**INFLUENCE OF MATERIAL TYPE, AGGREGATE SIZE,
AND UNCONFINED COMPRESSIVE STRENGTH ON
WATER JETTING OF CIDH PILE ANOMALIES**

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TITLE: Influence of Material Type, Aggregate Size, and
Unconfined Compressive Strength on Water Jetting of
CIDH Pile Anomalies

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ABSTRACT

Influence of Material Type, Aggregate Size, and Unconfined Compressive Strength on Water Jetting of CIDH Pile Anomalies

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Water jetting as a means for removing anomalous materials from cast-in-drilled-hole (CIDH) piles was examined. The primary objective of this research was to establish empirical relationships between different jetting parameters and the removal of commonly occurring anomalous zone materials, including low-strength concrete, slurry mixed concrete, grout, and clay soil. Also investigated was the current standard-of-practice used by water jetting contractors within California. The testing specimens consisted of typical anomalous material with unconfined compressive strengths between 5 and 6,000 psi. The experimental work consisted of water blasting submerged specimens using rotary jets, nozzles, and pumping equipment typically used in construction practice. Two testing protocols were developed. The first testing protocol called for the nozzle to be held stationary and the second allowed the nozzle to be cycled up and down across the anomaly. During testing, material removal rates were measured as a function of jet pressure and standoff distance. Water blasted specimens were cut apart after testing to confirm erosion measurements and to permit inspection of the water blasted surfaces. Based on the results, erosion rates and the effectiveness of water jetting are primarily influenced by unconfined compressive strength, when using standard test equipment and jetting pressures. Further, aggregate size and material type in the anomalous material does not appear to influence both total erosion and erosion rate.

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1 INTRODUCTION

Engineers use cast-in-drilled-hole (CIDH) piles to support a variety of civil engineering works, including buildings, towers, and bridges. Depending on the soil conditions and the location of the groundwater table, drilling slurries and/or steel casings may be used to help ensure borehole stability during construction. If borehole stability is not ensured, then anomalies can form within the pile during concrete placement. These anomalies have the potential to impact the design performance of the pile once it goes into service.

When CIDH piles are constructed in “wet” ground conditions using drilling slurries, it is common for engineers and owners to require quality control testing. This testing typically confirms the integrity of the pile, post-construction, by verifying the density of the concrete. If testing indicates the density of the pile differs significantly from that of sound concrete, the area in question is considered an anomaly.

When an anomaly is discovered within a CIDH pile, the engineer will document its location, size, and areal extent. The engineer will then determine if the anomaly compromises the integrity of the pile. If this is the case, the owner will be required to repair the anomaly or recommend an appropriate design alternative. The various methods that currently exist to repair anomalies within a CIDH piles will be briefly discussed in this thesis. Typically, the repair methods consist of a cleaning phase (to remove the deleterious material) and a grouting phase (to fill the void left after the cleaning phase). A repair method is typically selected by the contractor and approved by

the engineers and owners. The engineers or owners may require post-repair verification before a repair is finally accepted.

The California Department of Transportation (Caltrans) is typically the lead agency on freeway projects in California; however, city and county agencies may also be involved. Quality control methods have been extensively researched and are generally accepted by Caltrans. Brian Liebich, of Caltrans' Foundation Testing Division, has conducted much of this research. Caltrans currently has standards for two non-destructive testing (NDT) methods used to verify the quality of concrete. These methods will be briefly discussed herein.

As noted above, the repair method consists of a cleaning phase and a grouting phase. The grout phase may consist of cement grouting, which has been studied by other researchers. However, the cleaning phase has not been researched extensively. Due to limited research on water jetting and the unfamiliarity on the subject matter by local agencies, approval of the repair can take months. Delayed repairs then delay the rest of construction, costing both the agencies and the contractors' money.

1.1 Scope

The purpose of the proposed research is to establish an empirical relationship between water jetting pressure and the removal of commonly occurring anomalous materials and sound concrete. The research will consist primarily of laboratory testing under controlled conditions of which the result will lead to qualitative and quantitative relationships. One

goal of this research is to provide a basis for development of a “standardized” jetting technique, as well as increased confidence in water jetting as a repair procedure.

2 BACKGROUND AND LITERATURE REVIEW

It is important to define what an anomaly is with respect to this research. An anomaly is any material existing within the confines of a CIDH pile for which the CIDH pile was not originally designed. It is also important to note here that not all CIDH piles which contain anomalies are repaired. Only when an anomaly compromises the performance of the pile is it considered a defect (Skeen and Liebich 2004; Liebich 2002; O'Neill and Reese 1999). Piles with defects may then be repaired.

As noted, while water jetting of CIDH pile anomalies has been conducted by industry for years, very little direct research exists. However, considerable research does exist regarding the efficacy of water jet technology. The use of water jetting in other concrete and soil cutting applications are reviewed herein.

2.1 Anomalies in CIDH Piles

There are at least ten different types of commonly occurring anomalous materials (Liebich and Bonala 2007). These include: clay, sand, sand with small amounts of cement, gravel only, grout only, gravel and grout, a mix of 50% soil and 50% concrete, soil with bands of concrete, slurry-contaminated concrete, loose mortar mix, and varying strengths of concrete. However, it can be imagined that an infinite number of combinations of in-situ soil, groundwater, drilling slurry, and concrete are possible.

To identify the properties of defects it is imperative to first understand the causes of defects. Only defects relating to this research will be discussed. O'Neill (2005) provides a more detailed discussion of defects and their causes. He proposed the following five categories for defects in CIDH piles:

- Defects arising from general constructions problems;
- Defects arising from drilling problems;
- Defects arising from casing management problems;
- Defects arising from slurry management problems; and
- Defects resulting from design deficiencies.

2.1.1 Defects due to General Construction Problems

Three defects commonly arise from general construction problems. The first occurs when concrete is placed by free fall without directing the concrete down the middle of the CIDH pile. O'Neill and Reese (1999) note that concrete dropped in free fall from up to 80 feet can be placed without segregation. It was also noted that concrete can be placed in free fall from as little as 5 feet and maintain the design properties. If the concrete is placed off center, it may come into contact with the sidewalls of the CIDH pile or the reinforcing steel cage, thus reducing the velocity of the concrete when it strikes the bottom, as can be seen in Figure 2.1(a). Thus, the resulting concrete may not reach the desired density and cannot be relied upon for the design strength.

Another general construction defect may occur when a borehole is drilled closely to a recently concreted shaft (O'Neill 2005). Concrete can communicate between boreholes if a layer of weak cohesionless soil crosses both shafts. Concrete from the recently

concreted shaft may pour into the newly drilled borehole through the weak soil layer.

The results are voids in the recently concreted shaft and a weak soil-concrete mix in the new borehole, as can be seen in Figure 2.1(b).

The third general construction defect occurs when concrete is placed through water. This may occur when water has accumulated in the bottom of the borehole. The water may mix with the concrete or leach the cement out of the concrete (O'Neill 2005). Placing concrete through water is shown in Figure 2.1(c).

Figure 2.1: Defects due to General Construction Problems: (a) Segregation of Concrete during Free Fall; (b) Caving between Two CIDH Piles; and (c) Placing Concrete through Water (O'Neill 2005)

2.1.2 Defects due to Drilling Problems

Defects created by drilling problems include sloughing soil, surface material intrusion, and an unclean toe. Sloughing soil may occur during dry construction, depending on in-situ soils, if casing is not used during drilling or during the placing of concrete. Surface material may fall into the borehole during general construction activity as can be seen in

Figure 2.2(a). According to O'Neill (2005), this can be avoided by using temporary surface casing. Failing to clean the toe of a borehole may lead to defects and is of extra concern in end bearing piles, as shown in Figure 2.2(b).

Figure 2.2: Defects due to Drilling Problems: (a) Soil Sloughing into CIDH Pile; (b) Unclean Toe (O'Neill 2005)

2.1.3 Defects due to Casing Management Problems

O'Neill (2005) states that more defects occur from the use of temporary casing than from any other cause. When removing the casing, necking may occur if the casing is not removed before the concrete has had time to cure. Groundwater or soil may fill the void left by necking resulting in a defect. Another anomaly occurs by removing the casing after the concrete has cured. The concrete in the casing may arch, thus separating it from the concrete below resulting in a discontinuity, as shown in Figure 2.3(a). Another casing defect occurs when either an insufficient head of concrete or slurry is maintained above the concrete column, as shown in Figure 2.3(b). In either case, the groundwater

may mix with the concrete resulting in defects. Other defects arise from the incorrect use of casing; however, these defects are relatively rare and are not discussed herein.

Figure 2.3 Defects due to Casing Management Problems: (a) Necking and Arching within a CIDH Pile; (b) Insufficient Head Maintained within a CIDH Pile (O'Neill 2005)

2.1.4 Defects due to Slurry Management Problems

The first defect which arises from slurry management problems occurs when entrained material in slurry falls out of suspension. This material can settle to the bottom of the borehole. It may also attach itself to the reinforcing steel cage or borehole sidewall. In any case, the residual material may form a defect, as shown in Figure 2.4(a).

The slurry may not be weighted properly. This may cause the sidewalls to cave during concrete placement leading to a defect, as shown in Figure 2.4(b). Another defect may arise from insufficient mixing of slurry in the borehole resulting in small chunks of

unhydrated slurry fixed against the reinforcing steel cage or borehole sidewall (O'Neill 2005), as shown in Figure 2.4(c). Defects may occur if a tremie pipe is removed from the concrete then reinserted. Slurry may be forced into the concrete resulting in a defect shown in Figure 2.4(d).

Figure 2.4 Defects due to Slurry Management Problems: (a) Material Falling out of Suspension; (b) Borehole Instability; (c) Unhydrated Slurry Attached to Reinforcing Steel; (d) Slurry Mixed with Concrete (O'Neill 2005)

A recently debated cause of a defect results from the installation of construction joints. A construction or cold joint may be needed when changing from a wet method placement to a dry method placement, when the flow of concrete is interrupted long enough to allow the concrete to begin curing, or when a cutoff below grade is called out on the design plans (O'Neill 2005). Defects are likely to occur at the cold joint if the joint is not properly constructed.

2.1.5 Defects due to Design Problems

Finally, defects can arise from design problems. In order for concrete to reach the borehole side walls, sufficient space between reinforcing steel is needed. Small diameter shafts typically have less space between the reinforcing steel, which can impede concrete flow resulting in defects on the outside of the shaft.

Defects on the perimeter of the shaft may also arise from placement of PVC inspection tubes on the steel reinforcing cage. As will be discussed later, PVC pipes are used for quality control testing. These PVC inspection tubes reduce the clear space between reinforcing steel thus impeding the flow of concrete to the extremities of the borehole.

Previously, Caltrans specifications did not require PVC inspection tubes to be identified on design drawings which resulted in cluttered cages. Caltrans recently modified their design specifications to denote PVC inspection tubes on design drawings. Caltrans believes that this change will allow concrete to reach all areas of the pile, thus reducing defects caused by insufficient clear space.

2.2 Location of Anomalous Material

For the purpose of this discussion, defects will be simplified into two categories: location within the pile and location with respect to the groundwater table. With respect to location of defects in the pile, those occurring inside the reinforcing steel cage and those occurring outside the reinforcing steel cage in the cover area of the pile will be discussed.

The cover portion of a pile is considered the annular space between the reinforcing steel cage and the surrounding soil or rock. Large defects in the cover portion of the pile can reduce the skin friction of the pile and expose reinforcement. Most CIDH piles rely on skin friction to resist loads. Therefore, defects in this area can affect the load carrying capacity of the pile. Defects which occur in the cover portion of a pile may also increase the likelihood of corrosion of the reinforcing steel.

Skeen and Liebich (2004) state that a substantial number of anomalies occur outside the reinforcing steel cage. They also note that anomalous material can be trapped outside the steel reinforcing cage as concrete flows from the tremie toward the perimeter of the pile.

Typically, when a cover defect is above the groundwater table and the reinforcing steel is exposed, corrosion of reinforcing steel is a concern. Corrosion may be of concern below the groundwater as well if the groundwater or soil is corrosive in nature. If the environment is not corrosive, then corrosion of reinforcing steel may not occur.

Defects that occur in the pile interior (inside the steel reinforcing cage) are of main concern when the defect is in the moment carrying portion of the pile. The moment carrying portion of the pile is different for every pile; however, generally it is 5 to 10 diameters below the tip of the pile. The moment carried by the pile is resisted by in-situ soil and the pile itself. If the concrete within the pile is not continuous then the moment cannot be carried through the pile to the soil.

Defects in the core of the pile may also compromise the ability of the pile to carry load to the toe of the pile. If the pile is relying on base resistance, the inability of the pile to carry load to the toe may result in failure. Generally, if the defect is only in the core and cannot communicate with the groundwater table, corrosion of the reinforcing steel is not of concern.

2.3 Detection of Anomalous Material

Caltrans is at the forefront of research on detection of anomalies within CIDH piles, and in particular, Brian Liebich of Caltrans' Foundation Testing Division. As this research was funded by Caltrans, most of the information discussed herein regarding NDT relates to Caltrans standards.

To test for anomalies in CIDH piles, engineers will install inspection tubes along the entire length of the steel reinforcing cage. These tubes serve as a conduit through which NDT equipment passes that analyze the pile. Inspection tubes consist of two-inch inside diameter polyvinyl chloride (PVC) schedule 40 pipe. Caltrans requires one inspection tube per foot of diameter of the pile with a maximum spacing of 33 inches, but no less than two inspection tubes per pile (Caltrans 2008).

Currently, Caltrans employs both Crosshole Sonic Logging (CSL) and Gamma-Gamma Logging (GGL) to detect anomalies. CSL verifies the integrity of the concrete within the core of the pile whereas GGL verifies the integrity of the concrete around the perimeter of the inspection tube (Skeen and Liebich 2004). Caltrans established a standard protocol

for Gamma-Gamma Logging called “Method of Ascertaining the Homogeneity of Concrete in Cast-In-Drilled-Hole (CIDH) Piles Using the Gamma-Gamma Test Method,” or Caltrans Test 233.

GGL contains an internal nuclear source which emits gamma particles that supply a backscatter measurement to an internal receiver as it is pulled up the inspection tube (Skeen and Liebich 2004). Based on the backscatter, the density of the material can be determined and compared to that of the design concrete mix. If the variation in density is greater than twice the standard deviation of the test, then the area may be considered an anomaly (Liebich 2002). According to Skeen and Liebich (2004), the advantages of GGL include that it is a highly repeatable test and that it evaluates concrete outside the steel reinforcing cage. It is not a time dependent test and it requires the least amount of data interpretation.

Two distinct disadvantages of GGL are that it evaluates only 3 inches around the inspection tube and that strict federal regulations exist with respect to transportation of the equipment due to the nuclear source.

CSL uses two probes, a source probe which emits an ultrasonic pulse, and a receiver probe which records the pulse (Skeen and Liebich 2004). The two probes are lifted parallel to one another in different inspection tubes the entire length of the pile. Based on the arrival times of the ultrasonic pulse, the speed of sound in the material can be determined, which is a function of the material density (Skeen and Liebich 2004). If an

area shows, typically, slower arrival times than the surrounding media, than it may be an anomaly. According to Skeen and Liebich (2004), two advantages of CSL are that it can identify anomalies in the core of the pile and the lateral extent of the defect can be determined. Two disadvantages are that it is a time dependent test and that it will not work if the inspection tubes have debonded from the concrete. Another two disadvantages of CSL are that it cannot identify anomalous material outside of the reinforcing cage and data interpretation is not always repeatable because the data analyst subjectively selects the wave arrival time.

Caltrans only requires GGL for non destructive testing. However, CSL is used to verify the extent of the anomaly. Both are used when identifying a potentially anomalous area within a pile as the two methods complement each other. GGL and CSL give the engineer a more complete picture of the pile cross section, thus allowing the cleaning phase to be more subjective.

2.4 Mitigation of Anomalies

O'Neill and Reese (1999) discuss five types of repairs that may be used to repair defects in CIDH piles, namely grouting, hand repairs, underpinning with microshafts, removal and replacement, and straddle shafts. The hand repair methods discussed by O'Neill and Reese are still in use. They mention that it may be economically feasible to excavate around the pile and repair the anomalous area by hand. They also note that the designer should be consulted as to the quality of backfill placed around the top of the pile.

It is noted in Skeen and Liebich (2004) that removing an anomaly by water jetting and then grouting the resulting cavity is usually the least expensive option when the defect is deeper than three meters. Skeen and Liebich (2004) detail the typical water jetting process.

The “cleaning operation” begins with a high-pressure wash typically around 13,000 psi and 5 gpm which is intended to remove the PVC inspection tube and the anomaly within the CIDH pile. The authors suggest that water pressures around 15,000 – 20,000 psi will cut concrete and at 400 psi the PVC tube will be cut. The pressure wash typically begins 24 inches below and ends 24 inches above the identified defect to provide complete overlap. Water exiting the top of the inspection tube may provide an indication of the material being removed based on the suspended content, according to the authors. Water may flow out of a secondary tube during the water jetting operation which suggests that the tubes have hydraulic connectivity or communication. The authors state that good practice necessitates a second wash operation after the high pressure, low volume wash is complete. The secondary washing is low pressure, high volume. Typically around 45 gpm and 100 – 200 psi jetting will erode additional defective material that may not have been previously flushed.

Two methods for grouting are described in Skeen and Liebich (2004): “Replacement Grouting” and “Permeation Grouting.” First, during replacement grouting, any residual water left in the cavity is removed. Then, high-cement content structural grout is pumped into the cavity made during the water jetting process. If the cavity cannot be dewatered,

tremie placement of the grout is used. The authors recommend that replacement grouting occur under pressure, typically 230 psi.

Permeation grouting, is used when the surrounding soil has high permittivity. This method may prevent corrosion from occurring on reinforcing steel. First, a high-water, low-cement grout is pumped into the cavity, followed by increasingly higher-cement grouts until pressure reaches 230 psi. This creates a large grout bulb that replaces the cavity created by the water jetting operation.

ADSC developed a “Standard Mitigation Plan” cooperatively with the West Coast Chapter of the ADSC, Caltrans, and Holdrege and Kull. The plan was developed to serve as a template for mitigating anomalies within CIDH piles once the anomalies have been detected. The plan is broken into two parts: Standard Mitigation Plan “A” – Basic Repair and Standard Mitigation Plan “B” – Grouting Repair. Standard Mitigation Plan A focuses on manual removal of the anomaly and Standard Mitigation Plan B focuses on water jetting to remove anomalies. Although these plans exist, considerable uncertainties still remain. Thus, it is imperative to have a contractor who has experience in water jetting. The standard mitigation plans are discussed in more detail in Chapter 3.

2.5 Use of Water Jets in Soil

Atmatzidis and Ferrin (1983) evaluated the effects of specific soil properties, such as degree of saturation and dry density, time of exposure, and traversing velocity of the jet, with respect to cutting of soil. Four different soils, ranging from a clean sand to a clayey

soil, were subjected to a tungsten carbide water jet with a nozzle diameter of 1 mm (~0.039 inches) and pressures up to 8,000 psi sprayed through air. The nozzle was held approximately 6 inches from the initial cutting surface. The water jet traversed the samples at a constant velocity up to 1 inch per second.

The authors investigated the penetration of the water jet as a function of time and as a function of traversing velocity. However, traversing velocity is not pertinent to this research. They also investigated the effects of dry density, degree of saturation, permeability, and strength.

The authors found that the maximum penetration into soil occurs on the order of “tens of seconds,” with less for granular, cohesionless soils and more for cohesive soils. They found at a pressure of 1,000 psi the maximum depth of penetration was achieved in 15 to 20 seconds. The depth of penetration can be exponentially related to the cumulative time from initial impact using the general formula:

$$h = h_{\max}(1 - e^{-\frac{t}{\tau}})$$

where h is the penetration at time t after initial impact, $h_{\max}$ is the limiting penetration for infinite time, and τ is a constant (with units of time) which depends on jet and material properties.

Atmatzidis and Ferrin (1983) state that increasing the dry density at constant water content results in decreasing depth of jet penetration due to an increase in strength and a decrease in permeability of the soil. Similar results would be found, according to the

authors, if depth of penetration was plotted against void ratio or porosity. The results of this investigation can be found in Figure 2.5.

Figure 2.5 Idealized Relationships between Depth of Penetration and Time from Initial Impact (Atmatzidis and Ferrin 1983)

At full saturation, maximum penetration is achieved for fine grained soils and minimum penetration occurs around 40% to 50% saturation. For clean sand, the maximum penetration was obtained at a degree of saturation around 50%. The minimum penetration occurred at both dry and fully saturated conditions. According to the authors, the degree of saturation affects the permeability, to some extent, and the strength of a soil, which in turn affects the dry density and may affect the data for those investigations.

In general, particle size, void ratio, composition, fabric and degree of saturation affect the permeability. Based on the authors' observations the depth of penetration appears to vary linearly with soil permeability, or more exactly, the square of the void ratio. Changes in

permeability have little effect on the depth of penetration in clean sand; the effect of permeability becomes more prominent as grain size decreases.

With the increase in strength of the soil the depth of penetration decreases. The relationship between unconfined compressive strength and depth of penetration for fine grained material is not linear but does correlate to the degree of degree of saturation and dry density.

Finally, the authors observed that the volume of soil affected is actually larger than the hole created by the water jet. The area affected increases with increasing soil grain size. This may be due to the water permeating into the soil and/or due to the excess pore pressures generated by the water jet.

Rockwell (1981) summarizes research done under the authority of the Civil Engineering Laboratory (CEL) by both the CEL and Flow Industries, Inc. Rockwell (1981) investigated the ability of water jets to excavate trenches on the sea floor for cable and pipeline burial systems. The soil tested had a shear strength of 2 to 3 psi. The single water jet nozzles tested in this research varied between 1 and 4 inches, much larger than the 0.035 to 0.038 inch nozzles tips currently used for CIDH anomaly removal. The nozzles traversed across the top of the soil in excess of 100 feet per minute while removing 36 to 48 inch deep trenches. Although the nozzle types, nozzle diameters, flow rates, operating pressures and translational rates differ vastly from this research, some general observations are noteworthy. Rockwell (1981) states that, in general, increasing

the standoff distance resulted in shallower trench depths and increased trench width. The research team found that the water jet removed soil both in chunks and fine particles. It was also found that increased pressure resulted in increased trench depth. With pressure held constant, trench depth increased with larger nozzle sizes, according to the paper.

One relevant qualitative observation by Rockwell was that the jet did not produce a trench with straight walls. Also observed was that, in general, the trench width was equal to or larger than the nozzle diameter and the water jet did not cut in a smooth manner but often removed material in large chunks.

Under direction of the U.S. Army Corps of Engineers, the Repair, Evaluation, Maintenance, Rehabilitation Research Program (REMR) published a “Technical Note” addressing the use of water jet blasting for removal of deteriorated concrete on bridge decks. REMR (1988) notes that the advantage of water jet blasting is that it “removes the deteriorated concrete, leaving the sound concrete virtually intact and the steel reinforcement undamaged for reuse in the replacement concrete.” One limitation, they suggests, is that these systems use between 7,000 and 10,000 gallons of water during an operating shift. REMR (1988) also notes that “highly trained, experienced personnel are required to operate and maintain water-jet equipment.” Finally, they authors state that typical operating pressures are between 15,000 and 25,000 psi and operating flow rates between 20 and 26 gpm.

2.6 Water Jet Nozzle Research

Most available research describes the evaluation of the efficacy of water jets when tested through air in an un-submerged condition. Rockwell (1981) considered water jets in submerged conditions.

2.6.1 Water Jet Nozzle Research for Material Removal

Wright et al. (1997) examined the effectiveness of pressure, rotation speed, standoff distance, angle of jets, and type of target material. The authors tested two materials: concrete and neoprene rubber. The concrete contained only 3/8 inch coarse aggregate with a design strength of 3,000 psi. The authors state that in concrete, the jets erode the cement paste, leaving the aggregate intact. They also state that the jet cracks the concrete by pressurizing the microjoints. Standoff distances of 1, 4.5, and 10 inches were tested and rotation speeds between 125 to 3,000 rpm. Water pressures of 5,000, 10,000, 15,000, and 20,000 psi were investigated.

Wright et al. (1997) states that at 5,000 psi small amounts of cement paste began to be removed; however, the aggregate was not broken free. Their research found that once the jet pressure reaches the threshold pressure for the material, it is more effective to apply power through increased flow rate than through increased pressure. For the concrete tested and jet used, the material was removed effectively between 500 and 2,000 rpm. The authors observed that with successive passes the aggregate was removed as the jet penetrated deeper into the cement paste. They also found that the maximum depth of cut occurred around 1,000 rpm (22 ft/sec). They determined that it is more effective to make multiple high speed shallow passes than a single slow cut.

When the rotation speed was increased to 3,000 rpm (66 ft/sec) the jet effectiveness decreased due to the degradation of the jet stream as it moved through the air at higher rotational speeds. They also showed that smaller jet streams are more likely to degrade during high rotational speeds and with increased standoff distance. The optimum jet tip speed for the material tested was determined to be between 20 and 50 ft/sec. Finally, they found that increasing the standoff distance from 1 to 4 inches at a rotational speed of 10 ft/sec reduced the effectiveness by 40%.

Wright et al. (2001) tried to identify the optimum combination of pressure and flow for a given power at different standoff distances. According to the authors, most materials have a minimum pressure that must be attained before material is removed; this is known as the threshold pressure. The power of a jet is determined by the stagnation pressure and the mass flow rate at the surface of the material. The jet was shot through air and contacted a wax specimen with a threshold pressure between 5,000 and 6,000 psi and a cement and sand mixture with a threshold pressure around 1,500 psi. The standoff distances varied between 2 and 50 inches and the jet traversed at a rate of 1 foot per second. The results showed that an optimum pressure and flow for a given power at a given standoff distance exists and that it is not dependent on the threshold pressure of the material.

Figure 2.6 shows that the optimum standoff distance for nozzle diameters 0.035 to 0.038 inches is 3.9 to 5.3 inches.

Figure 2.6 Optimum Orifice Diameters at Standoff Distances (Wight et al. 2001)

Wright et al. (1999) analyzed the effectiveness of nine nozzles by varying the flow, pressure, upstream conditions, flow conditions, surface speed, and nozzle angle. A nozzle type commonly used for water blasting was tested in this research. The authors noted that jet quality is influenced by upstream flow path conditions and the nozzle design. They also state that as standoff distance is expressed in terms of nozzle diameters, deterioration across all sizes of nozzle is consistent. Wright et al. (1999) also restate that most materials have a minimum threshold pressure.

The authors found that poor upstream conditions reduced jet performance between 25% and 55% compared to good upstream conditions and that the deterioration increased with increasing flow rate. It was also found that when poor upstream conditions exist, the use

of a van type flow conditioner can improve the performance of the nozzle by up to 40%. Feeder tubes were found to improve jet performance; however, they are not applicable in water jetting of CIDH pile anomalies given the restricted access.

The jet angle was also investigated. Plowing occurs when travel is in the same direction as the jet angle; dragging occurs when travel is in the opposite direction as the jet angle. Jets were angled at 45° and 135° and were analyzed relative to the 90° jet. It was found that when the jets were plowing they performed 12% better and when the jets were dragging they performed 27% poorer than the 90° jets. Improved performance was also found with multiple, successive passes of the angled jets; however, single independent passes were not as effective as the 90° angle.

The surface speeds investigated in their research were between 5 and 40 feet per second, which are outside the range used for water jetting of CIDH pile anomalies. Shown in

Table 2.1 is a summary of the nozzle characteristics mentioned previously and their effect on material removal.

Table 2.1: Summary of Nozzle Characteristics

Nozzle Characteristic	Effect on Removal
Orifice Size	Large Orifices Produce Higher Forces.
Upstream Conditions	Nozzles with Poor Upstream Conditions are Less Efficient than Nozzles with Good Upstream Conditions.
Rotational Speed	High Rotation Speeds Degrade the Water Jet. An Optimum Rotation Speed Exists for Each Orifice Size.
Tranverse Speed	High Transverse Speeds Degrade the Water Jet. Multiple High Speed Passes Are More Effective Than a Single Slow Pass.
Standoff Distance	Increasing Standoff Distance Decreases Jet Efficiency. An Optimum Standoff Distance Exists for a Given Orifice Size.

2.6.2 Nozzle Type

Wright et al. (2007) tested five different tube nozzle designs and analyzed their performance. All nozzles were of the “unplugger” style. Unplugger style or nozzles typically have forward facing jets to break up any material in the path of the jet. Some unplugger style nozzles also have rear facing jets which aid the user by pushing the nozzle through the tube. Four of the nozzles were rotary while one was not. Three of the rotary nozzles operated at 25,000 rpm and one at 1,000 rpm. Nozzles C and E were

designed such that the water flows straight through the nozzle where as Nozzle A had a turbulent flow path. Please see Table 2.2 for complete description of the nozzles.

Table 2.2: Description of Nozzle Types Investigated in the Study by Wright et al. 2007 (after Wright et al. 2007)

Nozzle	Type	Diameter	Forward Jets	RPM
A	Rotary, Leak, Free Spin	25 mm	2	25000
B	Non-Rotary	19 mm	7	NA
C	Rotary, Leak, Free Spin	18 mm	3	25000
D	Rotary, Bearings, Seals	23 mm	3	1000
E	Rotary, Leak, Free Spin	24 mm	3	25000

The tests were conducted at 15,000 psi and 12 to 14 gpm and removal rates were time for each nozzle which are similar pressures and flow rates to those employed during water jetting of anomalies.

The results show that the nozzles with a straight through flow path allowed cleaning at a rate 13 times that of the nozzle with a turbulent flow path. The results also showed that the rotating nozzles removed material at a rate of 27 times that of the non-rotating nozzle. The nozzles commonly used for anomaly removal are straight through flow paths and are rotating nozzles which this paper showed to be the most effective.

Wright et al. (2003) examined three common nozzle material types: steel carbide, and sapphire. Of importance to this research are the results for steel nozzles as replaceable

steel nozzles are the most common nozzle material type used for anomaly removal. The authors state that higher quality steel nozzles may have useable lifetimes of 150 to 200 hours if the water is filter to 25 microns or better at pressures up to 20,000 psi. Wear in steel nozzles occurs in two ways: cavitation and abrasion. According to Wright et al. (2003), cavitation may not occur with replaceable nozzles. However, abrasive erosion will occur if the water contains abrasive particles. Typically abrasive erosion is very smooth and even. The authors recommend the used of steel replaceable nozzles in operating conditions where water is filtered to 25 microns or better and with pressures below 20,000 psi. These conditions are similar to those used in the water jetting studied herein.

3 INDUSTRY STANDARD OF PRACTICE

It is thought that approximately 20% of CIDH piles constructed under slurry have detectable anomalies (Liebich and Bonala 2007). ADSC and Caltrans have attempted to develop best practices for repairing defects within CIDH piles using hand repairs and water jetting, as will be discussed below. However, other options are available. Skeen and Liebich (2004) note that if the anomaly is determined to be unrepairable, the shaft must be partly augmented or replaced to meet the design standards. The partial augmentation may include reinforcing the pile with a steel tube insert or secondary rebar cage. Straddle shafts may be used as a replacement.

If the defect is repairable, Skeen and Liebich (2004) state that, if the shaft is within three-meters of the surface, the pile can be unearthed and repaired. Finally, if the anomaly lies deeper than three-meters below the ground surface, water jetting of the anomalous material through inspection tubes is used.

3.1 Standard Mitigation Methods

ADSC West Coast Chapter (2007) was developed to serve as a template for mitigating anomalies within CIDH piles once an anomaly has been detected. The plan is broken into two parts: Standard Mitigation Plan “A” – Basic Repair and Standard Mitigation Plan “B” – Grouting Repair as previously mentioned.

3.1.1 Standard Mitigation Plan “A”

Standard Mitigation Plan “A” – Basic Repair, also known as Simple Repair, involves the excavating around the pile to the depth of the anomaly, removal of the deleterious material, and the replacement of excavated material. Typically material is excavated along side of the CIDH pile by excavating at an approved slope, or by creating a shored shaft alongside the pile. The plan states that “mechanical removal will ‘chase’ all inclusions or compromised concrete until competent concrete is encountered.” If the anomalous material appears as competent concrete, then hammer testing or hand-chipping to one-inch depth will be conducted to prove deeper concrete is competent as well. If visual inspection and verification of the removal by the Engineer is approved then replacement of concrete shall proceed. Finally, the contractor must restore the earthen material to approximate “undisturbed, in-situ density” (ADSC West Coast Chapter 2007). This type of removal is most common for shallow defects as excavating to deeper depths is not economically viable for contractors.

3.1.2 Standard Mitigation Plan “B”

Standard Mitigation Plan “B” – Grouting Repair begins by removing the PVC inspection tube with a high pressure water jet. The PVC inspection tube is removed two-feet above and below the elevation range of the anomaly. Removal of the PVC inspection is verified. After which, the anomaly is removed with a high-pressure, low-volume wash. The mitigation plan states that water pressures typically range from 9,000 to 15,000 psi and flow rates between 10 and 15 gpm, although lower pressures may be used at the contractor’s discretion once the PVC inspection tube has been removed. During water

jetting, the contractor should monitor the returning effluent for solids periodically by straining. ADSC West Coast Chapter (2007) states that the contractor should also keep a log of “unanticipated communication between holes, water color, type of solids, and estimated solids content.” Washing of the anomaly should continue until the returning effluent is clear, unless the water jetting is deemed to have significantly disturbed the surrounding formation.

Once the anomaly has been removed, a high-volume, low-pressure flush is common. This is done to remove loose material before down-hole camera observation or grouting. Air, water or alternating injections of air and water are used to flush any remaining loose material in the cavity left by the water jetting operation. As with the water jetting operation, the contractor should monitor the returning effluent by straining for solids periodically while also keeping a log of “unanticipated communication between holes, water color, type of solids, and estimated solids content” (ADSC West Coast Chapter 2007). Flushing of the anomaly should continue until the returning effluent is clear, unless the flushing is deemed to have significantly disturbed the surrounding formation.

Down-hole camera observation is typically required after high-pressure water jetting and low-pressure flushing. The down-hole camera is used to verify that the water jetting operation has adequately removed the anomalous material. If the engineer verifies that the water jetting operation successfully removed the anomaly, grouting can begin.

The mitigation plan suggests permeation grouting or replacement grouting as effective methods to fill the cavity created by water jetting. Full description of the procedures used for both permeation grouting and replacement grouting can be found in ADSC West Coast Chapter (2007).

3.2 Current Water Jetting Methods

ADSC West Coast Chapter (2007) does not require specific pressures and flow rates. Instead, ranges are suggested for both cutting PVC and anomaly removal. It also does not specify water jetting equipment including water jet nozzles, water pump equipment, nozzle rotation speeds. The procedure for removing the PVC inspection tube and anomalies is not specified. Due to the competitive nature of the industry, some jetting contractors are reluctant to divulge details regarding their current practice.

Table 3.1 summarizes the water jetting practices employed by three different companies that currently perform mitigation work in California. As seen in Table 3.1, the range of pressures used closely matches with the values suggested by ADSC West Coast Chapter (2007). The volumetric flow rates vary from 10 to 24 gpm for PVC cutting and 10 to 20 gpm for anomaly removal, which are above the ranges suggested by ADSC West Coast Chapter (2007). As noted previously, ADSC West Coast Chapter (2007) does not specify nozzle or water pump type. Values seen in Table 3.1 were either given to the researchers (through a survey), or were inferred from Caltrans documents.

One company noted their standard of practice raises the jet nozzle $\frac{1}{4}$ -inch approximately every minute using a hand crank. For this testing, a Stoneage® Gopher GO-M9 water jet nozzle with 0.035-inch and 0.038-inch AP2 tips on the top and bottom respectively. The water pump was a diesel Jetstream® X3620 capable of 20,000 psi at 13 gpm. A 500 gallon water tank was used as a secondary source of water.

Table 3.1: Current Water Jetting Practices for Three Contractors in California

Nozzle						Water Pump					Operating Procedures			
											Cutting PVC		Jetting Anomaly	
Model	No. of Jets	Orifice Size (inches)	Rotational Speed (rpm)	Operating Pressure (ksi)	Operating Flow Rate (gpm)	Manufacturer	Model	Power Plant Type	Operating Pressure (ksi)	Operating Flow Rate (gpm)	Pressure (ksi)	Flow Rate (gpm)	Pressure (ksi)	Flow Rate (gpm)
Gopher	6	0.035 - 0.038	1000	5 - 25	8 - 25	Jetstream	X-3620	Diesel	Up to 20	Up to 13	10 - 15	20 - 24	10 - 15	20
Badger	5	Unknown	"High Speed"	Up to 15	5 - 15	Not Reported					9 - 12	10 - 15	10 - 15	10 - 15
ary	2	Unknown	Hand Turned	Unknown		NLB	Not Reported				10 - 15	13 - 18	10 - 15	13 - 18

4 MATERIALS

As previously noted, an infinite number of anomalies are possible. To ensure that created samples were representative of actual anomalies, the following case studies were examined.

4.1 Case Studies

Identified in Liebich and Bonala (2007) are at least ten different types of anomalous material including clay, sand, sand with small amounts of cement, gravel only, grout only, gravel and grout, a mix of 50% soil and 50% concrete, soil with bands of concrete, slurry-contaminated concrete, loose mortar mix, and varying strengths of concrete. Although these ten anomalous materials were identified, an infinite combination of anomalous material is possible.

Branagan et al. (2000) detail a case history where CSL detected a defect within a CIDH pile. Anomalies were located within two 7-foot-diameter, 68-foot-long CIDH piles in Overton, Nevada. The authors state that anomalous areas were cored which confirmed the presence of porous concrete, voids and non-cemented materials; the concrete defects appeared to be caused by water and/or soil intrusion. Recovered anomalous material was described as an unconsolidated sand, a mixture of sand and cement, poor-quality concrete, slurry and other contaminants mixed with concrete, and medium-dense to dense-sandy gravel to gravelly sand with some native soil. Other recovered anomalous material was also described as poor concrete and loose aggregates, soft sandy clay to

clayey sand with concrete fragments. Cores from the second pile labeled as “poor concrete” were tested for unconfined compressive strength. The strengths for three different anomalous materials were 350 psi, 1,150 psi, and 1,710 psi.

Skeen and Liebich (2004) discuss the Trabuco Creek Bridge Project where 13-foot-diameter CIDH piles were installed to an average depth of 62 feet. Initial permeation grouting was not effective therefore secondary shafts were excavated and NDT results were verified. The defective material was removed with a low-pressure wash.

Aaron Sykes of Pacific Coast Drilling Company provided cores of an anomaly from a CIDH pile. These cores appeared to be a weak concrete. The cores contain both coarse and fine aggregates. Some portions of the cores can be removed with a finger nail. Although unconfined compressive strength tests were not conducted on these samples, the qualitative descriptions of the cores indicate a likely strength range of 200 to 1500 psi.

Jason Wahleithner of Caltrans Foundation Testing Division provided unconfined compressive strength results for testing conducted on an anomaly detected on Thames Creek Bridge. Three samples broke below 4,000 psi. Their strengths were 680, 1250, and 1380 psi. Pictures of this material showed that the anomaly was fractured and contained both coarse and fine aggregates. These cores can be seen in Figure 4.1.

Figure 4.1: Cores from CIDH Pile from Thomes Creek Bridge

4.2 Test Material Selection

The above case studies provided a broad outline of anomalies observed in the field.

Given limitation of this research, such as budget, time frame, and available materials, it was determined that a small set of samples be created that would represent actual anomalies.

This research identified material type, grain size, and compressive strength as possible factors influencing water jetting of anomalies. These factors were considered when selecting samples for water jet testing. A testing program was developed to systematically investigate the influence of these different parameters. For this study, ten

samples were selected and tested in 23 separate tests. Table 4.1 summarizes the ten different materials examined in this study.

Table 4.1: Water Jetting Material Summary

Material Description	Maximum Aggregate Size (inches)	Aggregate Angularity	Design Compressive Strength (psi)
Semi-Cemented	0.375	Angular	500
Semi-Cemented	0.375	Angular	1250
Concrete	1	Angular	1000
Concrete	1	Angular	2000
Concrete	1	Angular	4000
Concrete	1	Angular	1500
Concrete	1	Rounded	1500
Slurry-Mixed Concrete	1	Angular	1250
Sand-Cement Grout	0.125	---	1250
Bentonite-Cement Mix	<0.002 mm	---	10

4.3 Initial Testing Matrix

Based on the selected materials discussed in the previous section, material type, grain size, and compressive strength were analyzed. Table 4.2 shows the proposed areas of influences studied. Seen in red, three strengths of concrete and two strengths of semi-cemented material were to be compared to determine the effect of unconfined compressive strength on erosion rate. Four materials with compressive strengths between 1,000 and 1,500 psi were to be compared to determine the effect of material type on erosion rate as is seen in gray. The three materials in the blue rectangle were designed to examine the effect of aggregate size on erosion rate. As is seen in green, two materials

were tested to determine the effect of aggregate angularity on erosion rate. Finally, a clay material was tested to determine the effect of the water jet on a weak soil.

Table 4.2: Initial Testing Matrix

Material Type	Target Unconfined Compressive Strengths(psi)				
	0 - 10	500 - 1000	1000 - 1500	2000 - 2500	4000 - 4500
Concrete: Angular Aggregate			✓	✓	✓
Semi- Cemented		✓	✓		
Grout			✓		
Slurry-Mixed Concrete			✓		
Concrete: Rounded Aggregate				✓	
Clay	✓				

4.4 Material Design and Sample Preparation

As little data was available regarding how effective the water jet eroded concrete, a weak material was selected initially for stationary jetting. Chapter 6 will discuss the different testing protocols used in this research.

During the placement of each material, save the sand-cement grout and the bentonite-cement mix, slump measurements were taken according to ASTM C143. The desired

slump of the all materials was between 7- to 9-inches. If the slump of the material was determined to be too low, either water or ADVA® 100 superplasticizer was added until the desired slump was achieved. After the desired slump was reached, the material was poured directly into sample molds. The sand-cement grout mixture was tested for flow and not slump due to the high water content associated with grouts. The bentonite-cement mix was not tested for slump or flow, instead, Marsh Funnel viscosity.

A representative sample was taken from each material and four-inch by eight-inch cylinder molds were made for unconfined compressive testing later. After all sample material was placed in the testing molds, the molds were covered with a plastic sheet and wooden board.

4.4.1 Semi-Cemented Materials

A mix similar to a controlled low strength material was determined to be the most easily placed. Batch testing of this material did not occur. The semi-cemented material consisted of 0.375-inch crushed-granite angular aggregate, sand, cement, and water. Air entrainers and superplasticizers were also added to this mix to achieve the high slump. This material was not vibrated after placement. The semi-cemented material was used for both the stationary and cyclic donut samples and the large sample mold

The semi-cemented material was ordered from Hanson Aggregates, a local concrete supplier, and delivered to the testing site by ready-mix truck. For the stationary testing, both four-inch by eight-inch and six-inch by twelve-inch cylinder molds were made.

Only four-inch by eight-inch cylinder molds were made for cyclic testing. Four cylinder molds (four-inch by eight-inch) were left to cure outdoors adjacent to the donut samples. These cylinder molds were placed in styrofoam to help insulate the material from the elements.

For stationary test, the semi-cemented material had an estimated 28 day strength of 400 psi and was water jetted near 560 psi after 56 days of curing. During cyclic testing, the semi-cemented material was replicated. However, it only cured for 28 days resulting in a strength around 440 psi. Also for cyclic testing, a semi-cemented material with a high unconfined compressive strength was created. The design compressive strength was 1,250 psi; however the as-tested compressive strength was nearly 2,200 psi. A summary of the semi-cemented materials is shown in Table 4.3.

In order to determine the as-tested strength of SCM-02 and SCM-03 cores were taken from the samples after testing. The results shown for SCM-03 represent the strength of the cores taken after testing.

Coring was tried on SMC-02 however the material was too weak to be cored effectively. Therefore, large sections of the sample were removed and cubes made. As will be discussed later, the sample molds were 18 inches tall; sections of the sample were cut from the top and middle six-inches. Approximately four-inch cubes were then cut from the inside and outside of the removed section.

Table 4.3: Summary of Semi-Cemented Materials

Material	SCM-01	SCM-02	SCM-03
Jetting Procedure	Stationary	Cyclic	
Target Strength (psi)	400	400	1250
As-Tested Strength (psi)	560	440	2160
Curing Time (days)	55 - 56	28	28
Coarse Aggregate Size	3/8" x #8		
Coarse Aggregate Type	Crushed Granite		
Cement (lbs/ yd³)	212	212	282
Fly Ash (lbs/ yd³)	70	70	94
Water (lbs/ yd³)	367	367	367
Coarse Aggregate (lbs/ yd³)	700	700	700
Sand (lbs/ yd³)	2302	2302	2219
Total (lbs/ yd³)	3651	3651	3662
Cement (ft³/ yd³)	1.079	1.079	1.435
Fly Ash (ft³/ yd³)	0.488	0.488	0.655
Water (ft³/ yd³)	5.874	5.874	5.874
Air (ft³/ yd³)	0.810	0.810	0.810
Coarse Aggregate (ft³/ yd³)	4.282	4.282	4.282
Sand (ft³/ yd³)	14.468	14.468	13.945
Total (ft³/ yd³)	27.001	27.001	27.001

The cubes were then tested for unconfined compressive strength. Mindess et al. (2002) states that cube tests are used as the standard compressive strength test in Great Britain, Germany, and other European nations. The standard is based on a six-inch cube. Due to the manner in which the cubes were cut, the cubes were around four-inches.

Compression tests have shown that cubes typically measure 25% stronger than cylinders; for weaker material, it is typically around 30% (Mindess et al, 2002). SCM-02 is considered a weaker material thus a reduction factor of 1.3 was applied to the measured cube strengths. This accounted for the difference in testing method. The factor was still applied even though the cubes were only four inches. A summary of the cube strengths are shown in Table 4.4.

Table 4.4: SCM-02 Cube Strength Summary

Specimen	6" Annulus						12" Annulus			
	Top 6 inches				Middle 6 inches		Top 6 inches		Middle 6 inches	
	Inside		Outside		Inside	Outside	Inside	Outside	Inside	Outside
	A	B	A	B	A	A	A	A	A	A
Date Tested	9/26/09	9/26/09	9/26/09	9/26/09	9/26/09	9/26/09	9/26/09	9/26/09	9/26/09	9/26/09
Time Tested	11:22 AM	11:25 AM	11:28 AM	11:32 AM	11:36 AM	11:39 AM	11:43 AM	11:45 AM	11:48 AM	11:51 AM
Age (days)	36.13	36.13	36.13	36.13	36.14	36.14	36.14	36.14	36.14	36.15
Weight in Air (g.)	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd
Weight Underwater (g.)	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd
Density (lbs/ft ³)	nd	nd	nd	nd	nd	nd	nd	nd	nd	nd
Length Pre Capping (in.)	4.000	3.875	3.875	4.250	3.750	3.188	4.125	4.000	4.000	4.000
Length Post Capping (in.)	4.167	4.094	4.052	4.370	3.893	3.331	4.345	4.356	4.247	4.191
Ave. Cap Thickness (in.)	0.08	0.11	0.09	0.06	0.07	0.07	0.11	0.18	0.12	0.10
Length of Sides (in.)	4.00	3.88	3.88	4.25	3.75	3.19	4.13	4.00	4.00	4.00
Area (in ²)	16.00	15.02	15.02	18.06	14.06	10.16	17.02	16.00	16.00	16.00
Max. Load (lbs)	9325	8275	9222	9777	9873	6716	10382	9812	10949	12017
Compressive Strength (psi)	583	551	614	541	702	661	610	613	684	751
Modified Cube Strength (psi)	448	424	472	416	540	508	469	472	526	578

4.4.2 Concrete Materials

Following Table 4.2, five concretes were designed. A partial summary of the properties for the five concrete materials are shown in Table 4.5.

Since the materials were all placed in the donut molds simultaneously, the strengths were varied by increasing the percent of fly ash as a function of total cementitious material.

This allowed the water to cementitious ratio to remain constant. By increasing the percent fly ash of total cementitious material, the strength decreases. CON-01 and CON-04 contained 75% cement (Permanent Type II/V) and 25% Fly Ash (Escalente). The cement to fly ash ratio was determined by analyzing data provided by Daniel Jansen.

The concrete was delivered by Hanson Aggregates in a ready-mix truck. After placement of the concrete in the donut molds all material was vibrated. Four by eight inch cylinders were made. Cylinders were left in Paso Robles to be cured and others were taken to Cal Poly and stored in the cure room.

A second set of concrete materials were also tested. Two concretes were made based on CON-01 from Table 4.5. The first material was identical to CON-01 and was delivered by Hanson Aggregates. The second material contained rounded coarse aggregate as opposed to angular. It also used Colton Type II/V Portland cement instead of Permanente Type II/V. This material was hand mixed as neither local ready-mix supplier had rounded aggregate available in Paso Robles. Instead, the rounded aggregate was provided by Hanson Aggregates and the concrete made using a drum-mixer. Each mix was poured two days prior to testing so that lower strengths would be achieved. Slumps were determined for each material when placed. Four- by eight-inch cylinders were made. Cylinders were left in Paso Robles to be cured and others were taken to Cal Poly and stored in the cure room.

As is shown in Table 4.5, CON-01, CON-02, and CON-03 varied greatly from the original design strengths. As-tested strengths were determined by coring the samples after testing. The mixes contained high amounts of fly ash as is shown in Table 4.5. The strength of CON-01 was not wholly unexpected. It is not unusual for concrete supplied by a ready-mix company to reach in excess of 6000 psi. It was unexpected to see the

strengths of CON-02 and CON-03 to be 33% and 79% higher than the cure room samples respectively.

The initial research, for which these mix designs were based, was conducted in laboratory settings. ASTM C 31 states that curing should occur at $73 \pm 3^{\circ}\text{F}$ and 100% humidity. Hence, the concrete materials were cured around these conditions. The average high temperature over the thirty-three days which these materials cured in Paso Robles was 96°F and the low was 53°F . The humidity averaged around 50% and the overall average temperature was 75°F . Obviously, the curing conditions were much more severe for before testing than for the initial research conducted on fly ash content.

Table 4.5: Summary of Concrete Materials

Mix #	CON-01	CON-02	CON-03	CON-04	CON-05
Target Strength (psi)	1000	2000	4000	2250	2250
As-Tested Strength (psi)	3910	4500	5790	655	2120
Curing Time (days)	34	33	33	2	2
Coarse Aggregate Size	1" x #4				
Coarse Aggregate Type	Angular				Rounded
Percent Fly Ash of Total Cementitious Materials	72	53	25	25	25
Cement (lbs/yd ³)	163	287	506	506	506
Fly Ash (lbs/yd ³)	420	324	169	169	169
Water (lbs/yd ³)	317	317	317	317	317
Coarse Aggregate (lbs/yd ³)	1736	1736	1736	1736	1736
Sand (lbs/yd ³)	1138	1143	1138	1138	1138
Total (lbs/yd ³)	3774	3807	3866	3866	3866
Cement (ft ³ /yd ³)	0.829	1.460	2.574	2.574	2.574
Fly Ash (ft ³ /yd ³)	2.925	2.258	1.177	1.177	1.177
Water (ft ³ /yd ³)	5.073	5.073	5.073	5.073	5.073
Air (ft ³ /yd ³)	0.405	0.405	0.405	0.405	0.405
Coarse Aggregate (ft ³ /yd ³)	10.619	10.619	10.619	10.619	10.619
Sand (ft ³ /yd ³)	7.149	7.186	7.152	7.152	7.152
Total (ft ³ /yd ³)	27.000	27.001	27.000	27.000	27.000

Gardner (1990) found that low-temperature fly ash-cement concretes have lower strength, especially at early ages (less than 14 days), than those cured at higher temperatures.

Haque and Kayyali (1989) found that hot-dry environments are more beneficial to the strength development of fly ash-cement concretes than cement-only concretes. Thus, the higher fly ash concretes (CON-02 and CON-03) were may have been more affected by the high temperatures and dry conditions than CON-01. This may have resulted in CON-02 and CON-03 being stronger than the initial data predicted.

4.4.3 Slurry-Mixed Materials

The slurry-mixed material consisted of the 4,000 psi base concrete mix combined with bentonite slurry. The 4,000 psi base concrete mix is CON-03 in Table 4.5. The concrete was prepared using a 2½ cubic foot electric drum mixer. The bentonite slurry was American Standard 200 mesh. The bentonite slurry mix proportions followed 80 pounds of slurry for every 100 gallons of water. This follows common recommendations for bentonite slurry used for fluid loss control in CIDH piles. The bentonite slurry was slowly added to water and mixed with a wand mixer connected to a power drill in a 32 gallon drum. The slurry was allowed to hydrate for at least 24 hours before mixing with concrete.

Initial batch testing was conducted to determine the mix ratio of concrete to bentonite slurry which would result in unconfined compressive strengths in the 1,000 to 1,500 psi range. The bentonite slurry was mixed by volume with the concrete. Mixes of 10%, 15%, 25%, 37.5%, and 50% bentonite slurry were made. Originally, only the 25%, 37.5%, and 50% mixes were made. Later, 10% and 15% mixes were made. Figure 4.2

shows the results from batch testing of the slurry mixed material for unconfined compressive strength using four by eight inch cylinders. Certain values for the 37.5% and 50% mixes are not seen as the samples either broke during capping or were too weak to provide accurate values during testing.

Figure 4.2 Results of Various Slurry Mixed Material Batch Testing

Two sets of testing were conducted. The first consisted of the SMM_Base, SMM_25, SMM_37.5, and SMM_50 corresponding to the volume of slurry mixed material to concrete (i.e. SMM_25 had 25% bentonite slurry). The second set consisted of the SMM_Base_1, SMM_10, and SMM_15. Due to the variable nature of the testing program, the selected strength would need to be between 14 and 28 days. From Figure 4.2, SMM_15 showed the ideal strength between 14 and 28 days, therefore, a 15% bentonite slurry by volume mix design was selected.

The slurry mixed material was prepared at the Case Pacific yard in Paso Robles. The bentonite slurry was pre-mixed and measured. Once the concrete was made, the bentonite slurry was added. The slurry mixed material was then shoveled into the modified cyclic mold (see Section 5.3.1). This material was not vibrated. Four- by eight- inch cylinders were made and cured both in Paso Robles and in the cure room. The as-tested strength was determined to be 1,850 psi.

4.4.4 Sand-Cement Grout Material

The sand-cement grout consisted of sand, Colton Type II/V Portland cement, and water. The sand-cement grout material was designed to investigate the effect on erosion of a material without any coarse aggregate. Batch trials were made based on total cement content per cubic yard. For example, GTB900 contained 900 pounds of cement per cubic yard. Each mix design contained 350 pounds of water per cubic yard and the amount of sand was iterated upon until each mix was 27 cubic feet absolute volume. Four percent entrapped air was assumed. Instead of testing the slump of this material, the flow was determined. To test the flow, ASTM C939 was followed. Three, seven, and twenty-eight day strengths can be seen in Figure 4.3.

Figure 4.3 Results of Grout Trial Batch Compressive Strength Testing

Given Figure 4.3, GTB500 was selected and placed. The sand-cement grout material was placed by CalPortland, a local concrete supplier, by ready-mix truck. This material was not vibrated and the flow was determined. Four by eight-inch cylinders were made and cured both in Paso Robles and in the cure room. The as tested strength was determined to be 1,900 psi.

4.4.5 Bentonite-Cement Material

The last material prepared was a bentonite-cement material. The clay material was developed to mimic weak, in-situ clay. The clay material was made by mixing Colton Type II/V Portland cement with American Standard 200 mesh bentonite clay or Hydro Drill Mud 220 (double yield) and water. Trial batches were made and tested for compressive strength. A summary of the trial materials is seen in Table 4.6.

Table 4.6: Bentonite-Cement Trial Batch Summary

	Bentonite Type	Water to Cement to Bentonite Ratio (by weight)	Marsh Funnel Viscosity (seconds)
Mix A	American Standard 200 Mesh	5:1:0.92	50
Mix B	Hydro Drill Mud 220 (double yield)	5:1:0.68	120
Mix C	Hydro Drill Mud 220 (double yield)	5:1:0.92	>200

The trial batches were made by mixing the cement and water initially in a 150 gallon tub. Wand mixers attached to the end of electric drills were used to combine the material. Next, the bentonite was slowly sifted into the tub through a number 60 sieve. A shovel was dragged across the bottom of the container to check for clumps. After all material was incorporated, the mixture sat for 5 to 10 minutes. Marsh Funnel Viscosity was checked before continuing. These mixing procedures were based on Mikkelsen (2002)

After sitting, the bentonite-cement mixture was poured into both four- by eight-inch cylinder molds and 2.5- by 6-inch open-ended sample molds. The 2.5- by 6-inch molds were selected as a piston extractor could be used to remove the samples. Unconfined compressive strength tests were measured after seven and fourteen days. Results are seen in Figure 4.4. Pocket Penetrometer measurements were also conducted on each mix. These results can be seen in Table 4.7.

Figure 4.4 Bentonite-Cement Trial Batch Compressive Strength Results

Table 4.7: Pocket Penetrometer Trial Batch Results

Mix	Unconfined Compressive Strength (psf)			
	7 day		14 day	
A	3000	3000	3500	4000
B	4000	5000	5000	6500
C	6000	---	9000	---

Based on these results, we saw that the smaller sample molds (2.5- by 6-inch) typically were 1.2 times as strong as the larger four- by eight-inch samples. This may be the result of the small samples drying faster than the larger.

As seen in Table 4.8, the clay sample should have had a compressive strength up to 10 psi or 1440 psf. Thus, Mix A was selected. Mix A was mixed and poured seven days

before water jet testing to ensure adequate strength was reached. The general mixing procedure used during batch creation was followed; however, the bentonite was not sieved. Only four- by eight-inch samples were prepared.

CLY-02 had a significantly higher strength than CLY-01 when determined after testing. A few causes may exist. One may have been due to the order in which the two clay materials were made. CLY-01 was mixed initially in a 150 gallon tub. After filling the six-inch mold with the bentonite-cement mixture, some excess mixture remained in the tub. The material was reused as opposed to discarding it. The CLY-02 material was then mixed with the remnants from CLY-01. As the remnant CLY-01 mixture had more time to hydrate than the new CLY-02 material, the resulting mixture would have been stronger.

4.5 As-Tested Study Areas

Due to the irregularities in unconfined compressive strength, the testing schedule in Table 4.2 was not realized. Instead, Table 4.8 represents the actual study areas.

Seen in red, three strengths of concrete and two strengths of semi-cemented material will be compared to determine the effect of unconfined compressive strength on erosion rate. Four materials with compressive strengths between 1,850 and 2,350 psi were created to analyze any effect of material type on erosion rate as is seen in gray. Two sets materials in the blue rectangles were tested to examine the effect of aggregate size on erosion rate. Finally, the clay material was tested to determine the effect of the water jet on a weak soil.

Table 4.8: As-Tested Study Areas

Material Type	Target Unconfined Compressive Strengths (psi)				
	0 - 10	250 - 750	1850 - 2350	4500	5800
Concrete: Angular Aggregate		✓		✓	✓
Semi- Cemented		✓	✓		
Grout			✓		
Slurry-Mixed Concrete			✓		
Concrete: Rounded Aggregate			✓		
Clay	✓				

The actual compressive strength represents the strength of cylinders which were cured in Paso Robles or the strength of core samples that were taken from samples after jetting. SCM-03, CON-01, CON-02, and CON-03 were all cored and tested. Five sample cores were not tested on the same day that the water jetting occurred. To account for this discrepancy, a curve was fit to the cure room data, and the percent increase between cores and the trend was determined. The percent increase was then used to estimate the as-tested strength of the material. This was done by applying the percent increase to the measured strength from when water jetting occurred. The five samples are shown in Table 4.9.

Table 4.9: Modification of Sample Strengths

Material	Date Water Jetted	Date Cores or Cubes were Tested	Age of Cores/ Cubes when Tested (days)	Logarithmic Trend of Cure Room Samples	Unconfined Compressive Strength (psi)			
					Based on Trend from Cure Room	Cores or Cubes	Percent Increase	Estimated As-Tested
CON-01	August 20, 2009	September 2, 2009	46	$y = 660.32\ln(x) + 19.775$	2565	4587	79%	3910
CON-02	August 19, 2009	September 2, 2009	47	$y = 880.74\ln(x) + 232.92$	3629	4814	33%	4500
CON-03	August 19, 2009	September 2, 2009	47	$y = 1030.1\ln(x) + 1410.1$	5383	6560	22%	5790
SCM-02	September 18, 2009	September 26, 2009	36	$y = 126.31\ln(x) - 12.76$	440	485	10%	442
SCM-03	September 18, 2009	September 26, 2009	33	$y = 601.91\ln(x) - 53.901$	2056	2350	14%	2156

5 TESTING EQUIPMENT

We had several options for testing the effectiveness of water jetting including actual constructed drilled shafts, drums, or a small controlled specimen. We decided that using a small controlled specimen would be the most feasible and allow for the maximum amount of tests. Thus, testing molds and a testing device were designed and constructed. Two sample molds were created using the procedure described below.

5.1 Stationary Testing Mold Design and Construction

Testing molds were created with several factors in mind. The first consideration was that intermediate erosion measurements must be taken during testing. Also, the samples must be easily stripped for post-testing inspection and the molds must be able to be submerged. Safety considerations necessitated that the samples be thick enough such that the water jet would not puncture through the sample. Finally, the molds must be relatively easy to construct and test and parts of the molds should be recycled to limit costs.

In cooperation with the ADSC, the Case Pacific Company of Paso Robles, California volunteered use of their construction yard, water jetting equipment, and operators for this research.

5.1.1 Stationary “Donut” Sample Mold

The first sample testing setup was a “donut” mold, as shown in Figure 5.1. This device was chosen for the simplicity of construction, ability to reuse the base boards, and relative ease of post-test investigation.

Figure 5.1: Stationary Donut Sample Setup

The main components of the stationary donut sample molds are the plywood baseboards and the round Sonotube® concrete forms. Plywood baseboards were made with 36-inch circular plywood cutouts fastened to 4-foot square plywood sheets. Twelve-inch tall, 36-inch round concrete forms attached to the plywood baseboards by steel angle brackets to form the outer ring of the mold. Inner rings consisted of two-, four-, six-, twelve-, and sixteen-inch cardboard tubes of were centered on the plywood baseboards.

The varying diameters were used to determine the effectiveness of the jet with regards to distance from the nozzle tip. The inner tubes were removed before testing and the water jet was fixed in the center of the donut and allowed to erode the material. Full construction plans including layout dimensions are included in Appendix A (Figures A-1 – A-2).

As most anomalies occur below the groundwater table, the decision was made to submerge the samples during water jetting. To do this, the samples were tested in large water tubs and effectively submerged under about 3 to 12 inches of water. The water

tubs were constructed by first laying two layers of 10-foot square, of six-mil thick black polyethylene sheeting under the donut samples. A 24-inch tall wood frame was used to form a tub with the plastic sheeting so that the samples were submerged. Two 24-inch tall, wooden forms were created to sit around the samples. Two by fours were screwed into the tops of the tubs to hold the plastic sheeting in place, as shown in Figure 5.2.

Figure 5.2: Wooden Tub and Plastic Sheeting with Test Sample

5.1.2 Donut Cover Plate Assembly

In order to fix the jet within the testing mold, a cover plate was created. The cover plate was designed to be quickly lifted on and off of the donut sample to allow for efficient intermediate erosion measurements. The water jet nozzle connected to a steel centralizer and the cover plate assembly attached to centralizer. The cover plate consisted of a 39-

inch square steel plate with a 2-inch hole cut into the center, as shown in Figure 5.3. A 2-inch steel collar was welded over the hold. Six set screw in the collar were used to lock the jet in place during testing. A screw cap was modified to allow the centralizer to extend out of the collar and to ensure the centralizer remained aligned. Triangular slits, not shown on the drawing, were cut out of the bottom of the collar to allow for effluent to escape during testing. Small steel plates were welded to the bottom of the cover plate with two set screws. These were added to ensure once the cover plate was aligned that it would not move during testing as shown in Figure 5.4. Finally, reinforcing steel was bent and welded to the steel plate to serve as handles or lifting points.

Figure 5.3 Cover Plate Assembly

Figure 5.4: Stationary Cover Plate

Full construction plans including layout dimensions and welding details are included in Appendix A (Figures A-3 – A-5).

5.1.3 Erosion Measuring Devices

A measuring device was created to conduct intermediate measurements. This device was designed to measure the inside diameter of the donut. It consisted of two threaded rods, one which was welded to a connector, the other extended out to the removal surface, as shown Figure 5.5. A small nut was used to mark the location of the extended rod. The measuring device was collapsed and brought out of the hole and re-extended. A measurement was then taken and recorded.

Figure 5.5: Inside Diameter Measurement Device

This measuring device worked initially, though, as deeper grooves were eroded during testing, the device could not accommodate the angle of the erosion into material. The device was then only used to determine the initial inside diameter. A new measuring device was created and consisted of one threaded rod with a small nut to measure the depth of erosion. The small nut was moved along the threaded rod until it corresponded with the uneroded surface. The new measuring device was then removed and the depth of erosion measured and recorded. The measurements were taken in the two grooves created by the upper and lower jets across the three cross-sections shown in Figure 5.6 resulting in twelve measurements.

Figure 5.6: Erosion Measurement Area Description (prepared by G. Fiegel)

To measure the pressure at the nozzle, a large, 4 ½ inch pressure gauge attached above the centralizer by a tee-connector. The gauge was filled with glycerin to reduce vibration of the needle and could measure pressures up to 20,000 psi in 200 psi increments. The tee-connector made by Jetstream of Houston can handle up to 20,000 psi.

5.2 Large Sample Mold Design and Construction

The second sample testing mold or “Large Sample Mold” (LSM) was similar to the donut mold however the LSM was design to mimic actual anomalous removal techniques.

5.2.1 Large Sample Mold

A four-foot tall, 36-inch round concrete form was placed on top of a plywood baseboard, assembled the same as in the stationary donut mold, and attached with multiple four-inch steel angle brackets. A two-inch schedule 40 PVC pipe was installed in the middle the LSM to simulate a PVC inspection tube as shown in Figure 5.7.

Figure 5.7: Large Sample Mold Setup

The PVC tube was fixed to plywood forms to ensure it remained centered, as shown in Figure 5.8. Measuring devices were installed in the LSM to detect erosion. These devices were installed in three areas where water jetting was anticipated.

Figure 5.8: Top view of Large Sample Mold

5.2.2 Large Sample Mold Measuring Devices

As intermediate erosion measurements would not be possible in this sample, two water detection devices were constructed to indicate when erosion had reached a specific depth.

As ADSC West Coast Chapter (2007) suggests, the effluent from the tests were monitored and collected. Full construction plans including layout dimensions for all measuring devices are included in Appendix A (Figures A-6 – A-10).

5.2.2.1 Vertical Water Detection Tubes

Water detection tubes were installed in the LSM to detect when erosion had reached six- and twelve-inches from the PVC inspection tube. Three vertical water detection tubes

were placed at both distances. Water detection tubes were created by taking ½-inch copper pipes and cutting slits into the pipe. In each copper pipe slits were only cut into one area where erosion was anticipated, as shown in Figure 5.9.

Figure 5.9: Vertical Water Detection Tubes with Slits

Connected to the top of the vertical water detection tubes were ½-inch rubber hosing which then connected to ¼-inch rubber hosing. The ¼-inch hosing was mounted to a manifold board. The rubber hosing had gradations to more easily show variations in water height. Once the manifold was connected, water was poured into the rubber hosing. The water served as a baseline for detection and was set at a fixed level indicated on the manifold board, as shown in Figure 5.10.

Figure 5.10: Water Detection Manifold Board

When placed into the LSM the slits on the vertical water detection tubes were covered with tape to ensure that the tubes did not fill with anomalous material when poured. The vertical water detection tubes were tied into a plywood form to assure they would remain vertical, as shown in Figure 5.8.

5.2.2.2 Stacked Water Detection Tubes

Stacked water detection tubes were installed in the LSM to detect when erosion had reached two-, six-, and twelve-inches from the PVC inspection tube. Stacked water detection tubes were set within each estimated removal area. The water detection tubes were created by soldering ½-inch copper pipes on top of each other, as shown in Figure 5.11.

Figure 5.11: Stacked Water Detection Tube

The ends of the stacked water detection tubes were covered with tape to assure that tubes remained open after the anomalous material was placed in the mold. Holes were cut into the concrete forms to hold the stacked water detection tubes along with wire connected to the baseboards and the plywood form at the top of the LSM, as shown in Figure 5.8.

5.2.2.3 Effluent Collection

To collect effluent during testing of the LSM a collection system was created. A tee-joint was connected to the top of the PVC inspection tube. A two-foot PVC tube attached to the horizontal outlet of tee joint. At the end of the horizontal tube, a 45° joint connected a 3½-foot PVC tube. Large trashcans were set under the end of the PVC tube. The angled PVC pipe was able to be rotated to trashcans which were lined up to switch out when a trashcan became full, as shown in Figure 5.12. A strainer with openings corresponding to a #20 sieve was placed across the trashcan to monitor the effluent.

Figure 5.12: Large Sample Mold and Effluent Collection System

5.3 Cyclic Testing Mold Design and Construction

After testing the stationary donut samples, the molds and procedures were revised. The result was the cyclic sample mold and procedure which examined how the water jet behaves when moved. The same factors were considered when creating the cyclic testing

molds as for the stationary donut samples. Also taking into account was that the water jet must be cycled up and down. Any removed material must have an area to discharge.

The cover plate should be redesigned to remain fixed and only the jet removed for intermediate measurements. In order to do this, the lifting device should be light enough to lift by hand.

5.3.1 Sample Mold Used During Cyclic Testing

As noted above, the stationary mold was redesigned. As the water jet was now cycled up and down, the height of the existing donut mold was increased to 18-inches. During stationary testing, the corners of the baseboards had a tendency to rip the plastic sheeting. To alleviate this problem, the base boards were modified by cutting off the corners. Only six- and twelve -inch cardboard tubes formed the inner rings. The remainder of the molds remained the same. This sample mold is shown in Figure 5.13(a).

As discussed in the Materials chapter, some anomalous material would have to be made by hand. Thus, some donut molds were made with a 24-inch concrete form between the outer and inner rings. Only the inner annular space was filled with anomalous material. This resulted in less material mixed and placed by hand. This mold is shown in Figure 5.13(b).

Figure 5.13: Cyclic Testing Molds: (a) Modified Sample Mold; (b) Modified Sample Mold with Inner Ring

Full construction plans including layout dimensions are included in Appendix A (Figures A-11– A-13).

Again, two layers (ten-foot square) of six mil black polyethylene sheeting were laid underneath of the donut samples. The 24-inch tall, wooden tubs were used again.

5.3.2 Cyclic Cover Plate Assembly and Cyclic Device

After stationary donut testing it was found that lifting the plate on and off took longer than desired, therefore the plate was redesigned to remain fixed to the donuts as is seen in Figure 5.14. A 14-inch diameter hole was cut into the middle of the plate to allow for measurements to be taken easily. The two small set screws which locked into the concrete forms were replaced with one large set screw for faster removal. Also added, were large bolts to which the new cyclic testing device mounted. To allow for eroded

material to flush out, five washers (approximately 0.14-inches each) were placed on top of the bolts to provide nearly two-inches of clearance.

Figure 5.14: Modified Cover Plate for Cyclic Testing

The cyclic testing device can be seen in Figure 5.15. The cyclic testing device consisted of an aluminum base plate, the collar assembly (a steel collar with set screws and two steel plates welded to the collar), an aluminum top plate, and four aluminum posts.

Figure 5.15: Cyclic Testing Device

The aluminum base plate bolted to the steel cover plate on the large bolts welded to the cover plate. The top aluminum plate had a U-shaped cut to allow the pressure gauge to fit. The four posts used lock washers and nuts to secure them to the aluminum plates. The steel collar was able to move up and down six-inches on the four posts between the two aluminum plates. Axle grease lubricated the posts to allow for smooth movement of the collar. Handles were bolted to the aluminum plate as well.

In order to lift the collar assembly, ropes were attached to the collar. The ropes were combined at a carabineer hook to a long rope and run through a pulley. The pulley was attached to a tripod held down by sand bags, as shown in Figure 5.16.

Figure 5.16: Full Cyclic Testing Setup including Tripod

In order for the nozzle to extend deep enough into the new sample, the steel collar was shortened and a seven-inch nipple was made by Jetstream of Houston. The nipple was designed to fit in line between the nozzle and the centralizer. Full construction plans

including layout dimensions and welding details are included in Appendix A (Figures A-14 – A-19).

5.3.3 Cyclic Testing Measuring Devices

As the water jet would now be moving over a six-inch area, grooves were no longer cut into the samples as had occurred in the stationary donut samples. Because of the different nozzle tip sizes, three distinct areas were removed. Thus, a thin piece of steel had three holes drilled into it corresponding to the middle of each layer to allow for consistent measurements in location. The measuring tool was place flush against the sample and rested on the bottom plywood baseboard, as shown in Figure 5.17. Each areas was approximately two-inches thick and will be discussed more in the Field Testing Results chapter herein.

Figure 5.17: 12-inch Donut Showing Erosion and Measuring Device

For each one direction shown in Figure 5.6, three vertical measurements were taken. A measurement was taken in each level of erosion. Thus, with six measurement spots and three measurement heights, eighteen total measurements were taken per testing interval. A depiction of the erosion measurements is shown in Figure 5.18.

Figure 5.18: Depiction of Erosions Measurements in Cyclic Experiments (by G.Fiegel)

The same measuring tool used to determine the erosion depth in the stationary donut was used for cyclic samples.

6 TESTING PROTOCOL

In cooperation with the ADSC, the Case Pacific Company in Paso Robles, California volunteered use of their construction yard, water jetting equipment, and operators for this research.

6.1 Stationary Donut Experimental Protocol

First, the inner cardboard ring was removed. The initial inside diameter was measured 8¼-inches below the top of the sample. The water jet was fixed into the cover plate assembly by setting the water jet on the bottom of the sample, then raising it 3¾-inches. The water jet was secured to the assembly plate by tightening the set screws in the collar. The nozzle was verified that it was perpendicular to the cover plate.

The cover plate was centered over the donut sample such that there was an equal overlap of the steel plate on opposite sides of the sample. It was then secured by tightening the set screws into the concrete forms. Next the sample was submersed by filling the tub with water. Two submersible pumps were placed into the tub. During the tests, excess water was pumped into the adjacent tub or into two auxiliary holding tubs. Pumping excess water into the second tub ensured that the next sample was submerged and ready to test when the first test was finished.

The diesel water pump was pressurized to approximately 10,000 psi and the water jet ran for 30 seconds. During each run, the pressure at the gauge above the nozzle was noted. Once testing was completed, enough water was pumped out of the tubs so that measurements could be made. The cover plate was removed and erosion depth in each groove was measured and recorded using the modified measuring device.

This process was repeated while increasing the time step. The time step was doubled each time and testing continued until the water jet no longer removed significant material. Once the erosion was no longer evident, testing ceased and the cover plate assembly and water jet were moved to the next sample. Also, the tub was removed and moved to the next sample to be tested. This protocol was repeated for each stationary donut sample.

6.1.1 Large Sample Mold Experimental Protocol

Three six inch areas of PVC were to be removed by water jet. The jet nozzle was lowered to the bottom of the sample and lifted 3¾-inch and the hose was marked with tape. The nozzle was lifted six-inches and the hose was marked with tape. This allowed to operator to know over which area to move the nozzle. First, the lowest six-inch section was to be removed and the effluent from the PVC inspection tube collected in a separate trash can.

The diesel water pump was pressurized to 10,000 psi. Once the operator concluded that the PVC inspection tube had been removed, the water jet was to be lowered to the bottom of the removed area. The water jet was to be lifted at ¼-inch per minute within the six-inch area or until water was seen exiting the 12 inch detection tubes. The effluent from

the “anomaly zones” was to be collected separately and labeled appropriately. The water detection tubes were monitored and the time noted when the change in the water level was seen. This procedure was to be repeated for the other two anomalous zones.

6.1.2 Cyclic Experimental Protocol

First, the inner cardboard ring was removed. The initial inside diameter was measured over the 3 different cross-sections and 3 heights of the sample and recorded. The cover plate was centered over the annulus sample such that there was an equal space between the steel plate and the inner ring of the sample. It was then secured by tightening the set screws into the concrete forms.

The cyclic testing assembly was placed onto the steel cover plate and bolted down. The water jet was set onto the bottom of the sample, and then raised 3 ¾-inches. The water jet was secured to the cyclic testing assembly by tightening the set screws in the collar. The nozzle was verified that it was perpendicular to the steel cover plate. The four post were greased to allow for smooth movement of the testing assembly

Next the sample was submerged by filling the tub with water. Two submersible pumps were placed into the tub. During the tests, excess water was pumped into the adjacent tub or into two auxiliary holding tubs. Pumping excess water into the second tub ensured that the next sample was submerged and ready to test when the first test was finished.

The diesel water pump pressurized to approximately 10,000 psi and the water jet ran for 30 seconds. When the testing began, a team member raised and lowered the cyclic testing assembly at a relatively constant rate of approximately 12 cycles per minute to assure that even jetting occurred along the length of the sample. The pressure at the gauge above the nozzle was noted during each test.

Once testing was completed, enough water was pumped out of the tubs so that measurements could be made. The cyclic testing assembly was removed by unscrewing the large bolts and lifting off the entire assembly. The erosion depth at each location was measured and recorded using the cyclic measuring devices. Once measurements were complete, water was pumped back into the tub so that the sample was submerged and the cyclic testing assembly was replaced.

This process was repeated with increasing time, usually 30, 60, 120, 240, 480, 960 seconds, doubling each time, until the water jet no longer removed significant material. Once erosion was no longer evident, testing ceased. The testing assembly and water jet were moved to the next sample. Also, the tub was removed and moved to the next sample to be tested. This protocol was repeated for each donut sample.

7 FIELD TESTING RESULTS

In total, twenty-three donut samples were tested. Only one LSM was tested. Of the twenty-three donut samples, five were stationary and eighteen were cyclic tests. The stationary tests were conducted to view the general erosive effect of the jet and to test the two different sample molds (stationary and LSM). A summary of all tested material, the as-tested compressive strength, and total jetting time is included in Table 7.1. All raw data can be found in Appendix B.

Table 7.1: As-Tested Material Summary

Test Series	Material Description	Maximum Aggregate Size (inches)	Actual Compressive Strength (psi)	Inside Diameter (inches)	Jetting Procedure	Total Jetting Time (minutes)	Average Jetting Pressure (psi)	Cycle Rate (cycles/minute)	Date Tested	Curing Time (days)
SCM-01	Semi-cemented	0.375, angular	560	2, 4, 6, 12, 16	Stationary	11, 11, 7, 4, 9	9,300	NA	June 25 - 26, 2009	55 - 56
SCM-02	Semi-cemented	0.375, angular	440	6, 12	Cyclic	18, 20	10,000	11 - 12	September 18, 2009	28
SCM-03	Semi-cemented	0.375, angular	2,160	6, 12	Cyclic	24, 24	9,700	11 - 12	September 18, 2009	28
CON-01	Concrete	1, angular	3,910	6, 12	Cyclic	24, 18	9,800	9 - 10	August 20, 2009	34
CON-02	Concrete	1, angular	4,500	6, 12	Cyclic	24, 18	9,700	9 - 10	August 19, 2009	33
CON-03	Concrete	1, angular	5,790	6, 12	Cyclic	32, 16	9,300	9 - 10	August 19, 2009	33
CON-04	Concrete	1, angular	655	6	Cyclic	32	10,500	11 - 13	November 19, 2009	2
CON-05	Concrete	1, rounded	2,120	6	Cyclic	32	10,300	11 - 13	November 19, 2009	2
SMX-01	Slurry-mixed concrete	1, angular	1,850	6, 12	Cyclic	32, 20	10,600	11 - 12	November 19 - 20, 2009	63 - 64
GRT-01	Sand-cement grout	0.125	1,900	6, 12	Cyclic	32, 22	10,600	12 - 13	November 19 - 20, 2009	43 - 44
CLY-01	Bentonite-cement mix	<0.002 mm	5	6	Cyclic	2	10,400	11 - 12	November 19, 2009	6
CLY-02	Bentonite-cement mix	<0.002 mm	9	12	Cyclic	8	10,600	12 - 14	November 20, 2009	7

7.1 Stationary Testing Results

SCM-01

The semi-cemented stationary samples were tested first. The samples were tested from largest to smallest inside diameter. Both the sixteen-inch and twelve-inch samples displayed unusual erosion rates. Initially, little erosion was observed followed by large removals. The six, four, and two-inch samples all displayed similar erosion rates where a large initial erosion was observed followed by decreasing incremental erosion. The experimental test results for SCM-01 are shown in Figure 7.1.

Figure 7.1: Stationary Donut Testing Results

Eroded material was found on top of the sample and at the bottom of the inner diameter hole. Material generally consisted of fine aggregates (sand and 3/8" rock). None of the aggregates appeared to be eroded or cut by the water jet itself.

The erosion pattern of the 16, 12, and 4-inch were all similar. As the jet was stationary, the jet cut two distinct slits into the samples. The slits were approximately one-inch wide at the mouth. In the two-inch donut sample, the jet nozzles were close enough to the face of the sample that they were eroding the same material initially. Figure 7.2 shows the different erosion patterns for the two-inch and twelve-inch samples.

Figure 7.2: Comparison of 2-inch and 12-inch Sample Erosion Patterns

Some coarse aggregate was observed to be lodged in the path of the jet. Observed behind the coarse aggregate was uneroded binder material, as is seen in Figure 7.3.

Figure 7.3: Uneroded Binder Material in 16-inch SCM Stationary Sample

LSM

It is important to note why the LSM is not included in Table 7.1. During testing of the LSM, the water jet cut through the PVC pipe and into the semi-cemented material within the first 90 seconds. After the cutting through the PVC pipe, the jet quickly eroded the sample material, as was seen in the two-inch donut mold. The jet became clogged by the fine aggregate nearly immediately. The jet was removed, cleaned, and testing resumed. During the second attempt, water was seen exiting from the bottom of the LSM and the entire mold lifted off the base. Jetting was terminated for the lower area. Jetting was continued for the middle and upper removal areas; however, the jet quickly became clogged again. It was observed with the down-hole camera that the water jet was not cutting the PVC out entirely. The testing on the LSM was concluded.

Figure 7.4: Removal Area of LSM

During the post-testing investigation of the sample, it was determined that if the PVC pipe would have had a cap on the end contacting the base boards, the sample may not have ruptured. Due to the large size of the LSM, post-testing investigation, including cutting the sample open for documentation, was very difficult. Given the inability to take intermediate erosion measurements and the difficulty of post-testing investigation, the LSM was determined to be an infeasible testing medium for further research.

The failure of the LSM shows the importance of removing the PVC inspection tube. The contractor operating the water jet equipment cannot verify the removal of the PVC. Instead, the contractor relies on his observation of the effluent rising out of the tube and as such, the PVC is not always removed fully, let alone efficiently. As mentioned before, ADSC West Coast Chapter (2007) only recommends a range of water pressures and flow

rates for PVC removal. Due to the difficulty in removing the PVC inspection tube and the lack of research, this remains an area of concern for both the contractors and owners.

7.2 Cyclic Testing Results

Erosion patterns of these samples were markedly different from that of the SCM-01 due to the cyclic testing procedure. The entire area over which the water jet moved was eroded. Figure 7.5 shows a typical erosion pattern of the jet when cycled up and down within the sample. Typically, it was observed that the middle section of the sample exhibited the most erosion, with the bottom two-inches exhibiting the least. For nearly all materials tested using the cyclic procedures, the erosion within the first five minutes of total jetting time accounted for almost half, if not more, of the total erosion. Also, it appeared that for all materials, which had both six- and twelve-inch inside diameters, more erosion occurred in the six-inch than in the twelve-inch.

Figure 7.5: 12-inch Semi-Cemented Material Erosion Pattern

SCM-02, SCM-03, and GRT-01

For these materials, the erosion patterns were very similar to that shown in Figure 7.5 even though the compressive strengths varied. The SCM-02 material had similar strength to the SCM-01 where as the SCM-03 was near 2,350 psi. The six-inch SCM-02 sample averaged almost 3.0 inches of total erosion and the twelve-inch averaged nearly 1.6 inches. The six-inch SCM-03 sample averaged almost 1.6 inches of total erosion and the twelve-inch averaged nearly 0.5 inches as shown in Figure 7.6.

Figure 7.6: 6-inch and 12-inch SCM-02 and SCM-03

GRT-01 behaved similarly to the described erosion patterns seen in other samples. The jet was held stationary during the final runs of both six-inch and twelve-inch GRT-01 samples. Total erosion averaged nearly 1.5 inches for the six-inch sample and nearly 0.5 inches for the twelve-inch sample. The experimental results are shown in Figure 7.7.

Figure 7.7: 6-inch and 12-inch GRT-01 Experimental Results

CON-01, CON-02, CON-03, CON-04, and CON-05

Overall, the material erosion patterns were not significantly different than those of other materials tested. The twelve-inch CON-02 did not erode as expected; very little erosion was observed in this sample, only 0.2 inches. The twelve-inch CON-01 averaged 0.3 inches total erosion and the twelve-inch CON-03 averaged over 0.5 inches. The six-inch samples for CON-01, CON-02, and CON-03 averaged around 1.3, 1.6, and 2.1 inches of total erosion respectively.

Figure 7.8: 6-inch Concrete Materials Testing Results

Figure 7.9: 12-inch Concrete Materials Testing Results

It was observed during post-testing investigation that the aggregate which was still fixed in the concrete matrix appeared polished where it was exposed to the jet.

Although the CON-04 was originally designed to have a compressive strength around 2,000 psi, the strength of the material was substantially less, near 655 psi. This may be due to the water to cementitious material ratio of being too high. Total cumulative erosion averaged 2.8 inches.

The erosion pattern of CON-05 was not significantly different from the general pattern observed during testing. After post-test investigation, the rounded aggregates that were protruding from the eroded surface were more easily knocked out than the angular aggregates of other samples. Total cumulative erosion averaged around 1.15 inches.

SMX-01

The six-inch SMX-01 followed the same erosion pattern which was seen throughout nearly all samples. Total erosion averaged 1.2 inches as is shown in Figure 7.10.

Figure 7.10: SMX-01 Experimental Test Results

What may be seen in Figure 7.11(a) and (b) is the difference in erosion surfaces between a materials with and without slurry. The aggregate in Figure 7.11(a) appears to be more exposed than those in Figure 7.11(b). Small slits and gaps appear around the aggregate which may be the result of a weakened binder material. The weakened binder material is due to the addition of bentonite slurry.

Figure 7.11: Comparison of Erosion Surfaces: (a) SMX-01; (b) CON-04

The jet was not cycled in the twelve-inch SMX-01 sample, instead it was fixed. After viewing the initial results from the six-inch SMX-01 sample, which only averaged 1.2 inches of total erosion, the jet was fixed to determine if more material could be removed. After twenty minutes of continuous jetting, only 0.35 inches of total erosion was evident.

CLY-01 and CLY-02

During testing, the effluent from CLY-01 was milky and thick. A large amount of erosion occurred in the first thirty seconds. The jet was then run for sixty seconds, during which much less erosion was seen and the sample was destroyed. The experimental test results are shown in Figure 7.12. A long slit was observed inside the sample. The sample itself was split into two pieces with the top piece slightly displacing. The testing was halted after the second run. Chunks of the bentonite-cement mixture were found in the bottom of the sample during post-testing investigation. Precise measurements were difficult. However, an estimated 5.75 inches of material was removed.

Figure 7.12: CLY-01 and CLY-02 Experimental Testing Results

CLY-02 behaved similarly to the described erosion patterns seen in other samples. Total cumulative erosion averaged nearly 2.5 inches. During testing, the effluent from CLY-02 was milky and thick. Chunks of the bentonite-cement mixture were found in the bottom of the sample during post-testing investigation. This was also observed by Rockwell (1981) when water jetting clay soils.

8 DISCUSSION

Materials were created and tested to determine the effect of unconfined compressive strength, aggregate size and angularity, material type, distance, and exposure time on water jetting efficacy. These parameters will be analyzed with respect to total erosion and erosion rate. Overall, twenty three tests were conducted on nine material types with twelve different strengths.

As was noted in Chapter 3, it is common for a contractor to raise the nozzle around ¼-inch every minute. During cyclic testing, the water jet was repeatedly raised and lowered over six-inches. Thus, had the industry standard been used to water jet the same six-inch area, testing would have lasted twenty-four minutes. Although water jetting for twenty-four minutes was feasible, it would not have allowed for intermediate measurements to be taken. Hence, twenty-four minutes of cyclic testing approximates the industry standard of practice.

It is important to note that the data was collected after water jetting with a Stoneage® Gopher at a pressure of approximately 10,000 psi. As noted earlier, contractors employ different testing methods using different jet nozzles. The observed data is only valid for tested nozzle and the stated pressure.

Much of the analyzed data has been fit with trend lines. Thus, trend lines were used to describe and assess the observed data.

8.1 Total Erosion Amount

Time Effects

As testing proceeded it appeared that total erosion amount was approaching a limit or maximum value. Figure 8.1 shows the testing results from the six-inch donut of CON-04. These results imply that as the amount of erosion occurring decreases as time increases. This trend was seen in all samples tested with the cyclic testing procedure.

Figure 8.1: 6-inch CON-04 Testing Results

Atmatzidis and Ferrin (1983) found depth of erosion and time were exponentially related. Figure 8.1 and all but two samples, which will be discussed next, followed logarithmic trends.

As more erosion occurred, the distance from the nozzle increased thus providing another variable. However, the testing procedure mimics the industry standard of practice and similar results have been observed in the field.

Figure 8.2 shows the results from stationary water jet testing. The six-, four-, and two-inch donuts followed the same logarithmic as is seen in the cyclic testing data. However, the sixteen-inch and twelve-inch donuts did not follow this trend. This variance may be due to the variation in uniformity of the materials placed in each mold. As these two donuts were the first samples tested, experimental error may have occurred. It is recommended that these tests be repeated to verify the results. Figure 8.2 also shows that the results from the six-inch donut samples follow the same trend as the two-inch donut.

Figure 8.2: Stationary Testing Results

Initial Distance from Nozzle

As shown in Figure 8.3, more erosion was evident in the six-inch donut than in the twelve-inch. This trend was seen in all materials which had both a six-inch and a twelve-inch donut. Wright et al. (1997) also found that increasing the standoff distance decreased the efficiency of the jet. This is due to degradation of the jet stream through air. It can be assumed that degradation of the jet stream through water would be even more rapid.

Figure 8.3: SCM-02 Comparison of 6-inch and 12-inch Donuts

The amount decrease in erosion from the six-inch annulus to the twelve-inch annulus varied from about 23% to 95% as is seen in

Table 8.1.

Table 8.1 is listed in decreasing compressive strength. Nearly all materials saw at least one-inch less erosion in the twelve-inch than the six-inch.

Table 8.1: Percent Decrease in Erosion from 6-inch to 12-inch Donut

Material	UCS (psi)	Amount Decrease (inches)	Percent Decrease
CON-03	5790	1.0000	78%
CON-02	4500	1.3889	86%
CON-01	3910	1.5833	75%
SCM-03	2160	1.1111	70%
GRT-01	1900	0.9807	67%
SMX-01	1850	1.1424	95%
SCM-02	440	1.3958	47%
CLY-01/02	5	1.3410	23%

Material Type

The four materials shown in Figure 8.4 have unconfined compressive strengths varying from 1,850 to 2,160 psi. The aggregate size varies from sand to one-inch rounded and angular. SMX-01 was mixed with 15% bentonite slurry by volume. Even though the material type varies, the relative erosion levels appear to follow the same trend.

Figure 8.4: Erosion of Different Materials with Similar Compressive Strength

Figure 8.5 shows the trend lines of Figure 8.4. Also seen in Figure 8.5 is an initial period of erosion with a steep slope. The trends appear to flatten after five minutes of water jetting. This will be discussed more in Section 8.2.

Figure 8.5: Trend Lines of 6-inch Donuts with Similar Compressive Strength

Aggregate Size and Angularity

Figure 8.5 shows three materials, CON-05, SCM-03, and GRT-01, which contain primarily coarse aggregates, fine aggregates, and sand, respectively. These materials have comparable compressive strengths. As seen in Figure 8.5, there does not appear to be a correlation between aggregate size and average erosion distance.

CON-04 and SCM-02 were made with one-inch and 3/8-inch aggregates, respectively. There appears to be little total difference between total average erosion distances for the two materials, as is shown in Figure 8.6 . The erosion predicted by the trendlines varies by less than 15 percent after 35 minutes.

Figure 8.6: Comparison of 6-inch Donut CON-04 and SCM-02

As both Figure 8.5 and Figure 8.6 suggest, there appears to be no relationship between aggregate size and average erosion distance.

As discussed earlier, two materials with similar strengths but different aggregate angularity was not achieved. Although one could think that CON-05 and SMX-01 had similar strengths with different aggregate angularity, the addition of bentonite slurry to SMX-01 renders these materials too dissimilar to compare. Thus, an observed correlation was not possible.

Unconfined Compressive Strength

The average inside diameter of the cavity created by water jetting was approximated for all materials. These diameters were then compared with unconfined compressive strength. Seen in Figure 8.7 and Figure 8.8 is the average inside diameter when plotted versus unconfined compressive strength on a semi-logarithmic scale for six-inch and twelve-inch donuts respectively.

It is interesting to note that the materials with compressive strengths between 1,850 and 2,160 psi group together in both figures. These materials had similar unconfined compressive strengths and varied material properties.

Figure 8.7: 6-inch Donut Average Diameter Varying with Compressive Strength

Figure 8.8: 12-inch Donut Average Diameter Varying with Compressive Strength

When comparing the trends, it appears that they may be parallel. Figure 8.9 shows that this is the case.

Figure 8.9: Comparison of Logarithmic Trend Lines

Four materials were tested which had similar aggregate size and angularity but varied compressive strengths. Figure 8.10 suggests that as unconfined compressive strength decreases, erosion increases. This agrees with results reported by Atmatzidis and Ferrin (1983). The logarithmic fit for these correlations appear appropriate and may approximated future values well.

Figure 8.10: Comparison of Four 6-inch Concrete Samples

There appears to be no correlation between percent decrease and unconfined compressive strength. These values can be seen in Table 8.1.

8.2 Erosion Rate

The erosion rate was determined by first fitting a logarithmic trend line to the observed data. As was noted earlier, the data appeared to have a steep initial erosion rate, usually around five minutes. An erosion value was estimated based on the trend line for both five minutes and twenty-four minutes. Twenty-four minutes was selected based on the cyclic testing procedure. The erosion values selected from the trendline were then used to estimate erosion rates.

A line was drawn from zero erosion to the respective erosion value. The slope of the resulting line, or erosion rate (in inches per minute), was approximated. As shown in Figure 8.11, the red arrow corresponds to the 5 minute erosion rate and the blue arrow correlates to the 24 minute erosion rate. Consequently, the erosion rate based on the five minute erosion value is seen as “Initial” in Table 8.2 and Table 8.3. The twenty-four minute erosion rate can also be seen in the tables.

Figure 8.11: Erosion Rate Diagram

Table 8.2: Erosion Rates of 6-inch Donut Samples

Material	Erosion Rate (inches/ minute)	
	Initial	24 minute
CON-03	0.1224	0.0429
CON-02	0.1897	0.0602
CON-01	0.2026	0.0802
CON-05	0.1297	0.0437
CON-04	0.4248	0.1151
SMX-01	0.1575	0.0469
SCM-03	0.1875	0.0604
SCM-02	0.5173	0.1335
GRT-01	0.1951	0.0564
CLY-01	1.2980	0.3102

Table 8.3: Erosion Rate of 12-inch Donut Samples

Material	Erosion Rate (inches/ minute)	
	Initial	24 minute
CON-03	0.0356	0.0135
CON-02	0.0389	0.0095
CON-01	0.0667	0.0226
SMX-01	---	---
SCM-03	0.0589	0.0180
SCM-02	0.2373	0.0637
GRT-01	0.0880	0.0225
CLY-02	0.8312	0.2096

Time Effects

As is suggested in Table 8.2 and

Table 8.3 the initial erosion rate is much larger than the long term erosion rate. This follows the logarithmic trend. It was also observed by Atmatzidis and Ferrin (1983) that the maximum erosion was achieved in 15 to 20 seconds. Their research was conducted

on material similar to CLY-01 and CLY-02. The erosion rate in these materials was much faster than in the rest of the tested samples.

Initial Distance from Nozzle

The initial erosion rates for the six-inch samples were much larger, often four times larger, than the twenty-four minute erosion rates. Note that Figure 8.12 and Figure 8.13 appear very similar but the vertical scale on Figure 8.13 is one-fourth that of Figure 8.12. For the twelve-inch donut samples, the initial erosion rate was more near three times as large as the twenty-four minute erosion rate; however, the variance was larger as well.

Figure 8.12: 6-inch Donut Initial Erosion Rates

Figure 8.13: 6-inch Donut 24 Minute Erosion Rates

As noted, increasing the distance of the jet nozzle from the erosion surface decreases the erosion rate. This is due to degradation of the jet stream through air. It can be assumed that degradation of the jet stream through water would be even more rapid.

Material Type

CON-05, SMX-01, SCM-03, and GRT-01 had varying material properties with comparable compressive strength values. These materials all had erosion rates between 0.1 and 0.2 inches per minute. It is interesting to see that the concrete materials had similar initial erosion rate values, although they had larger compressive strengths. The twenty-four minute erosion rate values for the concrete samples are, at times, higher than

lower strength material. This may be attributed to the threshold pressure of these materials.

As discussed by Wright et al. (2001) and Wright et al. (1997) the threshold pressure is the water pressure at which a jet begins erode a material. Wright et al. (1997) noted that a sand-cement mixture with a strength of 3,000 psi had a threshold pressure of 1,500 psi. Testing in this study occurred around a water pressure of 10,000 psi. Wright et al. (2001) state that once the threshold pressure is exceeded, it is more effective to use higher flow rates than to increase pressure. Since cutting occurred, the threshold pressure of the materials were exceeded. During the experiments, the flow rate was held constant throughout all tests.

Aggregate Size and Angularity

CON-05, SCM-03, and GRT-01 had varying aggregate size with comparable compressive strength values. As Figure 8.12 and Figure 8.13 show, these materials group together between an erosion rate of 0.1 and 0.2 inches per minute. This suggests that aggregate size may not influence removal amount. This follows Wright et al. (1997) and also suggests that water jets do not erode the aggregate, instead it erodes the cement paste. Although the compressive strength of the material is a function of the aggregate size and the binder cement paste, it appears that aggregate size does not influence erosion rate to as large of an extent as the strength of the cement paste.

Unconfined Compressive Strength

Initial and twenty-four minute erosions rates are compared to unconfined compressive strength in Figure 8.12 and Figure 8.13. A logarithmic regression appears to fit the trend very well. Again, the materials with compressive strengths between 1,850 and 2,160 psi clump together.

Table 8.4 shows that materials with an unconfined compressive strength greater than 1,850 psi had initial erosion rates from 0.12 to 0.20 inches per minute. These values only represent approximates and should be considered as such.

Table 8.4: Observed Erosion Rates for 6-inch Donut Samples

UCS(psi)	Erosion Rate (in/ min)	
	Initial	24 minute
5	1.3	0.31
440 to 655	0.4 to .05	0.11 to 0.13
1,850 to 5,790	0.12 to 0.20	0.04 to 0.08

9 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

9.1 Summary

The primary objective of this research was to establish an empirical relationship between jetting parameters and the removal of commonly occurring anomalous zone materials, including low-strength concrete, slurry mixed concrete, grout, and clay soil. Also established was the current standard-of-practice of water jetting contractors within California. During testing, material removal rates were measured as a function of jet pressure and standoff distance. Based on the results, erosion rates and the effectiveness of water jetting is primarily influenced by compressive strength, when using common test equipment and jetting pressures. Further, the size of aggregate and material type in the anomalous material does not appear to influence erosion.

9.2 Conclusions

Erosion rates and total erosion amounts were examined for all materials. The conclusions discussed below are general and apply to nearly all samples tested.

After a certain point, usually five minutes, the erosion rate decreases with increasing time of exposure. It was observed that the erosion pattern followed a logarithmic trend. Most of the total erosion occurs in first five minutes.

Distance from the jet nozzle to the material erosion surface plays role. Generally, the further away the water jet nozzle is from the material, the less erosion occurs. This is due, in part, to degradation of the jet stream through the water.

Material type and aggregate size did not seem to influence erosion amount or erosion rate observed during water jetting. Instead, unconfined compressive strength appears to have the greatest influence on total erosion and erosion rate. Although the compressive strength of the material is a function of the aggregate size and cement paste, it appears that aggregate size does not influence erosion rate to as large of an extent as the strength of the cement paste.

This research was funded with an eye towards verifying and improving upon current industry standards. Overall, this research verifies water jetting as mitigating method. There are limits to the effectiveness of this jet however. The distance between PVC inspection tubes is typically thirty-three inches. In order to removed anomalous material between the two inspection pipes the water jet would have to erode up to seventeen-inches of material. In general, the data from this research does not suggest that the water jet can erode seventeen-inches. However, contractors within the industry have stated that they can erode to such distances.

The data does suggest that the water jet can erode three-inches, the same depth as concrete cover over the reinforcing steel cage for most strengths. Water jetting used to mitigate anomalous material found in the cover portion should be feasible.

9.3 Recommendations for Future Testing

Due to the material difficulties encountered during this research, many of the samples had unconfined compressive strengths between 1,800 and 6000 psi. As discussed previously, anomalies have been observed with lower strengths. Thus, I recommend that more materials with strengths up to approximately 800 psi be examined.

As is seen in the Field Testing Results chapter of this thesis, unusual results for the 12 and 16 inch stationary donut samples were observed. I believe these samples should be recreated and water jetted again to verify the data obtained during this thesis work.

Only nine materials were tested. Thus, more testing should be conducted on materials including Caltrans' wet pile mix. Instead of varying compressive strength with water to cementitious materials ratio, vary compressive strength with the age of material. This will allow the testing group to better estimate the strengths on testing days. Careful planning with the contractor will be necessary for age-based strengths to be used.

Since the PVC inspection tubes are mounted to the reinforcing steel cages, it is important to determine if the reinforcing steel cage impinges upon the water jet's ability to erode anomalous material. Currently, investigating this possibility is schedule for the testing conducted under Phase II of the project.

Although the Large Sample Mold did not function properly, it could still be a viable testing method. If this mold is pursued for future research, it is imperative that a cap be

connected to the bottom of the PVC tube. This will force water out the top of the sample and should stop the sample from splitting apart.

This research was conducted with a specific jet model. However, two other water jet nozzles are used by industry. In order to determine the overall effectiveness of water jetting as a technique for mitigating anomalies, these other jets should be evaluated.

Identify the threshold pressure of different materials, including concrete. As is mentioned in the literature review, the threshold pressure is an important value to know in order to efficiently erode material. Once the threshold pressure is established, it is more efficient to increase flow rates than increase pressure. After threshold pressures are established investigation of different flow rates should be conducted.

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APPENDIX

A

Testing Equipment Construction Details

Figure A-1: Stationary Donut Mold

Figure A-2: Plan View of Stationary Donut Mold

Figure A-3: Stationary Cover Plate and Collar Assembly

Figure A-4: Plan View of Stationary Cover Plate and Collar Assembly

Figure A-5: Complete Stationary Testing Equipment

Figure A-6: Large Sample Mold

Figure A-8: Details of Stacked Water Detection Tubes

Figure A-9: Detail of Stacked Water Detection Tubes

Figure A-10: Vertical Water Detection Tubes

Figure A-11: Cyclic Donut Mold

Figure A-12: Plan View of Cyclic Donut Mold

Figure A-13: Plan View of Modified Cyclic Donut Mold

Figure A-14: Cyclic Cover Plate with Collar Assembly

Figure A-15: Detail of Cyclic Cover Plate

Figure A-16: Detail of Cyclic Cover Plate and Collar Assembly

Figure A-17: Plan View of Cyclic Cover Plate

Figure A-18: Complete Cyclic Testing Equipment in Down Position

Figure A-19: Complete Cyclic Testing Equipment in Up Position

APPENDIX

B

Complete Raw Data

16 inch Annulus Sample Inner Diameter Measurements						
Time Step (seconds)	Total Time (seconds)	Description	Inside Diameter (inches)			
			Direction 1	Direction 2	Direction 3	Average
0	0	6.25	16.2500	16.2500	16.2500	16.2500
		8.25	16.2500	16.2500	16.2500	16.2500
		10.25	16.3750	16.2500	16.5000	16.3750
		Average	16.2917	16.2500	16.3333	16.2917
30	30	Top	16.8750	16.3750	16.5625	16.6042
		Bottom	16.3125	16.3125	16.3125	16.3125
30	60	Top	16.4375	16.5000	16.8750	16.6042
		Bottom	16.3125	16.3125	16.3750	16.3333
60	120	Top	16.6875	16.8125	17.1250	16.8750
		Bottom	16.4375	16.2500	16.6250	16.4375
90	210	Top	16.6250	16.6875	17.1250	16.8125
		Bottom	16.3750	16.3750	16.6250	16.4583
120	330	Top	16.8125	16.8125	17.2500	16.9583
		Bottom	16.3750	16.3125	16.3750	16.3542
180	510	Top - Side 1	4.6250	5.3750	4.1875	4.7292
		Top - Side 2	4.7500	5.1250	6.1875	5.3542
		Top Total	25.6667	26.7500	26.7083	26.3333
		Bottom - Side 1	4.5000	3.2500	3.5000	3.7500
		Bottom - Side 2	3.3750	4.3750	4.1250	3.9583
		Bottom Total	24.1667	23.8750	23.9583	24.0000

Table B-1: 16-inch SCS-01 Raw Data

12 inch Annulus Sample Inner Diameter Measurements						
Time Step (seconds)	Total Time (seconds)	Description	Inside Diameter (inches)			
			Direction 1	Direction 2	Direction 3	Average
0	0	6.25	12.2500	12.2500	12.0000	12.1667
		10.25	12.2500	12.1875	12.1800	12.2058
		Average	12.2500	12.2188	12.0900	12.1863
30	30	Top - Side 1	0.4375	0.2500	0.4375	0.3750
		Top - Side 2	0.3750	0.3750	0.2500	0.3333
		Top Total	13.0625	12.8438	12.7775	12.8946
		Bottom - Side 1	0.2500	0.3125	0.3750	0.3125
		Bottom - Side 2	0.1875	0.1875		0.1875
		Bottom Total	12.6875	12.7188		12.7031
30	60	Top - Side 1	0.6250	0.3125	0.8750	0.6042
		Top - Side 2	0.7500	0.5000	0.3750	0.5417
		Top Total	13.6250	13.0313	13.3400	13.3321
		Bottom - Side 1	0.4375	0.3750	0.5000	0.4375
		Bottom - Side 2	0.3750	0.4375	0.3750	0.3958
		Bottom Total	13.0625	13.0313	12.9650	13.0196
60	120	Top - Side 1	3.3750	3.2500	3.1875	3.2708
		Top - Side 2	3.9375	3.8125	3.1250	3.6250
		Top Total	19.5625	19.2813	18.4025	19.0821
		Bottom - Side 1	2.7500	2.3750	2.1875	2.4375
		Bottom - Side 2	2.1250	2.0000	2.1250	2.0833
		Bottom Total	17.1250	16.5938	16.4025	16.7071
60	180	Top - Side 1	5.3125	5.1250	4.8750	5.1042
		Top - Side 2	5.2500	6.5000	5.9375	5.8958
		Top Total	22.8125	23.8438	22.9025	23.1863
		Bottom - Side 1	4.3750	4.8125	4.6875	4.6250
		Bottom - Side 2	4.2500	4.6250	4.7500	4.5417
		Bottom Total	20.8750	21.6563	21.5275	21.3529
60	240	Top - Side 1	6.1875	6.4375	7.3750	6.6667
		Top - Side 2	7.6250	6.3750	7.4375	7.1458
		Top Total	26.0625	25.0313	26.9025	25.9988
		Bottom - Side 1	5.3125	5.7500	5.0625	5.3750
		Bottom - Side 2	5.1875	5.6250	5.5000	5.4375
		Bottom Total	22.7500	23.5938	22.6525	22.9988

Table B-2: 12-inch SCS-01 Raw Data

6 inch Annulus Sample Inner Diameter Measurements						
Time Step (seconds)	Total Time (seconds)	Description	Inside Diameter (inches)			
			Direction 1	Direction 2	Direction 3	Average
0	0	6.25	6.2500	6.2500	6.2500	6.2500
		10.25	6.2500	6.2500	6.2500	6.2500
		Average	6.2500	6.2500	6.2500	6.2500
30	30	Top - Side 1	3.1250	3.1250	3.3750	3.2083
		Top - Side 2	3.6250	3.3125	3.0000	3.3125
		Top Total	13.0000	12.6875	12.6250	12.7708
		Bottom - Side 1	2.7500	2.8750	3.1250	2.9167
		Bottom - Side 2	2.8125	2.9375	2.5625	2.7708
		Bottom Total	11.8125	12.0625	11.9375	11.9375
30	60	Top - Side 1	3.5625	3.3750	3.5000	3.4792
		Top - Side 2	3.5000	3.3750	3.1875	3.3542
		Top Total	13.3125	13.0000	12.9375	13.0833
		Bottom - Side 1	2.9375	3.4375	3.3750	3.2500
		Bottom - Side 2	3.3750	3.6250	2.9375	3.3125
		Bottom Total	12.5625	13.3125	12.5625	12.8125
60	120	Top - Side 1	3.4375	3.4375	3.6875	3.5208
		Top - Side 2	3.6875	3.4375	3.5625	3.5625
		Top Total	13.3750	13.1250	13.5000	13.3333
		Bottom - Side 1	3.2500	3.1875	3.4375	3.2917
		Bottom - Side 2	3.3750	3.3750	3.1250	3.2917
		Bottom Total	12.8750	12.8125	12.8125	12.8333
60	180	Top - Side 1	4.0000	3.7500	4.0000	3.9167
		Top - Side 2	4.0625	3.8125	3.8125	3.8958
		Top Total	14.3125	13.8125	14.0625	14.0625
		Bottom - Side 1	3.3750	3.4375	3.1875	3.3333
		Bottom - Side 2	3.5000	3.8750	3.3750	3.5833
		Bottom Total	13.1250	13.5625	12.8125	13.1667
90	270	Top - Side 1	4.1875	4.3750	4.1250	4.2292
		Top - Side 2	4.5000	4.5625	4.6875	4.5833
		Top Total	14.9375	15.1875	15.0625	15.0625
		Bottom - Side 1	3.3125	3.8750	3.7500	3.6458
		Bottom - Side 2	3.6250	3.6250	3.3125	3.5208
		Bottom Total	13.1875	13.7500	13.3125	13.4167
120	390	Top - Side 1	4.2500	4.3125	4.0625	4.2083
		Top - Side 2	4.5625	4.6250	4.6250	4.6042
		Top Total	15.0625	15.1875	14.9375	15.0625
		Bottom - Side 1	3.5625	3.5000	3.8125	3.6250
		Bottom - Side 2	3.6875	3.7500	3.3750	3.6042
		Bottom Total	13.5000	13.5000	13.4375	13.4792
240	630	Top - Side 1	5.0000	5.0000	5.1250	5.0417
		Top - Side 2	5.5000	5.5000	5.5000	5.5000
		Top Total	16.7500	16.7500	16.8750	16.7917
		Bottom - Side 1	3.7500	3.6250	3.8750	3.7500
		Bottom - Side 2	3.8125	3.7500	3.5000	3.6875
		Bottom Total	13.8125	13.6250	13.6250	13.6875

Table B-3: 6-inch SCS-01 Raw Data

4 inch Annulus Sample Inner Diameter Measurements						
Time Step (seconds)	Total Time (seconds)	Description	Inside Diameter (inches)			
			Direction 1	Direction 2	Direction 3	Average
0	0	5.50	4.2500	4.2500	4.2500	4.2500
		7.00	4.1250	4.1250	4.1250	4.1250
		Average	4.1875	4.1875	4.1875	4.1875
30	30	Top - Side 1	3.4375	3.5000	3.4375	3.4583
		Top - Side 2	3.0000	3.1250	3.2500	3.1250
		Top Total	10.6250	10.8125	10.8750	10.7708
		Bottom - Side 1	3.3750	3.6875	3.3750	3.4792
		Bottom - Side 2	3.3125	2.8125	3.0625	3.0625
		Bottom Total	10.8750	10.6875	10.6250	10.7292
30	60	Top - Side 1	3.5625	3.8750	3.8750	3.7708
		Top - Side 2	3.8125	3.8125	3.6250	3.7500
		Top Total	11.5625	11.8750	11.6875	11.7083
		Bottom - Side 1	3.5625	3.5625	3.5625	3.5625
		Bottom - Side 2	3.4375	3.3125	3.5000	3.4167
		Bottom Total	11.1875	11.0625	11.2500	11.1667
60	120	Top - Side 1	4.0000	3.8125	4.0000	3.9375
		Top - Side 2	3.7500	3.8750	3.8750	3.8333
		Top Total	11.9375	11.8750	12.0625	11.9583
		Bottom - Side 1	3.8125	3.6875	3.6875	3.7292
		Bottom - Side 2	3.5625	3.5000	3.3125	3.4583
		Bottom Total	11.5625	11.3750	11.1875	11.3750
60	180	Top - Side 1	3.8125	4.0000	4.0625	3.9583
		Top - Side 2	4.0000	3.8125	3.9375	3.9167
		Top Total	12.0000	12.0000	12.1875	12.0625
		Bottom - Side 1	3.8125	3.7500	3.7500	3.7708
		Bottom - Side 2	3.8750	3.6875	3.9375	3.8333
		Bottom Total	11.8750	11.6250	11.8750	11.7917
240	420	Top - Side 1	4.1875	4.5000	4.5000	4.3958
		Top - Side 2	4.1250	4.0000	4.6250	4.2500
		Top Total	12.5000	12.6875	13.3125	12.8333
		Bottom - Side 1	3.7500	3.8125	4.0000	3.8542
		Bottom - Side 2	3.7500	3.8125	3.8125	3.7917
		Bottom Total	11.6875	11.8125	12.0000	11.8333

Table B-4: 4-inch SCS-01 Raw Data

2 inch Annulus Sample Inner Diameter Measurements						
Time Step (seconds)	Total Time (seconds)	Description	Inside Diameter (inches)			
			Direction 1	Direction 2	Direction 3	Average
0	0	5.50	2.1250	2.1250	2.1250	2.1250
30	30	Top - Side 1	4.3750	4.3750	4.1250	4.2917
		Top - Side 2	3.8750	4.6250	4.8750	4.4583
		Top Total	10.3750	11.1250	11.1250	10.8750
30	30	Top - Side 1	4.5000	4.3750	4.5000	4.4583
		Top - Side 2	3.6750	4.6250	4.6250	4.3083
		Top Total	10.3000	11.1250	11.2500	10.8917
480	510	Top - Side 1	5.7500	5.7500	5.6250	5.7083
		Top - Side 2	5.8125	5.8750	5.8125	5.8333
		Top Total	13.6875	13.7500	13.5625	13.6667
		Bottom - Side 1	5.3750	5.3750	5.5000	5.4167
		Bottom - Side 2	5.3125	5.4375	5.3125	5.3542
		Bottom Total	12.8125	12.9375	12.9375	12.8958

Table B-5: 2-inch SCS-01 Raw Data

12 inch SCM 400 Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
37	37	7	10000	A	1.1250	1.7500	1.0000
				B	1.1250	1.6250	1.0000
				C	1.1250	1.2500	0.7500
				D	0.6250	1.1250	0.6250
				E	0.8750	1.3750	1.8750
				F	1.0000	1.6250	0.7500
57	94	11	10000	A	1.3750	2.0000	1.0000
				B	1.2500	1.7500	1.1250
				C	1.0000	1.1250	0.8750
				D	0.8750	1.1250	0.7500
				E	1.0000	1.2500	0.7500
				F	1.1250	1.7500	1.1250
118	212	22	9700	A	1.5000	2.1250	1.1250
				B	1.3750	1.8750	1.2500
				C	1.0000	1.2500	1.0000
				D	0.8750	1.1250	0.6250
				E	0.8750	1.3750	0.7500
				F	1.3750	2.1250	1.1250
242	454	44	10100	A	1.5000	2.1250	1.1250
				B	1.5000	2.0000	1.3750
				C	1.0000	1.1250	0.7500
				D	0.8750	1.1250	0.7500
				E	1.0000	1.5000	0.8750
				F	1.2500	2.0000	1.1250
477	931	98	10000	A	2.3750	3.0000	1.8750
				B	2.3750	2.8750	1.8750
				C	1.2500	1.8750	1.1250
				D	1.0000	1.3750	1.0000
				E	1.5000	2.0000	1.0000
				F	2.0000	2.7500	1.6250
296	1227	52	10000	A	2.3750	3.0000	1.7500
				B	2.5000	2.8750	2.0000
				C	1.3750	1.8750	1.2500
				D	1.1250	1.5000	0.8750
				E	1.3750	1.8750	1.0000
				F	2.1250	2.7500	1.5000

Table B-6: 12-inch SCS-02 Raw Data

6 inch SCM 400 Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
32	32	7	10000	A	2.6250	2.6250	2.0000
				B	2.1250	2.1250	2.7500
				C	1.2500	1.3750	1.1250
				D	1.2500	1.3750	1.2500
				E	1.3750	1.7500	1.3750
				F	2.2500	2.2500	1.8750
30	62	6	9900	A	2.8750	3.0000	2.7500
				B	2.5000	2.6250	2.2500
				C	1.5000	1.8750	1.6250
				D	1.6250	2.0000	1.5000
				E	1.8750	2.1250	1.7500
				F	2.8750	2.8750	2.3750
60	122	12	10000	A	3.2500	3.5000	3.0000
				B	2.8750	3.2500	2.8750
				C	2.0000	2.2500	1.8750
				D	1.8750	2.2500	1.8750
				E	2.1250	2.5000	2.0000
				F	3.1250	3.3750	2.8750
122	244	24	10100	A	3.6250	3.7500	3.5000
				B	3.1250	3.5000	3.1250
				C	2.1250	2.6250	2.6250
				D	2.1250	2.5000	2.2500
				E	2.3750	2.7500	2.2500
				F	3.3750	3.7500	3.2500
159	403	31	10200	A	3.6250	3.8750	3.3750
				B	3.1250	3.5000	3.1250
				C	2.3750	2.6250	2.1250
				D	2.2500	2.5000	2.2500
				E	2.3750	2.8750	2.5000
				F	3.3750	3.8750	3.3750
162	565	32	10000	A	3.6250	4.0000	3.3750
				B	3.1250	3.6250	3.1250
				C	2.5000	2.7500	2.3750
				D	2.2500	2.6250	2.3750
				E	2.5000	2.7500	2.5000
				F	3.6250	3.8750	3.3750
481	1046	92	103000	A	3.7500	4.0000	3.5000
				B	3.3750	3.7500	3.5000
				C	2.6250	3.0000	2.6250
				D	2.6250	2.7500	2.6250
				E	2.7500	3.1250	3.0000
				F	3.7500	4.0000	3.5000

Table B-7: 6-inch SCS-02 Raw Data

12 inch SCM 1300 Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
34	34	7	10000	A	0.3750	0.5000	0.3750
				B	0.3750	0.3750	0.3750
				C	0.3750	0.3750	0.3750
				D	0.3750	0.3750	0.3750
				E	0.3750	0.5000	0.3750
				F	0.3750	0.3750	0.3750
66	100	13	10000	A	0.3750	0.7500	0.5000
				B	0.5000	0.5000	0.3750
				C	0.3750	0.5000	0.3750
				D	0.3750	0.3750	0.5000
				E	0.3750	0.5000	0.3750
				F	0.3750	0.6250	0.3750
116	216	24	10000	A	0.5000	0.7500	0.5000
				B	0.5000	0.3750	0.5000
				C	0.3750	0.5000	0.3750
				D	0.3750	0.3750	0.5000
				E	0.5000	0.3750	0.3750
				F	0.5000	0.3750	0.5000
242	458	48	10000	A	0.6250	0.7500	0.5000
				B	0.5000	0.6250	0.5000
				C	0.3750	0.5000	0.3750
				D	0.5000	0.6250	0.5000
				E	0.5000	0.6250	0.3750
				F	0.6250	0.6250	0.5000
478	936	94	10000	A	0.8750	1.1250	0.6250
				B	0.6250	0.8750	0.5000
				C	0.5000	0.7500	0.5000
				D	0.5000	0.7500	0.5000
				E	0.6250	0.8750	0.5000
				F	0.7500	0.7500	0.5000
	936	90+	9800	A	1.0000	1.1250	0.7500
				B	0.7500	0.8750	0.6250
				C	0.5000	0.7500	0.5000
				D	0.6250	0.7500	0.6250
				E	0.6250	0.8750	0.5000
				F	0.6250	0.8750	0.5000

Table B-8: 12-inch SCS-03 Raw Data

6 inch SCM 1300 Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
33	33	6	10000	A	0.6250	0.5000	0.6250
				B	0.5000	0.5000	0.3750
				C	0.6250	0.5000	0.3750
				D	0.5000	0.3750	0.3750
				E	0.3750	0.6250	0.3750
				F	0.5000	0.6250	0.5000
63	96	11	9500	A	0.7500	1.1250	1.0000
				B	0.7500	0.8750	1.0000
				C	0.6250	0.7500	0.6250
				D	0.8750	0.5000	0.6250
				E	0.7500	0.8750	0.6250
				F	0.8750	1.2500	0.8750
123	219	21	9600	A	1.1250	1.6250	1.3750
				B	1.0000	1.5000	1.1250
				C	0.7500	1.0000	0.8750
				D	0.8750	0.6250	0.8750
				E	0.8750	1.2500	0.6250
				F	1.2500	1.8750	1.1250
244	463	47	9600	A	1.6250	2.0000	1.3750
				B	1.3750	1.3750	1.2500
				C	0.8750	1.1250	1.0000
				D	1.0000	0.8750	0.7500
				E	0.8750	1.2500	0.7500
				F	1.6250	2.0000	1.3750
485	948	97	9700	A	2.0000	2.2500	2.0000
				B	1.3750	2.0000	1.5000
				C	1.0000	1.3750	1.1250
				D	0.8750	1.0000	1.1250
				E	1.0000	1.2500	0.8750
				F	1.8750	2.2500	1.6250
483	1431	90	9500	A	2.5000	2.8750	2.2500
				B	2.2500	2.5000	2.0000
				C	1.2500	1.6250	1.3750
				D	1.1250	1.2500	1.5000
				E	1.2500	1.5000	1.1250
				F	2.1250	2.5000	1.8750

Table B-9: 6-inch SCS-03 Raw Data

12 inch 1000 psi Concrete Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Inside Diameter (inches)		
					Direction 1	Direction 2	Direction 3
0	0	0	0	Top/Middle/Bottom	12.3750		
35	35	6	9100 - 9200	Top	12.3750	12.3750	12.5000
				Middle	12.5000	12.3750	12.5000
				Bottom	12.3750	12.3750	12.5000
61	96	9	9000	Top	12.6250	12.5000	12.6250
				Middle	12.7500	12.6250	12.6250
				Bottom	12.5000	12.5000	12.3750
118	214	18	9100	Top	12.6250	12.6250	12.7500
				Middle	12.7500	13.0000	12.6250
				Bottom	12.5000	12.5000	12.5000
214	428	35	9200	Top	12.6250	12.7500	12.8750
				Middle	12.7500	13.0000	13.0000
				Bottom	12.5000	12.5000	12.5000
230	658	34	10300	Top	12.8750	12.7500	13.0000
				Middle	13.0000	13.1250	12.7500
				Bottom	12.6250	12.6250	12.5000
474	1132	34	10200 - 10300	Top	13.1250	13.0000	12.8750
				Middle	13.0000	13.2500	12.7500
				Bottom	12.7500	12.7500	12.7500

Table B-10: 12-inch CON-01 Raw Data

6 inch 1000 psi Concrete Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	(inches)		
					Direction 1	Direction 2	Direction 3
0	0	0	0	Top/Middle/Bottom	6.3750		
31	31	1 1/2	10000	Top	No Measurements Taken		
				Middle			
				Bottom			
32	63	—	10200	Top	6.6250	6.5000	6.5000
				Middle	6.7500	6.7500	7.1250
				Bottom	6.5000	6.3750	6.5000
59	122	11	10300	Top	6.7500	7.0000	6.5000
				Middle	6.8750	7.3750	7.1250
				Bottom	6.5000	6.5000	6.5000
Measurement Change					Top	Middle	Bottom
120	242	21	10100	A	0.7500	0.8750	0.6250
				B	0.7500	0.8750	0.7500
				C	0.6250	0.6250	0.5000
				D	0.5000	0.7500	0.5000
				E	0.6250	0.7500	0.6250
				F	0.6250	0.7500	0.6250
240	482	37	10000 - 10050	A	1.0000	1.1250	0.7500
				B	0.8750	0.8750	0.7500
				C	0.7500	1.0000	0.6250
				D	0.7500	1.0000	0.6250
				E	0.6250	0.8750	0.6250
				F	0.8750	0.8750	0.7500
479	961	84	10200	A	1.5000	1.5000	1.1250
				B	1.2500	1.1250	1.0000
				C	0.8750	1.1250	0.8750
				D	0.7500	1.2500	0.7500
				E	1.0000	0.8750	0.7500
				F	1.1250	1.2500	0.8750
481	1442	79	10200	A	1.5000	1.6250	1.3750
				B	1.5000	1.5000	1.5000
				C	1.8750	1.2500	1.2500
				D	1.1250	1.2500	1.0000
				E	0.8750	1.2500	1.1250
				F	1.1250	1.2500	1.2500

Table B-11: 6-inch CON-01 Raw Data

12 inch 2000 psi Concrete Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	12.3750	12.2500	12.3750
88	88	14	9800	A	0.3750	0.3750	0.3750
				B	0.3750	0.3750	0.2500
				C	0.3750	0.2500	0.2500
				D	0.2500	0.3750	0.3750
				E	0.2500	0.3750	0.2500
				F	0.3750	0.3750	0.2500
358	446	56	9800	A	0.3750	0.3750	0.3750
				B	0.5000	0.3750	0.2500
				C	0.3750	0.3750	0.3750
				D	0.3750	0.3750	0.3750
				E	0.3750	0.3750	0.3750
				F	0.3750	0.3750	0.2500
960	1406	153	9500	A	0.3750	0.3750	0.3750
				B	0.3750	0.3750	0.2500
				C	0.3750	0.3750	0.2500
				D	0.3750	0.5000	0.2500
				E	0.3750	0.3750	0.3750
				F	0.3750	0.5000	0.2500
480	1886	Stationary Jet	9500	A	No Visible Erosion		
				B			
				C			
				D			
				E			
				F			

Table B-12: 12-inch CON-02 Raw Data

6 inch 2000 psi Concrete Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	6.3750		
30	30	6	9900	A	0.3750	0.5000	0.3750
				B	0.3750	0.3750	0.2500
				C	0.3750	0.3750	0.2500
				D	0.3750	0.5000	0.3750
				E	0.5000	0.3750	0.3750
				F	0.5000	0.5000	0.3750
60	90	10	9500	A	0.5000	0.5000	0.5000
				B	0.6250	0.5000	0.3750
				C	0.6250	0.6250	0.5000
				D	0.5000	0.6250	0.6250
				E	0.5000	0.5000	0.5000
				F	0.6250	0.6250	0.3750
122	212	19	10000	A	0.6250	0.5000	0.5000
				B	0.8750	0.6250	0.5000
				C	0.7500	1.0000	0.5000
				D	0.7500	0.7500	0.5000
				E	0.6250	0.7500	0.6250
				F	0.7500	0.7500	0.5000
237	449	41	10000	A	0.5000	0.6250	0.5000
				B	0.8750	0.5000	0.5000
				C	0.7500	1.1250	0.6250
				D	0.8750	0.7500	0.5000
				E	0.5000	0.7500	0.8750
				F	0.7500	0.8750	0.5000
477	926	____ + 49	10000	A	0.8750	1.0000	0.7500
				B	0.8750	0.7500	0.5000
				C	1.0000	1.2500	0.6250
				D	1.0000	1.1250	1.1250
				E	0.6250	1.1250	1.0000
				F	0.8750	0.8750	0.6250
477	1403	76	9500	A	1.0000	1.1250	0.8750
				B	1.2500	1.2500	0.7500
				C	1.3750	1.3750	0.8750
				D	1.0000	1.2500	1.3750
				E	0.7500	1.0000	0.8750
				F	1.0000	1.0000	0.8750

Table B-13: 6-inch CON-02 Raw Data

12 inch 4000 psi Concrete Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	Diameter (inches)		
					Direction 1	Direction 2	Direction 3
0	0	0	0	Top/Middle/Bottom	12.5000	12.0000	12.2500
16	16	0	9000	Top	No Measurements Taken		
				Middle			
				Bottom			
28	44	1/2	9000	Top	No Measurements Taken		
				Middle			
				Bottom			
118	162	16	9200	Top	12.5000	12.2500	12.5000
				Middle	12.5000	12.2500	12.3750
				Bottom	12.3750	12.1250	12.3750
117	279	20	9100	Top	12.3750	12.2500	12.5000
				Middle	12.6250	12.5000	12.3750
				Bottom	12.5000	12.3750	12.3750
234	513	40	9200	Top	12.5000	12.3750	12.5000
				Middle	12.6250	12.6250	12.6250
				Bottom	12.3750	12.2500	12.5000
445	958	70	9350	Top	12.6250	12.5000	12.5000
				Middle	12.7500	12.5000	12.6250
				Bottom	12.3750	12.3750	12.5000

Table B-14: 12-inch CON-03 Raw Data

6 inch 4000 psi Concrete Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Number of Cycles	Pressure at Nozzle (psi)	Description	(inches)		
					Direction 1	Direction 2	Direction 3
0	0	0	0	Top/Middle/Bottom	6.2500	6.3750	6.2500
30	30	5	9150	Top	6.2500	6.3750	6.3750
				Middle	6.2500	6.5000	6.3750
				Bottom	6.3750	6.3750	6.3750
50	80	9	9200	Top	7.0000	6.6250	6.5000
				Middle	6.6250	6.5000	6.3750
				Bottom	6.6250	6.5000	6.3750
121	201	20	9200	Top	7.2500	6.7500	7.0000
				Middle	6.8750	6.5000	6.8750
				Bottom	7.0000	6.5000	6.6250
Measurement Change					Top	Middle	Bottom
242	443	41	9300	A	0.5000	0.5000	0.3750
				B	0.7500	0.6250	0.3750
				C	0.6250	0.3750	0.2500
				D	0.3750	0.5000	0.3750
				E	0.7500	0.5000	0.6250
				F	0.6250	0.5000	0.3750
454	897	84	9400	A	0.6250	0.7500	0.3750
				B	0.8750	0.6250	0.5000
				C	0.6250	0.6250	0.5000
				D	0.7500	0.6250	0.5000
				E	0.7500	0.6250	0.6250
				F	0.7500	0.6250	0.5000
483	1380	69	10000	A	0.8750	0.7500	0.5000
				B	0.8750	1.1250	0.7500
				C	0.7500	0.7500	0.5000
				D	1.0000	1.0000	0.6250
				E	1.1250	0.8750	0.7500
				F	0.7500	0.7500	0.6250
480	1860	76	10100	A	0.8750	0.8750	0.6250
				B	0.8750	1.1250	0.7500
				C	1.0000	0.7500	0.6250
				D	1.0000	1.1250	0.7500
				E	1.1250	0.8750	0.7500
				F	1.2500	0.7500	0.8750

Table B-15: 6-inch CON-03 Raw Data

6 inch 4000 psi Mix with Angular Aggregate Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
32	32	10500	7	A	1.7500	2.0000	1.6250
				B	1.5000	1.7500	1.2500
				C	0.7500	0.5000	0.6250
				D	0.8750	1.0000	0.7500
				E	0.7500	1.2500	1.0000
				F	2.2500	2.1250	2.1250
60	92	11000	11	A	2.7500	3.2500	2.5000
				B	2.6250	2.6250	2.0000
				C	1.2500	1.6250	1.1250
				D	1.5000	1.7500	1.5000
				E	1.5000	1.6250	1.6250
				F	3.0000	3.1250	2.5000
123	215	10700	22	A	3.0000	3.2500	2.6250
				B	2.6250	2.7500	2.2500
				C	1.3750	1.5000	1.2500
				D	1.8750	1.8750	1.7500
				E	2.0000	2.0000	1.7500
				F	2.8750	3.6250	2.6250
242	457	10200	43	A	2.8750	3.5000	3.1250
				B	2.8750	3.0000	2.5000
				C	1.5000	1.6250	1.6250
				D	1.7500	2.0000	1.6250
				E	1.8750	2.1250	1.8750
				F	3.1250	3.5000	2.8750
478	935	10200	84	A	3.5000	4.0000	3.5000
				B	3.1250	3.5000	3.0000
				C	2.5000	2.3750	2.0000
				D	2.1250	2.3750	2.0000
				E	2.5000	2.6250	2.5000
				F	3.3750	3.6250	3.3750
960	1895	10200	184	A	3.5000	4.0000	3.7500
				B	3.6250	3.6250	3.1250
				C	2.3750	2.6250	3.2500
				D	2.1250	2.6250	2.1250
				E	2.3750	2.6250	3.3250
				F	3.2500	3.7500	3.6250

Table B-16: 6-inch CON-04 Raw Data

6 inch 4000 psi Mix with Rounded Aggregate Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
33	33	10200	7	A	0.6250	0.6250	0.3750
				B	0.7500	0.3750	0.3750
				C	0.3750	0.3750	0.3750
				D	0.3750	0.3750	0.3750
				E	0.3750	0.5000	0.5000
				F	0.6250	0.6250	0.3750
60	93	10200	12	A	0.8750	0.6250	0.6250
				B	1.0000	0.5000	0.3750
				C	0.3750	0.5000	0.3750
				D	0.5000	0.3750	0.3750
				E	0.3750	0.5000	0.3750
				F	0.6250	0.8750	0.3750
123	216	10400	23	A	0.8750	0.8750	0.8750
				B	1.0000	0.8750	0.6250
				C	0.6250	0.5000	0.5000
				D	0.6250	0.5000	0.3750
				E	0.3750	0.6250	0.3750
				F	0.7500	1.1250	0.7500
240	456	10300	46	A	1.0000	1.5000	1.1250
				B	1.0000	1.2500	0.8750
				C	0.6250	0.6250	0.5000
				D	0.6250	0.6250	0.5000
				E	0.5000	0.5000	0.5000
				F	1.0000	1.7500	2.0000
478	934	10600	92	A	1.2500	2.0000	1.6250
				B	1.8750	1.8750	1.1250
				C	0.7500	0.8750	0.5000
				D	1.0000	0.8750	0.7500
				E	1.5000	0.7500	1.0000
				F	1.0000	2.6250	2.3750
960	1894	10400	180	A	1.7500	2.2500	1.6250
				B	1.7500	1.8750	1.5000
				C	0.8750	0.8750	0.6250
				D	1.0000	1.0000	0.8750
				E	1.3750	1.1250	1.0000
				F	1.1250	2.5000	2.0000

Table B-17: 6-inch CON-05 Raw Data

12 inch SMX Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
1200	1200	10600	---	A	0.6250	-	0.5625
				B	0.6250	-	0.5000
				C	0.6250	-	0.5000
				D	0.6250	-	0.5000
				E	0.6250	-	0.5625
				F	0.8125	-	0.6875

Table B-18: 12-inch SMX-01 Raw Data

6 inch SMX Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
33	33	10800	11	A	0.7500	0.8750	0.3750
				B	0.3750	0.6250	0.6250
				C	0.8750	0.7500	0.5000
				D	0.5000	0.6250	0.6250
				E	0.5000	0.7500	0.5000
				F	0.7500	0.7500	0.5000
64	97	10500	12	A	0.6250	0.8750	0.5000
				B	0.6250	1.0000	0.8750
				C	0.5000	0.6250	0.7500
				D	0.5000	1.0000	0.7500
				E	0.6250	1.1250	0.5000
				F	0.6250	1.1250	0.5000
122	219	---	25	A	1.2500	1.0000	0.7500
				B	1.1250	1.1250	0.7500
				C	0.6250	0.8750	0.7500
				D	0.5000	1.0000	0.8750
				E	0.6250	1.2500	1.2500
				F	0.7500	1.1250	1.2500
238	457	10600	47	A	1.5000	1.7500	0.8750
				B	1.3750	1.2500	0.7500
				C	0.7500	0.8750	0.8750
				D	0.7500	1.0000	1.0000
				E	0.7500	1.0000	1.2500
				F	1.0000	1.5000	1.8750
483	940	10600	93	A	1.7500	1.6250	1.1250
				B	1.5000	1.3750	1.1250
				C	1.1250	1.0000	0.8750
				D	0.7500	1.0000	1.0000
				E	1.6250	1.3750	1.6250
				F	1.1250	1.7500	1.8750
960	1900	10500	185	A	2.0000	1.8750	1.2500
				B	1.5000	1.5000	1.1250
				C	1.2500	1.0000	0.8750
				D	1.1250	1.1250	1.1250
				E	1.3750	1.5000	1.5000
				F	1.8750	2.0000	2.1250

Table B-19: 6-inch SMX-01 Raw Data

12 inch Grout Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
31	31	11000	7	A	0.5000	0.6250	0.5625
				B	0.5000	0.6250	0.3750
				C	0.3750	0.5625	0.3750
				D	0.4375	0.3750	0.3750
				E	0.5000	0.6250	0.3750
				F	0.5000	0.5625	0.4375
62	93	10800	11	A	0.7500	0.9375	0.6875
				B	0.6750	1.0000	0.5625
				C	0.6875	0.5000	0.6250
				D	0.6250	0.6250	0.4375
				E	0.5000	0.9375	0.5000
				F	0.7500	0.8750	0.6250
240	333	10900	---	A	1.1250	-	-
				B	1.1875	-	-
				C	0.7500	-	-
				D	0.7500	-	-
				E	0.8750	-	-
				F	1.1250	-	-
960	1293	10700	---	A	1.2500	-	-
				B	1.1875	-	-
				C	0.7500	-	-
				D	0.6875	-	-
				E	0.8750	-	-
				F	1.2500	-	-

Table B-20: 12-inch GRT-01 Raw Data

6 inch Grout Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
33	33	---	8	A	1.1250	1.1250	0.7500
				B	1.0000	0.7500	0.6250
				C	0.5000	0.5000	0.3750
				D	0.5000	0.6250	0.5000
				E	0.5000	0.6250	0.5000
				F	1.6250	1.2500	0.8750
64	97	10400	13	A	1.2500	1.6250	0.8750
				B	1.2500	1.2500	0.6250
				C	0.5000	0.6250	0.3750
				D	0.6250	0.7500	0.5000
				E	0.6250	0.7500	0.6250
				F	1.7500	1.7500	1.0000
121	218	10400	24	A	1.2500	2.0000	1.2500
				B	1.6250	1.3750	1.0000
				C	0.6250	0.6250	0.5000
				D	0.6250	0.6250	0.5000
				E	0.8750	1.0000	0.7500
				F	2.1250	2.1250	1.3750
242	460	10400	50	A	1.2500	2.0000	1.2500
				B	1.7500	1.3750	1.1250
				C	0.6250	0.7500	0.5000
				D	0.6250	0.7500	0.6250
				E	0.8750	1.1250	0.7500
				F	2.1250	2.1250	1.6250
482	942	10600	97	A	2.2500	2.5000	1.7500
				B	2.2500	2.0000	1.7500
				C	1.0000	1.0000	0.6250
				D	1.0000	1.0000	0.7500
				E	1.1250	1.3750	1.0000
				F	2.5000	2.5000	2.0000
960	1902	---	---	A	2.8750	3.0000	2.7500
				B	2.8750	3.7500	2.5000
				C	2.2500	2.0000	1.3750
				D	2.1250	2.0000	1.3750
				E	2.1250	2.3750	1.7500
				F	2.5000	3.0000	3.2500

Table B-21: 6-inch GRT-01 Raw Data

6 inch Clay Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
30	30	10400	7	A	5.8750	5.8750	5.8750
				B	6.0000	4.7500	4.6250
				C	5.0000	4.7500	4.5000
				D	4.7500	4.3750	5.7500
				E	5.5000	5.2500	4.8750
				F	6.1250	6.1250	6.1250
Measurement Change					Diameter (inches)		
64	94	10400	13	A-B	18.0000	17.5000	17.5000
				C-D	18.7500	17.7500	17.0000
				E-F	19.0000	19.0000	17.5000

Table B-22: 6-inch CLY-01 Raw Data

12 inch Clay Annulus Sample Measurements							
Time Step (seconds)	Total Time (seconds)	Pressure at Nozzle (psi)	Number of Cycles	Description	Removal Depth (inches)		
					Top	Middle	Bottom
0	0	0	0	A through F	0.2500		
32	32	10600	8	A	2.5000	3.0000	1.2500
				B	2.6250	1.7500	1.0000
				C	1.3750	0.8750	0.3750
				D	1.6250	1.5000	0.5000
				E	2.0000	2.5625	1.6250
				F	2.9375	3.6250	2.0000
32	64	10600	6	A	2.7500	3.5000	1.8750
				B	2.5000	2.0625	1.0000
				C	1.5000	1.3750	0.5000
				D	1.8125	1.8750	0.8750
				E	2.6250	2.7500	1.7500
				F	3.2500	3.8125	2.6250
63	127	10600	13	A	2.7500	3.7500	3.0000
				B	2.5000	2.0000	2.0000
				C	1.5625	1.3750	0.7500
				D	2.1250	1.6250	1.0000
				E	2.6250	2.5000	1.5000
				F	3.2500	4.0000	2.5000
122	249	10600	24	A	3.2500	4.0000	3.1250
				B	2.6250	2.4375	2.2500
				C	1.7500	1.3750	1.2500
				D	2.2500	1.7500	1.1250
				E	2.8750	2.7500	1.7500
				F	3.2500	4.2500	2.7500
242	491	10600	46	A	3.5000	4.0000	3.1250
				B			
				C	2.1250	1.7500	1.6250
				D	2.5000	1.8750	1.2500
				E			
				F	3.6250	4.3750	2.8750

Table B-23: 12-inch CLY-02 Raw Data