		Area (in^2)	7.1963	
Load number (lb/6.62)	268		Load number (lb/6.62)	263	
Compressive strength (psi)	245.07		Compressive strength (psi)	241.93	**243.51**

Mix: M25RS 14 day

Height (in)	5.773		Height (in)	5.715	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.023	
Area (in^2)	7.1441		Area (in^2)	7.1773	
Load number (lb/6.62)	260		Load number (lb/6.62)	206	
Compressive strength (psi)	240.92		Compressive strength (psi)	190.00	**215.46**

Mix: M10RS 14 day

Height (in)	5.723		Height (in)	5.771	**AVG.**
Diameter (in)	3.01		Diameter (in)	3.014	
Area (in^2)	7.1157		Area (in^2)	7.1347	
Load number (lb/6.62)	186		Load number (lb/6.62)	234	
Compressive strength (psi)	173.04		Compressive strength (psi)	217.11	**195.08**

River Sand 28 day Strength

Mix: M85RS 28 day

Height (in)	5.603	Height (in)	5.669	**AVG.**
Diameter (in)	3.007	Diameter (in)	3.019	
Area (in^2)	7.1016	Area (in^2)	7.1584	
Load number (lb/6.62)	95	Load number (lb/6.62)	115	
Compressive strength (psi)	88.557	Compressive strength (psi)	106.35	**97.454**

Mix: M80RS 28 day

Height (in)	5.695	Height (in)	5.731	**AVG.**
Diameter (in)	3.01	Diameter (in)	3.015	
Area (in^2)	7.1157	Area (in^2)	7.1394	
Load number (lb/6.62)	176	Load number (lb/6.62)	185	
Compressive strength (psi)	163.73	Compressive strength (psi)	171.53	**167.64**

Mix: M75RS 28 day

Height (in)	5.723	Height (in)	5.777	**AVG.**
Diameter (in)	3.018	Diameter (in)	3.022	
Area (in^2)	7.1536	Area (in^2)	7.1726	
Load number (lb/6.62)	248	Load number (lb/6.62)	314	
Compressive strength (psi)	229.49	Compressive strength (psi)	289.8	**259.65**

Mix: M50RS 28 day

Height (in)	5.737	Height (in)	5.745	**AVG.**
Diameter (in)	3.026	Diameter (in)	3.019	
Area (in^2)	7.1916	Area (in^2)	7.1584	
Load number (lb/6.62)	247	Load number (lb/6.62)	202	
Compressive strength (psi)	227.36	Compressive strength (psi)	186.80	**207.09**

Mix: M25RS 28 day

Height (in)	5.706	Height (in)	5.704	**AVG.**
Diameter (in)	3.029	Diameter (in)	3.019	
Area (in^2)	7.2059	Area (in^2)	7.1584	
Load number (lb/6.62)	346	Load number (lb/6.62)	227	
Compressive strength (psi)	317.86	Compressive strength (psi)	209.92	**263.9**

Mix: M10RS 28 day

Height (in)	5.707	Height (in)	5.766	**AVG.**
Diameter (in)	3.022	Diameter (in)	3.031	
Area (in^2)	7.1726	Area (in^2)	7.2154	
Load number (lb/6.62)	236	Load number (lb/6.62)	262	
Compressive strength (psi)	217.81	Compressive strength (psi)	240.37	**229.1**

Foundry Sand 7 Day Strength

Mix: M85FS 7 day

Height (in)	5.424	Height (in)	5.525	**AVG.**
Diameter (in)	3.004	Diameter (in)	3.009	
Area (in^2)	7.0874	Area (in^2)	7.1110	
Load number (lb/2.75)	185	Load number (lb/2.75)	182	
Compressive strength (psi)	71.781	Compressive strength (psi)	70.383	**71.083**

Mix: M80FS 7 day

Height (in)	5.655	Height (in)	5.633	**AVG.**
Diameter (in)	3.024	Diameter (in)	3.016	
Area (in^2)	7.1821	Area (in^2)	7.1441	
Load number (lb/2.75)	265	Load number (lb/2.75)	303	
Compressive strength (psi)	101.46	Compressive strength (psi)	116.63	**109.05**

Mix: M75FS 7 day

Height (in)	5.686	Height (in)	5.709	**AVG.**
Diameter (in)	3.016	Diameter (in)	3.023	
Area (in^2)	7.1441	Area (in^2)	7.1773	
Load number (lb/2.75)	358	Load number (lb/2.75)	386	
Compressive strength (psi)	137.80	Compressive strength (psi)	147.89	**142.85**

Mix: M50FS 7 day

Height (in)	5.844	Height (in)	5.655	**AVG.**
Diameter (in)	3.032	Diameter (in)	3.033	
Area (in^2)	7.2201	Area (in^2)	7.2249	
Load number (lb/6.62)	189	Load number (lb/6.62)	195	
Compressive strength (psi)	173.28	Compressive strength (psi)	178.67	**175.98**

Mix: M25FS 7 day

Height (in)	5.715	Height (in)	5.849	**AVG.**
Diameter (in)	3.03	Diameter (in)	3.029	
Area (in^2)	7.2106	Area (in^2)	7.2059	
Load number (lb/6.62)	200	Load number (lb/6.62)	179	
Compressive strength (psi)	183.617	Compressive strength (psi)	164.44	**174.03**

Mix: M10FS 7 day

Height (in)	5.741	Height (in)	5.64	**AVG.**
Diameter (in)	3.031	Diameter (in)	3.029	
Area (in^2)	7.2154	Area (in^2)	7.2059	
Load number (lb/6.62)	186	Load number (lb/6.62)	170	
Compressive strength (psi)	170.65	Compressive strength (psi)	156.17	**163.41**

Foundry Sand 14 day Strength

Mix: M85FS 14 day

Height (in)	5.522		Height (in)	5.255	**AVG.**
Diameter (in)	3.015		Diameter (in)	2.999	
Area (in^2)	7.1394		Area (in^2)	7.0638	
Load number (lb/2.75)	200		Load number (lb/2.75)	216	
Compressive strength (psi)	77.036		Compressive strength (psi)	84.089	**80.563**

Mix: M80FS 14 day

Height (in)	5.566		Height (in)	5.6	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.008	
Area (in^2)	7.1441		Area (in^2)	7.1063	
Load number (lb/2.75)	374		Load number (lb/2.75)	450	
Compressive strength (psi)	143.96		Compressive strength (psi)	174.14	**159.05**

Mix: M75FS 14 day

Height (in)	5.6		Height (in)	5.689	**AVG.**
Diameter (in)	3.026		Diameter (in)	3.01	
Area (in^2)	7.1916		Area (in^2)	7.1157	
Load number (lb/2.75)	394		Load number (lb/2.75)	453	
Compressive strength (psi)	150.66		Compressive strength (psi)	175.06	**162.86**

Mix: M50FS 14 day

Height (in)	5.691		Height (in)	5.747	**AVG.**
Diameter (in)	3.018		Diameter (in)	3.021	
Area (in^2)	7.1536		Area (in^2)	7.1678	
Load number (lb/2.75)	623		Load number (lb/2.75)	444	
Compressive strength (psi)	239.49		Compressive strength (psi)	170.34	**204.92**

Mix: M25FS 14 day

Height (in)	5.531		Height (in)	5.544	**AVG.**
Diameter (in)	3.033		Diameter (in)	3.03	
Area (in^2)	7.2249		Area (in^2)	7.2106	
Load number (lb/6.62)	203		Load number (lb/6.62)	250	
Compressive strength (psi)	186.00		Compressive strength (psi)	229.52	**207.76**

Mix: M10FS 14 day

Height (in)	5.748		Height (in)	5.758	**AVG.**
Diameter (in)	3.036		Diameter (in)	3.028	
Area (in^2)	7.2392		Area (in^2)	7.2011	
Load number (lb/2.75)	576		Load number (lb/2.75)	488	
Compressive strength (psi)	218.80		Compressive strength (psi)	186.35	**202.58**

Foundry Sand 28 day Strength

Mix: M85FS 28 day

Height (in)	5.541	Height (in)	5.704	**AVG.**
Diameter (in)	3.019	Diameter (in)	3.014	
Area (in^2)	7.1584	Area (in^2)	7.1347	
Load number (lb/6.62)	85	Load number (lb/6.62)	55	
Compressive strength (psi)	78.606	Compressive strength (psi)	51.032	**64.82**

Mix: M80FS 28 day

Height (in)	5.705	Height (in)	5.515	**AVG.**
Diameter (in)	3.015	Diameter (in)	3.011	
Area (in^2)	7.1394	Area (in^2)	7.1205	
Load number (lb/2.75)	314	Load number (lb/2.75)	304	
Compressive strength (psi)	120.94	Compressive strength (psi)	117.40	**119.18**

Mix: M75FS 28 day

Height (in)	5.69	Height (in)	5.685	**AVG.**
Diameter (in)	3.021	Diameter (in)	3.008	
Area (in^2)	7.1678	Area (in^2)	7.1063	
Load number (lb/2.75)	478	Load number (lb/2.75)	457	
Compressive strength (psi)	183.38	Compressive strength (psi)	176.84	**180.12**

Mix: M50FS 28 day

Height (in)	5.789	Height (in)	5.766	**AVG.**
Diameter (in)	3.028	Diameter (in)	3.016	
Area (in^2)	7.2011	Area (in^2)	7.1441	
Load number (lb/6.62)	230	Load number (lb/6.62)	246	
Compressive strength (psi)	211.43	Compressive strength (psi)	227.95	**219.69**

Mix: M25FS 28 day

Height (in)	5.779	Height (in)	5.553	**AVG.**
Diameter (in)	3.023	Diameter (in)	3.027	
Area (in^2)	7.177	Area (in^2)	7.196	
Load number (lb/6.62)	278	Load number (lb/6.62)	234	
Compressive strength (psi)	256.41	Compressive strength (psi)	215.25	**235.83**

Mix: M10FS 28 day

Height (in)	5.68	Height (in)	5.751	**AVG.**
Diameter (in)	3.015	Diameter (in)	3.025	
Area (in^2)	7.1394	Area (in^2)	7.1868	
Load number (lb/6.62)	238	Load number (lb/6.62)	251	
Compressive strength (psi)	220.68	Compressive strength (psi)	231.20	**225.94**

Bottom Ash with 1% Cement 7 day Strength

sample: M90BA1C 7 day

Height (in)	5.583	Height (in)	5.598	**AVG.**
Diameter (in)	3.012	Diameter (in)	3.01	
Area (in^2)	7.1252	Area (in^2)	7.1157	
Load number (lb/2.75)	353	Load number (lb/2.75)	322	
Compressive strength (psi)	136.24	Compressive strength (psi)	124.44	**130.34**

Mix: M83BA1C 7 day

Height (in)	5.586	Height (in)	5.604	**AVG.**
Diameter (in)	3.027	Diameter (in)	3.033	
Area (in^2)	7.1963	Area (in^2)	7.2249	
Load number (lb/2.75)	439	Load number (lb/2.75)	532	
Compressive strength (psi)	167.75	Compressive strength (psi)	202.49	**185.13**

Mix: M75BA1C 7 day

Height (in)	5.71	Height (in)	5.745	**AVG.**
Diameter (in)	3.027	Diameter (in)	3.022	
Area (in^2)	7.1963	Area (in^2)	7.1726	
Load number (lb/2.75)	715	Load number (lb/2.75)	663	
Compressive strength (psi)	273.22	Compressive strength (psi)	254.19	**263.71**

River Sand with 1% Cement 7 day Strength

Mix: M85RS1C 7 day

Height (in)	5.761	Height (in)	5.739	**AVG.**
Diameter (in)	3.025	Diameter (in)	3.015	
Area (in^2)	7.1868	Area (in^2)	7.1394	
Load number (lb/6.62)	121	Load number (lb/6.62)	111	
Compressive strength (psi)	111.45	Compressive strength (psi)	102.924	**107.19**

Mix: M80RS1C 7 day

Height (in)	5.764	Height (in)	5.799	**AVG.**
Diameter (in)	3.026	Diameter (in)	3.024	
Area (in^2)	7.1916	Area (in^2)	7.1821	
Load number (lb/6.62)	136	Load number (lb/6.62)	176	
Compressive strength (psi)	125.18	Compressive strength (psi)	162.22	**143.71**

Mix: M75RS1C 7 day

Height (in)	5.786	Height (in)	5.769	**AVG.**
Diameter (in)	3.031	Diameter (in)	3.021	
Area (in^2)	7.2154	Area (in^2)	7.16789	
Load number (lb/6.62)	216	Load number (lb/6.62)	238	
Compressive strength (psi)	198.17	Compressive strength (psi)	219.80	**208.99**

Bottom Ash with 1% Cement 14 day Strength

Mix: M90BA1C 14day

Height (in)	5.585		Height (in)	5.622	**AVG.**
Diameter (in)	3.026		Diameter (in)	3.018	
Area (in^2)	7.1916		Area (in^2)	7.1536	
Load number (lb/6.62)	424		Load number (lb/6.62)	535	
Compressive strength (psi)	390.29		Compressive strength (psi)	495.08	**442.69**

Mix: M83BA1C 14day

Height (in)	5.636		Height (in)	5.637	**AVG.**
Diameter (in)	3.031		Diameter (in)	3.038	
Area (in^2)	7.2154		Area (in^2)	7.2487	
Load number (lb/6.62)	581		Load number (lb/6.62)	398	
Compressive strength (psi)	533.05		Compressive strength (psi)	363.47	**448.27**

Mix: M75BA1C 14 day

Height (in)	5.729		Height (in)	5.746	**AVG.**
Diameter (in)	3.042		Diameter (in)	3.042	
Area (in^2)	7.2678		Area (in^2)	7.2678	
Load number (lb/6.62)	620		Load number (lb/6.62)	878	
Compressive strength (psi)	564.73		Compressive strength (psi)	799.73	**682.23**

River Sand with 1% Cement 14 day Strength

Mix: M85RS1C 14 day

Height (in)	5.718		Height (in)	5.744	**AVG.**
Diameter (in)	3.018		Diameter (in)	3.012	
Area (in^2)	7.1536		Area (in^2)	7.1252	
Load number (lb/6.62)	157		Load number (lb/6.62)	116	
Compressive strength (psi)	145.28		Compressive strength (psi)	107.77	**126.53**

Mix: M80RS1C 14 day

Height (in)	5.745		Height (in)	5.808	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.024	
Area (in^2)	7.1441		Area (in^2)	7.1821	
Load number (lb/6.62)	174		Load number (lb/6.62)	230	
Compressive strength (psi)	161.23		Compressive strength (psi)	211.99	**186.62**

Mix: M75RS1C 14 day

Height (in)	5.804		Height (in)	5.791	**AVG.**
Diameter (in)	3.027		Diameter (in)	3.029	
Area (in^2)	7.1963		Area (in^2)	7.2059	
Load number (lb/6.62)	235		Load number (lb/6.62)	339	
Compressive strength (psi)	216.177		Compressive strength (psi)	311.436	**263.81**

Bottom Ash with 1% Cement 28 day Strength

Mix: M90BA1C 28 day

Height (in)	5.684		Height (in)	5.552	**AVG.**
Diameter (in)	3.024		Diameter (in)	3.033	
Area (in^2)	7.1821		Area (in^2)	7.2249	
Load number (lb/6.62)	484		Load number (lb/6.62)	524	
Compressive strength (psi)	446.11		Compressive strength (psi)	480.12	**463.12**

Mix: M83BA1C 28 day

Height (in)	5.668		Height (in)	5.753	**AVG.**
Diameter (in)	3.042		Diameter (in)	3.034	
Area (in^2)	7.2679		Area (in^2)	7.2297	
Load number (lb/6.62)	750		Load number (lb/6.62)	686	
Compressive strength (psi)	683.14		Compressive strength (psi)	628.15	**655.64**

Mix: M75BA1C 28 day

Height (in)	5.682		Height (in)	5.737	**AVG.**
Diameter (in)	3.026		Diameter (in)	3.034	
Area (in^2)	7.1916		Area (in^2)	7.2297	
Load number (lb/6.62)	868		Load number (lb/6.62)	1280	
Compressive strength (psi)	799.006		Compressive strength (psi)	1172.05	**985.53**

River Sand with 1% Cement 28 day Strength

Mix: M85RS1C 28 day

Height (in)	5.732		Height (in)	5.771	**AVG.**
Diameter (in)	3.022		Diameter (in)	3.021	
Area (in^2)	7.1726		Area (in^2)	7.1679	
Load number (lb/6.62)	163		Load number (lb/6.62)	129	
Compressive strength (psi)	150.44		Compressive strength (psi)	119.14	**134.79**

Mix: M80RS1C 28 day

Height (in)	5.797		Height (in)	5.852	**AVG.**
Diameter (in)	3.026		Diameter (in)	3.007	
Area (in^2)	7.1916		Area (in^2)	7.1016	
Load number (lb/6.62)	267		Load number (lb/6.62)	232	
Compressive strength (psi)	245.78		Compressive strength (psi)	216.27	**231.02**

Mix: M75RS1C 28 day

Height (in)	5.804		Height (in)	5.813	**AVG.**
Diameter (in)	3.023		Diameter (in)	3.031	
Area (in^2)	7.1774		Area (in^2)	7.2154	
Load number (lb/6.62)	196		Load number (lb/6.62)	266	
Compressive strength (psi)	180.77		Compressive strength (psi)	244.049	**212.41**

Foundry Sand with 1% Cement 7 day Strength

Mix: M85FS1C 7 day

Height (in)	5.594		Height (in)	5.512	**AVG.**
Diameter (in)	3.02		Diameter (in)	3.02	
Area (in^2)	7.1631		Area (in^2)	7.1631	
Load number (lb/2.75)	210		Load number (lb/2.75)	175	
Compressive strength (psi)	80.621		Compressive strength (psi)	67.184	**73.903**

Mix: M80FS1C 7 day

Height (in)	5.569		Height (in)	5.547	**AVG.**
Diameter (in)	3.031		Diameter (in)	3.016	
Area (in^2)	7.2154		Area (in^2)	7.1441	
Load number (lb/2.75)	340		Load number (lb/2.75)	368	
Compressive strength (psi)	129.58		Compressive strength (psi)	141.65	**135.62**

Mix: M75FS1C 7 day

Height (in)	5.692		Height (in)	5.731	**AVG.**
Diameter (in)	3.029		Diameter (in)	3.002	
Area (in^2)	7.2059		Area (in^2)	7.0780	
Load number (lb/3)	550		Load number (lb/3)	531	
Compressive strength (psi)	209.89		Compressive strength (psi)	206.30	**208.1**

Foundry Sand with 1% Cement 14 day Strength

Mix: M85FS1C 14 day

Height (in)	5.497		Height (in)	5.612	**AVG.**
Diameter (in)	3.012		Diameter (in)	3.025	
Area (in^2)	7.1252		Area (in^2)	7.1868	
Load number (lb/6.62)	89		Load number (lb/6.62)	120	
Compressive strength (psi)	82.689		Compressive strength (psi)	110.53	**96.612**

Mix: M80FS1C 14 day

Height (in)	5.605		Height (in)	5.633	**AVG.**
Diameter (in)	3.027		Diameter (in)	3.025	
Area (in^2)	7.1963		Area (in^2)	7.1868	
Load number (lb/6.62)	178		Load number (lb/6.62)	154	
Compressive strength (psi)	163.74		Compressive strength (psi)	141.85	**152.8**

Mix: M75FS1C 14 day

Height (in)	5.705		Height (in)	5.704	**AVG.**
Diameter (in)	3.034		Diameter (in)	3.029	
Area (in^2)	7.2297		Area (in^2)	7.2059	
Load number (lb/6.62)	186		Load number (lb/6.62)	209	
Compressive strength (psi)	170.31		Compressive strength (psi)	192.00	**181.16**

Foundry Sand with 1% Cement 28 day Strength

Mix: M85FS1C 28 day

Height (in)	5.677	Height (in)	5.487	**AVG.**
Diameter (in)	3.015	Diameter (in)	3.021	
Area (in^2)	7.1394	Area (in^2)	7.1678	
Load number (lb/6.62)	140	Load number (lb/6.62)	68	
Compressive strength (psi)	129.814	Compressive strength (psi)	62.802	**96.308**

Mix: M80FS1C 28 day

Height (in)	5.604	Height (in)	5.606	**AVG.**
Diameter (in)	3.03	Diameter (in)	3.022	
Area (in^2)	7.2106	Area (in^2)	7.1726	
Load number (lb/6.62)	168	Load number (lb/6.62)	152	
Compressive strength (psi)	154.23	Compressive strength (psi)	140.288	**147.26**

Mix: M75FS1C 28 day

Height (in)	5.709	Height (in)	5.779	**AVG.**
Diameter (in)	3.031	Diameter (in)	3.029	
Area (in^2)	7.2154	Area (in^2)	7.2059	
Load number (lb/6.62)	232	Load number (lb/6.62)	218	
Compressive strength (psi)	212.85	Compressive strength (psi)	200.27	**206.56**

Repeat Experiments

Mix: M75BA 7 day

Height (in)	5.815		Height (in)	5.84	**AVG.**
Diameter (in)	3.028		Diameter (in)	3.033	
Area (in^2)	7.2011		Area (in^2)	7.2249	
Load number (lb/6.62)	333		Load number (lb/6.62)	396	
Compressive strength (psi)	306.12		Compressive strength (psi)	362.84	**334.48**

Mix: M50BA 7 day

Height (in)	5.825		Height (in)	5.806	**AVG.**
Diameter (in)	3.041		Diameter (in)	3.035	
Area (in^2)	7.2631		Area (in^2)	7.2344	
Load number (lb/6.62)	656		Load number (lb/6.62)	581	
Compressive strength (psi)	597.91		Compressive strength (psi)	531.65	**564.78**

Mix: M75RS 7 day

Height (in)	5.803		Height (in)	5.779	**AVG.**
Diameter (in)	3.015		Diameter (in)	3.018	
Area (in^2)	7.1394		Area (in^2)	7.1536	
Load number (lb/6.62)	216		Load number (lb/6.62)	178	
Compressive strength (psi)	200.28		Compressive strength (psi)	164.72	**182.5**

Mix: M50RS 7 day

Height (in)	5.704		Height (in)	5.769	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.029	
Area (in^2)	7.1441		Area (in^2)	7.2059	
Load number (lb/6.62)	181		Load number (lb/6.62)	248	
Compressive strength (psi)	167.71		Compressive strength (psi)	227.83	**197.78**

Mix: M83BA1C 7 day

Height (in)	5.788		Height (in)	5.809	**AVG.**
Diameter (in)	3.025		Diameter (in)	3.023	
Area (in^2)	7.1868		Area (in^2)	7.1773	
Load number (lb/6.62)	203		Load number (lb/6.62)	199	
Compressive strength (psi)	186.98		Compressive strength (psi)	183.54	**185.27**

Mix: M75BA1C 7 day

Height (in)	5.724		Height (in)	5.72	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.022	
Area (in^2)	7.1441		Area (in^2)	7.1726	
Load number (lb/6.62)	297		Load number (lb/6.62)	303	
Compressive strength (psi)	275.20		Compressive strength (psi)	279.654	**277.43**

Repeat Experiments

sample: M75BA 14 day

Height (in)	5.813	Height (in)	5.875	**AVG.**
Diameter (in)	3.044	Diameter (in)	3.041	
Area (in^2)	7.2774	Area (in^2)	7.2631	
Load number (lb/6.62)	830	Load number (lb/6.62)	978	
Compressive strength (psi)	755.01	Compressive strength (psi)	891.40	**823.21**

Mix: M50BA 14 day

Height (in)	5.858	Height (in)	5.827	**AVG.**
Diameter (in)	3.02	Diameter (in)	3.02	
Area (in^2)	7.1631	Area (in^2)	7.1631	
Load number (lb/6.62)	776	Load number (lb/6.62)	***436***	
Compressive strength (psi)	717.15	Compressive strength (psi)	402.94	**560.05**

Mix: M75RS 14 day

Height (in)	5.857	Height (in)	5.858	**AVG.**
Diameter (in)	3.026	Diameter (in)	3.024	
Area (in^2)	7.1916	Area (in^2)	7.1821	
Load number (lb/6.62)	196	Load number (lb/6.62)	215	
Compressive strength (psi)	180.42	Compressive strength (psi)	198.17	**189.3**

Mix: M50RS 14 day

Height (in)	5.757	Height (in)	5.777	**AVG.**
Diameter (in)	3.042	Diameter (in)	3.033	
Area (in^2)	7.2678	Area (in^2)	7.2249	
Load number (lb/6.62)	355	Load number (lb/6.62)	286	
Compressive strength (psi)	323.35	Compressive strength (psi)	262.05	**292.7**

Mix: M83BA1C 14 day

Height (in)	5.815	Height (in)	5.803	**AVG.**
Diameter (in)	3.043	Diameter (in)	3.033	
Area (in^2)	7.2726	Area (in^2)	7.2249	
Load number (lb/6.62)	720	Load number (lb/6.62)	465	
Compressive strength (psi)	655.38	Compressive strength (psi)	426.06	**540.73**

Mix: M75BA1C 14 day

Height (in)	5.746	Height (in)	5.848	**AVG.**
Diameter (in)	3.039	Diameter (in)	3.038	
Area (in^2)	7.2535	Area (in^2)	7.2487	
Load number (lb/6.62)	802	Load number (lb/6.62)	946	
Compressive strength (psi)	731.94	Compressive strength (psi)	863.94	**797.94**

Repeat Experiments

Mix: M75BA 28 day

Height (in)	5.773	Height (in)	5.824	**AVG.**
Diameter (in)	3.035	Diameter (in)	3.031	
Area (in^2)	7.2344	Area (in^2)	7.2154	
Load number (lb/6.62)	1324	Load number (lb/6.62)	1044	
Compressive strength (psi)	1211.5	Compressive strength (psi)	957.84	**1084.7**

Mix: M50BA 28 day

Height (in)	5.899	Height (in)	5.873	**AVG.**
Diameter (in)	3.034	Diameter (in)	3.037	
Area (in^2)	7.2297	Area (in^2)	7.2440	
Load number (lb/6.62)	740	Load number (lb/6.62)	796	
Compressive strength (psi)	677.59	Compressive strength (psi)	727.43	**702.51**

Mix: M75RS 28 day

Height (in)	5.747	Height (in)	5.803	**AVG.**
Diameter (in)	3.024	Diameter (in)	3.016	
Area (in^2)	7.1821	Area (in^2)	7.1441	
Load number (lb/6.62)	210	Load number (lb/6.62)	239	
Compressive strength (psi)	193.56	Compressive strength (psi)	221.46	**207.51**

Mix: M50RS 28 day

Height (in)	5.78	Height (in)	5.8	**AVG.**
Diameter (in)	3.031	Diameter (in)	3.031	
Area (in^2)	7.2154	Area (in^2)	7.2154	
Load number (lb/6.62)	375	Load number (lb/6.62)	307	
Compressive strength (psi)	344.05	Compressive strength (psi)	281.66	**312.86**

Mix: M83BA1C 28 day

Height (in)	5.762	Height (in)	5.814	**AVG.**
Diameter (in)	3.034	Diameter (in)	3.049	
Area (in^2)	7.2297	Area (in^2)	7.3013	
Load number (lb/6.62)	944	Load number (lb/6.62)	1056	
Compressive strength (psi)	864.38	Compressive strength (psi)	957.45	**910.92**

Mix: M75BA1C 28 day

Height (in)	5.851	Height (in)	5.706	**AVG.**
Diameter (in)	3.038	Diameter (in)	3.036	
Area (in^2)	7.2487	Area (in^2)	7.239	
Load number (lb/6.62)	1494	Load number (lb/6.62)	XXX	
Compressive strength (psi)	1364.4	Compressive strength (psi)		**1364.4**

Bottom Ash with 2% Cement 7 day Strength

Mix: M40BA2C 7 day

Height (in)	5.760		Height (in)	5.715	**AVG.**
Diameter (in)	3.042		Diameter (in)	3.041	
Area (in^2)	7.268		Area (in^2)	7.263	
Load number (lb/6.62)	265.000		Load number (lb/6.62)	264.000	
Compressive strength (psi)	241.377		Compressive strength (psi)	240.624	**241**

Mix: M50BA2C 7 day

Height (in)	5.796		Height (in)	5.802	**AVG.**
Diameter (in)	3.038		Diameter (in)	3.037	
Area (in^2)	7.249		Area (in^2)	7.244	
Load number (lb/6.62)	245.000		Load number (lb/6.62)	263.000	
Compressive strength (psi)	223.748		Compressive strength (psi)	240.345	**232.05**

Mix: M60BA2C 7 day

Height (in)	5.742		Height (in)	5.818	**AVG.**
Diameter (in)	3.037		Diameter (in)	3.036	
Area (in^2)	7.244		Area (in^2)	7.239	
Load number (lb/6.62)	390.000		Load number (lb/6.62)	311.000	
Compressive strength (psi)	356.404		Compressive strength (psi)	284.397	**320.4**

Mix: M70BA2C 7 day

Height (in)	5.820		Height (in)	5.735	**AVG.**
Diameter (in)	3.029		Diameter (in)	3.025	
Area (in^2)	7.206		Area (in^2)	7.187	
Load number (lb/6.62)	392.000		Load number (lb/6.62)	400.000	
Compressive strength (psi)	360.127		Compressive strength (psi)	368.449	**364.29**

Mix: M80BA2C 7 day

Height (in)	5.711		Height (in)	5.776	**AVG.**
Diameter (in)	3.016		Diameter (in)	3.026	
Area (in^2)	7.144		Area (in^2)	7.192	
Load number (lb/6.62)	307.000		Load number (lb/6.62)	269.000	
Compressive strength (psi)	284.475		Compressive strength (psi)	247.618	**266.05**

<u>Bottom Ash with 2% cement 14 day Strength</u>

Mix: M40BA2C 14 day

Height (in)	5.649		Height (in)	5.767	**AVG.**
Diameter (in)	3.039		Diameter (in)	3.043	
Area (in^2)	7.254		Area (in^2)	7.273	
Load number (lb/6.62)	646.000		Load number (lb/6.62)	572.000	
Compressive strength (psi)	589.575		Compressive strength (psi)	520.667	**555.12**

Mix: M50BA2C 14 day

Height (in)	5.840		Height (in)	5.764	**AVG.**
Diameter (in)	3.046		Diameter (in)	3.034	
Area (in^2)	7.287		Area (in^2)	7.230	
Load number (lb/6.62)	845.000		Load number (lb/6.62)	655.000	
Compressive strength (psi)	767.653		Compressive strength (psi)	599.761	**683.71**

Mix: M60BA2C 14 day

Height (in)	5.841		Height (in)	5.757	**AVG.**
Diameter (in)	3.044		Diameter (in)	3.048	
Area (in^2)	7.277		Area (in^2)	7.297	
Load number (lb/6.62)	830.000		Load number (lb/6.62)	691.000	
Compressive strength (psi)	755.017		Compressive strength (psi)	626.926	**690.97**

Mix: M70BA2C 14 day

Height (in)	5.831		Height (in)	5.795	**AVG.**
Diameter (in)	3.044		Diameter (in)	3.052	
Area (in^2)	7.277		Area (in^2)	7.316	
Load number (lb/6.62)	1078.00		Load number (lb/6.62)	978.000	
Compressive strength (psi)	980.613		Compressive strength (psi)	884.989	**932.8**

Mix: M80BA2C 14 day

Height (in)	5.802		Height (in)	5.683	**AVG.**
Diameter (in)	3.034		Diameter (in)	3.049	
Area (in^2)	7.230		Area (in^2)	7.301	
Load number (lb/6.62)	603.000		Load number (lb/6.62)	788.000	
Compressive strength (psi)	552.146		Compressive strength (psi)	714.463	**633.3**

Bottom Ash with 2% cement 28 day Strength

Mix: M40BA2C 28 day

Height (in)	5.595	Height (in)	5.728	**AVG.**
Diameter (in)	3.037	Diameter (in)	3.044	
Area (in^2)	7.244	Area (in^2)	7.277	
Load number (lb/6.62)	765.000	Load number (lb/6.62)	463.000	
Compressive strength (psi)	699.101	Compressive strength (psi)	421.172	**560.14**

Mix: M50BA2C 28 day

Height (in)	5.768	Height (in)	5.781	**AVG.**
Diameter (in)	3.040	Diameter (in)	3.039	
Area (in^2)	7.258	Area (in^2)	7.254	
Load number (lb/6.62)	800.000	Load number (lb/6.62)	1051.000	
Compressive strength (psi)	729.644	Compressive strength (psi)	959.201	**844.42**

Mix: M60BA2C 28 day

Height (in)	5.900	Height (in)	5.783	**AVG.**
Diameter (in)	3.035	Diameter (in)	3.039	
Area (in^2)	7.234	Area (in^2)	7.254	
Load number (lb/6.62)	1028.000	Load number (lb/6.62)	820.000	
Compressive strength (psi)	940.684	Compressive strength (psi)	748.377	**844.53**

Mix: M70BA2C 28 day

Height (in)	5.840	Height (in)	5.807	**AVG.**
Diameter (in)	3.041	Diameter (in)	3.045	
Area (in^2)	7.263	Area (in^2)	7.282	
Load number (lb/6.62)	1401.000	Load number (lb/6.62)	1478.000	
Compressive strength (psi)	1276.949	Compressive strength (psi)	1343.594	**1310.3**

Mix: M80BA2C 28 day

Height (in)	5.753	Height (in)	5.824	**AVG.**
Diameter (in)	3.040	Diameter (in)	3.037	
Area (in^2)	7.258	Area (in^2)	7.244	
Load number (lb/6.62)	1141.000	Load number (lb/6.62)	1187.000	
Compressive strength (psi)	1040.655	Compressive strength (psi)	1084.749	**1062.7**

Bottom Ash with 3% Cement 7 day Strength

Mix: M40BA3C 7 day

Height (in)	5.698		Height(in) 2:	5.705	**AVG.**
Diameter (in)	3.042		Diameter(in) 2:	3.044	
Area (in^2)	7.268		Area(in^2) 2:	7.277	
Load number (lb/6.62)	314.0		Load number (lb/6.62)	356.00	
Compressive strength (psi)	286.009		Compressive strength (psi)	323.83	**304.92**

Mix: M50BA3C 7 day

Height (in)	5.769		Height (in)	5.655	**AVG.**
Diameter (in)	3.024		Diameter (in)	3.022	
Area (in^2)	7.182		Area (in^2)	7.173	
Load number (lb/6.62)	296.000		Load number (lb/6.62)	329.0	
Compressive strength (psi)	272.833		Compressive strength (psi)	303.65	**288.24**

Mix: M60BA3C 7 day

Height (in)	5.743		Height (in)	5.746	**AVG.**
Diameter (in)	3.029		Diameter (in)	3.026	
Area (in^2)	7.206		Area (in^2)	7.192	
Load number (lb/6.62)	318.0		Load number (lb/6.62)	283.0	
Compressive strength (psi)	292.144		Compressive strength (psi)	260.50	**276.32**

Mix: M70BA3C 7 day

Height (in)	5.744		Height (in)	5.816	**AVG.**
Diameter (in)	3.029		Diameter (in)	3.031	
Area (in^2)	7.206		Area (in^2)	7.215	
Load number (lb/6.62)	336.0		Load number (lb/6.62)	428.0	
Compressive strength (psi)	308.680		Compressive strength (psi)	392.68	**350.68**

Mix: M80BA3C 7 day

Height (in)	5.650		Height (in)	5.723	**AVG.**
Diameter (in)	3.018		Diameter (in)	3.013	
Area (in^2)	7.154		Area (in^2)	7.130	
Load number (lb/6.62)	445.0		Load number (lb/6.62)	356.0	
Compressive strength (psi)	411.803		Compressive strength (psi)	330.53	**371.17**

Bottom Ash with 3% cement 14 day Strength

Mix: M40BA3C 14 day

Height (in)	5.862	Height (in)	5.741	**AVG.**
Diameter (in)	3.024	Diameter (in)	3.040	
Area (in^2)	7.182	Area (in^2)	7.258	
Load number (lb/6.62)	574.000	Load number (lb/6.62)	381.000	
Compressive strength (psi)	529.074	Compressive strength (psi)	347.493	**438.28**

Mix: M50BA3C 14 day

Height (in)	5.749	Height (in)	5.741	**AVG.**
Diameter (in)	3.035	Diameter (in)	3.038	
Area (in^2)	7.234	Area (in^2)	7.249	
Load number (lb/6.62)	666.000	Load number (lb/6.62)	892.000	
Compressive strength (psi)	609.432	Compressive strength (psi)	814.624	**712.03**

Mix: M60BA3C 14 day

Height (in)	5.768	Height (in)	5.742	**AVG.**
Diameter (in)	3.038	Diameter (in)	3.041	
Area (in^2)	7.249	Area (in^2)	7.263	
Load number (lb/6.62)	998.000	Load number (lb/6.62)	1014.000	
Compressive strength (psi)	911.430	Compressive strength (psi)	924.215	**917.82**

Mix: M70BA3C 14 day

Height (in)	5.763	Height (in)	5.757	**AVG.**
Diameter (in)	3.048	Diameter (in)	3.048	
Area (in^2)	7.297	Area (in^2)	7.297	
Load number (lb/6.62)	919.000	Load number (lb/6.62)	731.000	
Compressive strength (psi)	833.784	Compressive strength (psi)	663.217	**748.5**

Mix: M80BA3C 14 day

Height (in)	5.801	Height (in)	5.791	**AVG.**
Diameter (in)	3.032	Diameter (in)	3.034	
Area (in^2)	7.220	Area (in^2)	7.230	
Load number (lb/6.62)	465.000	Load number (lb/6.62)	769.000	
Compressive strength (psi)	426.346	Compressive strength (psi)	704.147	**565.25**

Bottom Ash with 3% Cement 28 day Strength

Mix: M40BA3C 28 day

Height (in)	5.804	Height (in)	5.799	**AVG.**
Diameter (in)	3.041	Diameter (in)	3.041	
Area (in^2)	7.263	Area (in^2)	7.263	
Load number (lb/6.62)	491.000	Load number (lb/6.62)	746.000	
Compressive strength (psi)	447.524	Compressive strength (psi)	679.945	**563.73**

Mix: M50BA3C 28 day

Height (in)	5.742	Height (in)	5.710	**AVG.**
Diameter (in)	3.039	Diameter (in)	3.035	
Area (in^2)	7.254	Area (in^2)	7.234	
Load number (lb/6.62)	754.000	Load number (lb/6.62)	593.000	
Compressive strength (psi)	688.142	Compressive strength (psi)	542.632	**615.39**

Mix: M60BA3C 28 day

Height (in)	5.752	Height (in)	5.831	**AVG.**
Diameter (in)	3.037	Diameter (in)	3.038	
Area (in^2)	7.244	Area (in^2)	7.249	
Load number (lb/6.62)	723.000	Load number (lb/6.62)	911.000	
Compressive strength (psi)	660.719	Compressive strength (psi)	831.976	**746.35**

Mix: M70BA3C 28 day

Height (in)	5.850	Height (in)	5.817	**AVG.**
Diameter (in)	3.047	Diameter (in)	3.043	
Area (in^2)	7.292	Area (in^2)	7.273	
Load number (lb/6.62)	1345.000	Load number (lb/6.62)	1396.000	
Compressive strength (psi)	1221.084	Compressive strength (psi)	1270.719	**1245.9**

Mix: M80BA3C 28 day

Height (in)	5.785	Height (in)	5.726	**AVG.**
Diameter (in)	3.037	Diameter (in)	3.034	
Area (in^2)	7.244	Area (in^2)	7.230	
Load number (lb/6.62)	1185.000	Load number (lb/6.62)	1106.000	
Compressive strength (psi)	1082.921	Compressive strength (psi)	1012.726	**1047.8**

<u>Bottom Ash with 4% Cement 7 day Strength</u>

Mix: M40BA4C 7 day

Height (in)	5.766	Height (in)	5.680	**AVG.**
Diameter (in)	3.034	Diameter (in)	3.033	
Area (in^2)	7.230	Area (in^2)	7.225	
Load number (lb/6.62)	317.000	Load number (lb/6.62)	492.000	
Compressive strength (psi)	290.266	Compressive strength (psi)	450.805	**370.54**

Mix: M50BA4C 7 day

Height (in)	5.718	Height (in)	5.731	**AVG.**
Diameter (in)	3.030	Diameter (in)	3.035	
Area (in^2)	7.211	Area (in^2)	7.234	
Load number (lb/6.62)	425.000	Load number (lb/6.62)	495.000	
Compressive strength (psi)	390.186	Compressive strength (psi)	452.956	**421.57**

Mix: M60BA4C 7 day

Height (in)	5.813	Height (in)	5.751	**AVG.**
Diameter (in)	3.030	Diameter (in)	3.022	
Area (in^2)	7.211	Area (in^2)	7.173	
Load number (lb/6.62)	454.000	Load number (lb/6.62)	535.000	
Compressive strength (psi)	416.811	Compressive strength (psi)	493.779	**455.29**

Mix: M70BA4C 7 day

Height (in)	5.647	Height (in)	5.690	**AVG.**
Diameter (in)	3.017	Diameter (in)	3.021	
Area (in^2)	7.149	Area (in^2)	7.168	
Load number (lb/6.62)	457.000	Load number (lb/6.62)	512.000	
Compressive strength (psi)	423.188	Compressive strength (psi)	472.864	**448.03**

Mix: M80BA4C 7 day

Height (in)	5.797	Height (in)	5.799	**AVG.**
Diameter (in)	3.026	Diameter (in)	3.027	
Area (in^2)	7.192	Area (in^2)	7.196	
Load number (lb/6.62)	588.000	Load number (lb/6.62)	558.000	
Compressive strength (psi)	541.262	Compressive strength (psi)	513.307	**527.28**

Bottom Ash with 4% Cement 14 day Strength

Mix: M40BA4C 14 day

Height (in)	5.734		Height (in)	5.705	**AVG.**
Diameter (in)	3.027		Diameter (in)	3.037	
Area (in^2)	7.196		Area (in^2)	7.244	
Load number (lb/6.62)	802.000		Load number (lb/6.62)	914.000	
Compressive strength (psi)	737.764		Compressive strength (psi)	835.266	**786.52**

Mix: M50BA4C 14 day

Height (in)	5.686		Height (in)	5.803	**AVG.**
Diameter (in)	3.036		Diameter (in)	3.029	
Area (in^2)	7.239		Area (in^2)	7.206	
Load number (lb/6.62)	1018.000		Load number (lb/6.62)	1155.000	
Compressive strength (psi)	930.920		Compressive strength (psi)	1061.088	**996**

Mix: M60BA4C 14 day

Height (in)	5.872		Height (in)	5.722	**AVG.**
Diameter (in)	3.044		Diameter (in)	3.034	
Area (in^2)	7.277		Area (in^2)	7.230	
Load number (lb/6.62)	887.000		Load number (lb/6.62)	1042.000	
Compressive strength (psi)	806.868		Compressive strength (psi)	954.124	**880.5**

Mix: M70BA4C 14 day

Height (in)	5.815		Height (in)	5.747	**AVG.**
Diameter (in)	3.039		Diameter (in)	3.038	
Area (in^2)	7.254		Area (in^2)	7.249	
Load number (lb/6.62)	806.000		Load number (lb/6.62)	1074.000	
Compressive strength (psi)	735.600		Compressive strength (psi)	980.837	**858.22**

Mix: M80BA4C 14 day

Height (in)	5.780		Height (in)	5.759	**AVG.**
Diameter (in)	3.034		Diameter (in)	3.046	
Area (in^2)	7.230		Area (in^2)	7.287	
Load number (lb/6.62)	833.000		Load number (lb/6.62)	832.000	
Compressive strength (psi)	762.750		Compressive strength (psi)	755.843	**759.3**

Bottom Ash with 4% Cement 28 day Strength

Mix: M40BA4C 28 day

Height (in)	5.763	Height (in)	5.706	**AVG.**
Diameter (in)	3.030	Diameter (in)	3.034	
Area (in^2)	7.211	Area (in^2)	7.230	
Load number (lb/6.62)	952.000	Load number (lb/6.62)	753.000	
Compressive strength (psi)	874.017	Compressive strength (psi)	689.496	**781.76**

Mix: M50BA4C 28 day

Height (in)	5.558	Height (in)	5.721	**AVG.**
Diameter (in)	3.025	Diameter (in)	3.027	
Area (in^2)	7.187	Area (in^2)	7.196	
Load number (lb/6.62)	1029.000	Load number (lb/6.62)	1002.000	
Compressive strength (psi)	947.835	Compressive strength (psi)	921.745	**934.79**

Mix: M60BA4C 28 day

Height (in)	5.750	Height (in)	5.768	**AVG.**
Diameter (in)	3.029	Diameter (in)	3.025	
Area (in^2)	7.206	Area (in^2)	7.187	
Load number (lb/6.62)	1139.000	Load number (lb/6.62)	1220.000	
Compressive strength (psi)	1046.389	Compressive strength (psi)	1123.769	**1085.1**

Mix: M70BA4C 28 day

Height (in)	5.764	Height (in)	5.795	**AVG.**
Diameter (in)	3.029	Diameter (in)	3.030	
Area (in^2)	7.206	Area (in^2)	7.211	
Load number (lb/6.62)	xxxx	Load number (lb/6.62)	xxxx	
Compressive strength (psi)		Compressive strength (psi)		**xxxx**

Mix: M80BA4C 28 day

Height (in)	5.763	Height (in)	5.801	**AVG.**
Diameter (in)	3.021	Diameter (in)	3.025	
Area (in^2)	7.168	Area (in^2)	7.187	
Load number (lb/1)	12000.00	Load number (lb/1)	11900.00	
Compressive strength (psi)	1674.133	Compressive strength (psi)	1655.794	**1665**

Penetration Resistance Testing

M90BA

time	2hr	4hr	8hr	12hr	24hr	14day
area (in^2)	0.75	1	0.5	0.2	0.1	0.025
1	18	38	26	22	32	Over
2	12	35	22	30	30	mechanical
3	12	38	22	24	26	limit
avg (psi)	18.66667	37	46.66667	126.6667	293.3333	

M83BA

time	2hr	4hr	8hr	12hr	24hr	14day
Area (in^2)	1	1	0.5	0.2	0.1	0.025
1	26	34	34	25	26	Over
2	22	36	32	22	24	mechanical
3	23	42	30	23	25	limit
Avg (psi)	23.66667	37.33333	64	116.6667	250	

M75BA

time	2hr	4hr	8hr	12hr	24hr	14day
Area (in^2)	1	1	0.5	0.2	0.1	0.025
1	10	23	27	16	19	Over
2	10	25	28	18	19	mechanical
3	12	22	26	17	21	limit
Avg (psi)	10.66667	23.33333	54	85	196.6667	

M50BA

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1		12	32	58	Over
2	under	12	41	45	mechanical
3		14	37	52	limit
Avg (psi)		12.66667	36.66667	68.88889	

M25BA

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1			14	23	Over
2	under	under	10	16	mechanical
3			11	25	limit
Avg (psi)			11.66667	28.44444	

M10BA

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1			22	35	Over
2	under	under	18	31	mechanical
3			20	32	limit
Avg (psi)			20	43.55556	

Penetration Resistance Testing

M85RS

time	2hr	4hr	8hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.5	0.2	0.025
1	16	23	26	21	20	Over
2	15	26	27	20	20	mechanical
3	17	21	21	22	21	limit
avg (psi)	16	23.33333	32.88889	42	101.6667	

M80RS

time	2hr	4hr	8hr	12hr	24hr	14day
Area (in^2)	1	1	0.75	0.5	0.2	0.025
1		14	27	33	23	Over
2	under	19	29	35	22	mechanical
3		19	24	30	23	limit
Avg (psi)		17.33333	35.55556	65.33333	113.3333	

M75RS

time	2hr	4hr	8hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.25	0.025
1		9	22	21	24	Over
2	under	10	27	20	22	mechanical
3		10	25	26	24	limit
Avg (psi)		9.666667	24.66667	29.77778	93.33333	

M50RS

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1		10	29	40	Over
2	under	9	25	39	mechanical
3		10	25	38	limit
Avg (psi)		9.666667	26.33333	52	

M25RS

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1			15	26	Over
2	under	under	14	21	mechanical
3			14	22	limit
Avg (psi)			14.33333	30.66667	

M10RS

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1			16	31	Over
2	under	under	10	24	mechanical
3			10	20	limit
Avg (psi)			12	33.33333	

Penetration Resistance Testing

M85FS

time	2hr	4hr	8hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.5	0.2	0.025
1	22	24	18	19	10	64
2	20	30	20	20	11	84
3	18	29	20	20	11	70
avg (psi)	20	27.66667	25.77778	39.33333	53.33333	2906.667

M80FS

time	2hr	4hr	8hr	12hr	24hr	14day
Area (in^2)	1	1	0.75	0.5	0.2	0.025
1	21	34	36	21	17	Over
2	22	34	34	22	13	mechanical
3	23	31	31	26	18	limit
Avg (psi)	22	33	44.88889	46	80	

M75FS

time	2hr	4hr	8hr	12hr	24hr	14day
Area (in^2)	1	1	0.75	0.5	0.2	0.025
1		22	20	24	19	Over
2	under	22	20	20	18	mechanical
3		18	21	20	20	limit
Avg (psi)		20.66667	27.11111	42.66667	95	

M50FS

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1		10	23	34	Over
2	under	11	15	26	mechanical
3		11	19	30	limit
Avg (psi)		10.66667	19	40	

M25FS

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	1	0.025
1			13	26	Over
2	under	under	14	30	mechanical
3			11	27	limit
Avg (psi)			12.66667	27.66667	

M10FS

time	3hr	6hr	12hr	24hr	14day
Area (in^2)	1	1	1	0.75	0.025
1			17	26	Over
2	under	under	14	20	mechanical
3			14	19	limit
Avg (psi)			15	28.88889	

Penetration Resistance Testing

M90BA1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.25	0.025
1	19	39	34	32	Over
2	15	30	44	38	mechanical
3	18	32	34	28	limit
avg (psi)	17.33333	33.66667	49.77778	130.6667	

M83BA1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.25	0.025
1	17	29	30	30	Over
2	14	30	26	33	mechanical
3	14	36	30	24	limit
avg (psi)	15	31.66667	38.22222	116	

M75BA1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.25	0.025
1	11	34	40	44	Over
2	10	30	26	44	mechanical
3	11	29	24	38	limit
avg (psi)	10.66667	31	40	168	

M85RS1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.2	0.025
1	12	30	37	61	Over
2	8	28	38	19	mechanical
3	8	26	40	22	limit
avg (psi)	9.333333	28	51.11111	170	

M80RS1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.2	0.025
1	12	31	45	34	Over
2	8	29	36	36	mechanical
3	10	29	44	27	limit
avg (psi)	10	29.66667	55.55556	161.6667	

M75RS1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	0.75	0.2	0.025
1		20	30	28	Over
2	under	16	32	24	mechanical
3		20	34	28	limit
avg (psi)		18.66667	42.66667	133.3333	

Penetration Resistance Testing

M85FS1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	1	0.75	0.025
1	14	15	22	12	Over
2	11	15	21	14	mechanical
3	11	16	20	18	limit
avg (psi)	12	15.33333	21	19.55556	

M80FS1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	1	0.75	0.025
1		14	18	25	Over
2	under	14	18	22	mechanical
3		13	19	30	limit
avg (psi)		13.66667	18.33333	34.22222	

M75FS1C

time	3hr	6hr	12hr	24hr	14day
area (in^2)	1	1	1	0.75	0.025
1		10	20	22	120
2	under	10	18	19	120
3		11	14	22	110
avg (psi)		10.33333	17.33333	28	4666.667

Appendix C
Data Collected From Pipe Testing

Collected data for 6 inch Pipes
High strength In-Situ Soil with High Strength CLSM
9 inch trench

pressure	0psi	2	5	10	15	20		24hour
deflection 1	0.28	0.28	0.28	0.28	0.28	0.15		0.19
deflection 2	0.283	0.283	0.283	0.267	0.21	0.75		0.218
deflection 3	0.26	0.26	0.26	0.26	0.27	0.15		0.15
center stress	1.49	1.86	5.54	15.52	23.72	30.81		1.77
center stress	0.94	2.15	6.68	19.26	28.68	35.42		3.73
in-situ soil	1.28	2.28	4.8	11.76	18.03	24.63		1.37

Summary

pressure	0psi	2	5	10	15	20		24hour
max d	0	0	0	0.0016	0.0073	0.0208		0.011
avg. p	1.215	2.005	6.11	17.39	26.2	33.115	0	2.75

12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.284	0.284	0.325	0.558	0.762	0.955		0.955
deflection 2	0.401	0.401	0.156	0.085	-0.289	-0.48		-0.232
deflection 3	0.3	0.3	0.3	0.5	0.21	0.39		0.31
center stress	1.81	4.1	7.87	11.28	13.93	15.71		4.22
center stress	1.42	8.05	18.42	27	35.52	37.1		4.89
in-situ soil	0.43	4.57	8.28	12.21	16.06	20.04		1.09

Summary

pressure	0psi	5	10	15	20	25		24hour
max d	0	0	0.0245	0.0486	0.069	0.0881		0.0633
avg. p	1.615	6.075	13.145	19.14	24.725	26.405	0	4.555

15 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	#	#	#	#	#	#	#	#
deflection 2	0.784	0.749	0.713	0.522	0.414	0.305	0.21	0.388
deflection 3	0.01	0.01	0	0	-0.09	-0.21	-0.31	0.3
center stress	2.65	2.51	9.88	17.29	23.17	28.25	32.02	8.2
center stress	1.52	1.42	9.31	14.68	18.57	22.36	25.42	7.11
in-situ soil	0.45	3.47	7.45	11.25	15.28	19.67	24.25	1.31

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.0035	0.0071	0.0262	0.037	0.0479	0.0574	0.0446
avg. p	2.085	1.965	9.595	15.985	20.87	25.305	28.72	7.655

18 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.315	0.315	0.315	0.315	0.315	0.315	0.315	0.335
deflection 2	0.624	0.624	0.575	0.475	0.38	0.29	0.22	0.222
deflection 3	0.33	0.33	0.33	0.33	0.29	0.2	0.12	0.12
center stress	2.14	2.65	6.8	12.3	17.85	22.56	26.85	-2.66
center stress	0.47	1	6.47	10.89	14.21	16.89	19.21	5.47
in-situ soil	0	1.6	4.39	7	9.79	12.49	15.19	0.45

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0	0.0049	0.0149	0.0244	0.0334	0.0404	0.0402
avg. p	1.305	1.825	6.635	11.595	16.03	19.725	23.03	1.405

Collected Data for 6 inch Pipes
High Strength In-Situ Soil with Low Strength CLSM
9 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0121	0.0355	0.1138	0.1757	0.238	0.2855	0.3365	0.2855
deflection 2	-0.0242	-0.0483	-0.1194	-0.179	0.2421	-0.2909	-0.345	-0.2892
deflection 3	-0.003	0.008	0.057	0.109	0.166	0.213	0.264	0.205
center stress	##							
center stress	1.21	7.52	14.63	18.52	21.78	24.78	28.36	5.36
in-situ soil	0.22	3.61	7.41	10.93	14.82	18.12	22.1	2.05

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.024	0.1017	0.1636	0.2259	0.2734	0.3244	0.265
avg. p	1.21	7.52	14.63	18.52	21.78	24.78	28.36	5.36

12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0121	0.0081	0.0579	0.0945	0.1402	0.1778	0.2205	0.1646
deflection 2	0.0032	-0.0184	-0.065	-0.102	-0.1485	-0.1871	-0.2331	-0.1825
deflection 3	-0.004	-0.004	0.028	0.06	0.103	1441	0.184	0.156
center stress	1.95	3.77	6.29	8.81	11.32	13.7	16.27	3.03
center stress	1.84	7.84	14.15	18.57	21	23.84	26.15	6.89
in-situ soil	##	##	8.32	10.93	14	16.84	20.41	2.24

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.0216	0.07	0.1066	0.1523	0.1903	0.2362	0.1857
avg. p	1.895	5.805	10.22	13.69	16.16	18.77	21.21	4.96

15 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0274	0.0034	0.068	0.0975	0.1219	0.1464	0.1646	0.1402
deflection 2	-0.0154	-0.0294	-0.0627	-0.0903	-0.1121	-0.1339	-0.1506	-0.1135
deflection 3	0.029	0.032	0.065	0.097	0.123	0.149	0.17	0.139
center stress	2.47	7.55	16.41	22.7	27.17	30.45	33.24	7.27
center stress	1.15	6.63	18.15	28.63	36.57	43.26	48.63	8.36
in-situ soil	1.46	5.53	10.11	15.33	##			

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.014	0.0473	0.0749	0.0967	0.12	0.141	0.1128
avg. p	1.81	7.09	17.28	25.665	31.87	36.855	40.935	7.815

18 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0	0.0162	0.0487	0.0944	0.1504	0.2042	0.2631	0.2083
deflection 2	0.0132	-0.0177	-0.0508	-0.0975	-0.1538	-0.209	-0.2669	-0.1269
deflection 3	0.043	0.07	0.101	0.147	0.203	0.256	0.312	0.224
center stress	1.77	6.57	10.3	13.98	18.08	21.25	24.75	3.96
center stress	0.89	7.52	11.84	15.31	19.15	21.89	24.94	4.15
in-situ soil	0.68	3.52	6.59	10.61	15.14	20.54	25.3	0.73

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.0309	0.064	0.1107	0.167	0.2222	0.2801	0.2083
avg. p	1.33	7.045	11.07	14.645	18.615	21.57	24.845	4.055

Collected Data for 6 inch Pipes
Low Strength In-Situ Soil with High Strength CLSM
9 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0445	-0.052	-0.069	-0.092	-0.1094	-0.1265	-0.1465	-0.1281
deflection 2	-0.064	-0.0673	-0.0812	-0.1073	-0.1238	-0.1391	-0.1539	-0.1494
deflection 3	0.078	0.083	0.086	0.119	0.136	0.159	0.175	0.159
center stress	-3.64	6.85	13.75	19.11	22.93	26.8	31.51	7.1
center stress	-1.58	7.57	18.42	25.21	30.41	36.68	41.63	3.66
in-situ soil	-0.05	3.02	6.95	10.38	13.68	16.33	20.59	1.09

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.0245		0.0649		0.102	
avg. p			16.085		26.67		36.57	

12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0749	0.0783	0.0943	0.1032	0.1126	0.1249	0.1315	0.1154
deflection 2	0.0425	0.0449	0.0604	0.0691	0.0775	0.0877	0.0933	0.0814
deflection 3	0.083	0.083	0.093	0.104	0.113	0.125	0.131	0.13
center stress	1.72	5.54	14.07	19.62	24.94	29.88	33.14	6.99
center stress	-2	1.1	8	12.01	16.05	19.31	22.05	1.84
in-situ soil	0.73	5.35	9.61	14.64	19.49	25.26	29.97	0.86

Summary

pressure	0psi	5	10	15	20	25		24hour
max d			0.0194		0.0377		0.0566	
avg. p			11.04		20.5		27.6	

15 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0281	-0.02	0.0029	0.0127	0.0245	0.0344	0.0418	0.0348
deflection 2	0.037	0.0435	0.0576	0.0641	0.0717	0.0793	0.086	0.0773
deflection 3	-0.038	-0.038	-0.023	-0.007	0.006	0.018	0.028	0.028
center stress	1.07	4.42	8.01	11	13.33	15.29	17.15	5.54
center stress	-1.79	2.63	8.94	15.57	20.05	24	27.15	5.68
in-situ soil	0.5	3.66	6.59	10.16	13.77	17.43	20.96	0.86

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.031		0.0527		0.0699	
avg. p			8.43		16.91		24.06	

Collected Data for 6 inch Pipes
Low Strength In-Situ Soil with Low Strength CLSM
9 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0345	0.1574	0.2743	0.3648	0.4369	0.4938	0.5517	0.4126
deflection 2	-0.0514	-0.1708	-0.2872	-0.3747	-0.4435	-0.4991	-0.5561	-0.3666
deflection 3	0.041	0.152	0.261	0.344	0.407	0.459	0.512	0.366
center stress	2.61	8.53	11.79	15.19	17.94	20.32	22.84	4.94
center stress	-0.32	5.63	8.31	11.47	13.89	15.94	18.15	1.63
in-situ soil	#	#	#	#	#	#	#	#

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.2398		0.4024		0.5172	
avg. p			10.05		15.915		20.495	

12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0365	-0.006	#	0.1717	0.2601	0.3231	0.3882	0.2459
deflection 2	-0.135	-0.0967	#	0.0758	0.1621	0.2199	0.2799	0.1098
deflection 3	-0.0932	-0.0885	#	-0.0696	-0.0605	-0.0546	-0.0485	-0.0638
center stress	2.33	9.04	13.61	17.01	20.37	23.31	26.94	3.44
center stress	-0.16	5.73	9.68	12.94	16.21	19.05	22.63	0.47
in-situ soil	0.27	4.94	10.61	16.38	22.83	27.78	33.86	0.59

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d					0.2966		0.4247	
avg. p			11.645		18.29		24.785	

15 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0243	0.904	0.189	0.2814	0.3597	0.4408	0.5151	0.4095
deflection 2	0.0403	0.0287	0.1285	0.2215	0.3011	0.3827	0.4552	0.2115
deflection 3	-0.0879	-0.0815	-0.0714	-0.0623	-0.0546	-0.0496	-0.0396	-0.0607
center stress	2.61	7.27	10.81	14.07	17.48	20.51	23.58	4.14
center stress	0	4.31	7.05	9.63	12.42	14.89	17.31	1.57
in-situ soil	0.36	4.53	9.88	15.24	21	26.72	32.99	0.68

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.1647		0.3354		0.4908	
avg. p			8.93		14.95		20.445	

Collected data for 8 inch Pipes
High strength In-Situ Soil with High Strength CLSM
12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.3848	0.2336	0.2127	0.1051	0.0247	-0.0282	-0.0907	-0.0161
deflection 2	0.0966	0.0323	-0.0789	-0.1824	-0.265	-0.3186	-0.3825	-0.3107
deflection 3	-0.0688	-0.0003	0.112	0.2191	0.3008	0.3526	0.4132	3577
center stress	2.19	8.48	15.61	20.18	24	26.38	30.06	6.99
center stress	0	7.31	15.71	20.26	23.57	25.52	28.73	5.42
in-situ soil	0.27	4.02	9.47	15.24	21.09	25.4	31.94	0.73

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.1808		0.3696		0.482	
avg. p			15.66		23.785		29.395	

16 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.3411	0.3373	0.3088	0.2775	0.2438	0.2165	0.187	0.2141
deflection 2	-0.1258	-0.1258	-0.1064	-0.078	-0.0435	-0.0148	0.015	0.0052
deflection 3	-0.0921	-0.904	-0.0625	-0.0297	-0.0065	0.0368	0.0699	0.0362
center stress	2.05	3.77	7.41	9.69	10.91	11.98	13.42	3.77
center stress	-1	1.36	8.49	14.73	18.42	21.78	25.78	3.94
in-situ soil	0.41	3.43	7.09	10.98	16.06	20.86	27.76	0.45

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.0323		0.0973		0.162	
avg. p			7.95		14.665		19.6	

20 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0131	0.0131	0.0032	-0.0105	-0.0323	-0.0476	-0.0625	-0.0472
deflection 2	0.0384	0.0384	0.0384	0.0301	0.0096	-0.0057	-0.0204	-0.0074
deflection 3	-0.012	-0.012	-0.012	-0.006	0.017	0.033	0.05	0.048
center stress	1.44	2.51	6.24	9.04	10.67	11.98	12.86	3.54
center stress	-1.64	-0.69	2.84	5.05	6.42	7.31	8.36	0.57
in-situ soil	0.41	3.11	6.04	9.19	13.54	16.29	19.54	0.18

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.0099		0.0454		0.0856	
avg. p			4.54		8.545		10.61	

Collected Data for 8 inch Pipes
High Strength In-Situ Soil with Low Strength CLSM

12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0426	0.2073	0.4639	0.627	0.7316	0.8282		0.4969
deflection 2	-0.0614	-0.2435	-0.4843	-0.6785	-0.785	-0.8312		-0.5257
deflection 3	0.038	0.215	0.457	0.656	0.765	0.866		0.52
center stress	1.91	6.89	9.93	11.84	13.65	15.91		4.7
center stress	0.31	6.68	11.63	15.52	18.89	22.47		3.21
in-situ soil	0.66	4.34	6.4	11.25	15.78	20.64		0.13

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.1821	0.4229	0.618	0.727	0.828		0.482
avg. p	1.11	6.785	10.78	13.68	16.27	19.19		3.955

16 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0264	-0.0159	0.0254	0.0558	0.0975	0.128	0.1646	0.1178
deflection 2	-0.0123	-0.0359	-0.087	-0.1199	-0.1633	-0.1935	-0.2291	-0.171
deflection 3	0.005	0.005	0.009	0.027	0.07	0.101	0.137	0.097
center stress	2.65	11.14	18	21.07	22.33	24.38	26.06	8.2
center stress	0	4.94	10.84	14.73	17.31	29.15	22.63	5.21
in-situ soil	1.09	2.79	6.68	10.2	14.32	18.62	23.43	-0.14

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.0236	0.0747	0.1076	0.151	0.1812	0.2168	0.1587
avg. p	1.325	8.04	14.42	17.9	19.82	26.765	24.345	6.705

20 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0782	-0.0701	-0.0345	0.0091	0.0528	0.0955	0.1493	0.065
deflection 2	-0.0108	-0.0314	-0.0668	-0.1036	-0.1403	-0.1765	-0.2219	-0.1296
deflection 3	0.006	0.006	0.006	0.006	0.038	0.074	0.119	0.119
center stress	2.19	6.1	11.32	15.01	18.22	21.58	23.63	5.68
center stress	0.31	4.47	9.1	11.47	13.73	16.31	18.15	4.54
in-situ soil	1.83	3.43	5.4	7.91	10.7	14.18	18.16	0.36

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d	0	0.0206	0.056	0.0928	0.131	0.1737	0.2275	0.1432
avg. p	1.25	5.285	10.21	13.24	15.975	18.945	20.89	5.11

Collected Data for 8 inch Pipes
Low Strength In-Situ Soil with High Strength CLSM
12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	#	#	#	#	#	-0.024	-0.05	-0.041
deflection 2	-0.0149	-0.0149	-0.0149	-0.0448	-0.0874	-0.1217	-0.1476	-0.1477
deflection 3	0.01	0.01	0.039	0.085	0.137	#	#	#
center stress	1.86	6.43	12.58	17.29	20.83	23.44	27.27	4.05
center stress	-1.9	1.89	5.78	8.63	11.05	13.05	16.02	0.36
in-situ soil	1.05	5.44	9.97	14.87	20.13	24.8	30.02	1.14

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.029		0.127		0.1327	
avg. p			9.18		15.94		21.645	

16 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0004	-0.0098	-0.0355	-0.0629	-0.0918	-0.1183	-0.1421	-0.1173
deflection 2	-0.0469	-0.0469	-0.0665	-0.0935	-0.1209	-0.1462	-0.1698	-0.1698
deflection 3	0.0013	#	#	#	#	#	#	#
center stress	2.23	8.71	17.66	24.33	27.92	30.95	32.82	9.97
center stress	-1.48	4.73	13.94	21.52	25.73	29.31	31.52	5.42
in-situ soil	1.18	3.98	7.16	10.43	13.72	17.25	20.54	0.77

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.0359		0.0922		0.1425	
avg. p			15.8		26.825		32.17	

20 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.1114	0.1054	0.0845	0.0594	0.0443	0.0248	0.0078	0.0325
deflection 2	0.0853	0.0853	0.0721	0.0468	0.0329	-0.0139	-0.0031	-0.0034
deflection 3	-0.0986	-0.0971	-0.0828	-0.0612	-0.0476	-0.0285	-0.0106	-0.0145
center stress	2	5.78	13.56	17.01	21.11	23.49	25.87	8.06
center stress	-1.64	0.31	6.1	10.68	15.36	18.31	21.1	2.63
in-situ soil	0.64	4.3	8.69	12.81	17.52	22.56	27.91	0.32

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.0269		0.0671		0.106	
avg. p			9.83		18.235		23.485	

Collected Data for 8 inch Pipes
Low Strength In-Situ Soil with Low Strength CLSM
12 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0782	-0.0091	0.0914	0.2083	0.2825	0.3678	0.4562	0.2144
deflection 2	-0.0506	-0.1308	-0.2378	-0.3604	-0.4353	-0.5246	-0.6141	-0.353
deflection 3	0.065	0.155	0.277	0.4	0.481	0.573	0.661	0.432
center stress	2.14	7.31	11.56	14.96	18.64	21.35	24.52	2.84
center stress	0.47	5.78	8.84	11.1	13.94	16.15	18.89	1.42
in-situ soil	0.22	3.98	8.05	13.08	17.71	22.65	28.55	0.45

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.212		0.416		0.596	
avg. p			10.2		16.29		21.705	

16 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.0264	0.0711	0.1991	0.3211	0.4146	0.5182	0.5893	0.2855
deflection 2	0.0398	-0.0609	-0.1965	-0.3227	-0.4165	-0.5193	-0.5896	-0.2573
deflection 3	-0.058	0.047	0.185	0.314	0.397	0.481	0.551	0.257
center stress	3.03	8.34	11.65	14.31	16.59	18.64	20.13	5.31
center stress	0.52	5.15	8.52	11.57	14.31	16.94	19.05	2.42
in-situ soil	0.13	2.83	7.09	11.53	15.65	20.36	23.97	0.45

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.243		0.455		0.6294	
avg. p			10.085		15.45		19.59	

20 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	0.0264	0.063	0.1056	0.1839	0.2825	0.3455	0.4349	0.2439
deflection 2	-0.0281	-0.0621	-0.0976	-0.1648	-0.2562	-0.3145	-0.4023	-0.2097
deflection 3	0.018	0.046	0.082	0.149	0.235	0.288	0.367	0.209
center stress	1.53	2.75	3.35	3.82	4.28	4.7	5.36	1.95
center stress	-0.16	5	7.73	10.78	13.26	15.31	17.26	2
in-situ soil	0	5.17	9.42	15.42	22.24	27	33.13	0.68

Summary

pressure	0psi	5	10	15	20	25	30	24hour
max d			0.0792		0.2561		0.4085	
avg. p			7.73		13.26		17.26	

24 inch trench

pressure	0psi	5	10	15	20	25	30	24hour
deflection 1	-0.003	-0.0042	0.118	0.222	0.322	0.409	0.496	0.23
deflection 2	-0.0386	-0.0519	0.1251	0.2263	0.328	0.4115	0.5012	0.2326
deflection 3	#							
center stress	2.56	5.87	7.31	9.05	10.9	13.12	14.68	2.75
center stress	-0.85	5.36	7.78	11.17	13.36	16.37	19.26	0.73
in-situ soil	1.09	3.56	6.4	13.56	16.15	21.11	26.04	0.77

Summary

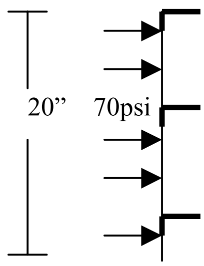
pressure	0psi	5	10	15	20	25	30	24hour
max d			0.121		0.325		0.499	
avg. p			7.545		12.13		16.97	

Appendix D
Calculations for the Pipe Testing Apparatus

Refer to Figure 3.1, Figure 3.2, and Figure 3.3 in chapter 3 for the schematic diagrams of the pipe testing apparatus.

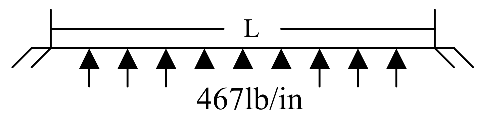
1.) Reinforcing Bars

Each side of the test box is reinforced with three steel angles. The deflection of the angles needs to be controlled. Each side can be modeled as a fixed-fixed connection with the load evenly distributed between the three reinforced angles.



W_{avg} = 70psi(20in)/3

W_{avg} = 467 lb/in

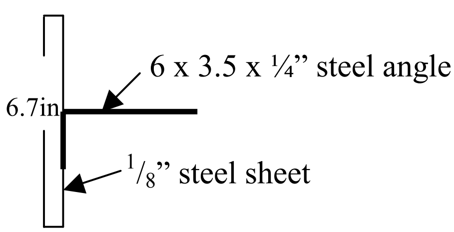


Using Equation 3.1

$$.y = (W)(L^4)/384(E)(I)$$

Using y=0.01 and E=29,000ksi, The minimum moment of inertia (I), that is required for the reinforcing members can be calculated.

<u>40 inch side</u> I_{min}=(467lb/in)(40^4)/384(29,000ksi)(0.01in)
I_{min}=10.74in^4



Using the equation for moment of inertia:
$I = \Sigma[bh^3/12 + Ad^2]$

I = 11.45in4

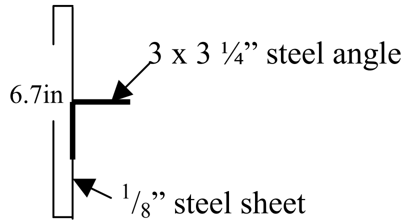
This number was substituted back into the original equation to find the expected deflections on the forty-inch side of the box.

$$.y = (467\text{lb/in})(40^4)/384(29{,}000\text{ksi})(11.45\text{in}^4)$$
$$.y = 0.0091\text{in}$$

25 inch side

I_{min}=(467lb/in)(25^4)/384(29,000ksi)(0.01in)

I_{min}=1.64in^4



Using the equation for moment of inertia:

$I = \Sigma[bh^3/12 + Ad^2]$

$I = 1.68in^4$

.y = (467lb/in)(25^4)/384(29,000ksi)(1.68in^4)

.y = 0.0098in

2.) Bottom and Lid Reinforcements

The bottom required similar reinforcement as the 25 inch side because it is effectively two pieces that are both 25 inch by 20 inch. However, the bottom was reinforced much stronger than it needs to be because extra reinforcement was used to accommodate a footing for the box. The two lid pieces are also the same size (25 inch by 20 inch) but they are supported with four 3 inch x 3 inch x ¼ inch thick members because they could not be spaced in a way to evenly distribute the load because of instrumentation that must pass through the lid. Both of these areas used the information in the above equations and are much stronger than they need to be.

3.) Lid Fasteners

The lid will be held on by ½ inch diameter bolts with a manufacturer specified yield strength equal to 70,000 psi. Using Equation 3.3 the average stress in all the bolts can be calculated.

$$\sigma = (P)(FS)/(A)(N)$$

A = cross sectional area of a bolt = $\pi/4(1/2)^2$ = 0.196in^2

N = number of bolts to be used

P = Total load = 70psi(40in)(25in) = 70k

FS = factor of safety (assumed) = 2.5

$\sigma_{design} = 2/3\ \sigma_{yield}$ = 2/3(70,000psi) = 46.7ksi

Solving for the number of bolts needed:

$N = (70k)(2.5)/(0.196in^2)(46.7ksi)$

$N = 19.12$ bolts

$\therefore$ Use 20 ½" diameter bolts

Next, by rearranging Equation 3.3 to obtain Equation 3.4, the actual factor of safety in the bolts can be solved as:

$FS = \sigma_{design}(A)(N)/(P)$

$FS = (46.7ksi)(0.196)(20)/70k$

$FS = 2.62$

4.) Weld Strength

The weld strength was calculated using the weld lengths of each side and the total force exerted on each side. Eq. (3.5) from the text, is the equation that was used:

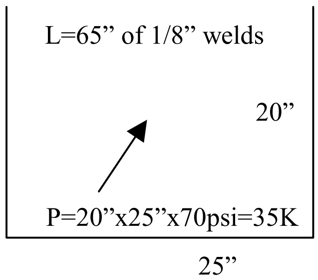
$$\sigma_{weld} = 0.3\ \sigma_{tensile} 0.707\omega$$

This equation specifies a strength per unit length of weld, where ω is the thickness of the member being welded, and $\sigma_{tensile}$ for the weld material is 60,000psi. Solving these equations for $^1/_8$" welds and ¼" welds, gives the following results.

σ_{weld} for 1/8" welds = 1590lb/in

σ_{weld} for ¼" welds = 3180lb/in

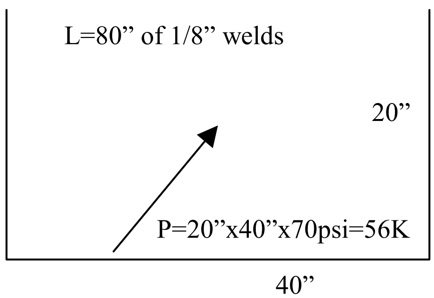
25 inch side



There is an additional 24 inch of ¼ inch weld from the reinforcing members that adds to the strength of this welded section. Using Equation 3.6:

$FS = \Sigma\sigma_{weld}*L/P$
FS = 1590lb/in*65in/35k + 3180lb/in*24in/35k
FS = 5.13

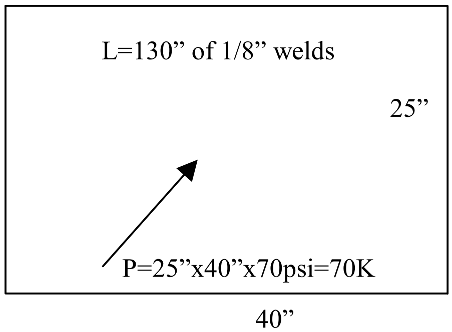
40 inch side



There is an additional 24 inch of ¼ inch weld from the reinforcing members that adds to the strength of this welded section. Using Equation 3.6:

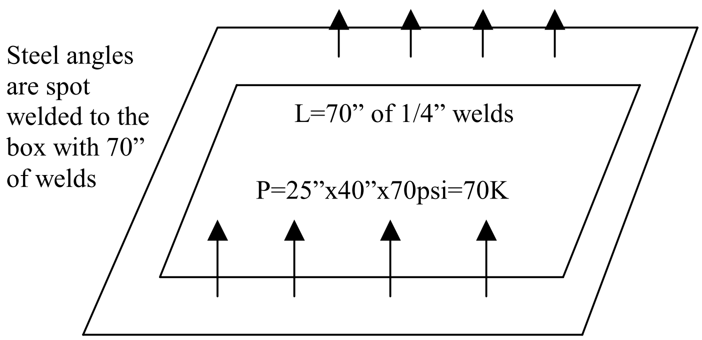
$FS = \Sigma\sigma_{weld}*L/P$
FS = 1590lb/in*80in/56k + 3180lb/in*24in/56k
FS = 3.63

Bottom



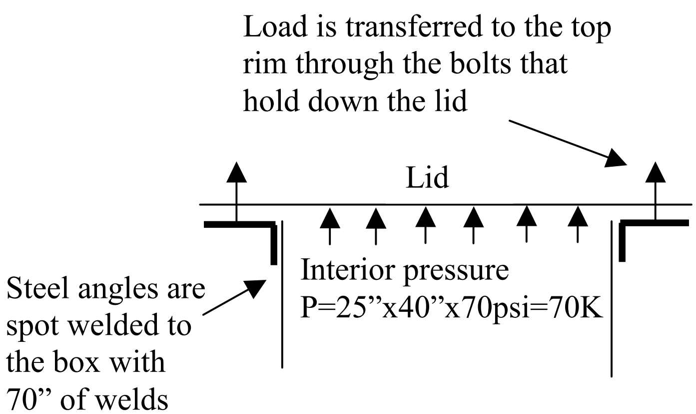
$FS = \Sigma\sigma_{weld}*L/P$
$FS = 1590 lb/in*130 in/70k$
$FS = 2.95$

Top Rim of Reinforcement



The 70k load is transferred to the welds of this section through the lid fastening bolts. This whole top rim of reinforcement could theoretically be pulled up from the box. Using the same equations as above:

$FS = \Sigma\sigma_{weld}*L/P$
$FS = 3180 lb/in*70 in/70k$
$FS = 3.18$



5. Design of the Testing Apparatus at 32 psi Operating Pressure

In the experimental procedures, the pressure did not exceed 32 psi. This means that the actual factors of safety, and the actual deflections in the experimental program will be much more conservative than the previously calculated numbers. All of the calculations have been linear with respect to the applied pressure, so the recalculation of all of these numbers is a simple process.

Expected deflections in the box on the 40-inch side.

From Eq. (3.1) $.y = (W)(L^4)/384(E)(I)$

$.y = 32psi(20/3)(40^4)/(384)(29{,}000ksi)(11.45in^4)$ = 0.004"

Expected deflections in the box on the 25-inch side.

From Eq. (3.1) $.y = (W)(L^4)/384(E)(I)$

$.y = 32psi(20/3)(25^4)/(384)(29{,}000ksi)(1.68in^4)$ = 0.004"

Factor of safety against failure of the lid bolts.

From Eq. (3.4) $FS = \sigma_{design}(A)(N)/(P)$

$FS = 46.7k(0.196in^2)(20)/(32psi)(40in \times 25in)$ = 5.72

Factor of safety against failure of the welds on the 25-inch side.

From Eq. (3.6) $FS = \Sigma\sigma_{weld}*L/P$

$FS = [1590lb/in(65in) + 3180lb/in(24in)]/(32psi)(25in \times 20in)$ = 11.22

Factor of safety against failure of the welds on the 40-inch side.

From Eq. (3.6) $FS = \Sigma\sigma_{weld}*L/P$

$FS = [1590lb/in(80in) + 3180lb/in(24in)]/(32psi)(40in \times 20in)$ = 7.94

Factor of safety against failure of the welds on the bottom of the box.

From Eq. (3.6) $FS = \Sigma\sigma_{weld}*L/P$

$FS = [1590lb/in(130in)]/(32psi)(40in \times 25in)$ = 6.45

Factor of safety against failure of the welds on the top rim of reinforcement.

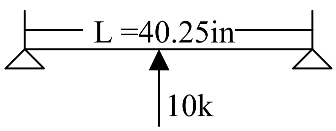
From Eq. (3.6) $FS = \Sigma\sigma_{weld}*L/P$

$FS = [3180lb/in(70in)]/(32psi)(40in \times 25in) = 6.96$

6. **Design of the Reaction Frame**.

The Reaction Frame is supported by four 3 inch x 3 inch x ¼ inch vertical steel angles that are bolted to the footing of the test box. It was designed to apply direct axial loading onto the soil. The load beam was designed in the same way as the reinforcing members, using the equation for deflection to solve for a minimum moment of inertia. The load beam can be modeled like a pin-pin beam with a center load of 10,000 lb, the maximum applied load. Equation 3.7 was used.

Choosing a load bar

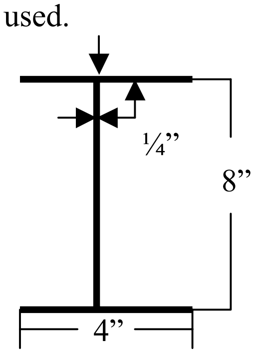


$.y = (P)(L^3)/48(E)(I)$

Rearranging to obtain

$I = (P)(L^3)/48(E)(y)$

A maximum deflection of y=0.02 inch and Young's modulus of E=29,000 ksi were used.



$I = (10k)(40.25in)^3/(48)(29{,}000ksi)(0.02in)$

$I = 23.4in^4$

A 4 x 8 x ¼" I-beam was used.

$I = \Sigma[bh^3/12 + Ad^2]$

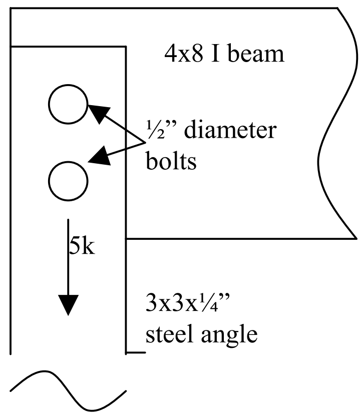
$I = 38.8in^4$

Substituting back into Equation 3.7, the maximum expected deflection can be calculated.

$.y = (10k)(40.25in)^3/(48)(29{,}000ksi)(38.8in^4)$

$.y = 0.012in$

Shearing of the bolts



The load bar has two ½" bolts at each connection, with a manufacturer specified yield strength of 70,000 psi. By considering the free body diagram, it can be shown that there is 5,000lb acting on each side, therefore, each bolt must hold 2,500lb in a double shear configuration. Using equations 8, 9, and 10 the factor of safety against bolt failure can be found.

The stress induced in the bolts is found with Equation 3.9

$$\sigma = P/2A$$

$$\sigma = 2{,}500\text{lb} / 2(0.196\text{in}^2)$$

$$\sigma = 6377.55\text{psi}$$

The design shear stress is found with Equation 3.8

$$\sigma_{shear} = 0.5\sigma_{design}$$

$$\sigma_{shear} = 0.5(2/3*70{,}000\text{psi})$$

$$\sigma_{shear} = 23{,}333.33\text{psi}$$

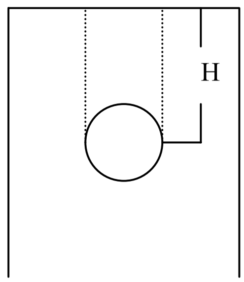
Factor of safety is the ratio of these two numbers as in Equation 3.10

$$FS = \sigma_{shear} / \sigma$$

$$FS = 23{,}333.33\text{psi}/6377.55\text{psi}$$

$$FS = 3.67$$

Shearing of the supports



The steel in the web is also subject to failure. It is only ¼” thick and must carry the load from the bolts. The spacing of the bolts is 2” and the specified yield strength of the steel is 60,000psi. Since this portion is not in double shear the equation for the induced shear stress is Equation 3.11, and the area is found with Equation 3.12.

The area of the potential failure is found with Equation 3.12

$$A = H(2)\ (\tfrac{1}{4}\ \text{in})$$

$$A = 2\text{in}(2)\ (\tfrac{1}{4}\ \text{in})$$

$$A = 1\text{in}^2$$

The induced stress is calculated with Equation 3.11

$$\sigma = P/A$$

$$\sigma = 2{,}500\text{lb} / 1\text{in}^2$$

$$\sigma = 2{,}500\text{psi}$$

The design shear stress is found with Equation 3.8

$$\sigma_{shear} = 0.5\sigma_{design}$$

$$\sigma_{shear} = 0.5(2/3 * 60{,}000\text{psi})$$

$$\sigma_{shear} = 20{,}000\text{psi}$$

Factor of safety is the ratio of these two numbers as in Equation 3.10

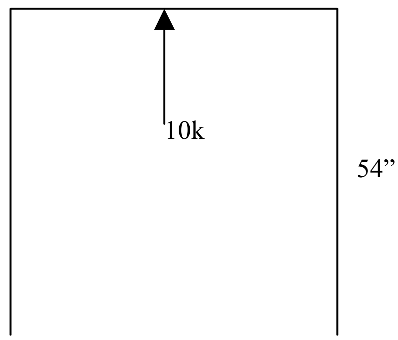
$$FS = \sigma_{shear} / \sigma$$

$$FS = 20{,}000\text{psi}/2{,}500\text{psi}$$

$$FS = 8$$

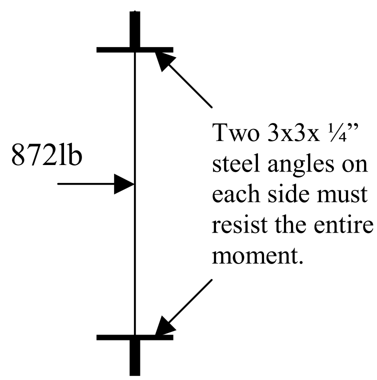
Horizontal bending of the load bar

View from side



The load bar has a 54” moment arm measured from the bottom of the bar to the point of the applied load. assuming a maximum twist in the load equal to 5^0, then the horizontal force that would cause an induced moment would be $\sin 5^0(10k)$. This horizontal force is used in the equation for deflections of a cantilever section (Eq. (3.13)).

View from top



$.y = (P)(L^3)/3(E)(I)$

$P = \sin 5^0(10k) = 872lb$
$E = 29{,}000ksi$
$I = \Sigma[bh^3/12 + Ad^2] = 10.36in^4$

$.y = (872lb)(54in)^3/3(29{,}000ksi)(10.36in^4)$

$.y = 0.15in$

This number seams high but this is an expected maximum. All measures were taken to insure that there was no horizontal deflection of the load bar.