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Composite Repair of Sandstone

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COMPOSITE REPAIR OF SANDSTONE

JAMES WELDON DOSSETT

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CHAPTER ONE: INTRODUCTION TO COMPOSITE REPAIR OF SANDSTONE

USE OF SANDSTONE AS A BUILDING MATERIAL

Sandstone has been one of the major building stones in North American since at least the middle part of the nineteenth century. In the Northeast, especially in cities such as New York, Boston, and Philadelphia, the brown sandstone – or simply brownstone – was preferred for its range of colors, tan to dark brown and red, first in ecclesiastical buildings and later in residential row houses (Matero and Teutonico 1982).

By the late nineteenth century, however, the brown sandstone had earned a reputation as an unsuitable building material for the Northeast. Buildings only a few years old began to exhibit excessive decay (Julien 1883).

SANDSTONE DECAY

There are three primary factors related to the decay of sandstone: *composition*, the physical make-up of the stone; *environment*, the stone's exposure to damaging agents such as water, salt and corrosive chemicals; and *use*, how the stone is quarried, shaped and installed in a structure. All three factors are interrelated and must be considered together to understand the decay processes for sandstone.

SANDSTONE COMPOSITION

Sandstone is a clastic sedimentary rock. Also known as layered rocks, sedimentary rocks are formed by one of two processes. Clastic sedimentary rocks form through the accumulation of rock particles by water or wind action. Organic/chemical sedimentary rocks form by accumulation of organic material or by chemical precipitates from ocean water (Winkler 1994). In most clastic sedimentary rocks - including sandstone - the grains are quartz. Quartz is a hard, chemical-resistant silicate mineral not directly attacked by most weathering agents; however, the size and shape of the grains is responsible for the pore size and shape of the rock. This pore space is a major factor in the durability of the stone. This is discussed in greater detail in the section on environment.

The grains of the sandstone are held together by one of four types of cementing matrix: 1) *siliceous*, in which silica or silicon dioxide is the binder; 2) *calcareous*, in which calcite is the binder; 3) *ferruginous*, which is cemented by iron oxides - usually limonite; and 4) *argillaceous*, in which clay is the binder. Some sandstones have more than one type of binder (Robertson 1982). The importance of the matrix in determining the durability of the stone was recognized as early as 1880 (Julien 1883). The matrix can be attacked by water

or by water- or air-borne corrosive agents (the relationship of the cementing matrix to decay is discussed in more detail in the section on environment).

The structure of the sandstone is the final aspect of composition related to decay. Sandstone is a sedimentary rock formed in layers as the grains are deposited. Structure is the type and thickness of the layers or beds (Winkler 1994). The thickness of the beds can range from less than one inch to many feet. The seams between the beds are a natural line of weakness in the stone (discussed in the section on use).

Sandstone's aggregate, matrix, and structure, along with its geographical source and color, combine to classify the sandstone. For example, the stone used in this study is from the Connecticut River Valley. It is a red/brown ferruginous-bound sandstone with primarily quartz aggregate. It has medium sized grains and beds of 2 – 18 feet thick, although tremendous variations in fabric exist.

ENVIRONMENT

The second major influence on the decay of sandstone is its environment. The stone is exposed to damaging agents like water, salt, wind, atmospheric pollution, and microorganisms. Human contact can damage a building through misuse, accidents, or acts of vandalism. The environment of any structure is a

very complicated system and it is rare that any single decay mechanism occurs alone. Water is the most crucial factor in sandstone decay because it is itself damaging, and it is the delivery agent for many other decay mechanisms.

The arrangement of the grains results in open spaces or pores in the stone. These pores allow liquid water and water vapor to travel through the stone. The water can come from the atmosphere in the form of rain or vapor or it can come from the ground. Water alone can damage sandstone through the processes of hydric dilatation, the volumetric expansion of a material in the presence of liquid water, and hygric dilatation, expansion in a humid environment (Snethlage and Wendler 1997). Argillacious or clay-bound sandstones are particularly vulnerable because clays swell a great deal (Robertson 1982). Hydric and hygric dilatation can damage the stone by breaking the bonds between grains and thereby dislodging the grains. Although the stone will contract to its original dimension when it dries, Snethlage and Wendler (1997) suggest that wet/dry cycling and its resultant expansion and contraction can cause material fatigue and more serious damage. The combination of water and freezing temperature results in a similar decay mechanism. Water expands when it freezes displacing grains and eventually dislodging them. The joints between the beds are a natural weak point in the stone and are particularly vulnerable to these types of decay.

Water also carries dissolved salts into the stone. Some soluble salts are present in the stone when it is quarried. Other sources of salts include groundwater, sea spray, deicing salts, and cements used in mortars (Winkler 1994). Lewin (1982) demonstrated that damage to sandstone from salt occurs when dissolved salts are deposited in the pores near the surface of the stone. The salts expand when they crystallize and cause a thin layer of the sandstone to blister or turn to powder. The decay will continue under the initial decay. Snethlage and Wendler (1997) showed how the type of decay - ranging from dislodging of individual grains to scaling up to 3 cm thick - could be explained. Salt concentrates and crystallizes in the part of the stone where water is retained in the pores the longest, not necessarily very near the surface. The depth of the zone of maximum water content varies according to the pore size and distribution of the particular stone, as well as conditions on the building. Surfaces exposed to sun and wind will dry quickly meaning the area that remains wet and attracts salts beneath the surface. The damage will be formation of scales. Surfaces that are sheltered and retain moisture will have granular detachment, or “sanding off” (Snethlage and Wendler 1997).

Some salts will hydrate, absorbing water into the crystal lattice of the salt. Hydration and dehydration depend on temperature and relative humidity.

Hydration increases the volume of the salt, putting pressure on the pore wall. The amount of expansion and the rate depend on conditions and the type of salt (Winkler 1994).

Plants and animals ranging from the smallest - bacteria, algae, lichens - to the largest - trees, birds, humans - will attack stone chemically and mechanically. Biodeterioration of stone is a complex and not fully understood process involving three stages of development : 1) the stone is colonized by air and water born organisms 2) decay is initiated 3) the damaged stone flakes or disintegrates and the process begins again on the new surface (Koestler et al. 1997). Certain bacteria attack minerals directly by producing corrosive chemicals. Algae, fungi and mosses as well as higher plants can mechanically attack stone by expanding within the pores or by producing organic acids that damage the minerals. They can also trap moisture within the stone. Scaling of sandstone similar to salt decay is possible through the action of algae that can penetrate deeply into the stone (Winkler 1994).

USE

The final major influence on the durability of sandstone is how the stone is tooled and set in place. Sandstone is a sedimentary rock, and bedding planes are a natural line of weakness. Quarrymen exploit these seams when cutting

sandstone (Bowles 1934). The stone is best set with the bedding planes parallel to the ground. If set on edge, water is more likely to get into the seams and split the stone. Each bed also has slightly different composition and different mechanical properties. If set on edge, differential stresses between the beds are created as the weight of the masonry above puts load on the stone. This creates shear stress along the bedding planes and can cause the stone to split (Harris 1998). The problem is worse if the stone is set with the bedding planes perpendicular to the ground *and* parallel with the face of the wall. This is known as face-bedding (Bowles 1934). The outermost bed is unsupported on one side and prone to pop off the wall.

There is no agreement on what agent is most responsible for sandstone decay. Julien (1883) claims that the stone is inferior and not suitable for the Northeast climate. Bowles (1934) believes there is nothing wrong with the stone as long as it is quarried, cut, and set correctly. In either case, there are many buildings in need of treatment today, due to poor stone selection and use.

SANDSTONE TREATMENTS

Composite repair is a method for replacing lost or damaged areas of stone by filling the loss area with a material that hardens in place and bonds to the substrate. Composite repair of sandstone is not an isolated treatment. In most

cases, it is just one element of a larger plan that can include many different treatments: cleaning, removal of soluble salts, consolidation, other types of repair including mechanical pinning, adhesive repair, replacement in kind, and replacement with artificial products such as cast stone.

CLEANING

If the treatment plan includes cleaning, it is best to clean before beginning composite repair (Ashurst and Dimes 1990). The repair material, once cured, must have color similar to the cleaned stone; if the repair is matched to soiled stone, the repair will stand out in contrast to the surrounding surface when the building is cleaned. Sometimes it is not possible to clean the building before installing composite repairs. In these cases, a test area can be cleaned for determining the color of the repair material, and then the repair is temporarily colored to blend with the surrounding soiled stone until the building is cleaned in the future. The method of cleaning[1] must be appropriate for the repair material as well as the stone. For example, an abrasive cleaning method that is suitable for the stone but too aggressive for a softer repair will etch away the repair and

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[1] Three general methods of stone cleaning are commonly used (Winkler 1994; Boyer 1988). Water washing methods include simple rinsing, misting, low to high pressure washing, and steam cleaning. Abrasive methods involve an abrasive material blown with or without water through a nozzle under pressure. The type of abrasive, the amount of pressure and amount of water vary with the nature of the stone and the soiling. Chemical cleaning methods use acidic or alkaline cleaners, and organic solvents.

create a relief between the stone and the repair. The relief can trap water and soiling leading to more rapid deterioration.

Due to the damage caused by salts, the removal of salts from the stone is often included in the treatment plan. There are three general methods of salt removal. The first is rinsing the surface with water to dissolve the salts. As long as there is sufficient water to keep the salt in solution, the dissolved salts are carried to the surface and away from the building. This method is not always practical because the large amount of water needed can create more problems, and because any salts left behind after the stone dries will crystallize at or just beneath the surface where it is likely to cause damage. The second method of salt removal is poulticing. It involves packing an absorbent material onto the surface of the stone. The dissolved salts are carried into the poultice instead of crystallizing on the surface. The third method combines rinsing the surface with a suction device to extract the water and the dissolved salts. This method is reportedly more efficient than poulticing and does not leave salts on or near the surface of the stone (Gauri et al. 1986). If the treatment plan includes salt removal, it should be done before composite repair to minimize the risk of crystallization at the interface between the stone and the repair.

CONSOLIDATION

Weathering of sandstone causes loss of grain to grain cohesion either at the surface or below the surface. If the loss of cohesion occurs at the surface the type of damage is individual aggregate dislodging from the stone, while loss of cohesion beneath the surface results in a flake or scale (Sasse and Snethlage 1997). Consolidation attempts to replace lost cementing material, increase grain to grain cohesion, adhere a decayed area to sound stone, and render the stone more resistant to future decay (Tabasso 1988).

The basic principle of stone consolidation is to introduce a compound that will penetrate into the stone and reestablish grain to grain cohesion either by forming a mineral bridge between grains, as with waterglass and ethyl silicates, or by forming a continuous membrane, as with organic resins (Sasse and Snethlage 1997). Stone consolidation is a very old practice and the list of materials used is long. The most common consolidant for sandstone is ethyl silicate (Winkler 1994). It was first reported as a stone consolidant in 1861 (Lewin 1988), and has been used successfully for several decades (Winkler 1994).

In many ways, the criteria for evaluating sandstone consolidation are similar to the criteria for composite repair. The consolidated stone should have enhanced

grain to grain cohesion, but must not be made stronger than the unconsolidated stone or the strengthened zone at the surface can scale off (Sasse and Snethlage 1997). The consolidated stone must be compatible with the unconsolidated stone in terms of color, water absorption, water vapor transmission, and thermal expansion just as composite repairs should be compatible with the surrounding stone. If the treatment plan calls for both consolidation and composite repair, consolidation - which inevitably alters the properties of the stone - almost always takes place before composite repair (Weber 1991). Consolidating first also helps ensure that the repair is applied to sound stone, and in some applications, consolidating can eliminate the need to chisel the area to be repaired down to sound stone.

REPAIR OF SANDSTONE

Composite repair is one of many types of treatments for replacing lost stone that fall under two headings: replacement and composite repair (Griswold and Uricheck 1998).[2] Replacement involves filling the loss area by attaching a piece of stone or cast material, or replacing the entire damaged masonry unit with stone or cast material. Composite repair is a method for replacing lost or damaged areas of stone by filling the loss area with a material that hardens in

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[2] Griswold and Uricheck point out that the terminology for stone repair methods differ. Fine arts conservators prefer "plastic repair" or "fill", while architectural conservators use "mortar repair," "composite repair" or "composite patch." They all refer to the same technique.

place and bonds to the substrate. While replacement with stone is an excellent technique, it has limitations. Carving a new stone piece is costly and time-consuming, and it requires skill that might not be available (Griswold and Uricheck 1998; Ashurst and Dimes 1990). It can be difficult to find matching stone (Weiss et al. 1982). Casting a replicate piece can be a cost effective alternative, but it requires an undamaged element to make a mold (if the lost element is carved), and some stones are very difficult to match. The most critical aspect of replacement is the joint between the new work and the old (Griswold and Uricheck 1998). The joint must be strong enough to bear the weight of the new work but not so strong that it damages the stone.

COMPOSITE REPAIR

Composite repair is generally reserved for shallow or isolated losses – less than two inches in depth – on flat wall areas, and for rebuilding corners, carvings or relief areas (Ashurst and Dimes 1990). The former condition is most common where the stone has been face-bedded. The latter is found on cornices, lintels, sills, and portals.

Proper application is required for a successful composite repair. Sources (Ashurst and Ashurst 1988; Ashurst and Dimes 1990; Hill and David 1995; Weiss et al. 1982) recommend chiseling out all deteriorated substrate, and/or

consolidating the area. The area to be repaired should be square cut without feathered edges. All foreign material must be removed and the surface cleaned. Mechanical keying for the composite material can be created by drilling or chiseling the surfaces, or inserting non-corroding anchors or pins. Depending on the type of repair material, the surface may need pre-wetting.

Composite repair consists essentially of a mixture of binders and aggregates, with pigments or other additives to give the material the necessary properties. There are many recent sources that discuss the properties of composite repair of stone (Ashurst and Ashurst 1988; Ashurst and Dimes 1990; Edison 1991; Iveson 1987; Hill and David 1995; Korpan 1982; Lynch 1984; Selwitz 1992; Torraca 1988; Weiss et al. 1982). The following is a summary of the properties that the sources indicate are critical for any composite repair.

COLOR AND TEXTURE

It is a given that a composite repair should resemble the stone itself. The degree to which a composite repair must duplicate the appearance of the stone depends on its location; a repair at ground level is subject to closer scrutiny than one on an upper level. Current conservation practice suggests that composite repair should closely match the color and texture of the stone and still be discernible from the stone. Given the difficulty of matching the complex color

variations of stone, especially for sandstone repair, keeping the repair discernible is less of a problem than color matching. The color of the composite is influenced by the aggregate, the binder and the use of pigments. The texture of the composite repair is just as important to the appearance of the repair. Texture is achieved by using aggregate that resembles the stone and by tooling the repair once it has set. While acknowledging their importance, this study does not address color or texture matching of the composites to the stone. Extensive research in this area has been done by Weiss et al. (1982). The composites tested in this experimental protocol use an aggregate that gives acceptable texture match with the assumption that they could be further pigmented to match the color.

WATER VAPOR TRANSMISSION AND WATER ABSORPTION

Water vapor transmission is the ability for water vapor to pass through the material. Water absorption is the ability of the material to absorb liquid water. It is important that the composite used can transmit liquid water and water vapor at rates equal to or preferably greater than the stone. Otherwise, water is forced into the stone where it damages the stone instead of the composite repair, or it is trapped at the interface between the repair and the stone, weakening the bond.

BOND STRENGTH

A composite repair should have sufficient bonding capability to adhere to the stone. The bond must not be so strong that the stone is damaged if the repair fails or must be removed.

STRENGTH

There are many ways to describe the strength of a material. For composite repair, the two most critical aspects of strength are *flexural strength* and *elasticity*. Most composite repairs are not structural and therefore not required to bear the weight of the masonry above. The stress endured by a composite repair is induced by differential movement of the repair and the stone through thermal expansion and swelling from water absorption. Flexural strength is a measure of the material's resistance to cracking as it bends. Elasticity is how stiff a material is, or how it resists bending. The composite repair material should have a high resistance to cracking and be flexible enough to absorb stress caused by movement. These properties are discussed in detail in the chapter on testing.

DIMENSIONAL STABILITY

This refers to how much a material shrinks as it cures, and to how much a material expands and contracts in reaction to changes in temperature and moisture content. The composite repair should have minimal shrinkage as it cures to prevent cracking. Once cured, the repair should have expansion and contraction properties similar to that of the stone, so that the repair moves with the stone and not against it.

DURABILITY

Broadly defined, durability of a composite repair is a measure of how well the composite repair withstands the corrosive or abrasive effects of the environment. A composite repair should not be durable at the expense of the stone. There are numerous ways to test durability, which are discussed in more detail in the chapter on testing.

COMPOSITION

The properties described above are controlled by manipulating the constituent parts of the composite repair material - aggregates, binders, and additives. Each part of the composite repair material plays a specific role in the properties of the material.

Aggregate is primarily responsible for the color and texture of the repair and must be carefully matched to the substrate. As with natural sandstone, the size and shape of the aggregate and the particle size distribution affect the pore size and distribution in the cured patch, and thereby the water absorption and water vapor transmission. In fine arts conservation, aggregates for composite repair of stone include a variety of materials: glass and ceramic microballoons, crushed stone, solid resins, crushed glass (Griswold and Uricheck 1998). For sandstone repair, the aggregate is either sand or crushed stone.

Binders, as the name suggests, cement the aggregates together. They also bond the repair to the substrate. The choice of binder is perhaps the most difficult decision because the binder can control so many essential properties. Binders can be divided into two broad categories: inorganic and organic. The inorganic binders include lime and gypsum, hydraulic lime, and natural and artificial hydraulic cements. Organic binders primarily include epoxies and polyester resins and acrylic emulsion admixtures.

Gypsum and lime both cure by the mixing and loss of water. None of the sources for composite repair of sandstone mentions gypsum. Gypsum is slightly water soluble after curing and its used for composite repair is limited to

indoor sculpture (Griswold and Uricheck 1998). Lime is widely cited as a binder for composite repair. Lime is made by burning limestone, calcium carbonate, to form calcium oxide or quicklime. Water is added to the quicklime in a process known as slaking to form lime putty, calcium hydroxide. Lime putty cures by absorbing carbon dioxide from the air to form calcium carbonate (Ashurst and Dimes 1990; Hill and David 1995). Lime has good plasticity and a long set time making application easier. Lime is less strong than Portland cement, but this can be an advantage if the substrate is deteriorated and a stronger repair could damage the stone. One of the biggest drawbacks to lime is that it has a long set time and will not cure in the presence of water. Hydraulic limes - limes that set in the presence of water - are produced by burning limestone that contains alumina and silica clays. During cure, the alumina and silica react with the hydrated lime.[3] Hydraulic lime can also be produced by adding sources of alumina and silica to lime. These additives include volcanic ash, brick dust, and fly ash (Teutonico et al. 1994).

Portland cement was developed in the nineteenth century and has become a standard binder for masonry mortar. It is widely used in combination with lime as a binder for composite repair. Portland cement has a very fast set time and high strength. It has low water vapor transmission and water absorption that

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[3] See Lea (1971) for a discussion of hydraulic lime and Portland cement chemistry.

can limit its application to porous stone. Its high strength can damage the substrate.

Organic binders are an entirely different class of materials than limes and cements. Organic binders consist of an organic resin that is dissolved, heated, or mixed with a catalyst. The resin cures by solvent evaporation, cooling, or chemical reaction (Griswold and Uricheck). Organic binders are widely used for stone repair in fine arts conservation but organic binders are not commonly used for composite repair of sandstone. Only two sources (Ashurst and Dimes 1990; Sramek and Eckert 1986) recommend organic binders. Both authors recommend epoxy mixed with stone dust for repairs in wet environments. Epoxies are resins combined with a hardener, and they cure by cross-linking polymer chains. They are strong, resistant to many solvents, and have minimal shrinkage. Their disadvantages are that they can discolor when exposed to ultra-violet light, they are difficult to remove and their strength can exceed that of the stone (Ashurst and Dimes 1990; Griswold and Uricheck 1998).

Many different materials can be added to cement and lime to alter the properties. The most common are acrylic emulsions to increase bond strength

and reduce water permeability, air-entraining agents to decrease density and increase porosity, and coloring agents such as pigments and stone dust.

SAMPLE FORMULAE FOR COMPOSITE REPAIR

The technical literature on masonry repair recommends a wide range of materials and formulations for composite repair of sandstone; some are general suggestions for sandstone repair and some are specifications for application. The following is a summary of published recommendations.

- Weiss et al. (1982) – For general sandstone repair, white Portland cement, hydrated lime, sand, and crushed stone in a ratio of 1:1:3:3 or 1:1:2:4, with Acryl® 60 (an acrylic latex admixture discussed in detail in the chapter on testing).
- Sramek and Eckert (1986) – For a fountain constantly exposed to water, epoxy resin, powdered silica and sand (no ratio given).
- Iveson (1987) – For general sandstone repair, cement, and stone dust (no proportions) with a bonding agent added to the mix water.

- Weiss (1987) – For the conservation of Trinity Church in New York, Weiss recommended three different mixes each for a different stone color:
    1) White Portland cement, hydrated lime, sand, and pigments in a ratio of 2:1:8:1/2, for gray-brown sandstone
    2) Gray Portland cement, white Portland cement, hydrated lime, sand, and pigment in ratio of ½ : 1½: 1:8: $^5/_8$ for red/brown sandstone.
    3) Gray Portland cement, white Portland cement, hydrated lime, sand, and pigment in a ratio of ½ : 1½: 1:8:¼, for yellow sandstone
- Ashurst and Ashurst (1988) – For general sandstone repair, hydraulic lime, sharp sand, and soft sand in a ratio of 1:2:1.
- Ashurst and Dimes (1990) – For general sandstone repair – masonry cement and aggregate in a 1:7 ratio, or Portland cement and aggregate in 1:8 ratio with a plasticizer added.
- Ashurst and Dimes (1990) – For certain sandstone repairs, epoxy resin and aggregate in a ratio of 1:12 or 1:16.
- Hill and David (1995) - For general sandstone repair, white cement, lime putty and aggregate in a 1:3:10 ratio with stone dust added for color.

In addition to these custom formulas, commercial products have been developed. These products have the advantage of arriving premixed so batch to batch consistency is guaranteed - something that is difficult to achieve with

custom mixes. Three commercial stone repair products available in the United States are Jahn Patching Mortars, Conproco MIMIC and Edison System 45. All three are Portland cement based products. Conproco and Edison materials contain acrylic latex admixture. All three are available in many standard mixes to match different types of stone, and all three will custom match a stone on request.

CHAPTER TWO: TESTING PROGRAM

MATERIALS

The selection of composite repair materials for the testing program was based upon three main criteria: 1) the ingredient materials must be representative of materials recommended or used in other case studies and published sources, 2) the mixes must have enough similarities to each other to make comparison possible, and 3) the material components must be commercially available.

BINDER

Three different binders were included for testing:

1) Lime putty: Niagara® Mature Lime Putty, supplied by GenLime Group, L. P., Genoa, OH 43430. Niagara lime putty is a high purity dolomitic lime putty made from dolomitic limestone quarried in Northwestern Ohio. It is available slaked and screened and ready for use. It complies with ASTM C 5 and C 207 (GenLime Group, n.d.)

2) Hydrated hydraulic lime: Riverton Corporation, Front Royal, Virginia. Riverton hydrated hydraulic lime has high-moderate to moderate hydraulicity. It complies with ASTM C 141 Riverton n.d.).

3) White Portland Cement (Type 1): Riverton Corporation, Front Royal, Virginia. It complies with ASTM C 150.

Lime putty has been used as a binder in mortar for thousands of years (Draffin 1943) and is a common ingredient in composite repair (Ashurst and Ashurst 1988; Weiss et al. 1982; Weiss 1987).[4] The dry hydrated lime is more convenient to use, and it has virtually replaced lime putty as an ingredient in mortar. It is a generally held belief that lime putty made by slaking calcined limestone gives a mortar better working properties and superior performance once cured (Peroni et al. 1982; Hill and David 1995).

Lime putty is now available slaked and ready for use in the United States. The lime putty is shipped in five gallon plastic buckets. There is several inches of excess water on top of the putty to keep the lime from carbonating. When using the putty it is possible to incorporate more water into test batches resulting in inconsistent amounts of water in the mortar mix. To avoid this, sufficient lime putty for all the samples was removed from the bucket and strained through cheese cloth to remove excess water. This putty was placed in a separate smaller container and kept covered in plastic to ensure that the putty in all the samples had approximately the same water content.

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[4] The use and properties of lime as a binder are well documented in many sources and will not be discussed in detail here.

Hydrated hydraulic lime can best be described as combining certain properties of Portland cement and hydrated lime. It is obtained by calcining naturally occurring limestone that contains silica and alumina to form quicklime, CaO, that can be hydrated to form $Ca(OH)_2$, while leaving un-hydrated calcium silicates to give the powder hydraulic properties (ASTM C 145-85). According to the Riverton Corporation, The Riverton hydrated hydraulic lime comes from limestone at Front Royal, Virginia, and it contains primarily dicalcium silicates as the hydraulic component (Hartman, 1998). Hydraulic limes are not widely used in construction or conservation in the United States – they were replaced by Portland cements – and are only recently being considered as a material for conservation (English Heritage 1997). Hydraulic limes have been used as a grout for reattachment of lime plasters to earthen walls (Matero and Bass 1995), and some testing of hydraulic lime mortars has been undertaken by the Smeaton Project (Teutonico et al. 1995). Hydraulic lime is included here to compare its properties to those of lime putty mortars and Portland cement /lime mortars, and to see how it responds to the addition of acrylic polymer.

Portland cement has been a standard material for masonry since its development in the late nineteenth century (Draffin 1943)[5]. White Portland cement was chosen over ordinary gray Portland cement because the white is easier to color with pigments than gray Portland cement and it is more appropriate for a custom mortar that may require tinting (Weiss et al. 1982). White Portland cement is not as strong in compression as gray cement (ASTM C 150); compressive strength is not an important factor for the type of composite repair assumed for this study.

Organic resin binders were not included in this study because of their limited application in composite repair of sandstone, according to published sources. Organic binders require proportioning, mixing, and curing conditions very different from inorganic binders making it impossible to compare the test results between the samples.

SAND AGGREGATE

A commercially available sand aggregate, Schofield # 236, was chosen for all the mortar mixes. The choice was based on several factors. First, mortars made with Schofield #236 display an acceptable match in color and texture to

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[5] See Draffin (1943 and Lea (1971) for discussion of the history and properties of Portland cement.

the Portland brown sandstone, the model sandstone used for this study. Secondly, the sand is readily available, and it was one of the sands (then known as Schofield # 147) used in the *Sandstone Restoration Study* by the New York Landmarks Conservancy (Weiss et al. 1982). This study does not include the addition of masonry pigments to refine the color of the mortar.

According to the manufacturer, the sand meets the requirements of ASTM C 144-93*: Standard Specification for Aggregate for Masonry Mortar.* Prior to use, the particle size distribution of the sand was confirmed by sieving the sand with a mechanical sieve shaker according to ASTM C 136-92*: Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates.* The results of the sieve analysis are in Figure 1. The sand was found to meet the grading requirements of ASTM C 144.

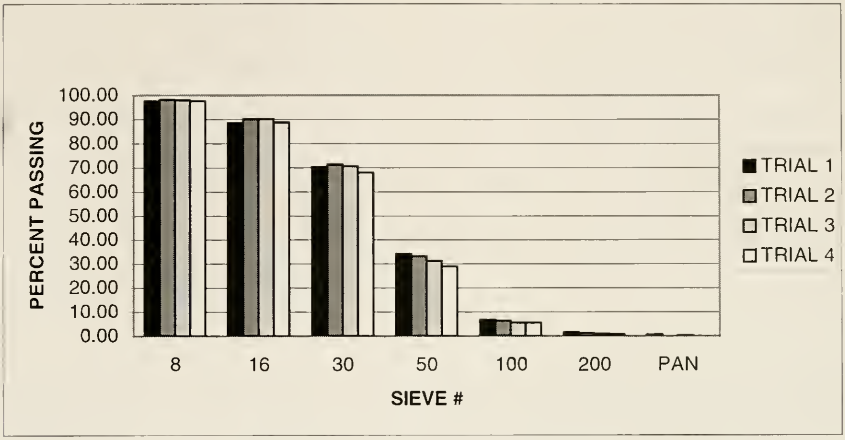

Figure 1 Sieve Analysis of Schofield #236 Sand

ACRYLIC ADMIXTURE

The admixture included in the testing program was Acryl® 60, an acrylic polymer emulsion manufactured by Harris Specialty Chemical for use as an internal admixture for Portland cement mortars, plasters, stuccos, and concrete mixes for improved adhesion, cohesion, compressive and flexural strength (Harris 1993). Acryl® 60 is a one-component, water based acrylic emulsion. It is a milky-white liquid slightly more viscous than water. It has a density of 1.04 kg/L and a solids content of 28% by volume (Harris 1993).

Acrylic polymer admixtures for concrete have been in use for almost 40 years (Lavelle 1986). The process of acrylic emulsions' reaction with Portland cement is documented in Ohama (1984). Essentially, the acrylic emulsion coalesces and forms a film over the cement hydrate particles. The film prevents micro-crack propagation in the hardened cement pates and results in a strong cement hydrate-aggregate bond.

Acrylic polymer admixtures such as Acryl® 60 are widely used for unit masonry and repair of stone because they increase bond strength between new and old work and dissimilar materials, and because they improve the overall durability of the work (Weiss et al. 1982). Acrylic polymer admixtures are added to the mixing water in ratios that vary according to application. The manufacturer of Acryl 60, Harris Specialty Chemical Inc. (1993), recommends a ratio of 1 part Acryl® 60 to 3 parts water. The *Sandstone Restoration Study* (Weiss et al. 1982) recommended a 1:5 ratio, based on experiments with ratios from 1:4 to 1:9. Higher concentrations of Acryl® 60 gave the composite repair an artificial appearance that was considered unacceptable (Weiss et al. 1982).

Acrylic latex emulsions are usually combined with the mix water, and in practice the amount of acrylic can vary according to the amount of water needed to give each mortar the right consistency. Since one of the aims of this study was to

measure the effects of the admixture, the same amount of acrylic emulsion was added to each mortar and the balance made up with plain water (see the section on ratio below). The amount of acrylic was constant, but the acrylic to water ratio varied slightly among the mortars, depending on the amount of total liquid (Acryl® 60 plus water) needed for the correct consistency. The ratio of Acryl® 60 to water was about 1:5 by volume.

JAHN M 70 #2

One commercial composite repair material, Jahn M 70 #2®, was chosen for comparison with the custom formulations. Jahn Restoration Mortars® are manufactured in the Netherlands by Jahn International and distributed in the United States by Cathedral Stone Products Inc. Jahn repair mortars are single component cementitious mortars mixed with water and applied according to the manufacturer's specifications. Cathedral Stone strictly controls the application of Jahn products by requiring that the installer complete a seminar on Jahn products before purchasing any of the repair materials. Jahn has a variety of materials to match the properties of the stone or concrete to be repaired. Jahn M70 #2 brownstone repair mortar is an off-the-shelf mix (Cathedral Stone will also custom match any stone or concrete) that approximately matches the middle color and texture range of Portland Connecticut brownstone (Cathedral Stone n.d.).

STONE

The brown sandstone from the Portland Brownstone Quarries, Portland, Connecticut was chosen for tests that required comparison of the properties of the repair material with those of the substrate stone. This sandstone stone was widely used as a building stone in the United States in the 18th, 19th and early 20th centuries (Bowles 1934; Gannet 1883), and recently the quarry was re-opened. [6] The large number of structures exhibiting deterioration make it is a logical choice for this study.

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[6] The Portland Brownstone Quarries, Cheshire, Connecticut. Michael Meehan, owner.

COMPOSITE REPAIR FORMULAE

All samples were formulated with a 1:3 binder to sand ratio by volume. This is a common proportion for masonry mortar (Hill and David 1995), and it has been recommended for composite repair in numerous sources (Ashurst and Ashurst 1988; Weiss et al. 1982; Weiss 1987). The hybrid binder of slaked lime and Portland cement employed proportions of 1:1:6 and 1:2:9 Portland cement, lime, sand. Mortars for unit masonry and composite repair are typically mixed by volume. Measuring the volume of relatively small amounts of dry material such as sand or Portland cement is not accurate enough for testing purposes. The density of each material was obtained either from the manufacturer or by measuring in the lab. The volumetric proportions were then converted to more easily and accurately measured weight proportions. For example, a 1:3 lime putty to sand mix is 220 ml of lime putty and 660 ml of sand. The material was measured as 288.2 g of lime putty and 1042.8 grams of sand. For each mortar formula, the amount of water needed to attain the desired consistency was determined, and that volume of water was constant for each sample of a given formula (the methodology for determining consistency is discussed in the section on testing).

The amount of Acryl® 60 needed for all the test samples was established by mixing Acryl® 60 and water in a 1:5 ratio and observing how was needed for the mortar formulation to reach the correct consistency. The amount of admixture mixture for sample C, 1:3 hydrated hydraulic lime and sand, was arbitrarily chosen as the basis for proportioning the Acryl® 60. Sample C required 169 ml of total liquid to reach the desired consistency, and a 1:5 ratio gives 28 ml of Acryl® 60. All the samples that contained Acryl® 60 were given 28 ml and the balance of the liquid was made up with plain water.

	PROPORTIONS				
SAMPLE DESIGNATION	White Portland Cement	Hydrated Hydraulic Lime	Lime Putty	Sand Schofield # 236	Acryl 60
A	-	-	1	3	-
B	-	-	1	3	~1:5
C	-	1	-	3	-
D	-	1	-	3	~1:5
E	1	-	1	6	-
F	1	-	1	6	~1:5
G	1	-	2	9	-
H	1	-	2	9	~1:5
I	Jahn M70 #2 mixed according to manufacturer's specifications				
S	Sandstone from the Portland Brownstone Quarries, Connecticut				

Figure 2 Description of Repair Mortars and Stone Tested

To ensure uniform mixing, all samples were mixed in a Hobart model C-100 mixer according to ASTM C 305-94: *Standard Practice for Mixing of Hydraulic Cement Pastes and Mortars.*

A suitable arrangement for curing the samples required some compromise because the different binders require different curing conditions. Portland cement is best cured in a moist environment for maximum hydration (ASTM C 109), while lime needs a drier environment with plenty of air circulation for carbonation to occur (Gillmore 1874; Hill and David 1992). The solution was to place the samples in a glass tank with the lid propped open slightly. The arrangement controlled the humidity and allowed air circulation; it also buffered the samples from rapid changes in temperature when the exterior door to the laboratory was opened. While the samples were curing in the tank the relative humidity and temperature were monitored daily. The relative humidity range was 62 – 80% with an average of 67.2%. The temperature ranged from 20 - 23º C, with an average of 22.3º C.

METHODOLOGY OF TESTING

The tests are divided into three general types: workability, compatibility and durability. Workability refers to the properties of the composite repair material in its plastic state and as it sets and cures. Tests include consistency, set time, and drying shrinkage. Compatibility tests measure the properties of the composite repair against those of the stone. These tests include water vapor transmission, water absorption capacity and linear thermal expansion.

Durability tests measure the cured composite repairs' ability to resist degradation from various sources in the building's environment. Tests include flexural strength, bond strength, frost resistance, salt crystallization resistance, abrasion resistance, and accelerated weathering.

CONSISTENCY

Consistency is related to the stiffness of the wet mortar. This is an important factor in composite repair mortar because the mortar must be plastic enough to be pushed into the area to be filled, but stiff enough to stay where it is applied, and long enough to set. There is no standard for stiffness of mortar; it varies according to application, and mortars are mixed to an appropriate stiffness based on the conditions. In this study, there are two reasons for measuring the consistency of the mortars: 1) to ensure that the mixes of the same sample are the same batch to batch, and 2) to observe how adding Acryl® 60 affects the consistency of the mortar.

The first step was to establish a target consistency based on simply observing how the mortars - mixed with varying amounts of water - behaved in the mixer, on the trowel, and in test repairs on stone. Once an appropriate consistency was established, the next step was to find the best way to quantify that consistency so that it could be repeated for each test.

ASTM has two general techniques for measuring the consistency of cement pastes and mortars. One is based upon measuring the mortar's resistance to penetration. ASTM C 187–85, *Standard Test Method for Normal Consistency of Hydraulic Cement* is designed to determine the amount of water required to prepare hydraulic cement pastes for testing. The test uses the Vicat apparatus shown in Figure 4, but instead of the 1 mm needle it uses the 10 mm diameter plunger. The consistency is measured by dropping the plunger from a certain height onto the surface of a mortar sample contained in a ring mold and measuring the depth of penetration. This test failed in this application. The test is apparently designed for very wet pastes and not the relatively stiff mortars required for composite repair. The plunger did not penetrate at all into the stiffer mortar mixes.

The second way to measure consistency is by measuring how much a sample of mortar spreads out when it is placed on a flat surface and the surface is raised and dropped a certain number of times through a specified height. The lab was not equipped with a flow table that complies with ASTM C 230-90 *Standard Specification for Flow Table for Use in Tests of Hydraulic Cement.* Therefore a device that approximates this flow table was assembled out of threaded black pipe, flanges, medium density fiberboard, plywood, and Plexiglas

(Figure 3). The top consists of a sheet of ¼ inch Plexiglas attached to a sheet of plywood. The shaft for the top is a 1 inch by 5 inch black pipe threaded on both ends and attached to the top with a threaded flange. The base is constructed similarly with a 1¼ inch by 5 inch black pipe shaft. The top shaft fits inside the larger diameter base shaft and the table top is raised and dropped by a handle attached to the pipe. The height through which the top can be lifted is controlled by a hose clamp attached to the pipe. The mold is the same mold used for the Vicat apparatus.[7] It has an inside diameter of 70 mm at the base and 60 mm at the top and a height of 40 mm.

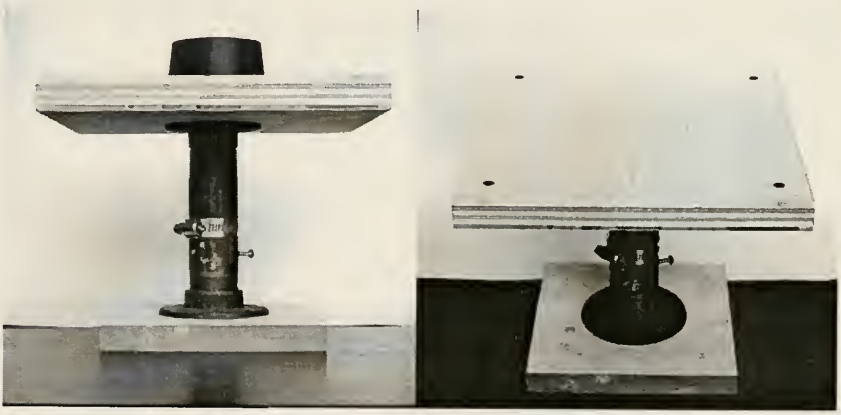

Figure 3 Flow Table

The testing procedure followed is from ASTM C 109-90 *Standard Test Method for Compressive Strength of Hydraulic Cement Mortars, Section 10.3*

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[7] Humboldt Manufacturing Company, part number H.3080.

Determination of Flow. The flow table is clamped to the bench and the mold is given a light coat of mineral oil as a release agent. Immediately after mixing the mold is placed in the center of the table and filled with mortar. With a trowel, the top of the mortar is cut off flush with the top of the mold, and the mold is lifted away. The table top is dropped through a height of 13 mm, 25 times in 15 seconds. A caliper is used to measure the base diameter of the mortar mass at four equally spaced axes. The flow is the average increase in base diameter, expressed as a percentage of the original base diameter - the inside diameter of the mold, 70 mm.

$$\text{FLOW} = \frac{(\text{AVERAGE BASE DIAMETER AFTER TEST}) - (\text{ORIGINAL BASE DIAMETER})}{(\text{ORIGINAL BASE DIAMETER})} \times 100$$

TIME OF SETTING

The time of setting is the rate at which the mortar goes from freshly mixed to final set - defined as when a 1 mm needle can no longer penetrate the surface of the mortar. The purpose of this test is to compare how different binders and different ratios of binders affect set time as well has how the addition of Acryl® 60 affects set time.

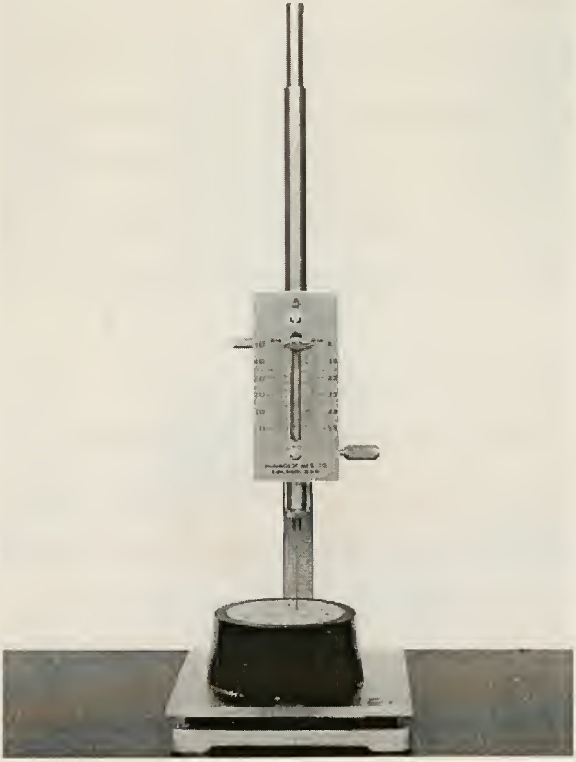

Figure 4 Vicat Apparatus with mortar sample

The time of setting of each sample was determined by ASTM C 191 - 92 *Standard Method for Time of Setting of Hydraulic Cement by Vicat Needle.* The test measures the depth of penetration of a 1 mm needle at different time intervals. The procedure for preparing the sample is as follows: immediately after mixing, take a ball of mortar and toss it six times between hands held 6 inches apart; throw the ball into the larger end of the ring mold that has been coated with mineral oil, completely filling the mold; swipe the excess mortar off the bottom of the mold and place the mold on a Plexiglas plate; level the top of the mortar with a single stroke of a trowel held perpendicular to the surface. To determine the time of setting, place the sample under the needle and lower the needle until the tip touches the surface, tighten the set screw, adjust the indicator to zero, release the set screw, and record the depth of penetration.

The procedure for measuring depth of penetration must be repeated at intervals until the penetration is zero. The spacing of the intervals will vary according to the type of mortar, and must be determined by pre-testing. For example, in this study, the lime putty mortars required over two days to reach full set while the high cement mortars needed only a few hours. At every interval three different readings were recorded on each sample and the value averaged. Three samples were tested for each mortar. Between readings, the samples were stored in the glass tank described above.

SHRINKAGE DURING CURING

Shrinkage during cure is a critical property of composite repair mortars. A repair that shrinks too much will crack, pull away from the edges of the stone, or, even worse, damage the stone. Either situation will allow water in the crack and accelerate decay of the repair.

ASTM C 531-95 *Linear Shrinkage and Coefficient of Thermal Expansion of Chemical-Resistant Mortars, Grouts, Monolithic Surfacings, and Polymer Concretes*, ASTM C 96-96 *Drying Shrinkage of Mortar Containing Hydraulic Cement*, and ASTM C 1148-92a *Measuring the Drying Shrinkage of Masonry Mortar* are all similar tests that use a mold to make test bars with studs imbedded in the ends allowing the bar to be accurately measured in a length comparator (Figure 5). The length comparator does not actually measure the length of the test bars. The comparator measures the test bars relative to standard steel reference bar, and the change in length of the test bars can be calculated at any time against the reference bar. The tests differ in how the samples are prepared and cured, but all follow the general procedure in ASTM C 490-93a *Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete.*

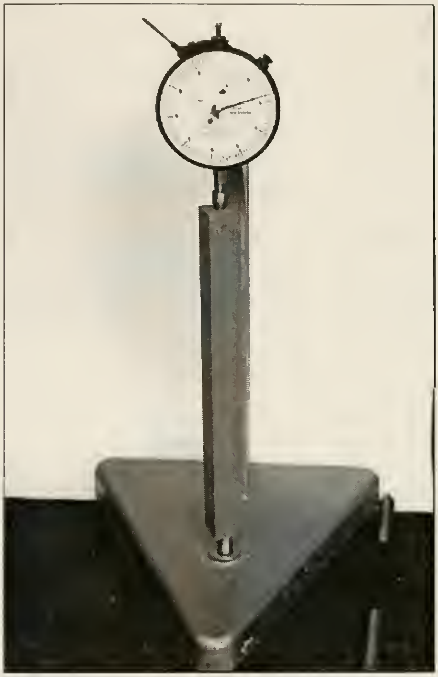

Figure 5 Length Comparator

The procedure for this study was based upon ASTM C 490 with modifications to accommodate the wide variety of mortars tested. The mold is assembled and coated with mineral with the gage studs held in place. The distance between the studs is the gage length and is a nominal 250 mm. The mold is filled in two pours with careful tamping after each pour. The top is struck off level with a trowel and the mold is placed in the tank and left for 48 hours. The long set

time was necessary for the lime putty sample to harden enough to be removed. The reference bar is placed in the comparator and the apparatus is adjusted until the dial indicator reads 0.2000 inches (the dial indicator has a range of 0.4000 inches). The test bar is then placed in the comparator and the reading of the dial indicator is recorded. This procedure is repeated every 24 hours for 28 days. Length change is calculated according to the formula:

$$L = \frac{(Lx - Li)}{G} \times 100$$

where:

L = change in length at x age
Lx = comparator reading of test bar minus comparator reading of reference bar at x age.
Li = comparator reading of test bar minus comparator reading of reference bar at 48 hours.
G = Gage length = 250 mm

THERMAL EXPANSION

There are several tests for thermal expansion applicable to stone and concrete including: ASTM D 5335-92 *Linear Coefficient of Thermal Expansion of Rock Using Bonded Electric Resistance Strain Gages*, ASTM E 228-95 *Linear Thermal Expansion of Solid Materials With a Vitreous Silica Dilatometer*, ASTM D 4535-85e1 *Measurement of Thermal Expansion of Rock Using a Dilatometer.* All of these tests require sophisticated equipment and all assume a very high temperature range that is not necessary for this study. There is no reason to exceed 60° C, the temperature experienced on the exterior of a building on a warm day (Clifton et al. 1975). RILEM Test No. VI.3 *Thermal Expansion* is applicable in that it measures thermal expansion over a temperature range of 10° to 50°C - a reasonable range for approximating conditions on a building. However, RILEM VI.3 requires equipment unavailable for this study.

ASTM C 531-95 *Linear Shrinkage and Coefficient of Thermal Expansion of Chemical-Resistant Mortars, Grouts, Monolithic Surfacings, and Polymer Concretes* was the only standard test method found designed for mortar. This test is relatively simple. The sample test bars used for shrinkage during cure are heated in an oven from ambient temperature up to the desired extreme temperature and periodically measured with a length comparator. ASTM C 531 differs from ASTM C 490 in that it requires that the distance between the gage

studs be measured directly with a 250 mm reference bar instead of assuming a nominal 250 mm gage length. The mold used in this study is not equipped with a 250 mm reference bar, so the coefficient of thermal expansion cannot be calculated exactly. It is possible to obtain relative measurements for thermal expansion and compare the different mortar samples to each other using the same formula for length change used for shrinkage during cure.

WATER VAPOR TRANSMISSION

The testing follows ASTM E 96-95 *Standard Methods for Water Vapor Transmission of Materials* Section 12. "Procedure for Water Method." A disk-shaped sample is sealed over the mouth of a cup containing water. The assembly is placed in a chamber with desiccant. As water vapor passes through the sample and is absorbed by the desiccant, the weight of the assembly decreases.

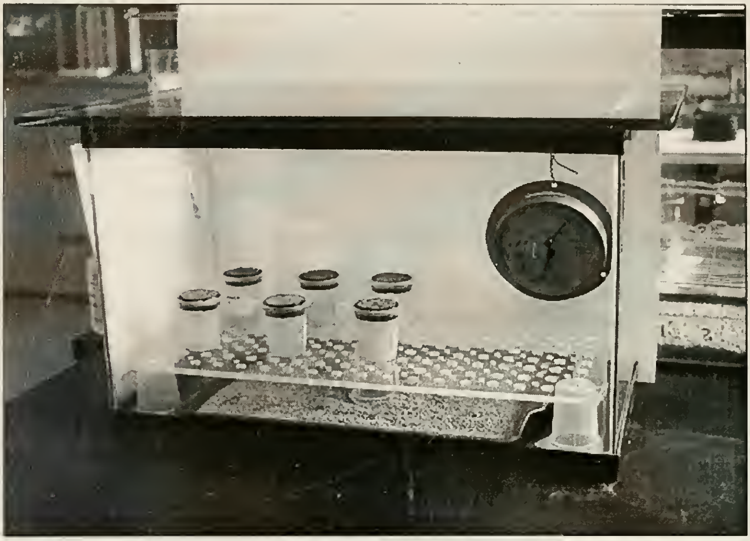

Figure 6 Samples for Water Vapor Transmission Test in the Controlled Chamber

Suitable samples of the stone were obtained by first cutting a cylinder of stone with a 7/8 inch diameter diamond masonry coring bit mounted in an electric drill. The cylinder was cut perpendicular to the bedding plane. The cylinder was then cut into disks 13.5 mm thick and 41 mm in diameter with a table saw equipped with a diamond masonry blade. These dimensions were based on another standard, NORMAL 21/85 *Water Vapor Permeability*, which requires that the diameter must be three times the thickness. The samples were rinsed with tap water and brushed with a stiff brush to remove any cutting dust, and then dried to a constant weight at 60°C.

Samples of the mortars were molded in rings cut from PVC pipe with a thickness of 13.5 mm and a diameter of 40.5 mm (the slight difference in size between the stone samples and the mortar samples is not relevant because the calculations for water vapor transmission and permeance take into account the dimensions of the samples). The molds were placed on small pieces of wood and filled with mortar in one pour. The tops were leveled with a single stroke of a trowel to minimize working the surface which could affect the permeability. The samples were placed in the tank and allowed to cure 48 hours in the molds before they were removed from the molds and placed on wire racks in the tank. The samples were allowed to cure 45 days, and then they were dried to a constant weight at 60°C.

The samples were wrapped with PVC electrical tape to help create a good seal and pressed into 50 ml polypropylene beakers containing about 20 ml of water and a small amount of cotton to prevent the water from touching the bottom of the sample when the assemblies are moved. The samples were sealed with melted paraffin, weighed, and placed in the controlled chamber. The assemblies were weighed every 24 hours (this interval was determined through pre-testing), and the temperature and relative humidity inside the chamber were

recorded prior to every weighing. The test was run until the change in weight became relatively constant.

The results were graphed and the water vapor transmission and permeance calculated.

WATER ABSORPTION BY TOTAL IMMERSION

Water absorption by total immersion, according to NORMAL 7/81 - *Water Absorption by Total Immersion - Imbibition Capacity* is the amount of water absorbed by a sample when immersed in de-ionized water at room temperature. It is expressed as a percentage of the dry weight of the sample. Imbibition capacity is the maximum amount of water absorbed by a material. A composite repair materials' ability to absorb water is important to its durability. A repair that has a much greater or lower water absorption than the surrounding stone can fail from water becoming trapped at the bond interface where it can weaken the adhesive bond by freezing expansion or by depositing soluble salts.

The samples are placed upright on glass rods in a container, and the container is filled with de-ionized water until the samples are covered by 2 cm of water. The samples are removed, blotted to remove excess water, and weighed in air at regular intervals established by pre-testing. The test runs until the weight

increase between two successive weightings is less than 1% of the weight of the sample, and at that point the test is ended. NORMAL 7/81 specifies that at this point the samples are dried to constant weight to determine the weight of the sample after the test. This value is to be used to calculate the imbibition capacity:

$$\text{Imbibition Capacity} = \frac{M_{MAX} - M_{AF}}{M_{AF}} \times 100$$

M_{max} = maximum weight of the sample at the end of the test

M_{AF} = weight of sample after drying to a constant weight at the end of the test

This step was omitted so the samples could be could be used in the frost resistance test (see below). Imbibition capacity was calculated using the value for M_0, the initial dry weight of the sample. The amount of water absorbed at each weighing interval "t" is calculated as a percentage of the initial weight of the dry sample:

$$\text{Amount of water absorbed} = {}^{\Delta M}/_{M}\ \% = \frac{M_i - M_0}{M_0} \times 100$$

M_i = weight of sample at t

M_0 = weight of dry sample at beginning of test.

The ${}^{\Delta M}/_{M}$ % is plotted on a graph as a function of time.

FLEXURAL STRENGTH AND MODULUS OF ELASTICITY

Flexural strength is a measure of a material's resistance to cracking under bending stress. Modulus of elasticity is a measure of the stiffness of a material or how much it deforms under load. Lower modulus materials have more capacity to absorb stress. It can be measured in compression or in flexure (bending). It is assumed in this study that the mortars are not expected to endure much compression stress so the modulus of elasticity was measured in flexure.

ASTM C 580-93 *Flexural Strength and Modulus of Elasticity of Chemical-Resistant Mortars, Grouts, Monolithic Surfacings, and Polymer Concretes* is the only standard for both flexural strength and modulus of elasticity related to mortar. In general, a rectangular prism is supported at both ends, and a load is applied at the center until the beam cracks. The load on the sample and its deformation or displacement are simultaneously recorded and graphed. Some modifications to the test were made. In the interest of efficiency, the mortar samples left over from the bond strength test were cut into rectangular prisms 57 mm long by 19 mm by 19 mm. This size does not meet the length requirements of the standard. Also, the span between the supports was reduced from 3 times the depth to 2 times the depth of the beam.

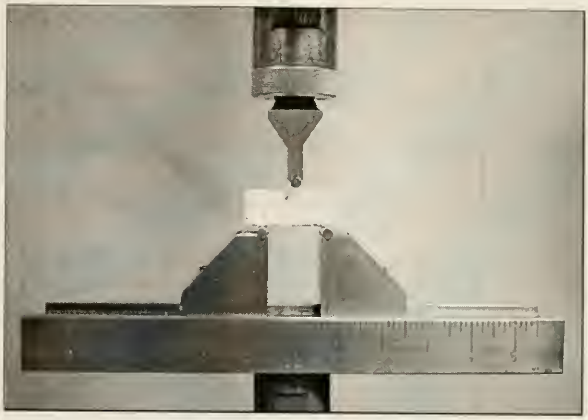

Figure 7 Three-Point Bending Test

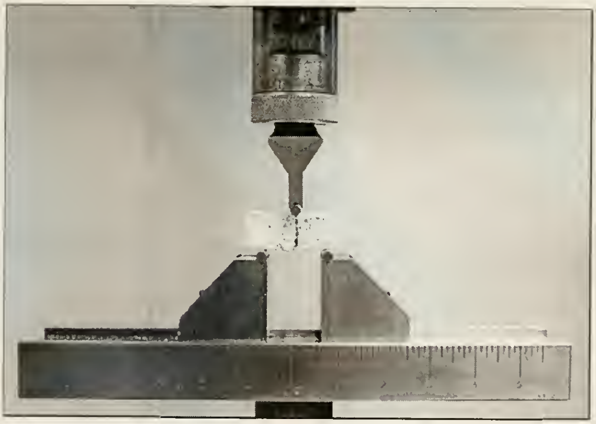

Figure 8 Three-Point Bending Test, after failure

An Instron Testing Machine Model 1331 was used in a strobe control ramp test. The cross head speed was 0.1 inches per minute. Three replicates were tested for each sample, and the flexural strength and modulus of elasticity calculated for each sample and the results averaged

BOND STRENGTH

ASTM standards for bond strength of masonry unit are not applicable because they assume unit size equivalent to a standard brick or cement block (C952-91 *Bond Strength of Mortar to Masonry Units*; E518-80(1993)*e1 Flexural Bond Strength of Masonry*). They require making prisms of multiple units and the testing apparatus is too large for the stone samples available. Tests for bond strength of cement or concrete are not applicable because they require samples impossible to replicate in solid stone.

A test for bond strength of adhesives was modified. ASTM D 905-89 *Standard Test Method for Strength Properties of Adhesive Bonds in Shear By Compression Loading* is intended primarily as an evaluation of adhesives for wood. The test specimen consists of two blocks 2" by 1 ¾" bonded with a ¼" offset. In this study, a block of stone has a quantity of wet mortar applied to it and allowed to cure. The stone is 50 mm by 57 mm block 19 mm thick with all faces sawn. One face is scored to a depth of 2 mm in a grid pattern to provide a mechanical key for the mortar. This is in accordance with recommended procedures for composite repair (Ashurst and Ashurst 1988; Ashurst and Dimes 1990; Edison 1991). The stone is put in a wooden mold constructed for this purpose and a 19 mm thick layer of mortar applied so that the mortar offsets the stone 7 mm (Figure 9).

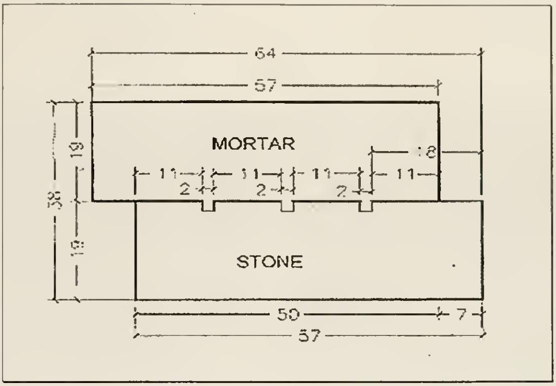

Figure 9 Drawing of Assembly for Bond Strength Test

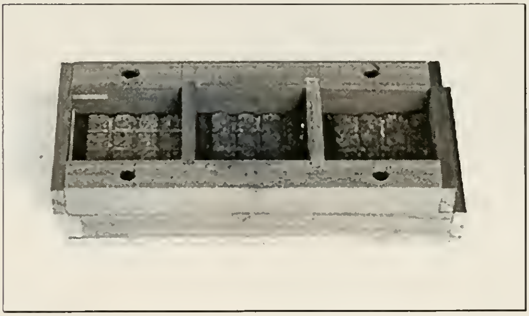

Figure 10 Mold for Bond Strength Assemblies. The prepared stone samples are in the bottom and the top is ready to be filled with mortar.

After curing, the sample is put in a testing machine that holds one half on the assembly and applies load to the other half. The bond strength is defined as the shear stress at failure in psi based on the bond area. The score lines are not taken into account in calculating the bond area thus the bond area is 2500 mm.

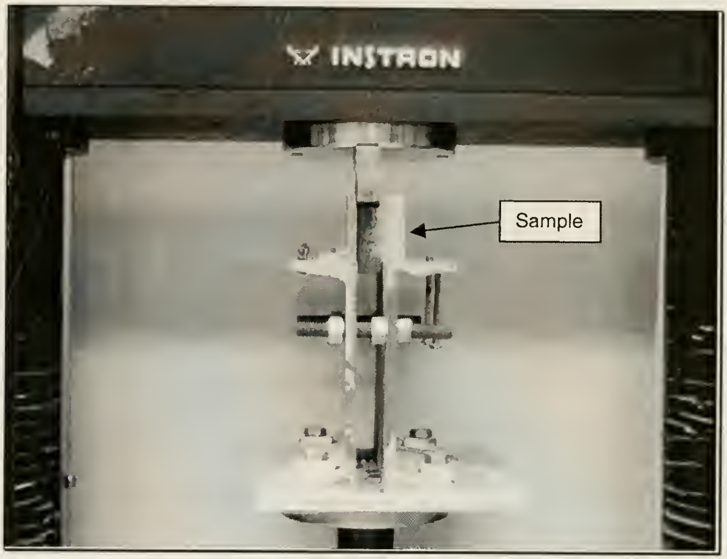

Figure 11 Bond Strength Test, with custom support assembly and sample in place

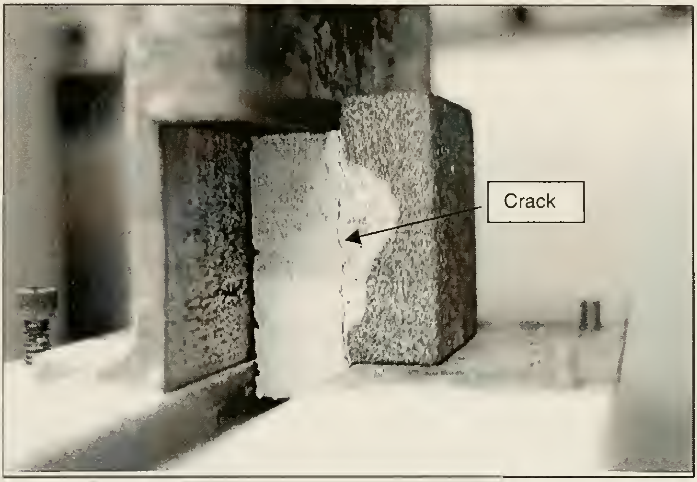

Figure 12 Close-up of Sample after Bond Strength Test. Note the hairline crack at the bond line

FROST RESISTANCE

This test measures the samples' resistance to decay due to cycles of freezing and thawing. RILEM V.3 *Frost Resistance* is designed for building stone, and was applied to the mortar samples with minor modification.

Wooden molds were constructed that can be disassembled without disturbing the sample. The mold were made of poplar and coated with mineral oil. The molds were filled according to the procedure for filling 50 mm cube molds in ASTM C 109-90 *Standard Test Method for Compressive Strength of Hydraulic Cement Mortars* (Section 10.4.4), and allowed to cure in the tank chamber for 48 hours before un-molding. The samples were then cured on wire racks in the tank for 60 days. The cubes were cut in half with one half of the samples for the frost resistance test and the other half for the salt crystallization test.

The basic procedure for the frost resistance test is as follows: the samples are immersed in water for 6 hours (the water absorption capacity was measured at this time, see above), then frozen in air at -15°C for at least 6 hours, and then thawed in water for 6 hours, then frozen again. At the start of the test and after every three cycles, the bulk volume of the samples is measured by hydrostatic weighing - weighing the sample suspended from a wire in water (Figure 13) - and subtracting that from the weight of the sample in air, after excess water is

blotted from the sample. Resistance to freeze-thaw decay is determined by the change in bulk volume expressed as a percentage of the original bulk volume.

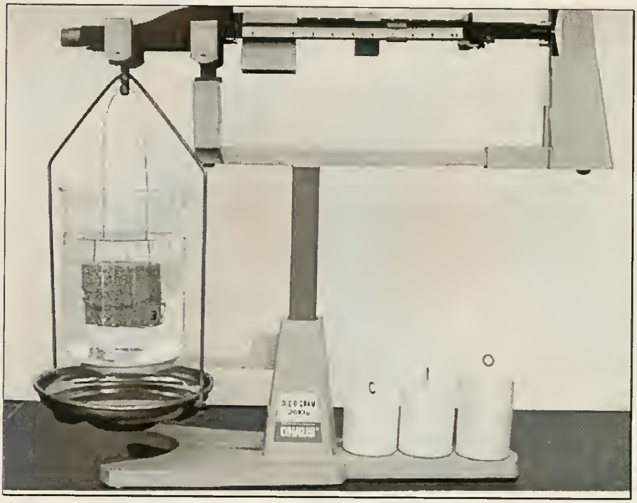

Figure 13 Hydrostatic Weighing

Three replicates of each mortar mix were tested. The samples measured 50 mm long by 23 mm wide by 50 mm high. The samples were placed in containers supported on glass rods to ensure water and air circulation around the samples. The samples were immersed in water for 6 hours. The freeze-thaw cycle consisted of 18 hours freezing in air at -15°C followed by 6 hours thaw in water. The test deviated from the standard in one respect. RILEM V.3 specifies that the thawing water be maintained at 5°C ± 2°C. Lab temperature

tap water was used for the thaw cycle. After thawing the water was evacuated with a siphon to avoid disturbing the samples. The test was run for 15 cycles

SALT CRYSTALLIZATION

This test measures the samples' resistance to decay by salt crystallization. RILEM V.2 *Crystallization by Partial Immersion* is designed to assess the damage caused by salts to treated stone versus untreated stone. This is not unlike comparing the effects of salt on mortar samples with and without admixture, or with different binders.

The test is based on the destructive effect of the volume increase of sodium sulfate as it re-crystallizes from Na_2SO_4 to $Na_2SO_4 \bullet 10H_20$ (sodium sulfate decahydrate) at 32.7°C. The procedure as specified is simple. 50 mm sample cubes are dried to a constant weight at 60°C, and placed in 1 cm deep 10 % $Na_2SO_4 \bullet 10H_20$ for 2 hours, dried at 60° C for 19 hours, cooled for 3 hours and weighed before being replaced in fresh salt solution. The test continues for 15 cycles or less if the sample disintegrates. At the end of the test the samples are immersed for 7 days in often renewed tap water to remove the salt from the samples, and the samples are dried to constant weight to measure how much material has been lost.

Some modifications were made to the test. The sample size was 50 mm long by 21 mm wide by 50 mm high. The samples were soaked for 1 hour instead of 2. This shorter soaking period was chosen because, during pre-testing, the samples absorbed the solution very readily and the level of capillary rise reached the top of some of the samples. This condition made it difficult to observe the decay of the sample at the top of the capillary rise where most decay would occur. With a one hour soaking period the level of capillary rise did not reach the top of the samples. The drying period was extended to 20 hours to maintain a convenient 24 hour cycle.

ACCELERATED WEATHERING

The purpose of this test was to evaluate the materials' resistance to ultraviolet light degradation and wet/dry cycling, particularly the samples containing Acryl 60, because some acrylic polymers are known to discolor in UV light (Ohama 1984).

The same samples that had been used for the water vapor transmission test were used for the accelerated weathering test. Two samples of each mortar were tested with one of each identified as a control. The samples were attached with coated wire to 75 by 300 mm sample holders – two samples to each holder

– and placed in a QUV/SE Accelerated Weathering Tester (manufactured by the Q-Panel Company) fitted with UV-A 340 lamps[8]. The QUV uses fluorescent UV lamps to simulate sunlight. The effects of atmospheric moisture are simulated by water vapor – produced by heating water in a pan inside the testing chamber – condensing on the samples. The procedure followed was that of ASTM G 53: *Standard Practice for operating Light- and Water Exposure Apparatus (Fluorescent UV-Condensation Type) for Exposure of Nonmetallic Materials*. The test conditions were: 4 hours UV at 60º C, 4 hours condensation at 50º C. The irradiance level was set at 0.77 W/m^2/nm (the manufacturer's recommended setting for this lamp type and temperature). The samples were exposed for 360 hours (45 cycles of UV and condensation) and rotated according to ASTM G 53; the extreme left and right samples were rotated to the center every day, and half way through the test the samples were rotated vertically.

At the end of the test the samples were compared to the unexposed controls for color change, texture change, friability, and loss of material. Loss of material was evaluated by weighing the samples before and after the test (after drying to a constant weight). Color change was evaluated visually without the aid of

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[8] UV-A lamps produce an energy spectrum from 315 to 400 nanometers with a peak emission of 340 nm.

standards.[9] Change in texture and friability was evaluated by touch, which is an effective method to compare the roughness of samples when a suitable instrument is not available (Ashurst, N. 1994).

ABRASION RESISTANCE

Abrasion resistance is a measure of the grain-to-grain cohesive strength of the mortar. Testing can predict how a mortar will withstand various abrasive conditions in the environment - wind, rain, human contact, and testing can serve as a general comparative indicator of overall durability.

The procedure for testing abrasion resistance is one used in the Architectural Conservation Laboratory at the University of Pennsylvania (Kopelson 1996). It is loosely based on ASTM C 418 *Standard Test Method for Abrasion Resistance of Concrete by Sandblasting.*

In general, a sample is blasted with abrasive powder at sufficient pressure and distance to create a loss. The amount of material lost is indirectly measured by weighing the amount of fine aggregate needed to fill the loss area.

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[9] More accurate and repeatable results for color change analysis are possible with the aid of a chroma meter or spectrophotometer (Young 1993).

The samples used were the same samples used in the bond strength test. The samples were cured for 45 days and the test was carried out on the face of the sample not in contact with the stone or the mold. Three replicates were tested for each mortar and three for the stone. The size of the samples was 57 mm long by 30 mm wide by 19 mm thick.

The equipment used was an S. S. White Model K Airbrasive Jet Machine Unit with a 1 mm diameter tip abrasive pen, an Englo air compressor, an S. S. White Airbrasive Chamber with a wooden jig to hold the sample and pen at the correct distance, and 50 micron aluminum oxide abrasive powder.

The procedure was to blast each pre-weighed sample for 2 minutes at a distance of 1 cm at 75 psi. This gave a hole that could be filled without destroying the sample.

Each sample was cleaned by blasting with air at 40 psi to remove any abrasive powder and weighed. The loss area was carefully filled with 250 micron sand and the sample weighed again. The amount of loss is the weight of sand required to fill the loss.

CHAPTER THREE: RESULTS AND CONCLUSIONS

CONSISTENCY

The results of the consistency test are presented below. All of the custom mixes displayed similar flow (average 30.20 %). The only exception is sample I, the Jahn M70. This is because the Jahn material was mixed according to the manufacturer's specifications to achieve optimal working properties. If mixed to the same flow as the other samples, the Jahn material is much too wet to work properly, most likely because it has much smaller aggregate.

Sample	diameter of mortar after test (mm)					
	1	2	3	4	Average	**FLOW**
A	89.55	88.30	87.00	90.75	88.90	**27.00%**
B	90.35	91.50	92.70	91.05	91.40	**30.57%**
C	90.80	88.85	89.25	89.50	89.60	**28.00%**
D	91.70	92.85	91.00	91.90	91.86	**31.23%**
E	91.90	94.50	92.80	93.25	93.11	**33.02%**
F	91.65	92.80	91.05	92.00	91.88	**31.25%**
G	92.70	92.50	90.45	91.85	91.88	**31.25%**
H	90.60	90.95	89.85	90.66	90.52	**29.31%**
I	83.35	82.90	83.45	84.35	83.51	**19.30%**

Figure 14 Results of Flow Test

The flow test made it possible to assess the effect of the Acryl® 60 on the consistency of the mortar, by comparing the total amount of liquid required to achieve target flow between mortars with the same components and proportions with and without Acryl 60.

Sample	total liquid (water + Acryl 60) ml	percent difference
A	100	
B	85	15.0%
C	169	
D	143	15.4%
E	130	
F	108	16.9%
G	113	
H	98	13.3%
I	138	N/A

Figure 15 Effect of Acryl® 60 on Water Content

Comparing two sample sets with same components and proportions - one without and one with Acryl® 60 - revealed interesting data. In every case, Acryl® 60 reduced the total amount of liquid required to achieved target flow by an average of 15.1%. Ohama (1984) noticed similar results with acrylic polymers in concretes and Portland cement mortars. He attributed it to the lubricating properties of the acrylic emulsions. The colloidal particles in the acrylic emulsion act like ball bearings to increase the lubricity of the mortar. This observation is significant because the amount of water added to a mortar is related to the amount of shrinkage the mortar will undergo as it dries – adding more water during mixing results in more potential shrinkage during cure (Hill and David 1995). Since the addition of Acryl® 60 reduces the total amount of liquid needed to achieve optimal consistency, Acryl® 60 could, in effect, decrease the shrinkage during cure.

TIME OF SETTING

The results of the time of setting test are in Figure 16.

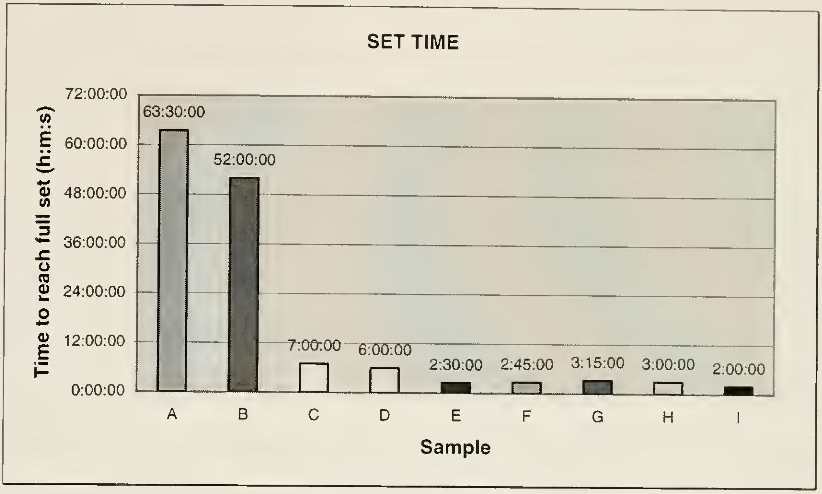

Figure 16 Average Set Time

The lime putty mortars (samples A and B) had the longest set time, as expected. The hydraulic lime (samples C and D) had the next longest set time, and the cement/lime mortars and the Jahn are all closely grouped with the shortest set times. The 1:1:6 cement, lime, sand mortars (samples E and F) had shorter set times than the 1:2:9 cement lime sand mortars (samples G and H) because the 1:1:6 mortars had a higher proportion of Portland cement. The short set time of the Jahn material (sample I) is attributed to its high Portland

cement content and the low amount of water needed to achieve optimal consistency. The very long set time for lime mortar could be a problem in some applications because mortar could slump before it has time to set, and the repair would need to be protected from inclement weather during set time. A shorter set time is an advantage if the surface of the repair is to be tooled, etched, or left unprotected.

The effects of Acryl® 60 are inconsistent. It was expected that the addition of Acryl® 60 would decrease set time because less total liquid is needed for mortars containing Acryl 60. Adding Acryl® 60 to the mortar caused a 22% decrease in set time for lime mortar and a 16% decrease for hydraulic lime. Acryl® 60 had an insignificant effect on the cement/lime mortars (It *increased* the set time for the 1:1:6 mortar). It is possible that the acrylic polymer emulsion retained moisture in the mortar and thereby increased the set time.

A source of error for this test is the large aggregate in the mortars. Some of the aggregate is substantially bigger than the 1 mm needle on the Vicat apparatus, and if the needle struck a large aggregate near the surface it could not penetrate as far. A possible alternative test is to modify the Vicat apparatus with a larger diameter needle.

SHRINKAGE DURING CURE AND THERMAL EXPANSION

These tests were eliminated from the program because of difficulty with the specimens. The test requires daily handling of the specimens and they are long, thin and quite fragile in the early stages of cure. Many of the samples began to break after a few days of cure. The breakage did not appear to be due to any flaws in molding - there were no gaps or voids visible. The most likely explanation is that the specimens were jarred too much when they were removed from the mold after 48 hours of cure. The jarring introduced micro-cracks which increased during cure and handling until the sample broke This conclusion is based on the observation that the cement mortars - which should have been more durable - were the first to break. They also required the most force to remove from the mold. There is a device for de-molding specimens described in ASTM C 157-93 *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*. This device allows the mortar to be removed with minimal jarring.

Too few samples survived for any conclusions to be drawn, except that - if the de-molding problem can be solved - the test appears to be a good method for comparing shrinkage and thermal expansion.

WATER VAPOR TRANSMISSION AND WATER ABSORPTION

Sample	Average Water Vapor Transmission (g/m m^2)	Average Permeance (g/Pa s m^2)	Absorption Capacity (% dry weight)
A	9.22	2.09E-06	9.76
B	9.37	2.12E-06	15.23
C	8.14	1.85E-06	9.45
D	7.11	1.61E-06	7.86
E	4.85	1.10E-06	8.84
F	5.88	1.33E-06	9.77
G	5.75	1.30E-06	10.54
H	6.78	1.54E-06	10.12
I	4.89	1.11E-06	9.93
S	1.17	2.64E-07	N/A

Figure 17 Water Vapor Transmission and Absorption Capacity

Figure 17 shows the values for water vapor transmission (WVT) and permeance (calculated according to formulae in ASTM E 96) along with the values for absorption capacity. The most significant observation is that all mortar samples have a WVT and permeance greater than that of the stone (sample S). The Jahn sample (sample I) had the lowest WVT but it was still well above that of the stone. None of these composite repair materials runs the risk of trapping water vapor at the interface and thereby accelerating decay. It must be noted that the test was carried out on fresh un-weathered stone. Weathered stone could have a significantly different WVT and permeance.

A surprising observation was the effect of Acryl 60. Acryl® 60 should have decreased WVT and permeance according to reported tests (Harris 1993; Lavelle 1983; Ohama 1984), especially in the cement mortars. In all cases the Acryl® 60 had minimal effect, and in all but the hydraulic lime mortar, the Acryl® 60 *increased* WVT and permeance. A possible explanation for this is that there was not enough Acryl® 60 to form a continuous film in the matrix and thereby significantly change WVT and permeance, and the air entraining property of the acrylic polymer emulsion discussed by Ohama (1984) and Ramakrishnan (1992) could have created a more porous material with greater WVT and permeance.

The results of the water absorption test support this hypothesis. The results are given in Figures 18 and 19. Figure 18 shows the rate of water absorption and Figure 19 shows the total absorption of imbibition capacity. As shown in Figure 17, in most cases there is a correlation between increase in WVT and permeance and an increase in absorption capacity when Acryl® 60 is added to the mix.

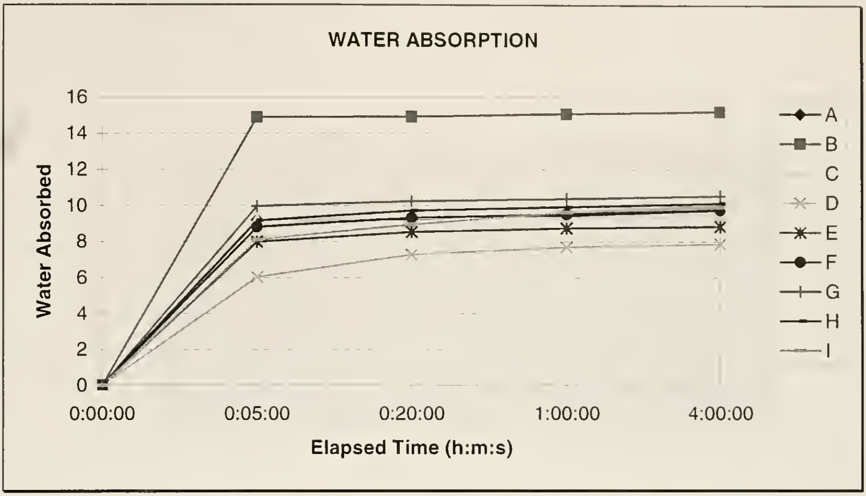

Figure 18 Water Absorption Curves

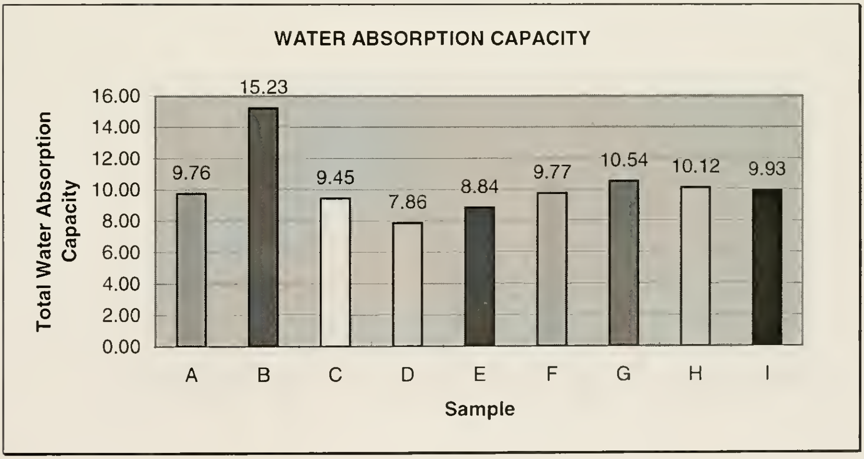

Figure 19 Water Absorption Capacity

FLEXURAL STRENGTH AND MODULUS OF ELASTICITY

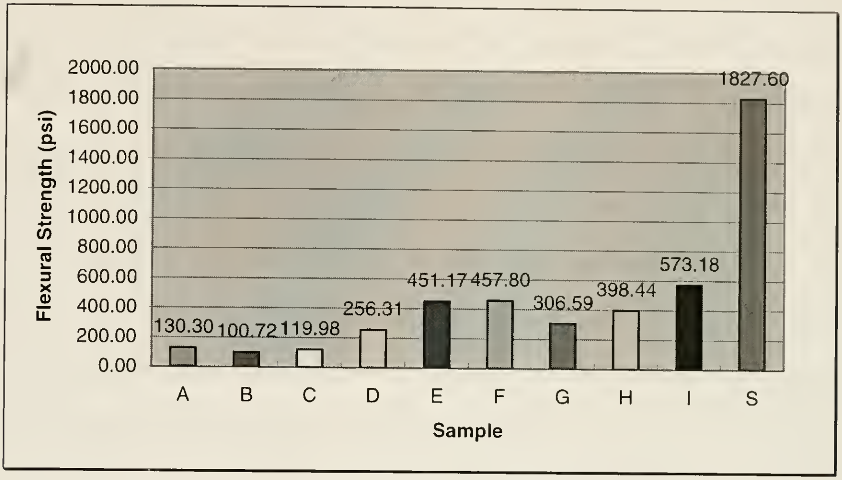

Figure 20 Flexural Strength

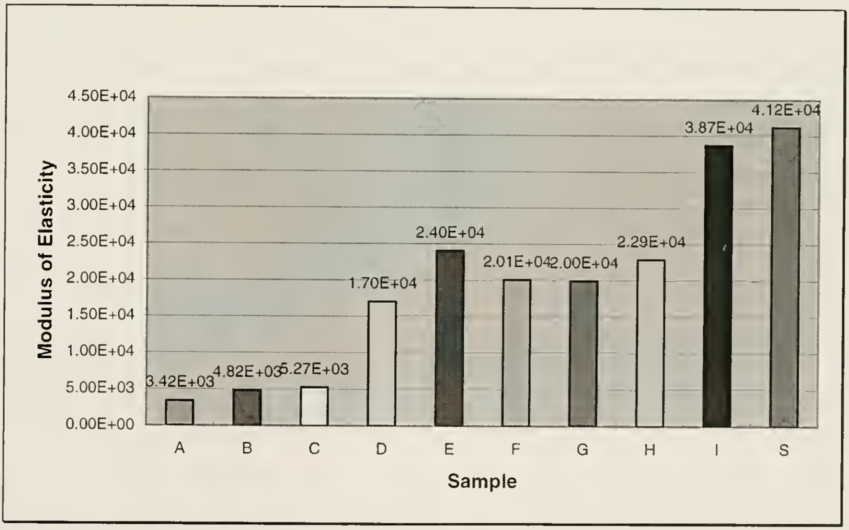

Figure 21 Modulus of Elasticity

The results for flexural strength - the material's resistance to cracking under load - are as expected. The stone is by far the strongest. The lime mortar is weakest, and the mortars with the most cement are strongest. It was expected that the addition of Acryl® 60 would increase the flexural strength of the mortars (Ohama 1984 Lavelle 1986). This held true for all the mortars except the lime putty, where the addition of Acryl® 60 decreased the flexural strength. The values for modulus of elasticity are more ambiguous. The Acryl® 60 dramatically increased the modulus of elasticity for the hydrated hydraulic lime, while having insignificant effect on the other mixes.

Modulus of elasticity is a measure of the stiffness of the material. In a composite repair it is desirable to have a repair with a lower modulus than that of the stone so that stress caused by expansion will be absorbed by the repair and not cause the repair to crack or pop out.[10] The ideal composite repair would have high flexural strength and low modulus of elasticity. All the composites have a modulus less than that of the stone. The lime mortars had the lowest modulus. Adding Acryl® 60 had inconsistent effect. As with the flexural strength, Acryl® 60 raised the modulus for the hydraulic lime mortar

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[10] This is assuming a shallow repair - less than 2 inches. With deeper repairs, where the material is expected to bear more of the weight of the masonry above, the repair should have a modulus of elasticity – measured in compression instead of bending - compatible with that of the stone (Harris 1998)

considerably but had less pronounced effect on the other formulations. Due to the long cure time of lime, the lime based formulations had not reached their full strength yet.

One problem identified with this test is that the surface of the sample in contact with the supports or the loading nose was in some cases friable. This condition altered the initial portion of the load deflection curve as the loading nose crushed the surface slightly instead of bending the sample. It was problematic because the slope of the curve is needed to calculate the modulus of elasticity (the equation is given in Appendix A). The friable surface probably was a result of cutting samples from larger blocks of mortar instead of molding the samples the appropriate size and shape. The sides of the sample that were in contact with the mold are smoother and less friable than a sawn surface. The problem could be solved by preparing the samples so that that all faces are smooth.

The results for the stone could be misleading because the samples were relatively slender. With such a large-grained stone, larger samples of the same stone could yield much greater flexural strength and modulus of elasticity.

BOND STRENGTH

The results of the bond strength test are difficult to assess because there is no standard for making and testing the assemblies used in the test. The evaluation of whether the test is an effective test for bond strength of composite repair material to stone is as important as the evaluation of the results.

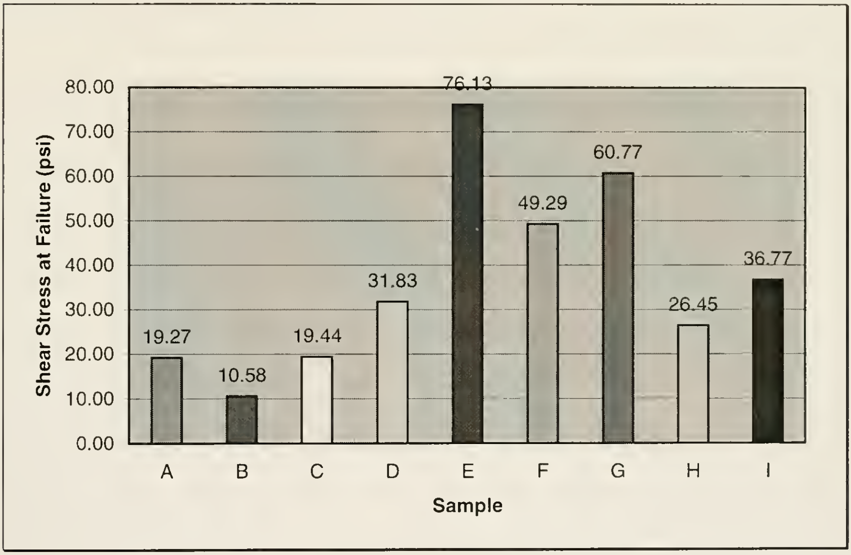

Figure 22 Graph of Average Values for Bond Strength in Shear

Examination of the samples after the test (photos are in Appendix C) show that all samples had good coverage indicating that the mortar was sufficiently tamped into the molds. None of stone was damaged during the test. This

shows that none of the mortars had a bond strength so great that the stone broke before the bond. All of the samples broke at the bond line with very little mortar left on the surface of the stone. The only exception was the Jahn samples (sample I) which left, on average, 10% of the stone surface covered with mortar. An interesting observation is that the samples with Acryl® 60 left less mortar stuck in the grooves of the stone. This suggests that the Acryl® 60 increases the cohesive strength of the mortar.

The most significant observation is that Acryl® actually decreased the bond strength in every sample except the hydrated hydraulic lime The Acryl® 60 was *not* applied to the stone surface prior to applying the mortar as the manufacturer specifies (Harris Chemical 1993). Every source on polymer modified mortars indicates that acrylic emulsions increase bond strength. Apparently this is true only if used as a surface treatment as well as an admixture. There is no good explanation of why the Acryl® 60 would decrease bond strength. It could be related to the presence of pure lime putty in the mortar - only the hydrated hydraulic lime sample did not have any lime putty. Another possible explanation is that mortars with the Acryl® 60 had more entrained air and therefore less cementing material in contact with the stone.

FROST RESISTANCE

The results of the frost resistance test are expressed in Figure 23 as the amount of material - the bulk volume - retained after every three cycles. Figure 24 compares each sample after 15 cycles. Sample A and B are not shown because the percent retained was zero. The highest percent bulk volume retained corresponds to the highest resistance to freeze-thaw decay.

	% BULK VOLUME RETAINED					
Sample	Start	3 Cycles	6 Cycles	9 Cycles	12 Cycles	15 Cycles
A	100.00	98.00	58.45	26.98	0.00	0.00
B	100.00	83.66	22.80	4.86	0.00	0.00
C	100.00	99.37	99.34	98.98	98.49	97.79
D	100.00	99.72	99.05	98.94	97.54	94.48
E	100.00	100.17	100.27	101.11	99.79	99.89
F	100.00	100.12	99.97	100.50	100.20	99.79
G	100.00	98.11	98.22	98.38	98.46	98.16
H	100.00	99.70	99.87	100.12	99.76	99.49
I	100.00	99.80	99.92	100.22	99.60	99.34

Figure 23 Frost Resistance

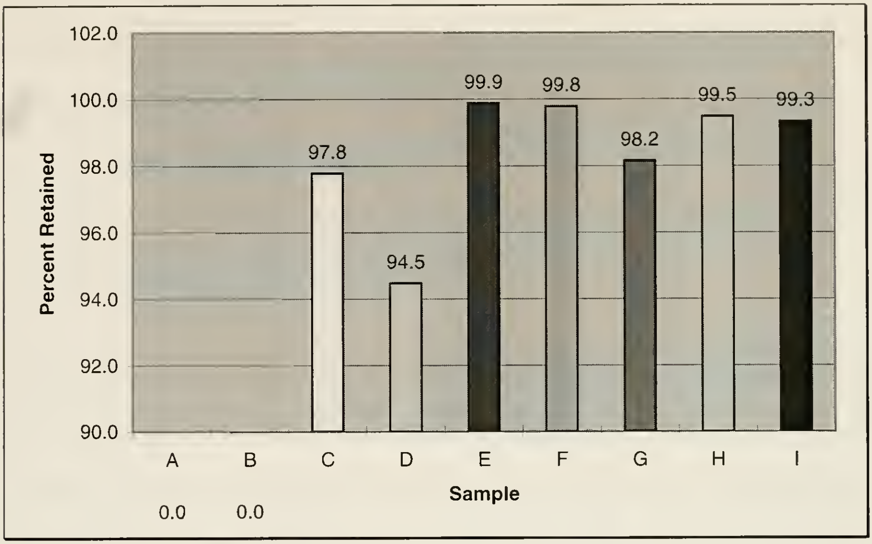

Figure 24 Frost Resistance - percent bulk volume retained after 15 cycles

The most durable mortars were the mortars containing Portland cement. The lime putty mortars (samples A and B) disintegrated before the end of the test. There is no apparent pattern to the effect of Acryl® 60. Adding Acryl® 60 dramatically decreased frost resistance in lime mortar (sample B, lime putty with Acryl® 60, disintegrated more quickly that sample A without Acryl® 60). Acryl® 60 slightly decreased frost resistance in hydrated hydraulic lime mortar and had insignificant effect on the other samples. Nothing in the literature suggests anything to explain why Acryl® 60 would decrease the frost resistance of mortar. In the lime mortars (samples A and B) Acryl® 60 increased the absorption

capacity (Figure 17); this could explain why the lime mortar with Acryl® 60 (Sample A) had lower frost resistance that the lime mortar without Acryl® 60 (Sample A). Possibly the constant exposure to water - in this test the samples are never allowed to dry - caused the coalesced polymer film to swell and induced more internal stress in the mortar samples in addition to the expansion from freezing.

There are no standards for evaluating the results. The test is very aggressive in that the samples are fully saturated before freezing - a condition that rarely occurs in practice. A 15 cycle test duration does not appear to be sufficient to test the most durable samples. Pure lime mortar should not be used in an environment subject to heavy rain and rapid freezing but could be appropriate in more sheltered areas. The other samples demonstrated acceptable durability.

SALT CRYSTALLIZATION

The test chosen is not fully applicable to mortars. The test was designed for stone which is generally less porous and absorptive than mortar. As the mortars absorbed the solution and salt crystallization occurred, the mortars gained weight. The soaking period at the end of the test did not remove all of the salt, so even visibly decayed samples weight more at the end of the test than at the beginning.

The mortars were evaluated by visual examination only. Each sample was ranked on a scale of one to ten, with ten indicating the best resistance to decay. The results are given in Figure 24.

Sample	Salt Decay Rating
A	4
B	5
C	7
D	9
E	10
F	10
G	7
H	9
I	10

Figure 25 Salt Crystallization

Only general conclusions can be drawn. The pure lime sample (sample A) was the worst performer as expected, since lime mortars are generally known to

have less cohesive strength than Portland cement mortars (Hill and David 1992). Lime also takes a long time to cure fully and its durability could increase with longer curing time. Comparing mortars with 1:1:6 cement, lime, sand ratio (samples E and F) with mortars with 1:2:9 cement lime sand ratio indicates that higher lime content in cement/lime mortar decreased resistance to salt attack.

Acryl® 60 improved the performance of every mortar, probably by increasing the grain to grain cohesive strength as demonstrated in the abrasion resistance test.

ACCELERATED WEATHERING

The samples in the accelerated weathering test were evaluated for loss of material, and they were compared to the controls - which had been stored flat in ambient air out of direct sunlight - for color change, texture change, and friability.

The weathered samples showed no significant weight change. Texture and friability were evaluated by touch and none of the samples exhibited any discernible change. None of the samples had any color change, with one exception. The Jahn samples (sample I) took on a slightly blotchy white appearance. Most of this could be removed with 5% acetic acid, indicating that it was cement laitance and not color loss. But the weathered samples maintained a slightly mottled appearance compared to the control. A similar phenomenon had been observed in a cemetery conservation project in Hartford, Connecticut (Matero 1998). It might be necessary to extend the test period beyond 360 hours to evaluate the color stability of the mortars.

ABRASION RESISTANCE

The results of the abrasion resistance are given in Figures 26 and 27

Sample	Weight of sand need to fill loss (g)
A	0.45
B	0.26
C	0.50
D	0.16
E	0.18
F	0.15
G	0.20
H	0.14
I	0.25
S	0.07

Figure 26 Abrasion Resistance

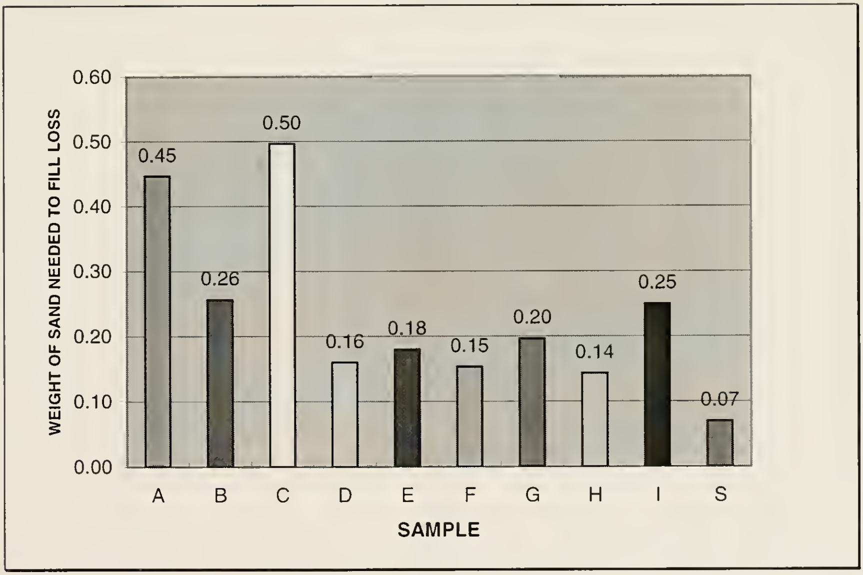

Figure 27 Abrasion Resistance Graph

The lowest peak indicates the smallest loss and therefore the best resistance to abrasion. Hydrated hydraulic lime (Sample C) was the most friable. This is surprising since, in other durability tests, the lime mortars (samples A and B) were less durable than the hydrated hydraulic lime. In every case, the addition of Acryl® 60 dramatically increased abrasion resistance. This is especially true of the hydrated hydraulic lime, where adding Acryl® 60 brought the abrasion resistance to a level comparable to the cement/lime mortars. All of the mortars had lower abrasion resistance than the stone. This is important because the repair should weather faster than the surrounding stone or it will eventually project from the surface of the wall, trapping water and soil (Ashurst 1988).

SUMMARY CONCLUSIONS

The tests data are summarized in Figures 28 and 29

There is no perfect composite repair material for sandstone. The critical properties of the repair always depend on the nature and condition of the particular stone to be repaired, the environment, and the application. The following summary is not intended to identify which material tested will yield the best repair. It highlights how the various components of the repair materials can affect the working properties, permeance, strength and durability of a composite repair. Depending on the type of stone, its condition, location, and environment, composite repairs can be formulated by varying their components to provide optimal performance.

Sample	Flow	Total Liquid (water + Acryl 60) ml	Time to reach full set (h:m:s)	Average Water Vapor Transmission (g/m m^2)	Average Permeance (g/Pa s m^2)	Absorption Capacity (% dry weight)
A	27.00%	100	63:30:00	9.22	2.09E-06	9.76
B	30.57%	85	52:00:00	9.37	2.12E-06	15.23
C	28.00%	169	7:00:00	8.14	1.85E-06	9.45
D	31.23%	143	6:00:00	7.11	1.61E-06	7.86
E	33.02%	130	2:30:00	4.85	1.10E-06	8.84
F	31.25%	108	2:45:00	5.88	1.33E-06	9.77
G	31.25%	113	3:15:00	5.75	1.30E-06	10.54
H	29.31%	98	3:00:00	6.78	1.54E-06	10.12
I	19.30%	138	2:00:00	4.89	1.11E-06	9.93
S				1.17	2.64E-07	

Figure 28 Summary of Test Results

Sample	Flexural Strength (psi)	Modulus of Elasticity (psi)	Bond Strength - shear stress at failure (psi)	Frost Resistance (% bulk volume retained)	Salt Crystallization Rating (compariative visual analysis)	Abrasion Resistance - indirect weight of loss (g)
A	130.30	3.42E+03	19.27	0	4	0.45
B	100.72	4.82E+03	10.58	0	5	0.26
C	119.98	5.27E+03	19.44	97.8	7	0.50
D	256.31	1.70E+04	31.83	94.5	9	0.16
E	451.17	2.40E+04	76.13	99.9	10	0.18
F	457.80	2.01E+04	49.29	99.8	10	0.15
G	306.59	2.00E+04	60.77	98.2	7	0.20
H	398.44	2.29E+04	26.45	99.5	9	0.14
I	573.18	3.87E+04	36.77	99.3	10	0.25
S	1827.60	4.12E+04				0.07

Figure 29 Summary of Test Results cont.

LIME PUTTY BASED FORMULATIONS

The lime putty formulations (Samples A and B), while having very high permeability, were the least durable of all the materials tested, as determined by frost resistance, resistance to salt crystallization, and abrasion resistance. Their long set time limits their potential application in wet or humid environments or where prolonged protection is not possible. Low durability suggests poor performance in severe exposure situations.

The high WVT and permeance are good in that the repair would not be likely to trap moisture at the bond interface; however, the high water absorption capacity means that the repair is easily saturated encouraging decay from frost, salt crystallization, and biological and chemical related deterioration.

The lime formulations had low flexural strength combined with a relatively high modulus of elasticity, meaning the material has low resistance to cracking and is very brittle. The bond strength is the lowest of the materials tested. The low strength could be an advantage for repairs of very friable stone where a stronger, more adhesive repair could damage the stone.

The frost resistance and salt decay resistance were low, but where these decay factors can be controlled, the lime putty formulations could have acceptable durability and performance.

Adding Acryl® 60 to the lime putty repair formulation is not generally recommended because of its inconsistent effects on the properties of the repair. Acryl® 60 slightly increased the permeance of the lime, but the permeance of the lime formulations without Acryl® 60 was so high that the slight increase is irrelevant. Acryl® 60 made the lime formulation even more prone to cracking, and it decreased the bond strength. The resistance to salt crystallization was improved but this was offset by a decrease in frost resistance.

RIVERTON HYDRATED HYDRAULIC LIME BASED FORMULATIONS

The Riverton hydrated hydraulic lime (samples C and D) met its expectation of combining certain properties of lime and Portland cement. The set time of the hydrated hydraulic lime was somewhat longer than the Portland cement formulations, but it was dramatically shorter than the lime putty formulations. The permeance of the hydrated hydraulic lime was nearly as high as that of the lime putty. The hydrated hydraulic lime without Acryl® 60 (sample C) had lower flexural strength and higher modulus of elasticity than the lime putty, and it had nearly identical bond strength.

The durability tests gave inconsistent results. The hydrated hydraulic lime formulations had much higher resistance to frost and salt crystallization than the lime formulations, which would suggest better cohesive strength. Yet the hydrated hydraulic lime without Acryl® 60 (sample C) had one of the lowest resistance to abrasion.[11]

Of all the repair materials tested, hydrated hydraulic lime responded most dramatically to adding Acryl® 60, especially in strength and durability. The flexural strength of the hydrated hydraulic lime doubled when Acryl® 60 was added while the modulus of elasticity decreased. Thus the material was more resistant to cracking and more flexible with Acryl® 60. The hydrated hydraulic lime was the only formulation tested whose bond strength increased with the addition of Acryl® 60, and Acryl® 60 significantly increased the hydrated hydraulic lime's resistance to abrasion. The increases in flexural strength, bond strength, and durability came with only a very slight decrease in permeance.

HYBRID FORMULATIONS - PORTLAND CEMENT AND LIME PUTTY

The 1:1:6 cement, lime sand formulations without (sample E) and with (sample F) Acryl® 60 were the strongest, least permeable (although still more permeable

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[11] Recent inquiries with the manufacturer revealed that the Riverton hydrated hydraulic lime includes air entraining agents which would increase frost resistance and resistance to salt crystallization.

than the stone), and most durable of all the custom mixes tested. All of these properties can be attributed to the significant proportion of Portland cement. Cementitious repairs would be appropriate for sound stone where there is minimal risk of damaging the stone, and where maximum durability is required; however, removal would be difficult, and there is the risk of soluble salts from the cement damaging the surrounding stone.

The 1:1:6 formulation generally benefited from the addition of Acryl® 60. It slightly increased the permeance while increasing the flexural strength and decreasing the modulus of elasticity. It also increased abrasion resistance; however, Acryl® 60 cut the bond strength of the 1:1:6 formulation in half. Acryl 60 had nearly identical effects on the 1:2:9 cement, lime, sand formulation.

Comparing the test results for the 1:1:6 formulations (samples E and F) with the 1:2:9 cement, lime, sand (sample G, without Acryl® 60 and sample H, with Acryl® 60) indicates that a higher proportion of lime in the hybrid mix increases permeance and absorption capacity, and decreases flexural strength, bond strength and overall durability.

JAHN M 70

The Jahn repair material (sample I) is difficult to compare to the custom mixes because the ingredient materials are different. The Jahn has a much smaller particle size aggregate and contains air entraining agents and pigments in addition to Portland cement (the exact composition of the Jahn repair material is proprietary). The Jahn material had a fast set time which is an advantage if the surface of the repair is to be tooled dry. The permeance of the Jahn material was the lowest of all the formulations tested, although it was still higher than the test stone. Permeance could be an issue with the Jahn material if the stone being repaired has a higher permeance than the stone tested in this study.

The Jahn material had the highest flexural strength, which indicates that the material is very resistant to cracking; however, the modulus of elasticity was also the highest, meaning the material is very stiff and less likely to absorb stresses from differential movement. The bond strength is difficult to compare because some of the replicates in the Jahn sample set tested broke within the repair and not at the bond line. In terms of overall durability the Jahn material ranked among the most durable of the materials tested.

Although the testing program did not address color and texture matching, it should be noted that, although the Jahn material displayed an acceptable color

match to the Portland sandstone, the stone has a much coarser and variable texture than the Jahn material. The custom mixes contained more large aggregate and more closely matched the texture of the test stone.

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C 230-90 Standard Specification for Flow Table for Use in Tests of Hydraulic Cement

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APPENDIX A: TEST RESULTS

This appendix contains complete test data for the testing program. Special considerations for statistical evaluation are given, where applicable, for each test along with sample calculations. All samples are labeled according to sample matrix below (the same table is in the section on testing):

	PROPORTIONS				
SAMPLE DESIGNATION	White Portland Cement	Hydrated Hydraulic Lime	Lime Putty	Sand	Acryl 60
A	-	-	1	3	-
B	-	-	1	3	~1:5
C	-	1	-	3	-
D	-	1	-	3	~1:5
E	1	-	1	6	-
F	1	-	1	6	~1:5
G	1	-	2	9	-
H	1	-	2	9	~1:5
I	Jahn M70 #2 mixed according to manufacturer's specifications				
S	Stone from the Portland Brownstone Quarries, Connecticut				

Figure 30 Sample Matrix

A number next to the letter (example: A1) indicates a replicate of that sample.

CONSISTENCY

Sample	diameter of mortar after test (mm)					
	Axis 1	Axis 2	Axis 3	Axis 4	Average	**FLOW**
A	89.55	88.30	87.00	90.75	88.90	**27.00%**
B	90.35	91.50	92.70	91.05	91.40	**30.57%**
C	90.80	88.85	89.25	89.50	89.60	**28.00%**
D	91.70	92.85	91.00	91.90	91.86	**31.23%**
E	91.90	94.50	92.80	93.25	93.11	**33.02%**
F	91.65	92.80	91.05	92.00	91.88	**31.25%**
G	92.70	92.50	90.45	91.85	91.88	**31.25%**
H	90.60	90.95	89.85	90.66	90.52	**29.31%**
I	83.35	82.90	83.45	84.35	83.51	**19.30%**

Figure 31 Data for Flow Test

Flow is calculated as the average increase in diameter of the base of the mortar mass after 25 drops through 13 mm in 25 seconds expressed as a percentage of the original diameter.

$$\text{FLOW} = \frac{(\text{AVERAGE BASE DIAMETER AFTER TEST}) - (\text{ORIGINAL BASE DIAMETER})}{(\text{ORIGINAL BASE DIAMETER})} \times 100$$

For sample A,

$$\text{FLOW} = \frac{(88.9) - (70)}{(70)} \times 100 \quad = \mathbf{27.0\%}$$

TIME OF SETTING

Elapsed Time (hours)	0	17.5	24	30	40	58	60.5	71.5
A1	40.0	40.0	38.0	32.0	25.0	0.0	0.0	0.0
A2	40.0	40.0	37.0	30.0	26.0	5.0	0.0	0.0
A3	40.0	40.0	38.0	37.0	30.0	5.0	2.0	0.0
Elapsed Time (hours)	0	15	24	30	40	50	51	52
B1	40.0	38.0	33.0	20.0	8.0	2.0	0.0	0.0
B2	40.0	40.0	30.0	35.0	17.0	6.0	3.0	0.0
B3	40.0	40.0	25.0	26.0	12.0	4.0	4.0	0.0
Elapsed Time (hours)	0	2	4	6.5	7	8		
C1	40.0	21.0	17.0	0.0	0.0	0.0		
C2	40.0	21.0	9.0	0.0	0.0	0.0		
C3	40.0	34.0	18.0	12.0	5.0	0.0		
Elapsed Time (hours)	0	2.5	3	4	5	5.5	6	6.5
D1	31.0	25.0	30.0	7.0	0.0	0.0	0.0	0.0
D2	40.0	40.0	40.0	35.0	20.0	16.0	1.0	0.0
D3	40.0	40.0	30.0	18.0	18.0	11.0	3.0	0.0
Elapsed Time (hours)	0	1.5	2	2.25	2.5	2.75		
E1	40.0	28.0	5.0	1.0	0.0	0.0		
E2	40.0	35.0	5.0	0.0	0.0	0.0		
E3	40.0	40.0	25.0	8.0	6.0	0.0		
Elapsed Time (hours)	0	1.5	1.75	2	2.75	2.75		
F1	40.0	35.0	22.0	4.0	1.0	0.0		
F2	40.0	40.0	30.0	10.0	7.0	0.0		
F3	40.0	40.0	28.0	8.0	1.0	0.0		
Elapsed Time (hours)	0	1.5	2	2.5	2.75	3	3.25	3.5
G1	40.0	40.0	38.0	20.0	11.0	8.0	6.0	0.0
G2	40.0	39.0	28.0	25.0	8.0	8.0	3.0	0.0
G3	40.0	35.0	30.0	19.0	21.0	11.0	0.0	0.0
Elapsed Time (hours)	0	1.5	2	2.5	2.75	3		
H1	40.0	19.0	17.0	6.0	2.0	0.0		
H2	40.0	35.0	28.0	12.0	1.0	0.0		
H3	40.0	21.0	16.0	4.0	4.0	0.0		
Elapsed Time (hours)	0	1	1.25	1.5	1.75	2		
I1	40.0	20.0	18.0	18.0	4.0	0.0		
I2	40.0	17.0	12.0	8.0	1.0	0.0		
I3	40.0	21.0	16.0	6.0	2.0	0.0		

Figure 32 Elapsed Time and Depth of Penetration Measurements for Set Time Test

WATER VAPOR TRANSMISSION

Elapsed Time (h:m:s)	0:00:00	21:00:00	45:00:00	69:00:00	93:25:00	117:07:00	141:15:00	165:10:00	189:10:00
Sample	Weight of sample (g)								
A1	57.38	57.12	56.84	56.54	56.21	55.92	55.67	55.37	55.09
A2	58.83	58.57	58.30	58.00	57.68	57.39	57.15	56.85	56.60
A3	57.78	57.52	57.23	56.91	56.58	56.31	56.04	55.74	55.48
B1	56.56	56.29	55.98	55.67	55.36	55.08	54.81	54.51	54.25
B2	58.14	57.90	57.60	57.30	56.99	56.70	56.44	56.13	55.88
B3	57.34	57.06	56.75	56.44	56.10	56.80	55.52	56.21	54.93
C1	59.93	59.67	59.41	59.12	58.85	58.58	58.33	58.07	57.84
C2	60.28	60.02	59.78	59.52	59.25	59.00	58.76	58.51	58.28
C3	62.47	62.21	61.97	61.71	61.42	61.18	60.95	60.68	60.45
D1	60.58	60.36	60.15	59.90	59.66	59.43	59.23	59.00	58.79
D2	59.23	59.01	58.79	58.55	58.31	58.08	57.87	57.63	57.42
D3	59.94	59.74	59.54	59.31	59.07	58.87	58.67	58.45	58.25
E1	62.18	62.07	61.92	61.75	61.55	61.41	61.27	61.11	60.97
E2	63.94	63.82	63.68	63.52	63.34	63.20	63.07	62.91	62.77
E3	63.82	63.70	63.56	63.40	63.23	63.09	62.95	62.82	62.68
F1	60.79	60.61	60.44	60.23	60.03	59.86	59.70	59.51	59.34
F2	62.19	62.02	61.84	61.65	61.44	61.28	61.11	60.94	60.77
F3	59.63	59.42	59.23	59.02	58.81	58.62	58.44	58.24	58.06
G1	60.86	60.71	60.53	60.34	60.14	59.96	59.80	59.62	59.45
G2	61.66	61.51	61.33	61.15	60.93	60.76	60.60	60.41	60.24
G3	60.98	60.84	60.67	60.48	60.27	60.11	59.95	59.77	59.61
H1	59.62	59.45	59.24	59.03	58.77	58.56	58.35	58.15	57.94
H2	58.60	58.41	58.22	58.02	57.78	57.59	57.40	57.21	57.02
H3	59.30	59.12	58.90	58.68	58.44	58.23	58.04	57.82	57.62
I1	54.40	54.27	54.12	53.95	53.76	53.62	53.49	53.34	53.18
I2	54.00	53.83	53.71	53.54	53.35	53.21	53.07	52.92	52.77
I3	54.56	54.42	54.29	54.13	53.95	53.82	53.69	53.54	53.40
S1	74.56	74.56	74.52	74.51	74.47	74.43	74.40	74.37	74.32
S2	75.38	75.37	75.35	75.33	75.29	75.25	75.22	75.18	75.13
S3	76.42	76.41	76.37	76.33	76.29	76.25	76.21	76.17	76.12
S4	76.91	76.91	76.89	76.86	76.83	76.81	76.77	76.74	76.70
S5	75.40	75.39	75.36	75.33	75.30	75.27	75.24	75.20	75.15
S6	76.73	76.72	76.69	76.66	76.63	76.60	76.57	76.53	76.49
RH (%)		50.00%	51.00%	49.00%	55.00%	51.00%	57.00%	55.00%	55.00%
TEMP (C)		22.0	22.0	22.0	22.0	22.5	22.0	22.0	21.5

Figure 33 Water Vapor Transmission Data. The shaded area indicates data used to calculate WVT and permeance

Sample	WVT (g/m m2)	Average WVT	Standard Deviation	Permeance (g/Pa s m2)	Average Permeance	Standard Deviation
A1	9.37			2.12E-06		
A2	9.06	9.22	0.16	2.05E-06	2.09E-06	3.69E-08
A3	9.24			2.09E-06		
B1	9.17			2.08E-06		
B2	9.17	9.37	0.34	2.08E-06	2.12E-06	7.61E-08
B3	9.76			2.21E-06		
C1	8.27			1.87E-06		
C2	8.01	8.14	0.13	1.82E-06	1.85E-06	2.93E-08
C3	8.14			1.85E-06		
D1	7.17			1.63E-06		
D2	7.30	7.11	0.23	1.65E-06	1.61E-06	5.28E-08
D3	6.85			1.55E-06		
E1	5.04			1.14E-06		
E2	4.85	4.85	0.19	1.10E-06	1.10E-06	4.39E-08
E3	4.65			1.05E-06		
F1	5.75			1.30E-06		
F2	5.69	5.88	0.28	1.29E-06	1.33E-06	6.38E-08
F3	6.20			1.41E-06		
G1	5.75			1.30E-06		
G2	5.88	5.75	0.13	1.33E-06	1.30E-06	2.93E-08
G3	5.62			1.27E-06		
H1	7.04			1.60E-06		
H2	6.46	6.78	0.30	1.46E-06	1.54E-06	6.71E-08
H3	6.85			1.55E-06		
I1	4.97			1.13E-06		
I2	4.97	4.89	0.15	1.13E-06	1.11E-06	3.38E-08
I3	4.72			1.07E-06		
S1	1.20			2.72E-07		
S2	1.26			2.86E-07		
S3	1.32	1.17	0.17	3.00E-07	2.64E-07	3.78E-08
S4	1.01			2.29E-07		
S5	1.13			2.57E-07		
S6	1.07			2.43E-07		

Figure 34 Values for WVT and Permeance with averages and standard deviations

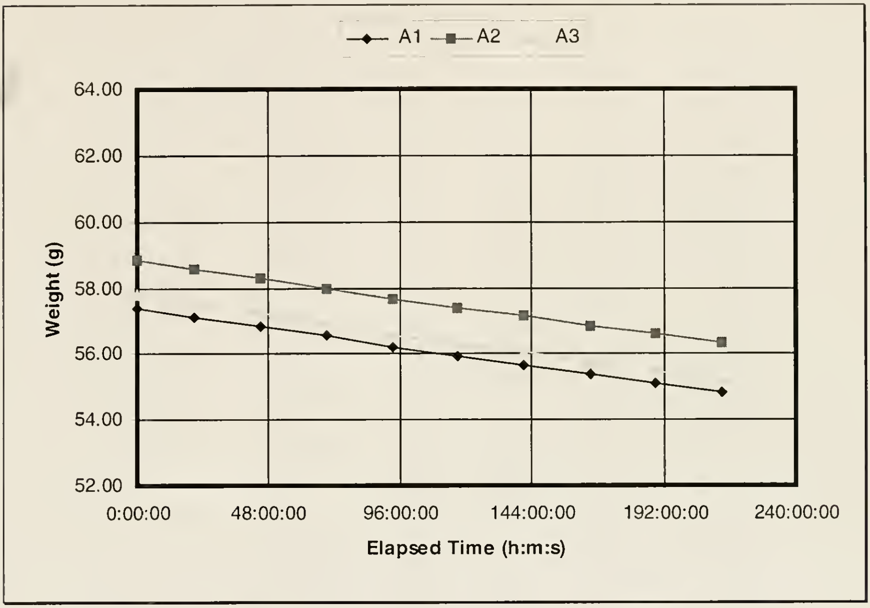

Figure 35 Example of a Water Vapor Transmission Curve. A straight line indicates a constant rate of vapor transmission through the sample

WATER ABSORPTION

	M0	M1	M2	M3	M4
Elapsed Time (h:m:s)	0:00:00	0:05:00	0:20:00	1:00:00	4:00:00
Sample	Weight of sample (g)				
A1	94.61	103.31	103.58	103.53	103.84
A2	99.13	108.29	108.12	108.58	108.67
A3	99.82	108.98	109.00	109.17	109.69
B1	86.51	99.47	99.57	99.70	99.80
B2	89.66	103.25	103.28	103.55	103.64
B3	90.80	104.04	103.98	103.99	104.18
C1	102.72	111.97	112.07	112.13	112.37
C2	103.87	113.60	113.70	113.80	113.86
C3	103.71	113.27	113.33	113.36	113.39
D1	100.34	106.88	108.00	108.24	108.44
D2	100.25	106.52	107.68	108.14	108.23
D3	101.19	106.54	108.07	108.61	108.83
E1	114.92	123.93	124.63	124.88	124.97
E2	113.22	122.69	123.11	123.39	123.40
E3	112.71	121.43	122.18	122.38	122.60
F1	101.48	110.49	111.19	111.34	111.65
F2	99.44	108.31	108.70	108.95	109.24
F3	100.73	109.44	109.90	110.08	110.23
G1	108.92	119.53	119.85	120.09	120.19
G2	109.18	119.95	120.39	120.50	120.74
G3	108.07	119.19	119.36	119.44	119.62
H1	101.03	110.17	110.78	110.90	111.21
H2	102.69	111.90	112.54	112.84	113.05
H3	100.84	110.38	110.87	111.06	111.13
I1	87.77	94.81	95.60	96.29	96.74
I2	86.78	94.00	94.69	95.19	95.31
I3	85.60	92.43	93.14	93.74	93.95

Figure 36 Data for Water Absorption Test

FLEXURAL STRENGTH

Sample	L - gage length or span (mm)	L (in)	b - width of beam (mm)	b (in)	d - depth of beam (mm)	d (in)	P - max load at failure (lb)	Flexural Strength (psi)
A1	38.00	1.496	19.90	0.784	19.90	0.784	27	125.74
A2	38.00	1.496	19.65	0.774	19.70	0.776	28	134.81
A3	38.00	1.496	19.80	0.780	19.60	0.772	27	130.34
Average								**130.30**
St Dev								4.54
B1	38.00	1.496	19.75	0.778	19.60	0.772	21	101.64
B2	38.00	1.496	19.90	0.784	19.20	0.756	20	100.16
B3	38.00	1.496	20.10	0.792	19.55	0.770	21	100.36
Average								**100.72**
St Dev								0.80
C1	38.00	1.496	19.75	0.778	19.50	0.768	25	122.26
C2	38.00	1.496	20.10	0.792	19.65	0.774	27	125.34
C3	38.00	1.496	20.10	0.792	19.75	0.778	24	112.35
Average								**119.98**
St Dev								6.79
D1	38.00	1.496	19.60	0.772	20.40	0.804	54	242.83
D2	38.00	1.496	19.85	0.782	20.30	0.800	62	278.00
D3	38.00	1.496	20.00	0.788	20.25	0.780	53	248.09
Average								**256.31**
St Dev								18.97
E1	38.00	1.496	20.30	0.800	20.45	0.806	105	453.39
E2	38.00	1.496	19.00	0.748	20.65	0.812	70	318.51
E3	38.00	1.496	20.20	0.796	20.65	0.812	105	448.96
Average							**93**	**451.17**
St Dev							20.207	76.62

Figure 37 Flexural Strength Values for 3-point Bending Test, samples A through E. Shaded values had % difference from mean greater than 15% and were eliminated to calculate the final average

	L - gage length or span (mm)	L (in)	b - width of beam (mm)	b (in)	d - depth of beam (mm)	d (in)	max load at failure (lb)	Flexural Strength (psi)
F1	38.00	1.496	20.30	0.800	20.30	0.800	103	451.45
F2	38.00	1.496	20.50	0.808	20.30	0.800	103	446.98
F3	38.00	1.496	20.40	0.804	20.40	0.804	110	474.97
Average								**457.80**
St Dev								15.04
G1	38.00	1.496	20.20	0.796	20.25	0.798	90	398.44
G2	38.00	1.496	20.35	0.802	20.10	0.792	66	294.42
G3	38.00	1.496	20.40	0.804	20.15	0.794	72	318.77
Average							76	**306.59**
St Dev							12.490	54.41
H1	38.00	1.496	20.10	0.792	20.25	0.798	83	369.31
H2	38.00	1.496	20.10	0.792	20.40	0.804	98	429.57
H3	38.00	1.496	20.20	0.796	20.30	0.800	90	396.45
Average								**398.44**
St Dev								30.18
I1	38.00	1.496	20.25	0.798	20.45	0.806	128	554.09
I2	38.00	1.496	19.80	0.800	20.45	0.806	126	544.07
I3	38.00	1.496	20.05	0.790	20.30	0.800	140	621.39
Average								**573.18**
St Dev								42.05
S1	38.00	1.496	20.75	0.816	20.45	0.806	410	1735.66
S2	38.00	1.496	20.45	0.806	19.60	0.772	420	1962.10
S3	38.00	1.496	20.60	0.812	20.00	0.788	420	1869.31
S4	38.00	1.496	20.25	0.798	18.50	0.728	340	1804.08
S5	38.00	1.496	20.25	0.798	19.00	0.748	360	1809.41
Average								**1836.11**
St Dev								84.86

Figure 38 Flexural Strength Values for 3-point Bending Test, samples F through S. Shaded values had % difference from mean greater than 15% and were eliminated to calculate the final average

Sample	L - gage length or span (mm)	L (in)	b - width of beam (mm)	b (in)	d - depth of beam (mm)	d (in)	slope	E - Modulus of Elasticity (psi)
A1	38.00	1.496	19.90	0.784	19.90	0.784	14000	4427.75
A2	38.00	1.496	19.65	0.774	19.70	0.776	146000	44204.79
A3	38.00	1.496	19.80	0.780	19.60	0.772	8000	2403.40
Average								**3415.57**
St Dev								23571.41
B1	38.00	1.496	19.75	0.778	19.60	0.772	16000	4794.48
B2	38.00	1.496	19.90	0.784	19.20	0.756	12000	3402.93
B3	38.00	1.496	20.10	0.792	19.55	0.770	16000	4842.92
Average								**4818.70**
St Dev								817.76
C1	38.00	1.496	19.75	0.778	19.50	0.768	24000	7080.52
C2	38.00	1.496	20.10	0.792	19.65	0.774	16000	4918.79
C3	38.00	1.496	20.10	0.792	19.75	0.778	18000	5619.88
Average								**5269.33**
St Dev								1102.88
D1	38.00	1.496	19.60	0.772	20.40	0.804	54000	18137.17
D2	38.00	1.496	19.85	0.782	20.30	0.800	48000	16088.23
D3	38.00	1.496	20.00	0.788	20.25	0.780	54000	16904.18
Average								**17043.19**
St Dev								1031.52
E1	38.00	1.496	20.30	0.800	20.45	0.806	70000	24546.16
E2	38.00	1.496	19.00	0.748	20.65	0.812	36000	12068.76
E3	38.00	1.496	20.20	0.796	20.65	0.812	66000	23545.91
Average								**24046.03**
St Dev								6933.15

Figure 39 Results for Modulus of Elasticity from Three-point Bending Test, Samples A through E. Shaded results were eliminated from the average

Sample	L - gage length or span (mm)	L (in)	b - width of beam (mm)	b (in)	d - depth of beam (mm)	d (in)	slope	E
F1	38.00	1.496	20.30	0.800	20.30	0.800	56000	19201.64
F2	38.00	1.496	20.50	0.808	20.30	0.800	84000	29090.48
F3	38.00	1.496	20.40	0.804	20.40	0.804	60000	20987.75
Average								**20094.69**
St Dev								5269.94
G1	38.00	1.496	20.20	0.796	20.25	0.798	36000	12190.31
G2	38.00	1.496	20.35	0.802	20.10	0.792	48000	16009.64
G3	38.00	1.496	20.40	0.804	20.15	0.794	94000	31669.10
Average								**19956.35**
St Dev								10321.74
H1	38.00	1.496	20.10	0.792	20.25	0.798	48000	16172.06
H2	38.00	1.496	20.10	0.792	20.40	0.804	86000	29633.44
H3	38.00	1.496	20.20	0.796	20.30	0.800	144000	49128.77
Average								**22902.75**
St Dev								16570.16
I1	38.00	1.496	20.25	0.798	20.45	0.806	96000	33579.14
I2	38.00	1.496	19.80	0.800	20.45	0.806	112000	39273.85
I3	38.00	1.496	20.05	0.790	20.30	0.800	128000	43340.84
Average								**38731.28**
St Dev								4903.42
S1	38.00	1.496	20.75	0.816	20.45	0.806	192000	68673.13
S2	38.00	1.496	20.45	0.806	19.60	0.772	136000	42219.80
S3	38.00	1.496	20.60	0.812	20.00	0.788	132000	43903.48
S4	38.00	1.496	20.25	0.798	18.50	0.728	132000	34022.12
S5	38.00	1.496	20.25	0.798	19.00	0.748	160000	44731.97
Average								**41219.34**
St Dev								12993.39

Figure 40 Results for Modulus of Elasticity from Three-point Bending Test, Samples F through S. Shaded results were eliminated from the average.

The equation for for modulus of elasticiticy is $\mathbf{E = L^3M_1/4bd^3}$, where M_1 equals the slope of tangent to the initial straight line portion of the of the load-deformation curve (ASTM C 580-93).

The values for modulus of elasticity were less consistent that the values for flexural strength because of the problem described in Chapter Three. ASTM C 580-93 requires that if any value within a sample that differs from the mean by more than 15%, it must be eliminated from the mean and the mean recalculated. This procedure was followed for the flexural strength values. For the modulus of elasticity values, any value that differed from the mean by more than 25% was eliminated.

BOND STRENGTH

BOND AREA = 50X50mm or 3.875 in^2
CROSS HEAD SPEED: 0.02 in/min
CHART SPEED: 2 in/min

Sample	max load at failure (lb)	shear stress at failure (psi)	% diff from mean
A1	82	21.16	9.82%
A2	72	18.58	3.57%
A3	70	18.06	6.25%
Average	75	**19.27**	
St Dev	5.25	1.35	
B1	30	7.74	26.83%
B2	38	9.81	7.32%
B3	55	14.19	34.15%
Average	41	**10.58**	
St Dev	10.42	2.69	
C1	46	11.87	38.94%
C2	96	24.77	27.43%
C3	84	21.68	11.50%
Average	75	**19.44**	
St Dev	21.31	5.50	
D1	130	33.55	5.41%
D2	112	28.90	9.19%
D3	128	33.03	3.78%
Average	123	**31.83**	
St Dev	8.06	2.08	
E1	285	73.55	3.39%
E2	210	54.19	28.81%
E3	390	100.65	32.20%
Average	295	**76.13**	
St Dev	73.82	19.05	
F1	133	34.32	30.37%
F2	270	69.68	41.36%
F3	170	43.87	10.99%
Average	191	**49.29**	
St Dev	57.87	14.93	
G1	258	66.58	9.55%
G2	269	69.42	14.23%
G3	202	52.13	14.23%
Average	243	**60.77**	
St Dev	29.34	7.57	
H1	131	33.81	27.80%
H2	120	30.97	17.07%
H3	85	21.94	17.07%
Average	112	**26.45**	
St Dev	19.61	5.06	
I1	125	32.26	12.28%
I2	260	67.10	82.46%
I3	25	6.45	82.46%
Average	137	**36.77**	
St Dev	96.29	24.85	

Figure 41 Values for Shear Bond Strength with averages and standard deviations

The formula used for bond strength in shear is:

$$\text{Shear Bond Strength (psi)} = \frac{\text{Maximum Load at Failure (lbs)}}{\text{Bond Area (in}^2\text{)}}$$

A source or error in the calculation is that the bond area was not measured directly but was based on the dimensions of the molds which were built to tolerances of ± 0.25 mm. Another source of error is that, although the samples were set in the machine so that the load was parallel to the bond line, the samples could have move slightly creating some torsion stress as well as shear.

There is no procedure for the statistical manipulation of the data in the referenced standard, ASTM D 905-94; therefore, all the data is presented in Figure 41. No values were eliminated from the average for each sample.

FROST RESISTANCE

Cycles	0			3			6		
Sample	weight in water (g)	weight in air (g)	bulk volume (g)	weight in water (g)	weight in air (g)	bulk volume (g)	weight in water (g)	weight in air (g)	bulk volume (g)
A1	54.20	103.79	49.59	54.10	102.43	48.33	37.40	54.02	16.62
A2	56.80	108.42	51.62	56.90	107.48	50.58	48.60	82.99	34.39
A3	57.40	109.25	51.85	57.20	108.30	51.10	50.40	89.40	39.00
B1	49.90	99.84	49.94	45.20	84.43	39.23	15.00	16.02	1.02
B2	51.70	103.43	51.73	46.30	89.89	43.59	24.50	39.69	15.19
B3	52.10	103.91	51.81	47.70	93.38	45.68	24.40	43.57	19.17
C1	59.10	112.30	53.20	59.30	112.17	52.87	59.00	111.89	52.89
C2	59.80	114.15	54.35	59.90	113.81	53.91	59.80	113.69	53.89
C3	59.70	113.61	53.91	59.80	113.47	53.67	59.60	113.22	53.62
D1	53.35	108.52	55.17	54.90	110.51	55.61	54.90	110.08	55.18
D2	53.30	108.50	55.20	54.10	109.43	55.33	54.90	109.78	54.88
D3	53.70	109.96	56.26	55.00	110.20	55.20	55.70	110.67	54.97
E1	68.40	125.17	56.77	68.30	125.09	56.79	68.30	125.08	56.78
E2	67.50	123.59	56.09	67.20	123.38	56.18	67.10	123.48	56.38
E3	67.10	122.70	55.60	66.70	122.48	55.78	66.80	122.55	55.75
F1	54.70	111.70	57.00	55.30	112.54	57.24	55.80	112.92	57.12
F2	53.00	109.30	56.30	53.50	109.89	56.39	54.00	110.44	56.44
F3	53.90	110.57	56.67	54.90	111.44	56.54	55.60	111.96	56.36
G1	65.50	123.37	57.87	65.60	120.22	54.62	65.50	120.15	54.65
G2	65.70	120.66	54.96	65.60	120.58	54.98	65.60	120.60	55.00
G3	64.80	119.72	54.92	64.70	119.57	54.87	64.60	119.60	55.00
H1	55.50	111.53	56.03	56.60	111.93	55.33	56.80	112.27	55.47
H2	55.70	113.20	57.50	sample broke and was eliminated					
H3	55.70	111.31	55.61	55.90	111.87	55.97	56.10	112.12	56.02
I1	40.50	96.76	56.26	40.60	96.85	56.25	40.80	97.14	56.34
I2	39.80	95.47	55.67	39.70	95.12	55.42	39.70	95.25	55.55
I3	39.20	94.06	54.86	38.90	93.69	54.79	38.90	93.67	54.77

Figure 42 Results of Frost Resistance Test

Cycles	9			12			15		
Sample	weight in water (g)	weight in air (g)	bulk volume (g)	weight in water (g)	weight in air (g)	bulk volume (g)	weight in water (g)	weight in air (g)	bulk volume (g)
A1	sample disintegrated		0.00			0.00			0.00
A2	"		0.00			0.00			0.00
A3	"		0.00			0.00			0.00
B1	"		0.00			0.00			0.00
B2	"		0.00			0.00			0.00
B3	"		0.00			0.00			0.00
C1	59.00	111.86	52.86	59.00	111.67	52.67	58.00	109.79	51.79
C2	59.90	113.58	53.68	59.90	113.26	53.36	59.80	113.14	53.34
C3	59.60	112.87	53.27	59.50	112.49	52.99	59.50	112.27	52.77
D1	56.30	111.13	54.83	55.10	108.95	53.85	51.90	101.59	49.69
D2	56.00	110.90	54.90	56.00	109.81	53.81	55.50	109.33	53.83
D3	56.20	111.33	55.13	57.20	112.07	54.87	56.20	110.12	53.92
E1	68.30	125.24	56.94	68.50	125.54	57.04	68.50	125.27	56.77
E2	67.10	123.55	56.45	67.40	122.68	55.28	67.40	123.27	55.87
E3	66.80	123.73	56.93	66.90	122.68	55.78	67.20	122.83	55.63
F1	55.80	113.34	57.54	56.00	113.33	57.33	56.40	113.40	57.00
F2	54.00	110.53	56.53	54.50	110.99	56.49	54.50	110.78	56.28
F3	55.60	112.36	56.76	56.00	112.49	56.49	56.00	112.34	56.34
G1	65.50	120.36	54.86	65.50	120.45	54.95	65.50	120.37	54.87
G2	65.80	120.78	54.98	65.70	120.74	55.04	65.90	120.78	54.88
G3	64.60	119.68	55.08	64.70	119.77	55.07	65.00	119.81	54.81
H1	56.70	112.47	55.77	56.90	112.39	55.49	57.10	112.48	55.38
H2									
H3	56.30	112.30	56.00	56.70	112.58	55.88	56.70	112.39	55.69
I1	40.90	97.59	56.69	41.50	97.50	56.00	41.70	97.63	55.93
I2	39.80	95.53	55.73	40.00	95.50	55.50	40.00	95.42	55.42
I3	39.10	93.85	54.75	39.20	93.83	54.63	39.60	93.95	54.35

Figure 43 Result of Frost Resistance Test continued

SALT CRYSTALLIZATION

Cycles	0	1	2	3	4	5	6	7
Sample	Weightof Sample (g)							
A1	105.84	106.28	106.42	106.96	107.21	107.75	105.72	103.90
A2	103.52	103.93	104.05	104.61	104.83	105.33	105.49	106.13
A3	102.40	102.84	103.02	103.56	103.79	104.37	104.18	101.82
B1	94.54	92.20	92.36	92..86	93.04	93.71	93.70	89.82
B2	93.41	94.07	94.28	94.77	94.85	95.52	95.63	95.56
B3	93.73	94.40	94.61	95.01	95.01	95.67	95.65	95.89
C1	107.34	107.73	107.87	108.58	109.11	109.69	110.53	112.50
C2	106.89	107.29	107.44	108.10	108.73	109.23	110.31	112.02
C3	106.76	107.11	107.27	107.57	108.57	109.08	110.01	111.41
D1	102.00	102.26	102.25	102.57	102.76	103.03	103.21	103.48
D2	104.42	104.56	104.55	104.78	104.93	105.14	105.28	105.47
D3	100.95	101.17	101.16	101.41	101.53	101.76	101.91	102.11
E1	113.83	113.61	113.56	114.36	115.17	115.29	116.26	118.01
E2	115.65	115.49	115.39	116.32	117.16	117.33	118.22	119.61
E3	115.21	115.20	115.12	116.10	117.01	117.20	118.31	119.61
F1	102.60	102.36	102.32	102.67	102.90	103.03	103.38	103.57
F2	102.45	102.94	102.36	102.77	103.02	103.17	103.61	103.74
F3	103.07	103.05	102.92	103.30	103.53	102.67	104.09	104.15
G1	112.97	113.05	113.22	114.87	116.12	115.85	117.89	119.39
G2	112.82	112.97	113.18	114.82	116.22	116.06	118.56	119.89
G3	113.34	113.53	113.84	116.08	117.52	117.43	119.52	120.21
H1	104.26	104.32	104.37	104.83	105.11	105.31	105.74	105.98
H2	103.26	103.42	103.47	103.93	106.17	104.37	104.81	104.99
H3	103.46	103.51	103.54	103.94	104.15	104.37	104.75	104.84
I1	92.34	90.44	91.61	92.14	92.36	92.51	92.86	93.29
I2	90.89	91.76	90.26	90.76	90.95	91.08	91.45	91.70
I3	91.36	90.79	90.80	91.14	91.44	91.56	91.96	92.13

Figure 44 Results of Salt Crystallization Test

Cycles	8	9	10	11	12	13	14	15	16*
Sample	Weightof Sample (g)								
A1	103.82	104.08	104.39	88.20	88.17	87.32	77.36	77.84	63.34
A2	106.42	86.12	86.42	81.05	80.42	79.27	71.11	69.67	60.76
A3	101.77	92.09	92.48	92.75	93.02	93.27	84.42	80.99	69.74
B1	88.49	88.60	88.66	80.05	80.30	80.33	80.04	74.72	67.64
B2	95.54	95.70	95.55	95.75	95.11	95.09	82.81	77.57	70.00
B3	95.42	93.48	88.06	88.40	87.28	87.27	86.30	84.79	71.30
C1	112.99	113.19	113.08	114.32	115.21	115.34	115.83	115.77	109.46
C2	112.51	113.78	113.14	113.53	114.58	114.67	115.26	115.23	106.70
C3	111.75	112.11	112.51	112.69	114.02	114.26	114.83	114.87	104.07
D1	103.67	103.82	104.23	104.29	104.49	104.69	104.99	105.19	103.59
D2	105.61	105.72	106.07	106.07	106.24	106.40	106.61	107.79	105.86
D3	102.27	102.40	102.75	102.76	102.94	103.11	103.36	103.55	102.31
E1	118.28	118.83	118.82	119.18	120.03	120.08	120.08	120.51	115.52
E2	119.81	120.44	120.52	120.96	121.81	121.85	121.85	122.43	117.50
E3	119.85	120.40	120.46	120.95	121.94	121.99	121.99	122.47	116.92
F1	103.71	103.84	104.35	104.30	104.57	104.72	104.72	105.10	103.60
F2	103.92	104.12	104.70	104.64	104.95	105.12	105.12	105.63	103.89
F3	104.34	104.52	105.11	105.05	105.35	105.54	105.54	105.98	104.43
G1	119.53	119.37	118.27	119.08	119.93	120.05	120.05	113.89	98.88
G2	120.11	120.36	119.46	119.74	121.68	121.67	121.67	111.12	97.13
G3	120.39	120.74	119.64	120.64	121.61	121.51	121.51	112.72	99.87
H1	106.21	106.35	106.91	106.92	107.18	107.41	107.41	107.90	105.70
H2	105.25	105.39	105.97	105.98	106.23	106.45	106.45	107.02	104.25
H3	105.06	105.14	106.65	105.65	105.82	106.04	106.04	106.45	94.38
I1	93.43	93.51	94.09	94.01	94.25	94.40	94.40	94.74	93.66
I2	91.86	91.91	92.46	92.38	92.61	92.70	92.73	93.05	92.16
I3	92.27	92.33	92.88	92.79	93.01	93.15	93.35	93.46	92.60

Figure 45 Results of Salt Crystallization Test – continued. Cycle 16 refers to the weight of the sample after the soaking and rinsing period.

ABRASION RESISTANCE

Sample	weight of sample before test (g)	weight of sample after test (g)	weight of sample plus sand (g)	weight of material lost (g)	weight of sand needed to fill loss (g)
A1	53.27	52.60	53.10	0.67	0.50
A2	49.46	48.99	49.46	0.47	0.47
A3	50.22	49.90	50.27	0.32	0.37
Average				0.49	**0.45**
St Dev				0.18	0.07
B1	48.21	47.94	48.26	0.27	0.32
B2	45.03	44.81	45.07	0.22	0.26
B3	43.61	43.41	43.60	0.20	0.19
Average				0.23	**0.26**
St Dev				0.04	0.07
C1	52.74	52.14	52.74	0.60	0.60
C2	51.65	51.12	51.64	0.53	0.52
C3	50.95	50.58	50.95	0.37	0.37
Average				0.50	**0.50**
St Dev				0.12	0.12
D1	54.18	54.00	54.19	0.18	0.19
D2	53.67	53.51	53.70	0.16	0.19
D3	53.26	53.17	53.27	0.09	0.10
Average				0.14	**0.16**
St Dev				0.05	0.05
E1	61.55	61.43	61.61	0.12	0.18
E2	53.28	53.03	53.28	0.25	0.25
E3	60.33	60.27	60.38	0.06	0.11
Average				0.14	**0.18**
St Dev				0.10	0.07
F1	51.72	51.63	51.84	0.09	0.21
F2	55.76	55.70	55.83	0.06	0.13
F3	54.72	54.68	54.80	0.04	0.12
Average				0.06	**0.15**
St Dev				0.03	0.05
G1	62.97	62.71	62.95	0.26	0.24
G2	64.85	64.71	64.89	0.14	0.18
G3	64.46	64.35	64.52	0.11	0.17
Average				0.17	**0.20**
St Dev				0.08	0.04
H1	54.92	54.80	54.97	0.12	0.17
H2	53.08	53.00	53.13	0.08	0.13
H3	55.08	55.01	55.14	0.07	0.13
Average				0.09	**0.14**
St Dev				0.03	0.02
I1	47.06	46.84	47.14	0.22	0.30
I2	45.00	44.83	45.05	0.17	0.22
I3	45.30	45.14	45.37	0.16	0.23
Average				0.18	**0.25**
St Dev				0.03	0.04
S1	132.88	132.78	132.85	0.10	0.07
S2	133.92	133.84	133.90	0.08	0.06
S3	132.80	132.70	132.78	0.10	0.08
Average				0.09	**0.07**
St Dev				0.01	0.01

Figure 46 Results of Abrasion Resistance Test with averages and standard deviations

APPENDIX B: PHOTOS OF SALT CRYSTALLIZATION

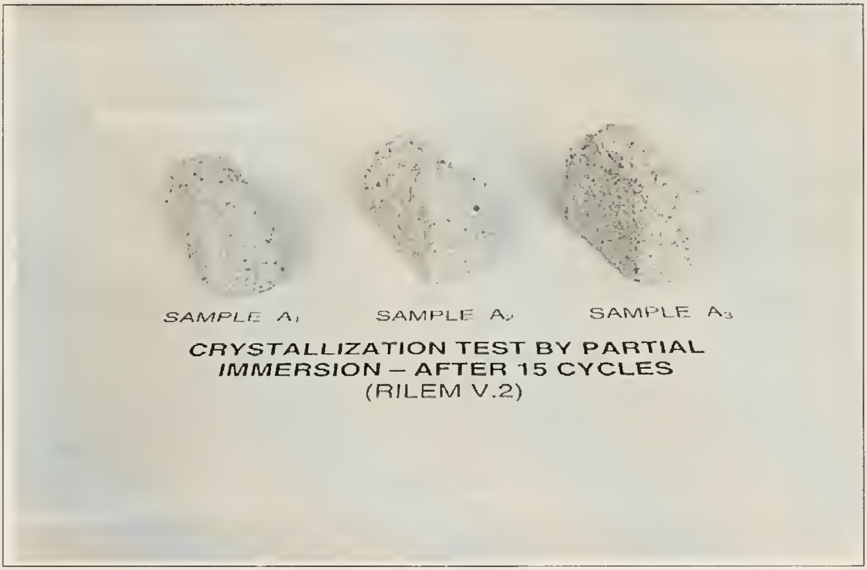

Figure 47 Salt Crystallization Test Sample A. This sample set had the highest level of deterioration mostly at the top of the sample. The capillary rise of the salt solution reached almost to the top.

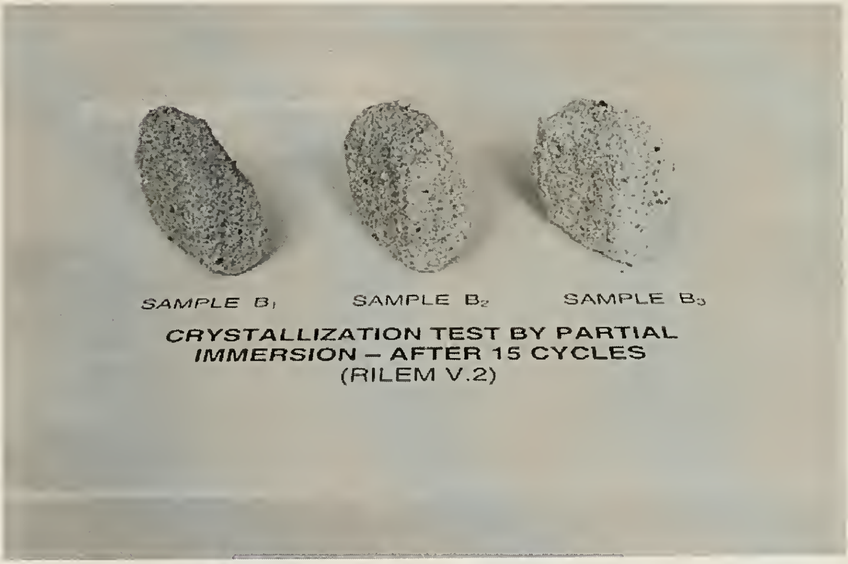

Figure 48 Salt Crystallization Test Sample B. This sample set had less deterioration indicating that Acryl® 60 increased the resistance to salt attack.

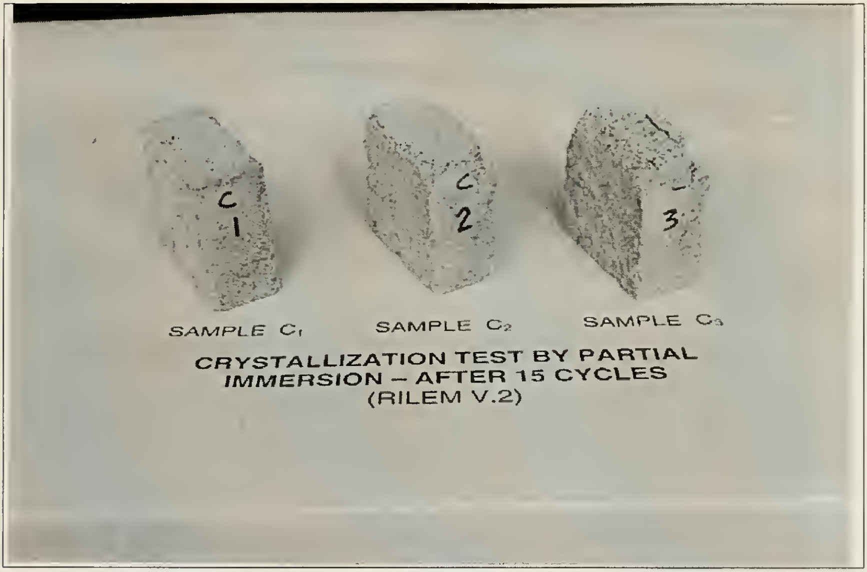

Figure 49 Salt Crystallization Test Sample C. This sample set had only minor deterioration at the edges, mostly in replicate 3.

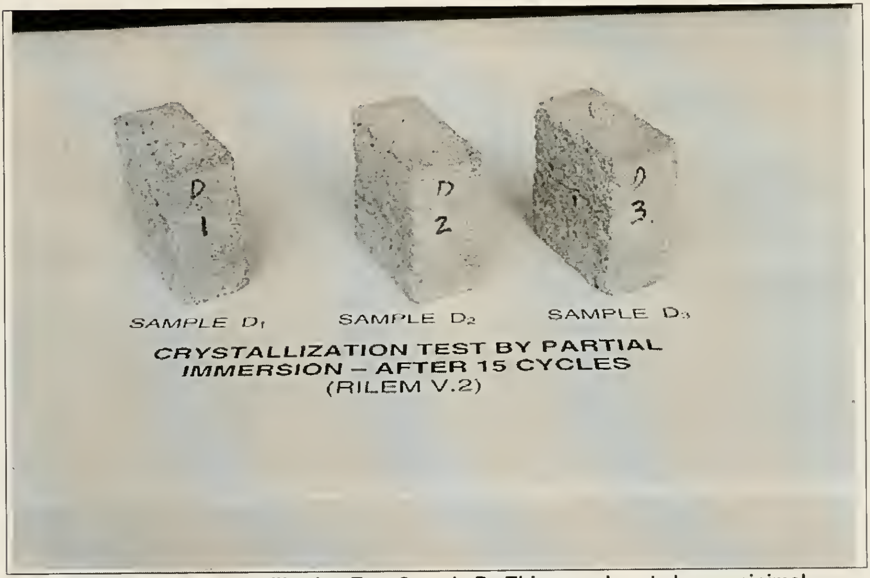

Figure 50 Salt Crystallization Test Sample D. This sample set shows minimal deterioration. There was some dissolution of binder as shown by the dark aggregate visible in replicate 3. Acryl® 60 improved the durability of the hydrated hydraulic lime.

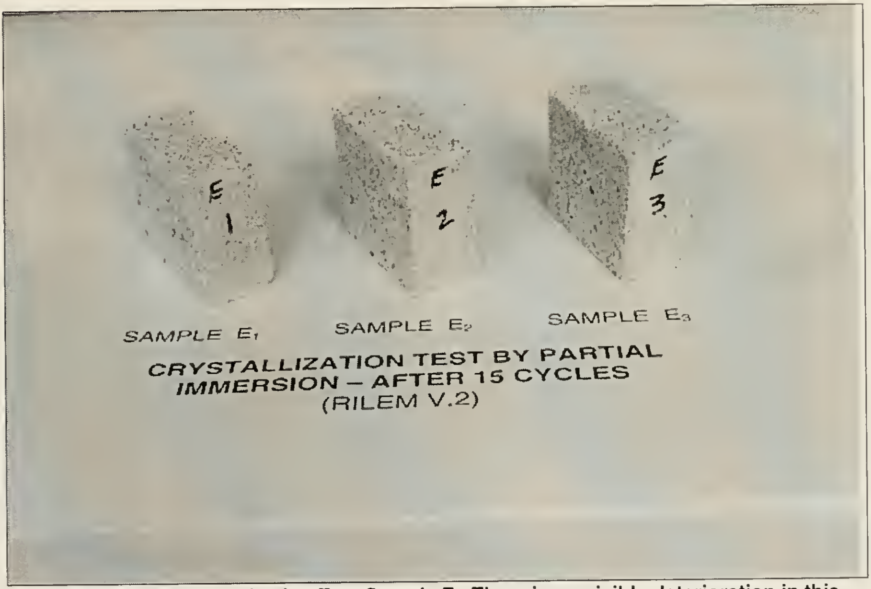

Figure 51 Salt Crystallization Test Sample E. There is no visible deterioration in this sample set.

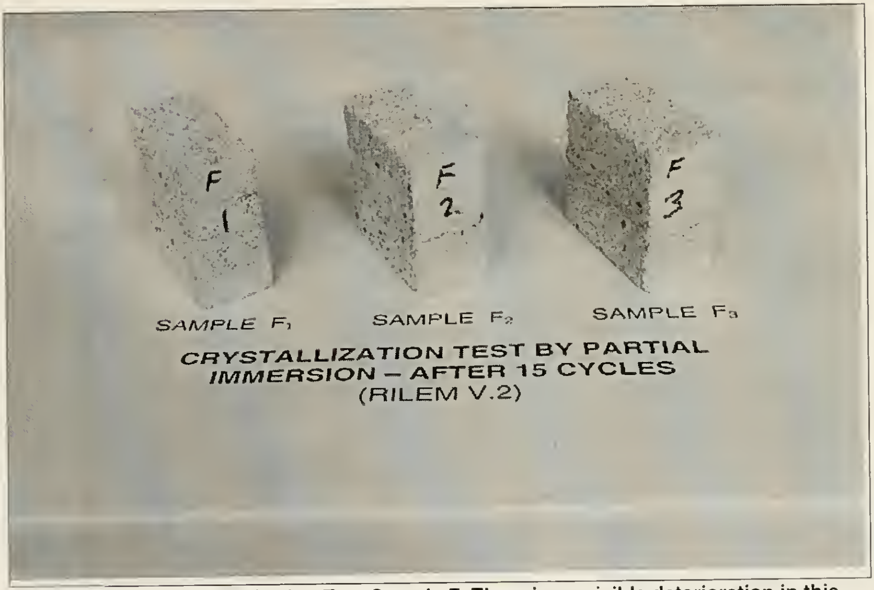

Figure 52 Salt Crystallization Test Sample F. There is no visible deterioration in this sample set, except for some darkening at the level of capillary rise at the midpoint of the samples.

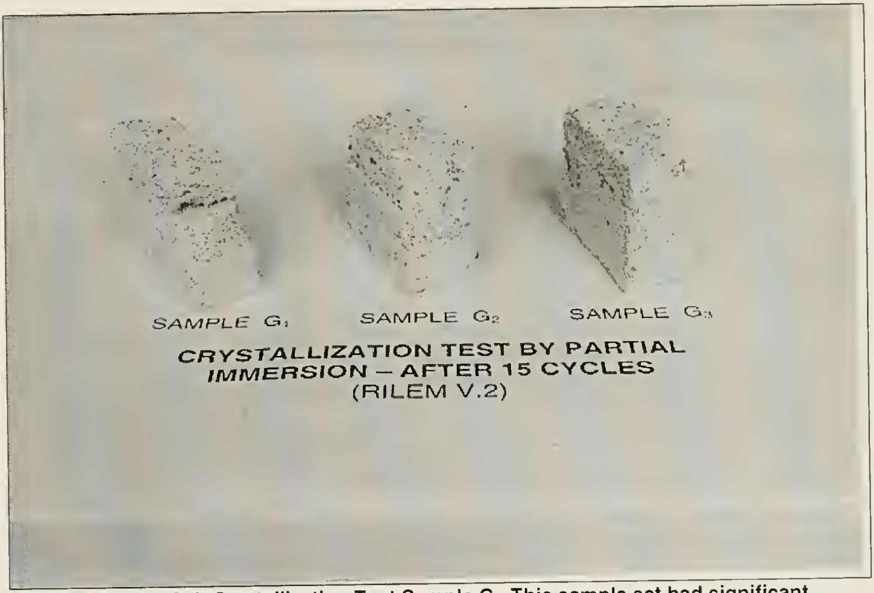

Figure 53 Salt Crystallization Test Sample G. This sample set had significant deterioration at the level of capillary rise at the midpoint of the samples. The decay has a cove pattern resulting in an hourglass shape.

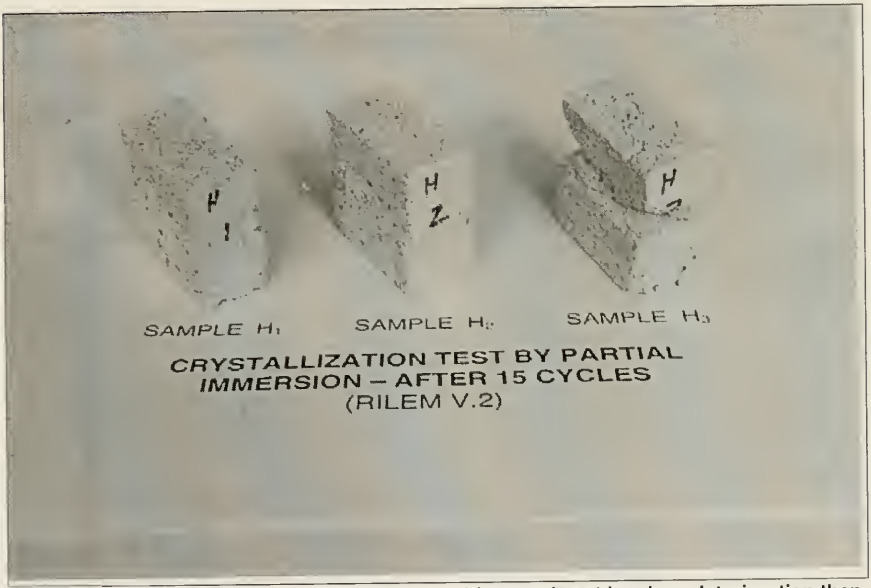

Figure 54 Salt Crystallization Test Sample H. This sample set has less deterioration than Sample G, indicating again that Acryl® 60 improves resistance to salt decay. The excessive decay of replicate 3 could be due to a line of weakness introduced during molding.

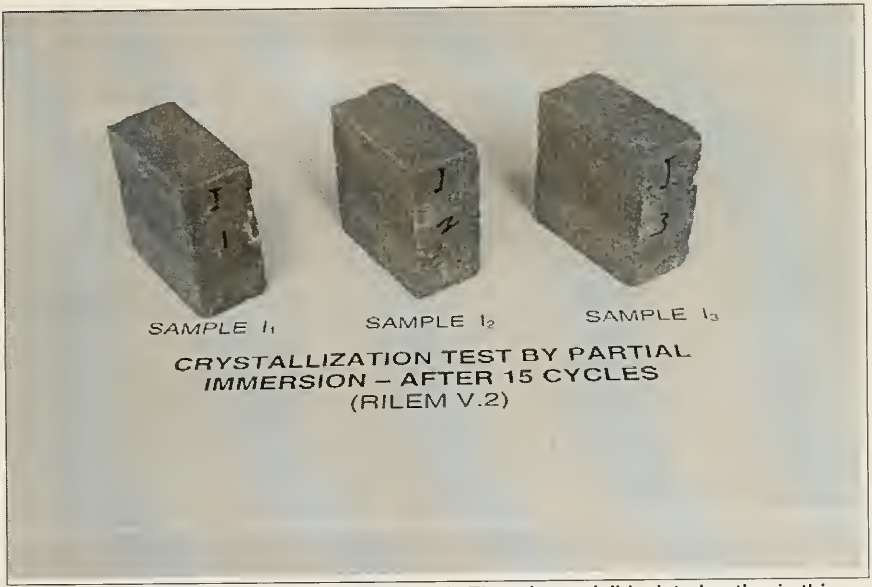

Figure 55 Salt Crystallization Test Sample I. There is no visible deterioration in this sample set. The salt remaining after the soaking and rinsing period is clearly visible.

APPENDIX C: PHOTOS OF BOND STRENGTH TEST

Figure 56 Bond Strength Test Sample A. There is a clean break at the bond line with no stone loss. About 10 % of the mortar originally in the grooves is retained on the mortar. There was good coverage during molding

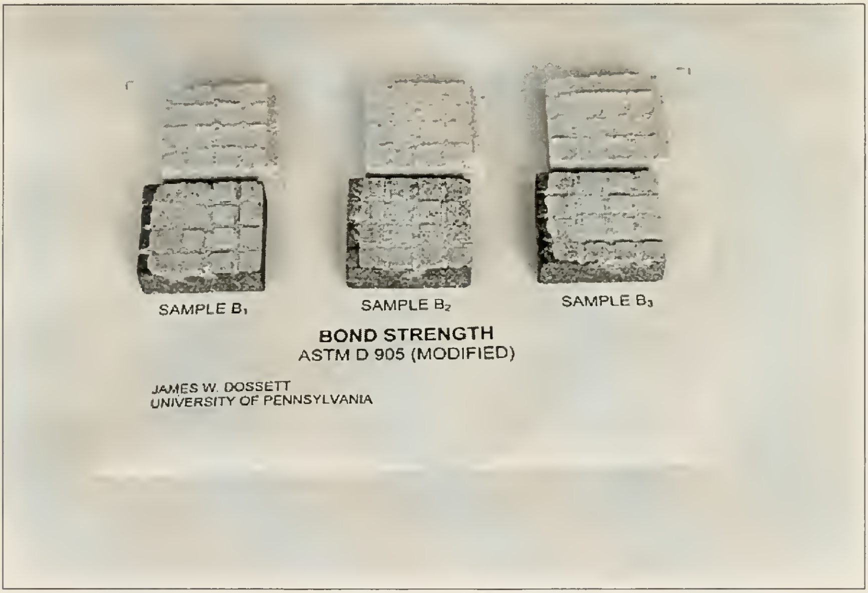

Figure 57 Bond Strength Test Sample B. There is a clean break at the bond line with no stone loss. About 25 % of the mortar originally in the grooves is retained on the mortar, indicating that the Acryl® 60 improved the cohesive strength of the mortar. There was good coverage during molding.

Figure 58 Bond Strength Test Sample C. There is a clean break at the bond line with no stone loss. About 10 % of the mortar originally in the grooves is retained on the mortar. There was good coverage during molding except for one small void (2 mm^2) near the center of each replicate. This was probably caused by air pockets not removed by tamping.

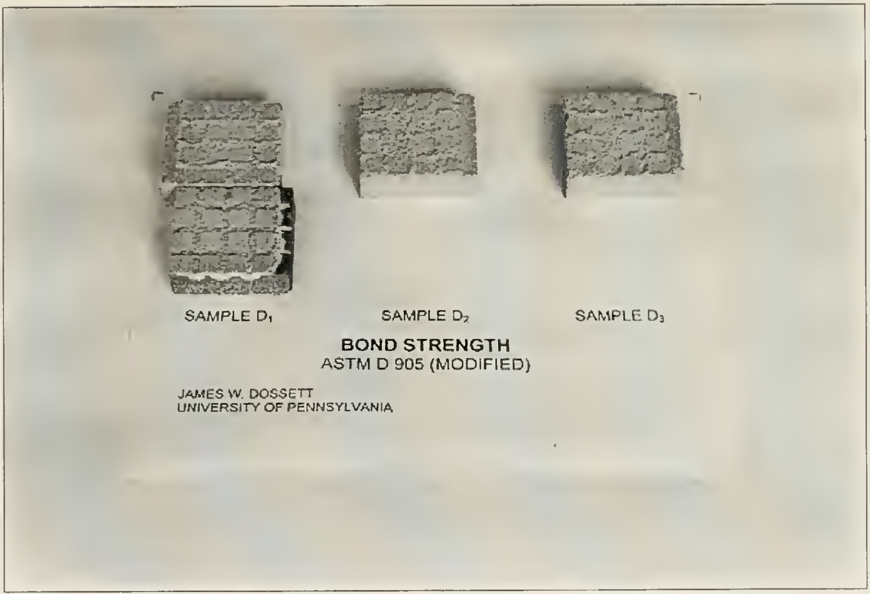

Figure 59 Bond Strength Test Sample D. There is a clean break at the bond line with no stone loss. About 10 % of the mortar originally in the grooves is retained on the mortar, the same as Sample C. There was good coverage during molding. (The stone portions of replicates 2 and 3 are not shown because they were inadvertently discarded before the picture was taken).

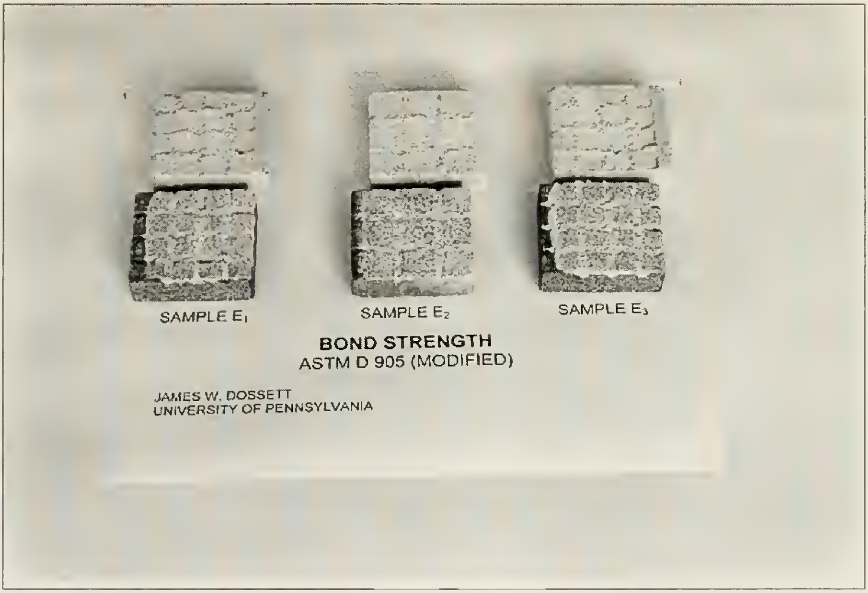

Figure 60 Bond Strength Test Sample E. There is a clean break at the bond line with no stone loss. About 10 % of the mortar originally in the grooves is retained on the mortar. There was good coverage during molding.

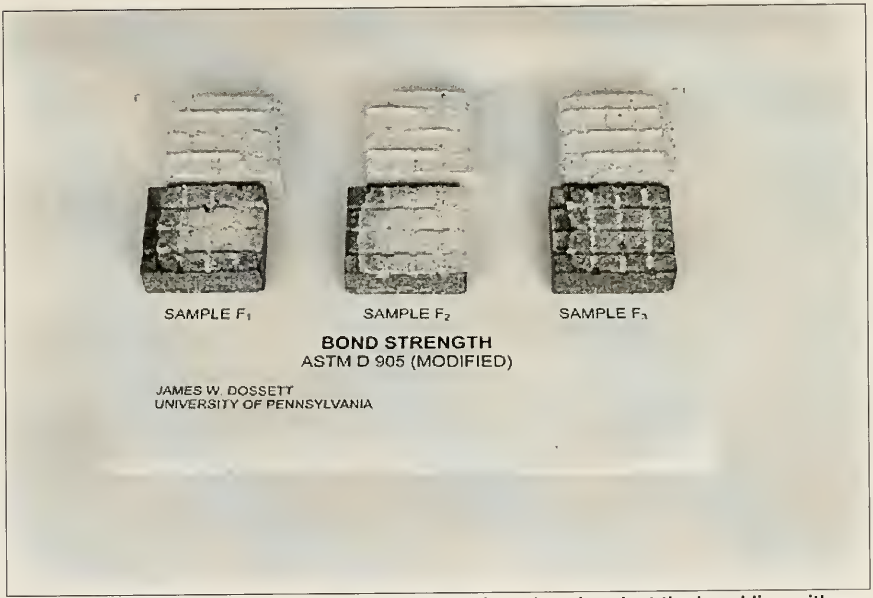

Figure 61 Bond Strength Test Sample F. There is a clean break at the bond line with a very small (less than 1 mm^3 size particle) stone loss. The loss occurred at the intersection of 2 groves and is probably due to cutting. About 40 % of the mortar originally in the grooves is retained on the mortar. Note that the mortar retained is parallel to the line of shear. The surface of the stone is covered but there is a thicker coating on replicate 2. The difference could be due to inconsistent molding or to differences in the stone surface.

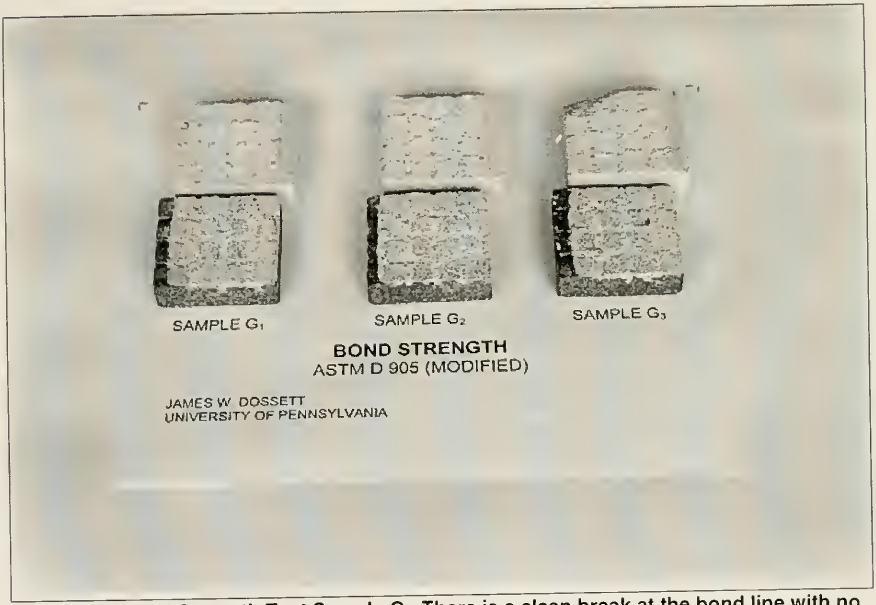

Figure 62 Bond Strength Test Sample G. There is a clean break at the bond line with no stone loss. About 10 % of the mortar originally in the grooves is retained on the mortar. The corner of the mortar in replicate 3 was dislodged during the test. There was good coverage.

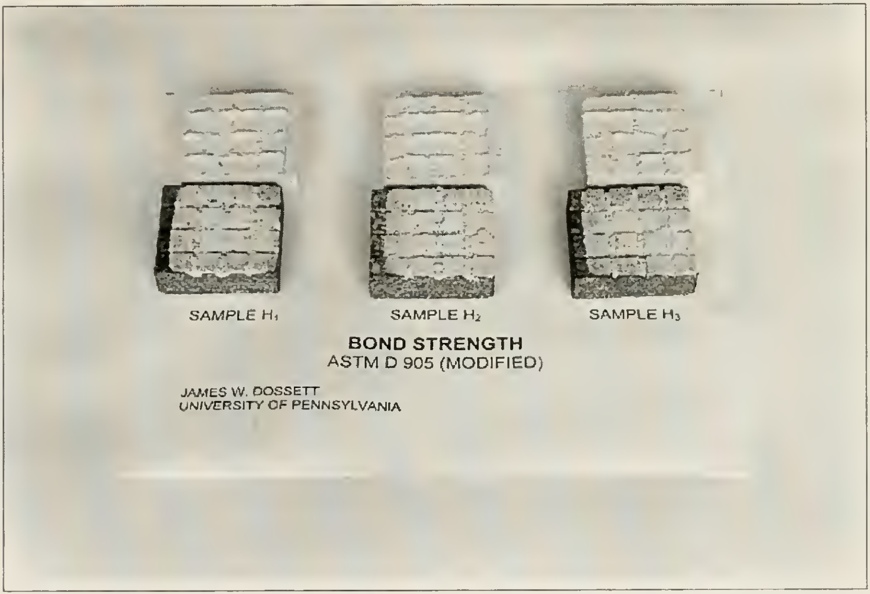

Figure 63 Bond Strength Test Sample H. There is a clean break at the bond line with no stone loss. About 45 % of the mortar originally in the grooves is retained on the mortar. Again note the direction of the retained grooved mortar - parallel to the direction of shear. There was good coverage.

Figure 64 Bond Strength Test Sample I. This sample set has the most difference between replicates. Replicate 1 broke cleanly at the bond line with almost none of the mortar in the grooves retained on the mortar. Replicate 2 broke mostly at the bond line but left 25% of the stone surface still covered with mortar indicating that the bond strength was greater than the cohesive strength of the mortar. Replicate 3 left 10% of the stone still covered with mortar.

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