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Concrete Repair Guide

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This document provides guidance on the selection and application of materials and methods for the repair, protection, and strengthening of concrete structures. An overview of materials and methods is presented as a guide for making a selection for a particular application. References are provided for obtaining in-depth information on the selected materials or methods.

Keywords: anchorage; cementitious; coating; concrete; joint sealant; placement; polymer; reinforcement; repair.

CONTENTS

Exhibit 22

CHAPTER 1—INTRODUCTION

1.1—Use of this document

This document provides guidance on selection of materials and application methods for the repair, protection, and strengthening of concrete structures. The information is applicable to repairing damaged or deteriorated concrete structures, correcting design or construction deficiencies, or upgrading a structure for new uses or to meet more restrictive building codes.

This guide summarizes current practices in concrete repair and provides sufficient information for the initial planning of repair work and for selecting suitable repair materials and application methods for specific conditions. Many of the topics covered in this guide are more extensively covered in other ACI committee documents. Readers of this guide should refer to the appropriate documents of other ACI committees and other industry resources for additional information.

1.2—Definitions

corrosion—destruction of metal by chemical, electrochemical, or electrolytic reaction within its environment.

dampproofing—treatment of concrete or mortar to retard the passage or absorption of water, or water vapor, either by application of a suitable admixture or treated cement, or by use of preformed film, such as polyethylene sheets, placed on grade before placing a slab.

excavation—steps taken to remove deteriorated concrete, sound concrete, or both, designated for removal.

nonstructural repair—a repair that addresses local deterioration and is not intended to affect the structural capacity of a member.

protection—the procedure of shielding the concrete structure from environmental and other damage for the purpose of preserving the structure or prolonging its useful life.

repair—to replace or correct deteriorated, damaged, or faulty materials, components, or elements of a structure.

repair systems—the combination of materials and techniques used in the repair of a structure.

strengthening—the process of restoring the capacity of damaged components of structural concrete to its original design capacity, or increasing the strength of structural concrete.

structural repair—a repair that re-establishes or enhances the structural capacity of a member.

surface preparation—steps taken after removal of deteriorated concrete, including conditioning of the surface of substrate at bond line and the cleaning of existing reinforcing steel.

waterproofing—prevention of the passage of water, in liquid form, under hydrostatic pressure.

1.3—Repair methodology

A basic understanding of the causes of concrete deficiencies is essential to perform meaningful evaluations and successful repairs. If the cause of a deficiency is understood, it is much more likely that an appropriate repair system will be selected and, consequently, that the repair will be successful and the maximum life of the repair will be obtained.

Symptoms or observations of a deficiency should be differentiated from the actual cause of the deficiency, and it is imperative that causes and not symptoms be dealt with wherever possible or practical. For example, cracking can be a symptom of distress that may have a variety of causes such as drying shrinkage, thermal cycling, accidental overloading, corrosion of embedded metal, or inadequate design or construction. Only after the cause or causes of deficiency are determined can rational decisions be made regarding the selection of a proper repair system and the implementation of the repair process (Fig. 1.1).

1.3.1 *Condition evaluation*—The first step in concrete repair is to evaluate the current condition of the concrete structure. This evaluation may include a review of available design and construction documents, structural analysis of the structure in its deteriorated condition, a review of available test data, a review of records of any previous repair work, review of maintenance records, a visual inspection of the structure, an evaluation of corrosion activity, destructive and nondestructive testing, and review of laboratory results from chemical and petrographic analysis of concrete samples. Upon completion of this evaluation step, the personnel responsible for the evaluation should have a thorough understanding of the condition of the concrete structure and be able to provide insights into the causes of the observed deterioration or distress. Additional information on conducting surveys can be found in ACI 201.1R, 222R, 224.1R, 228.2R, 364.1R, and 437R.

1.3.2 *Determination of causes of deterioration or distress*—After the condition evaluation of a structure has been completed, the deterioration mechanism that caused the behavior to evaluate and design a structural repair, strengthening procedure, or both. Some design considerations follow and are discussed throughout this guide.

1.4.1 *Current load distribution*—In a deteriorated state, a structural member or system distributes dead and live loads differently than first assumed when the structure was new. Cracking, deteriorated concrete and corroded reinforcement alter the stiffness of members, which leads to changes in shear, moment, and axial load distribution. As concrete and reinforcement are removed and replaced during the repair operation, these redistributed forces are further modified. To understand the final behavior of the structural system, the engineer should evaluate the redistribution of the forces. To fully re-establish the original load distribution, a member should be relieved of the load by jacking or other means. The repaired member and the repair itself supports the loads differently than would be assumed in the original or a new structure.

1.4.2 *Compatibility of materials*—If a repair and the member have the same stiffness—for example, modulus of elasticity—the analysis of the repaired member may be the same as a new section. If the stiffness varies, however, then the composite nature of the repaired system should be considered. A mismatch of other material characteristics further exacerbates the effects of thermal changes, vibrations, and long-term creep and shrinkage effects. Different coefficients of thermal expansion of the repair and original material results in different dimensional changes. The engineer should design for the different movements, or the repair system should be similar to the thermal and dimensional characteristics of the original material.

1.4.3 *Creep, shrinkage, or both*—Reduction in length, area, or volume of both the repair and original materials due to creep, shrinkage, or both, affect the structures serviceability and durability. As an example, compared with the original material, high creep or shrinkage of repair materials results in loss of stiffness of the repair, redistributed forces, and increased deformations. The engineer should consider these effects in the design.

1.4.4 *Vibration*—When the installed repair material is in a plastic state or until adequate strength has been developed, vibration of a structure can result in reduced bonding of the repair material. Isolating the repairs or eliminating the vibration may be a design consideration.

1.4.5 *Water and vapor migration*—Water or vapor migration through a concrete structure can degrade a repair. Understanding the cause of the migration and controlling it should be part of a repair design consideration.

1.4.6 *Safety*— The contractor is responsible for construction safety. Nevertheless, as the engineer considers a repair design, which may involve substantial concrete removal, steel reinforcing cutting, or both, he or she should notify the contractor of the need and extent of shoring and bracing. The local repair of one small section can affect a much larger area, of which the contractor may not be aware.

1.4.7 *Material behavior characteristics*—When new and innovative materials and systems are used for repair and strengthening, the structural behavior of the repaired section can differ substantially from the behavior of the original section. For example, if a beam's steel reinforcement has corroded extensively and lost part of its load-carrying capacity, the steel reinforcement may be replaced by carbon fiber-reinforced polymer (CFRP) applied to the external bottom face of the beam. The original yielding behavior of the steel bar is replaced by FRP that is stronger, but has a more elastic and brittle behavior. The behavior assumptions of codes like ACI 318 are no longer valid. The engineer should consider the behavior and performance of the new repair under the actual service and ultimate load, and design the repair to provide at least an equivalent level of safety to the original design. Such a design is outside the scope of ACI 318.

1.5—Format and organization

Chapter 2 discusses removal of deteriorated concrete, preparation of surfaces to receive repair materials, general methods for concrete repair, and repair techniques for reinforcing and prestressing steel. Chapter 3 discusses various types of repair materials that may be used. The reader is urged to use Chapters 2 and 3 in combination when selecting the repair material and method for a given situation. Chapter 4 describes materials and systems that may be used to protect repaired or unrepaired concrete from deterioration. Chapter 5 provides methods for strengthening an existing structure when repairing deficiencies, improving load-carrying capabilities, or both. Chapter 6 provides references, including other appropriate ACI documents and industry resources.

CHAPTER 2—CONCRETE REMOVAL, PREPARATION, AND REPAIR TECHNIQUES

2.1—Introduction and general considerations

This chapter covers removal, excavation, or demolition of existing deteriorated concrete, preparation of the concrete surface to receive new material, preparation and repair of reinforcement, methods for anchoring repair materials to the existing concrete, and various methods that are available to place repair materials. The care that is exercised during the removal and preparation phases of a repair project can be the most important factor in determining the longevity of the repair, regardless of the material or technique used.

Specific attention should be given to the removal of concrete around prestress strands, both bonded and unbonded. The high-energy-impact tools, such as chipping hammers, should avoid contact with the strand because this will reduce the strands' load-carrying capacity and may cause the wire(s) to rupture, which may lead to strand failure.

2.2—Concrete removal

A repair project usually involves removal of deteriorated, damaged, or defective concrete. In most concrete repair projects, the zones of damaged concrete are not well defined. Most references state that all damaged or deteriorated material should be removed, but it is not always easy to determine when all such material has been removed or when too much good material has been removed. A common recommendation is to remove sound concrete for a defined distance beyond the delaminated area; thereby, exposing the reinforcing steel beyond the point of corroded steel.

Removal of concrete using explosives or other aggressive methods can damage the concrete that is intended to remain in place. For example, blasting with explosives or the use of some impact tools heavier than 12 kg (30 lb) can result in additional delamination or cracking. Delaminated areas can be identified by using a hammer to take soundings. In most cases, such delaminations should be removed before repair materials are placed.

Removal of concrete using impact tools may result in small-scale microcracking damage (termed bruising) to the surface of the concrete left in place. Unless this damaged layer is removed, a weakened plane may occur in the parent concrete below the repair material bondline. This condition can result in a low tensile rupture (bond) strength between the parent concrete and repair material. Thus, a perfectly sound and acceptable replacement material may fail due to improper surface preparation. All damaged or delaminated concrete, including bruising, at the interface of the repair and the parent concrete should be removed before placing the repair material. This may require one type of aggressive removal for gross removal followed by another type of removal for bruising.

In all cases in which concrete has been removed from a structure by primary means such as blasting or aggressive impact methods, the concrete left in place should be prepared by using a secondary method, such as chipping, abrasive blasting, or high-pressure water jetting to remove any remaining damaged surface material. Careful visual inspections of the prepared surfaces should be conducted before placing repair materials. Wetting the surface may help to identify the presence of cracking. Determination of the tensile strength (ACI 503R, Appendix A) by pull-off testing is advisable on prepared surfaces to determine the suitability of the surface to receive repair material.

Removal of limited areas of concrete in a slab, wall, or column surface requires saw-cutting the perimeter of the removal area, providing an adequate minimum thickness of repair material at the edge of the repaired area, and mitigating the advancement of undetected incipient cracking. Feathering of repair materials generally should be avoided. The preparation for shotcreting is an exception. ACI 506R recommends tapered edges around the perimeter of such patches. Saw cutting can also improve the appearance of the repaired area. The general shape of the repaired areas should be as symmetrical as possible (ICRI 03730). Reentrant corners should be avoided. Large variations in the depth of removal in short distances should also be avoided. The texture of the prepared surface should be appropriate for the intended repair material (ICRI 03732).

Every precaution should be made to avoid cutting underlying reinforcement. Reviewing design drawings and using a covermeter or similar device provides data as to the location and depth of reinforcement. In addition, the removal of small areas of concrete is commonly used to confirm the location and depth of bars before saw cutting.

Sections 2.2.1 through 2.2.18 present descriptions of many of the commonly used concrete removal techniques to help in the selection process.

2.2.1 *General considerations*—Concrete removal addresses deteriorated and damaged material. Some sound concrete, however, may be removed to permit structural modifications and to ensure that all unsound material is removed. The effectiveness of various removal techniques can differ for deteriorated and sound concrete. Some techniques may be more effective in sound concrete, while others may work better for deteriorated concrete.

Concrete removal techniques selected should be effective, safe, economical, environmentally friendly, and minimize damage to the concrete left in place. The removal technique chosen may have a significant effect on the length of time that a structure will be out of service. Some techniques permit a significant portion of the work to be accomplished without removing the structure from service. The same removal technique, however, may not be suitable for all portions of a given structure. In some instances, a combination of removal techniques may be more appropriate to speed removal and limit damage to the remaining sound concrete. Trial field testing various removal techniques can help confirm the best procedures.

In general, the engineer responsible for the design of the repair should specify the objectives to be achieved by the concrete removal, and the repair contractor should be allowed to select the most economical removal method, subject to the engineer's acceptance. In special circumstances, the engineer may also need to specify the removal techniques to be used and those that are prohibited.

The mechanical properties of the concrete and the type and size of aggregate to be removed provide important information to determine the method and cost of concrete removal. The mechanical properties include compressive and tensile strengths. This information is also necessary for the engineer to specify the prepared surface condition and select the repair material, and it should be made available to contractors for bidding purposes.

2.2.2 *Monitoring and shoring during removal operations*—It is essential to evaluate the removal operations to limit the extent of damage to the structure and to the concrete that remains. Structural elements may require shoring, removal of applied loads, or both, before concrete removal to prevent structural deformations, possible collapse, buckling, or slippage of reinforcement. Care should be used during removal of concrete to avoid cutting and damaging reinforcing steel. Because reinforcement is often misplaced, unanticipated damage may occur when saw cutting, impacting, or removing concrete.

Careful monitoring is required throughout the concrete removal operation. This can be accomplished by visual inspection, sounding, use of a covermeter, or other means to locate reinforcement. The project specifications should assign responsibilities for the inspection of the prepared concrete.

Sounding is an excellent means to detect delaminated concrete adjacent to the outermost layers of reinforcing steel. Subsurface cracks, the extent of deterioration, or other internal defects, however, may not be identified by this method alone. Other means of evaluation should be used to properly identify the extent of concrete to be removed. In addition, sounding usually does not indicate near-surface microcracking or bruising. Only microscopic examination or bond testing may disclose near-surface damage.

Subsurface evaluation (examination of the substrate) can provide valuable information about the condition of the concrete. This information may be obtained by the following methods before, during, or after concrete removal (ACI 228.2R):

a) Taking cores for visual examination, microscopic examination, compressive strength tests, and splitting-tensile strength tests;

b) Pulse-velocity tests;

c) Impact-echo tests;

d) Bond tests (pull-off testing, ACI 503R Appendix A);

e) Covermeter or similar equipment to locate reinforcement and determine its depth below the surface;

f) Infrared thermography; and

g) Ground-penetrating radar (GPR).

Many of these methods are discussed in ACI 228.2R.

2.2.3 *Quantity of concrete to be removed*—In most repair projects, all damaged or deteriorated concrete should be removed; however, the quantity of concrete to be removed is directly related to the elapsed time between preparation of the estimate and actual removal. Substantial overruns are common. Estimating inaccuracies can be minimized by a thorough condition survey as close as possible to the time the repair work is executed. Potential quantity overruns, based on field-measured quantities, should be taken into account. When, by necessity, the condition survey is done far in advance of the repair work, the estimated quantities should be increased to account for continued deterioration. Because most concrete repair projects are based on unit prices, repair areas should be accurately measured before forms are installed. This is usually done jointly by the engineer and the contractor. It is not uncommon for estimated quantities to increase significantly between the detectable quantities and the actual quantity removed. ICRI 03735 provides guidelines for methods of measurement for concrete repair work.

2.2.4 *Classification of concrete removal methods*—Removal and excavation methods can be categorized by the way in which the process acts on the concrete. These categories are blasting, cutting, impacting, milling, hydrodemolition, presplitting, and abrading. Table 2.1 provides a general description of these categories, lists the specific removal techniques within each category, and provides a summary of information on each technique. The techniques are discussed in detail in the following sections.

2.2.5 *Blasting methods*—Blasting methods generally employ rapidly expanding gas confined within a series of bore holes to produce controlled fracture and removal of the concrete. The only blasting method addressed in this report is explosive blasting.

Explosive blasting is the most cost-effective and expedient means for removing large quantities of concrete—for example, portions of large mass concrete foundations or walls. This method involves drilling bore holes, placing an explosive in each hole, and detonating the explosive. Controlled-blasting techniques minimize damage to the material that remains after blasting. One such technique, cushion blasting, involves drilling a line of 75 mm (3 in.) diameter or smaller bore holes parallel to the removal face, loading each hole with light charges of explosive (usually detonating cord) distributed along its length, cushioning the charges by stemming each hole completely or in the collar with wet sand, and detonating the explosive with electric blasting caps. The uniform distribution and cushioning of the light charges produce a relatively sound surface with little overbreak.

Blasting machines and electrical blasting-cap delay series are also used for controlled demolition and employ proper timing sequences to provide greater control in reducing ground vibration. Controlled blasting has been used successfully on numerous repair projects. The selection of proper charge weight, borehole diameter, and borehole spacing for a repair project depends on the location of the structure, the acceptable degree of vibration and damage, and the quantity and quality of concrete to be removed. If at all possible, a pilot test program should be implemented to determine the optimum parameters. Because of the inherent dangers in the handling and usage of explosives, all phases of the blasting project should be performed by qualified, appropriately licensed personnel with proven experience and ability.

2.2.6 *Cutting methods*—Cutting methods generally employ mechanical sawing, intense heat, or high-pressure water jets to cut around the perimeter of concrete sections to permit their removal. The size of the sections that are cut free is governed by the available lifting and transporting equipment. The cutting methods include high-pressure water jets, saw cutting, diamond wire cutting, mechanical shearing, stitch drilling, and thermal cutting.

a) *High-pressure water jet (without abrasives)*—A high-pressure water jet uses a small jet of water driven at high velocities, commonly producing pressures of 69 to 310 MPa (10,000 to 45,000 psi). A number of different types of water jets are currently being used. The most promising are the ultra high-pressure jet and the cavitating jet. Section 2.2.10 describes using a water jet as a primary removal method. Water jets used with abrasives are described in Section 2.2.11.

b) *Saw cutting*—Diamond or carbide saws are available in sizes ranging from small (capable of being hand-held) to large (capable of cutting depths of up to 1.3 m [52 in.]).

c) *Diamond wire cutting*—Diamond wire cutting is accomplished with a wire containing nodules impregnated with diamonds. The wire is wrapped around the concrete mass to be cut and reconnected with the power pack to form a continuous loop. The loop is spun in the plane of the cut while being drawn through the concrete member. This system can be used to cut a structure of any size as long as the wire can be wrapped around the concrete. The limits of the power source determines the size of the concrete structure that can be cut. This system provides an efficient method for cutting up and dismantling large or small concrete structures.

d) *Mechanical shearing*—The mechanical shearing method employs hydraulically powered jaws to cut concrete and reinforcing steel. This method is applicable for making cutouts through slabs, decks, and other thin concrete

Table 2.1— Summary of features and considerations/limitations for concrete removal methods

Category	Features	Considerations/Limitations
2.2.5 *Blasting* Uses rapidly expanding gas confined within a series of boreholes to produce controlled fracture and removal of concrete.	*Explosives* Most expedient method for removing large volumes where concrete section is 10 in. (250 mm) thick or more. Produces good fragmentation of concrete debris for easy removal.	Requires highly skilled personnel for design and execution. Stringent safety regulations must be complied with regarding the transportation, storage, and use of explosives due to their inherent dangers. Blast energy must be controlled to avoid damage to surrounding improvements resulting from air blast pressure, ground vibration, and flying debris.
2.2.6 *Cutting* Uses perimeter cuts to remove large pieces of concrete.	*High-pressure water jet (without abrasives)* Applicable for making cutouts through slabs, decks, and other thin concrete members. Cuts irregular and curved shapes. Makes cutouts without overcutting corners. Cuts flush with intersecting surfaces. No heat, vibration, or dust is produced. Handling of debris is efficient because bulk of concrete is removed in large pieces.	Cutouts for removal limited to thin sections. Cutting is typically slower and more costly than diamond blade sawing. Moderate levels of noise may be produced. Controlling flow of waste water may be required. Additional safety precautions are required due to the high water pressure produced by the system.
2.2.6 *Cutting* (continued)	*Diamond saw* Applicable for making cutouts through slabs, decks, and other thin concrete members. Makes precision cuts. No dust or vibration is produced. Handling of debris is efficient because bulk of concrete is removed in large pieces.	Cutouts for removal limited to thin sections. Performance is affected by type of diamonds and the diamond-to-metal bond in blade segments (segment selection is based on aggregate hardness). The higher the percentage of steel reinforcement in cuts, the slower and more costly the cutting. The harder the aggregate, the slower and more costly the cutting. Controlling flow of waste water may be required.
2.2.6 *Cutting* (continued)	*Diamond wire cutting* Applicable for cutting large and/or thick pieces of concrete. The diamond wire chain can be infinitely long. No dust or vibration is produced. Large blocks of concrete can be easily lifted out by a crane or other mechanical methods. The cutting operation can be equally efficient in any direction.	The cutting chain must be continuous. Access to drill holes through the concrete must be available. Water must be available to the chain. Controlling the flow of waste water may be required. The harder the aggregate and/or concrete, the slower and more costly the cutting. Performance is affected by the quality, type, and number of diamonds as well as the diamond-to-metal bond in the chain.
2.2.6 *Cutting* (continued)	*Mechanical shearing* Applicable for making cutouts through slabs, decks, and other thin concrete members. Steel reinforcement can be cut. Limited noise and vibration are produced. Handling of debris is efficient because bulk of concrete is removed in large pieces.	Limited to thin sections where an edge is available or a hole can be made to start the cut. Exposed reinforcing steel is damaged beyond reuse. Remaining concrete is damaged. Extremely rugged profile is produced at the cut edge. Ragged feather edges remain after removal.
2.2.6 *Cutting* (continued)	*Stitch drilling* Applicable for making cutouts through concrete members where access to only one face is feasible. Handling of debris is more efficient because bulk of concrete is removed in large pieces.	Rotary-percussion drilling is significantly more expedient and economical than diamond core drilling; however, it results in more damage to the concrete that remains, especially at the point of exit from the concrete. Depth of cuts is dependent on accuracy of drilling equipment in maintaining overlap between holes with depth and diameter of the boreholes drilled. The deeper the cut, the greater borehole diameter required to maintain overlap between adjacent holes and the greater the cost. Uncut portions between adjacent boreholes will hamper or prevent the removal. Cutting reinforced concrete increases the cutting time and increases the cost. Aggregate toughness for percussion drilling and aggregate hardness for diamond coring will affect cutting cost and rate. Personnel must wear hearing protection due to high noise levels.

members. It is especially applicable where total demolition of the member is desired. The major limitation of this method is that cuts should be started from free edges or from holes made by hand-held breakers or other means.

e) *Stitch drilling*—The stitch-drilling method employs the use of overlapping boreholes along the removal perimeter to cut out sections for removal. This method is applicable for making cutouts through concrete members where access to only one face is possible, and the depth of cut is greater than can be economically cut by the diamond-blade method. The primary drawback of stitch drilling is the potential for costly removal complications if the cutting depth exceeds the accuracy of the drilling equipment, so that uncut concrete remains between adjacent holes.

f) *Thermal cutting*—This method requires powder torch, thermal lance, and powder lance, which develop intense heat generated by the reaction between oxygen and powdered metals to melt a slot into concrete. The thermal device's ability for removing concrete from structures mainly depends on the rate at which the resulting slag can flow from the slot. These devices use intense heat and are especially effective for cutting reinforced concrete; however, they are considered slow, relatively expensive, and are not widely used.

Table 2.1 (cont.)—Summary of features and considerations/limitations for concrete removal methods

Category	Features	Considerations/Limitations
2.2.6 *Cutting* (continued)	*Thermal cutting* Applicable for making cutouts through heavily reinforced decks, beams, walls, and other thin to medium concrete members. An effective means of cutting reinforced concrete. Cuts irregular shapes. Produces minimal noise, vibrations, and dust.	Limited availability commercially. Not applicable for cuts where slag flow is restricted. Remaining concrete has thermal damage with more extensive damage occurring around steel reinforcement. Produces smoke and fumes. Personnel must be protected from heat and hot slag produced by cutting operation.
2.2.7 *Impacting* Uses repeated striking of the surface with a mass to fracture and spall the concrete.	*Hand-held breakers* Applicable for limited volumes of concrete removal. Applicable where blow energy must be limited. Widely available commercially. Can be used in areas of limited work space. Produces relatively small and easily handled debris.	Performance is a function of concrete soundness and aggregate toughness. Significant loss of productivity occurs when breaking action is other than downward. Removal boundaries will likely require saw cutting to avoid feathered edges. Concrete that remains may be damaged (microcracking). Produces high levels of noise, dust, and vibration.
	Boom-mounted breakers Applicable for full-depth removal from slabs, decks, and other thin concrete members and for surface removal from more massive concrete structures. Can be used for vertical and overhead surfaces. Widely available commercially. Produces easily handled debris.	Blow energy delivered to the concrete may have to be limited to protect the structure being repaired and the surrounding structures from damage due to high cyclic energy generated. Performance is a function of concrete soundness and aggregate toughness. Damages remaining concrete. Damages reinforcing steel. Produces feathered edges. Produces high level of noise and dust.
2.2.7 *Impacting* (continued)	*Scabblers* Low initial cost. Can be operated by unskilled labor. Can be used in areas of limited work space. Removes deteriorated concrete from wall or floor surfaces efficiently. Readily available commercially.	High cyclic energy applied to a structure will produce fractures in the remaining concrete surface area. Produces high level of noise and dust. Limited depth removal.
2.2.8 *Milling* Uses scarifiers to remove concrete surfaces.	*Scarifier* Applicable for removing deteriorated concrete surfaces from slabs, decks, and mass concrete. Boom-mounted cutters are applicable for removal from wall and ceiling surfaces. Removal profile can be controlled. Method produces relatively small and easily handled debris.	Removal is limited to concrete without steel reinforcement. Sound concrete significantly reduces the rate of removal. Can damage concrete that remains (microcracking). Noise, vibration, and dust are produced.
2.2.9 *Hydrodemolition* Uses high-pressure water to remove concrete.	Applicable for removal of deteriorated concrete from surfaces of bridges and parking decks and other deteriorated surfaces where removal depth is 6 in. (150 mm) or less. Does not damage the concrete that remains. Steel reinforcing is left clean and undamaged for reuse. Method produces easily handled, aggregate-sized debris.	Productivity is significantly reduced when sound concrete is being removed. Removal profile will vary with changes in depth of deterioration. Method requires large source of potable water to meet water demand. Waste water may have to be controlled. An environmental impact statement may be required if waste water is to enter a waterway. Personnel must wear hearing protection due to the high level of noise produced. Flying debris is produced. Additional safety requirements are required due to the high pressures produced by these systems.

2.2.7 *Impacting methods*—Impacting methods are the most commonly used concrete removal systems. They repeatedly strike a concrete surface with a high-energy tool or a large mass to fracture and spall the concrete. The use of these methods in partial-depth concrete removal can result in microcracking on the surface of the concrete left in place. Extensive microcracking results in a weakened plane below the bond line. Currently, the committee is unable to provide definitive guidelines to prevent such damage when using impact methods; however, factors such as the weight and size of the equipment should be considered to minimize microcracking. Determination of the tensile strength by pull-off testing is recommended to determine the suitability of the surface to receive repair materials. Additionally, after impacting secondary methods, such as sandblasting, abrasive blasting, and water blasting, may be required to remove excessive microcracking.

a) *Hand-held breakers*—The hand-held breaker or chipping hammer is probably the best known of all concrete removal devices. Hand-held breakers are available in various sizes with different levels of energy and efficiency. These tools are generally defined by weight and vary in size from 3.5 to 41 kg (8 to 90 lb). (Note: the larger the hammer, such as 14 kg [30 lb] and larger, the greater the potential for microcracking.) The smaller hand-held breakers, such as 7 kg (15 lb) and smaller, are used in partial removal of unsound concrete or concrete around reinforcing steel because they do little damage to surrounding concrete. Larger breakers are used for complete removal of large volumes of concrete or delaminations. Care should be exercised when selecting the size of

(0.1 to 4 in.). Milling operations usually leave a sound surface with less microfractures than impact methods (Virginia Transportation Research Council 2001).

2.2.9 *Scarifier*—A scarifier is a concrete cutting tool that employs the rotary action and mass of its cutter bits to rout cuts into concrete surfaces. It removes loose concrete fragments (scale) from freshly blasted surfaces and removes concrete that is cracked and weakened by an expansive agent. It also is the sole method of removing deteriorated and sound concrete in which some of the concrete contains form ties and wire mesh. Scarifiers are available in a range of sizes. The scarifier is an effective tool for removing deteriorated concrete on vertical and horizontal surfaces. Other advantages include well-defined limits of concrete removal, relatively small and easily handled concrete debris, and simplicity of operation.

2.2.10 *Hydrodemolition*—High-pressure water jetting (hydrodemolition) can be used to remove concrete to preserve and clean the steel reinforcement for reuse and to minimize microcracking to the remaining in-place concrete. The method also has a high efficiency. Hydrodemolition disintegrates concrete, returning it to sand and gravel-sized pieces. This process works on sound or deteriorated concrete and leaves a rough profile. Hydrodemolition punches through the full depth of slabs in small areas when the concrete is unsound or when full-depth patches are inadequately bonded to the side walls. Hydrodemolition should not be used in structures with unbonded tendons, except under the direct supervision of a structural engineer.

High-pressure water jets in the 70 MPa (10,000 psi) range require 130 to 150 L/min (35 to 40 gal./min). As the pressure increases to 100 to 140 MPa (15,000 to 20,000 psi), the water demand varies from 75 to 150 L/min (20 to 40 gal./min) (Nittenger 1997). The equipment manufacturer should be consulted to confirm the water demand. Ultra-high-pressure equipment operating at 170 to 240 MPa (25,000 to 35,000 psi) has the capability of milling concrete to depths of 3 to 150 mm (0.1 to 6 in.). Containment and subsequent disposal of the water are requirements of the hydrodemolition process. Many localities require this water to be filtered and then treated to reduce the alkalinity and particulates before the water can be released into a storm or wastewater system.

Water jet lances operating at pressures of 70 to 140 MPa (10,000 to 20,000 psi) and having a water demand of 75 to 150 L/min (20 to 40 gal./min) are available. They are capable of cutting sections of concrete or selectively removing surface concrete in areas that are difficult to reach with larger equipment (ICRI 03737).

2.2.11 *Presplitting methods*—Presplitting methods use hydraulic splitters, water pressure pulses, or expansive chemicals used in boreholes drilled at points along a predetermined line to induce a crack plane for the removal of concrete. The pattern, spacing, and depth of the boreholes affect the direction and extent of the crack planes that propagate. Presplitting is generally used in mass concrete structures or unreinforced concrete.

a) *Hydraulic splitter*—The hydraulic splitter is a wedging device that is used in predrilled boreholes to split concrete into sections. This method has potential as a primary means for removal of large volumes of material from mass concrete structures. Secondary means of separating and handling the concrete, however, may be required where reinforcing steel is involved.

b) *Water-pulse splitter*—The water-pressure pulse method requires that the boreholes be filled with water. A device, or devices, containing a very small explosive charge is placed into one or more holes, and the explosive is detonated. The explosion creates a high-pressure pulse that is transmitted through the water to the structure, cracking the concrete. Secondary means may be required to complete the removal of reinforced concrete. This method does not work if the concrete is so badly cracked or deteriorated that it does not hold water in the drill holes.

c) *Expansive product agents*—Commercially available cementitious expansive product agents, such as those containing aluminum powder, when correctly mixed with water, exhibit a large increase in volume over a short period of time. By placing the expansive agent in boreholes located in a predetermined pattern within a concrete structure, the concrete can be split in a controlled manner for removal. This technique has potential as a primary means of removing large volumes of material from concrete structures and is best suited for use in holes of significant depth. Secondary means may be required to complete the separation and removal of concrete from the reinforcement. A key advantage to the use of expansive agents is the relatively nonviolent nature of the process and the reduced tendency to disturb the adjacent concrete.

2.2.12 *Abrading blasting*—Abrading blasting removes concrete by propelling an abrasive medium at high velocity against the concrete surface to abrade it. Abrasive blasting is typically used to remove surface contaminants and as a final surface preparation. Commonly used methods include sandblasting, shotblasting, and high-pressure water blasting.

2.2.13 *Sandblasting*—Sandblasting is the most commonly used method of cleaning concrete and reinforcing steel in the construction industry. The process uses common sands, silica sands, or metallic sands as the primary abrading tool. The process may be executed in one of three methods.

2.2.14 *Dry sandblasting*—Sands are projected at the concrete or steel in a stream of high-pressure air in the open atmosphere. The sand particles are usually angular and may range in size from passing a 212 to a 4.75 mm (No. 70 to a No. 4) sieve. The rougher the required surface condition, the larger the sand particle size.

The sand particles are propelled at the surface in a stream of compressed air at a minimum pressure of 860 kPa (125 psi). The compressor size varies, depending on the size of the sandblasting pot. Finer sands are used for removing contaminants and laitance from the concrete and loose scale from reinforcing steel. Coarser sands are commonly used to expose fine and coarse aggregates in the concrete by removing the paste or tightly bonded corrosion products from reinforcing steel. Although sandblasting has the ability to cut quite deeply into concrete, it is not economically practical to remove more than 6 mm (0.25 in.) from the concrete surface.

Exhibit 23: Hydrodemolotion of Concrete Surfaces and Reinforced Concrete

Hydrodemolition of Concrete Surfaces and Reinforced Concrete,
Andreas Momber
2005

Fundamentals of Hydrodemolition

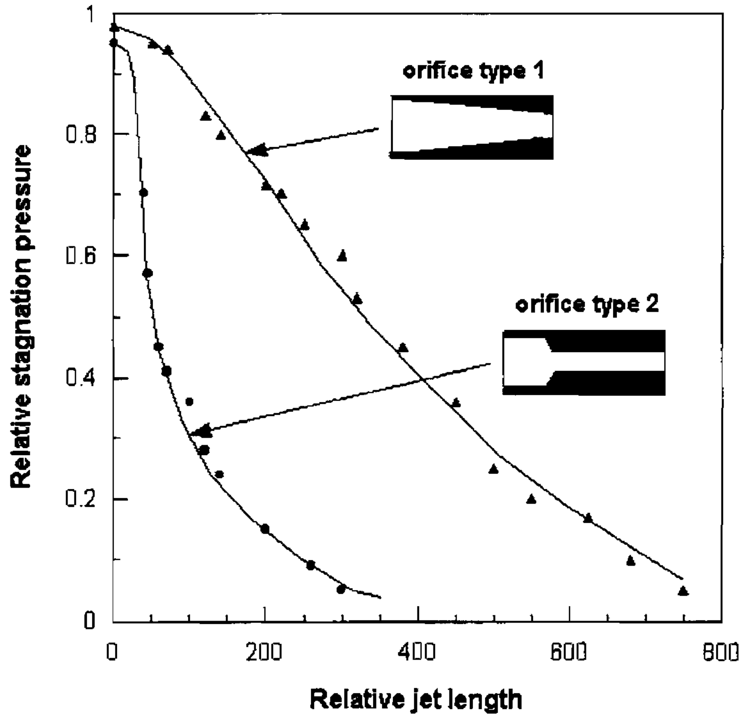


Figure 2.8 Orifice type effects on stagnation pressure profiles (Leach and Walker, 1966)

$$\frac{p_S(x)}{p_S(0)} = \left(\frac{x_C}{x}\right)^{K^*} \tag{2.23}$$

for $x<3\cdot x_C$: $K^*=0.27+0.075\cdot(x/x_C)^2$.
for $x>3\cdot x_C$: $K^*=0.3$.

The loading duration is given through the exposure time which is for a moving jet:

$$t_E = \frac{d_N}{v_T} \tag{2.24}$$

The difference between the stagnation pressure at the surface and the pressure inside the target material forces a certain volume of water to penetrate the structure. This volume is

$$Q_S = \varpi^* \cdot \dot{Q}_A \cdot t_E \tag{2.25}$$

where $\omega^*=0$ is the limiting case for a completely non-permeable material, and $\omega^*=1$ is the limiting case when the whole volume delivered by the nozzle penetrates into the material. For $\omega^*>0$, the following three cases can be distinguished:

(i)	the water flows into a crack and creates a corresponding stress at the crack tip;
(ii)	the water flows into a capillary which results in pressure amplification;
(iii)	the water flows through an open pore system and creates friction forces to the structural elements (e.g. grains).

Case (i) was experimentally investigated by Mazurkiewicz et al. (1986) whose results are illustrated in Fig. 2.9. Although the experiments were restricted to comparatively low water pressures, a linear relationship between pump pressure and pressure developed at the crack tip could be noted. If jet velocity is considered instead of pump pressure, the following relationship is valid:

$$p_R = C_1 \cdot v_J^2 \tag{2.26}$$

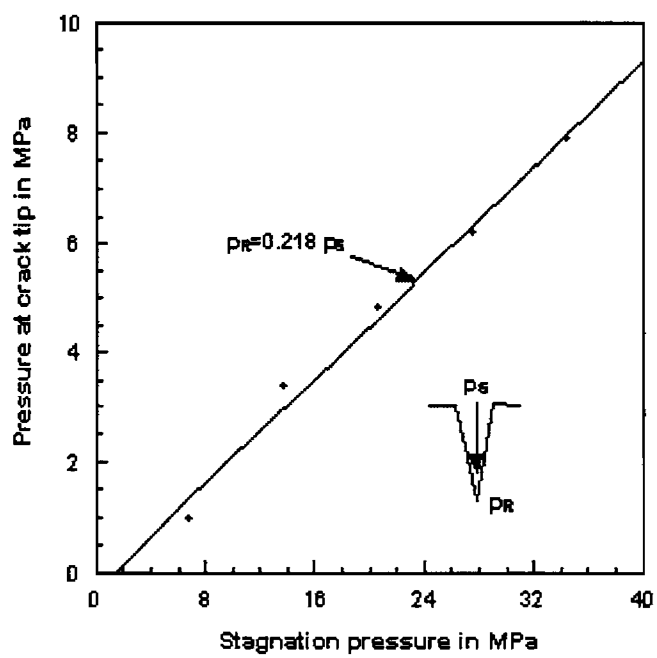


Figure 2.9 Water flow into a crack (Mazurkiewicz et al., 1986)

The constant was found to be C_1=0.22 which corresponded closely to values estimated by Momber and Kovacevic (1995) who found 0.19<C_1<0.21 based on an LEFM-model. Lin et al. (1996) used a finite element code to investigate the influence of the water jet velocity on principal stresses as well as stress intensity at the crack tip. Some of their results are shown in Fig. 2.10. The calculated points (filled circles) can be approximated by a square-root relationships which verifies Eq. (2.25). The open circles in Fig. 2.10 are experimental points estimated by Witzel (1998) on rocks. However, the calculated points in Fig. 2.10 can be approximated by an almost linear function in the range of low jet velocities up to 300 m/s. A

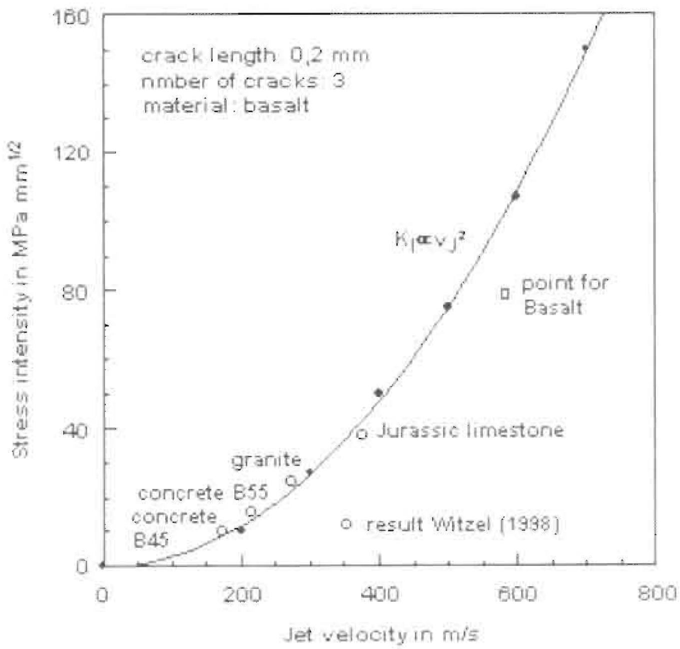


Figure 2.10 Relationship between jet velocity and stress intensity (Lin et al., 1996)

mercury intrusion study performed by Momber (1992) on cementitious composites showed clearly that the fracture started in the interfacial zone between cement matrix and aggregate which is known to be the weakest link in conventional concrete. Moreover, fracture propagation was mainly affected by aggregate size and distribution. A detailed microscopic study on crack-aggregate interactions in concrete samples eroded by water jets were made by Momber (2003b) who found clear evidence of crack deflection, crack stopping, crack tip bluntness, but also of crack bridging and crack face friction. Some of these features are illustrated in Fig. 2.11.

Case (ii) corresponds to capillary-like micropores. A model for pressure intensification in "blind", air filled tubes was developed by Evers et al. (1982). A

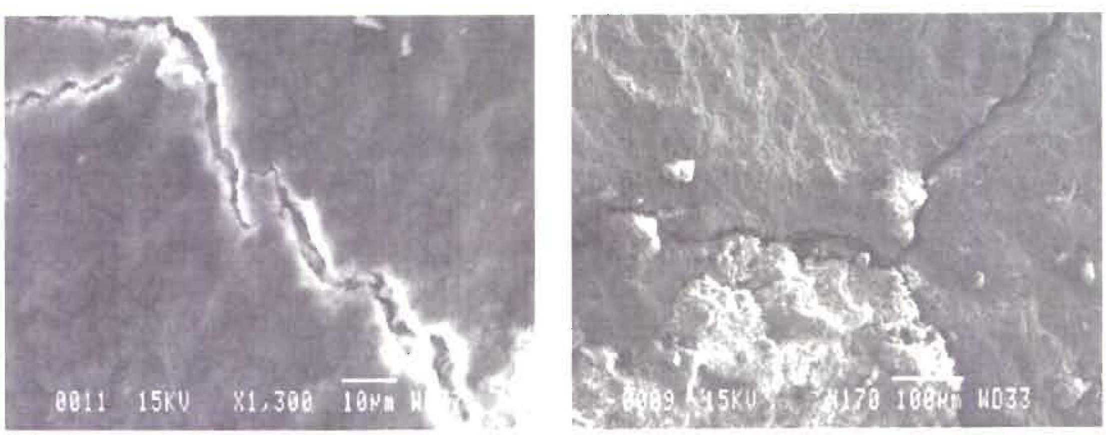


Figure 2.11 Microscopic features of concrete eroded by high-speed water jets (Momber, 2003b)

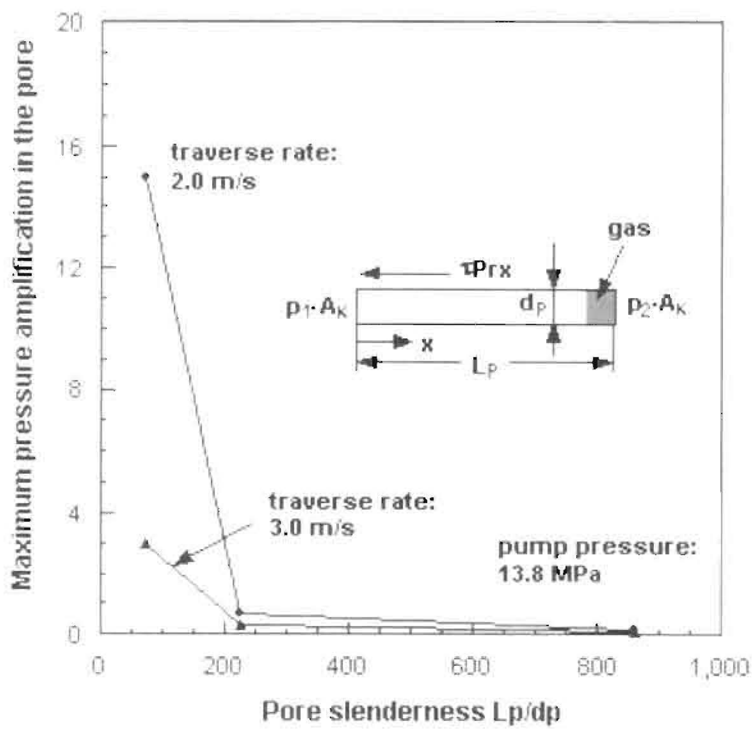


Figure 2.12 Pressure amplification in a pore subjected by a water jet (Evers et al., 1982)

liquid jet, that strikes a pore opening, transports liquid into this capillary and displaces the air. A force balance as shown in Fig. 2.12 delivers the following:

$$p_1 \cdot A_C = \tau_S \cdot Pr \cdot x + p_2 \cdot A_C \tag{2.27}$$

Thus, pressure intensification depends on shear stress, pore geometry and perimeter of the liquid column. The approach was later modified by Evers and Eddingfield (1984) by considering compressibility effects. The capillary model was verified experimentally for rather large pores and low pressures. For a pore with a diameter of 0.2 mm and a length of 38 mm where a water jet with a velocity of 71 m/s traversed over with a speed of 6.0 m/s (corresponding exposure time would be $3.3 \cdot 10^{-5}$ s), a pressure intensification of 3.5 was estimated.

Case (iii) was in detail investigated by Rehbinder (1977) for porous solids. Based on a known pressure gradient, the speed, the liquid penetrates the pore system at, can be estimated with Darcy's Law:

$$v_F = -\frac{k_P}{\mu_W} \cdot \text{grad}\, p_S \tag{2.28}$$

The frictional force acting on an individual grain due to the liquid flow can be approximated for low Reynolds numbers and spherical particles as follows:

$$F_F = C \cdot d_M \cdot \mu_W \cdot v_F \tag{2.29}$$

Further treatment – especially the replacement of the constant C – delivers the following relationship (Rehbinder, 1977):

$$F_F = \frac{V_M}{1 - P_M} \cdot \text{grad}\, p_S \tag{2.30}$$

If this frictional force exceeds the cohesion force to neighbouring grains, the grain in question will be removed.

2.2.2 Material response

Important information about the response of concrete to water jet loading is stored in the structure of fracture faces. Depending on loading regime and material structure, two general types of macroscopic failure can be distinguished:

- type I: sections without brittle fracture features;
- type II: sections with dominant brittle fracture features.

Both types, that were probably first distinguished by Nikonov (1971) for coal cutting with water jets, are illustrated in Fig. 2.13. It can be noted that type-II failure occurred always near larger aggregate particles. Investigations on rock materials have shown that the transition from type-I to type-II failure depended on jet velocity and exposure time. The transition jet velocity was a function of the

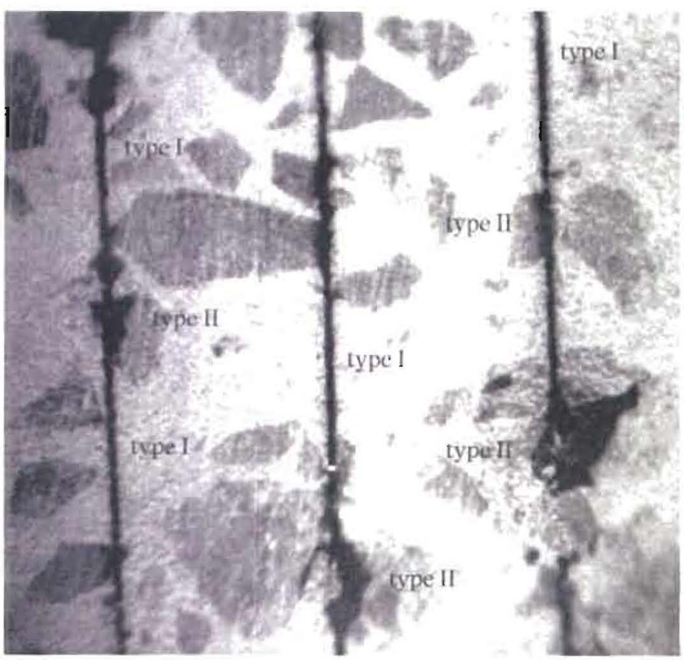


Figure 2.13 Failure types in concrete during hydrodemolition (Momber, 2004a)

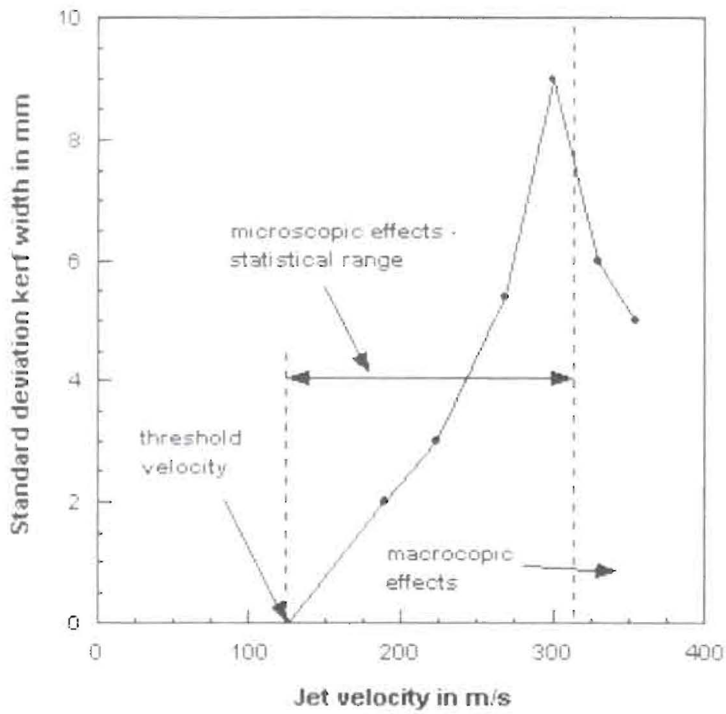


Figure 2.14 Effect of jet velocity on kerf width variation in concrete (Momber and Kovacevic, 1995)

Figure 2.15 Fracture surface of a cement sample cut with a high-speed water jet (Momber and Kovacevic, 1994)

tensile strength, whereas the transition exposure time followed a more complex relationship (Sugawara et al., 1998). Momber and Kovacevic (1995) have investigated the influence of jet velocity on the fracture statistics of concrete surfaces. The results illustrated in Fig. 2.14 show that the standard deviation of the kerf width started to drop at a certain jet velocity (at about 300 m/s). If this velocity was exceeded the failure process was more homogeneous. Effects due to the microstructure (e.g. microcrack distribution) were eliminated, and a macroscopic material property, lets say tensile strength, determined the material response. The average kerf width was always larger for a plain matrix material (cement matrix) than for a composite (mortar or concrete) due to the rather unrestrained fracture propagation in the matrix. This is illustrated in Fig. 2.15 showing the very smooth fracture face in a cement matrix. It may, however, be noted that the roughness of the surface increased as the fracture propagated (from top to bottom). Such effects are known from other brittle material as well (Hull, 1999; Schönert, 1972) and is considered to be a result of crack acceleration. If two concrete materials were compared, a water jet formed wider kerfs in the material with the coarser aggregates (Momber, 1998b; Werner, 1991a).

2.2.3 Material resistance parameters

Conventional properties of concrete, namely strength parameters, can not characterise the resistance against water jet erosion. This was found in very detailed studies performed by Kauw (1996) and Werner (1991a); an illustrative example is shown in Fig. 2.16. Figure 2.17 shows the situation if the compressive strength in Fig. 2.16 is replaced by the characteristic length. The characteristic length is a fracture parameter originating from a fracture model introduced for concrete by Hillerborg et al. (1976); see Section 1.4.2. The relationship between volumetric erosion rate and characteristic length is:

$$\dot{V}_M \propto L_{ch} = C_M \cdot \frac{d_A}{\sigma_C^{0.3}} \tag{2.31}$$

whereby the very right term expresses an empirical relationship between aggregate size, compressive strength, and characteristic length (Hilsdorf and Brameshuber, 1991). It was shown that Eq. (2.31) also holds for other impact situations, namely for the comminution of concrete in a jaw breaker (Momber, 2002b). The supporting effect of coarse aggregates on concrete hydrodemolition, as expressed in Eq. (2.31), was verified by Werner (1991a). The proportionality coefficient C_M is considered to be a machine parameter.

Rehbinder (1978) defined a so-called 'specific erodability' to evaluate the resistance of porous solids against water jet erosion. This parameter is defined as follows:

$$R_E = \frac{k_P}{\mu_W \cdot d_M} = \frac{\Delta h_M}{\Delta p} \cdot \frac{1}{t_E} \tag{2.32}$$

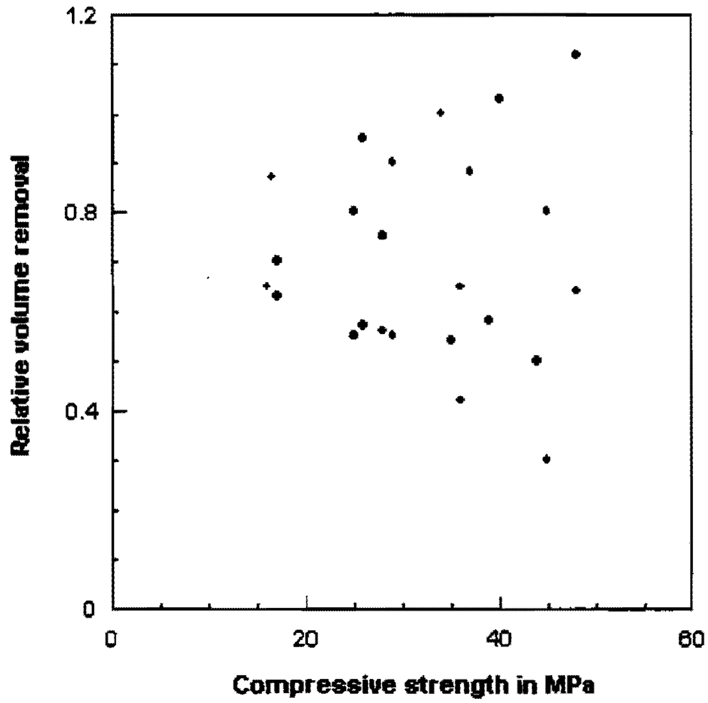


Figure 2.16 Relationship between removal rate in concrete and compressive strength (Werner, 1991)

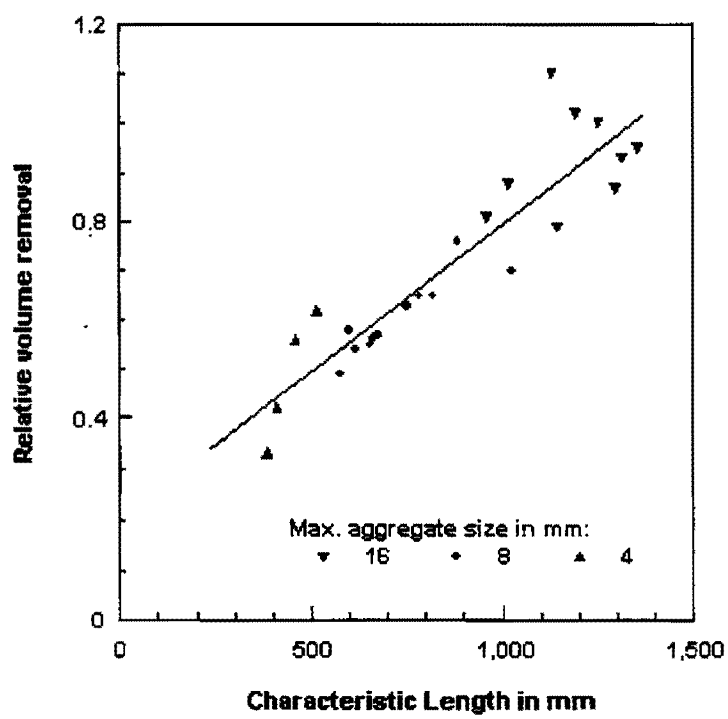


Figure 2.17 Relationship between removal rate in concrete and characteristic length (Momber, 2003c); values correspond to Fig. 2.16

The higher specific erodability, the lower the resistance. The physical unit of this parameter is $[m^3/N \cdot s]$. The right term of Eq. (2.32) allows the experimental estimation of R_E, whereby $\Delta h_M/\Delta p$ is simply the progress of an erosion depth-pressure function. Specific erodability increases as grain size or viscosity decreases, and as permeability increases. It was in fact shown by Kolle and Marvin (2000) that the resistance of rock materials was higher for water as a liquid compared to liquefied carbon dioxide (having a lower viscosity). If, however, viscosity is a constant value, erosion resistance depend only on pore structure (Rehbinder, 1980):

$$R_E \propto \frac{1}{d_M} \cdot \left(\frac{d_M}{d_O}\right)^2 \tag{2.33}$$

Thus, resistance is proportional to pore slenderness and it decreases if – for a given pore slenderness – grain size decreases. These results are partly in agreement with experimental results obtained by Evers et al. (1982) on rocks.

2.3 Process parameter effects on material removal

2.3.1 Parameter definitions

Basic target parameters include thickness of removed layers (h_M), volume removal (V_M), volumetric removal rate ($\dot{V}_M$), and removal width (w_M). They are illustrated in Fig. 2.18. For the erosion with a stationary water jet, these parameters are related through the following approximation:

$$V_M = \frac{\pi \cdot w_M^2 \cdot h_M}{4} \tag{2.34}$$

For a given removal width, a certain concrete volume must be removed to completely erode a layer of given thickness. A maximum volume removal is desired. The energy efficiency of the demolition process is given by the specific energy:

$$E_S = \frac{E_J}{V_M} \tag{2.35}$$

This parameter should be as low as possible; its physical unit is $[kJ/m^3]$. The volumetric removal rate is the mass removed in a given period of time:

$$\dot{V}_M = \frac{V_M}{t_E} \tag{2.36}$$

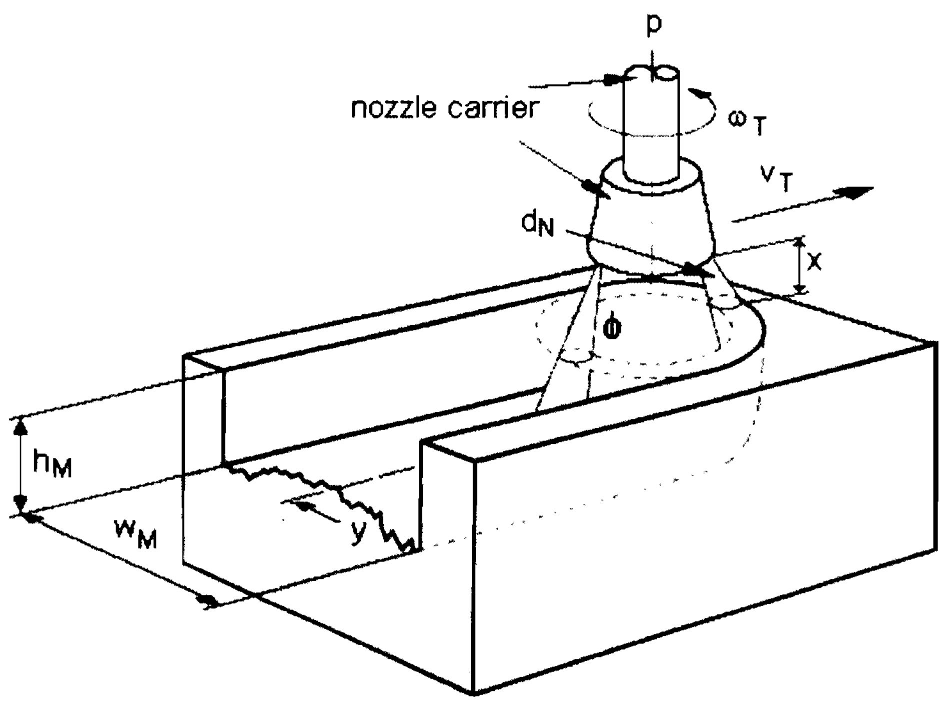


Figure 2.18 Target and process parameters for hydrodemolition

Volumetric removal rate should also be maximum; its physical unit is [m^3/h]. Other target parameters that may focus on the surface quality, such as roughness or cleanliness, are not considered in this paragraph. Hydrodemolition process parameters are summarised in Fig. 2.18. They can be subdivided into hydraulic parameters and performance parameters. Hydraulic parameters characterise the pump-nozzle-system; they include the following:

- operating pressure (p);
- volumetric flow rate ($\dot{Q}_A$);
- nozzle diameter (d_N).

Typical relationships between these parameters are described in Chapter 3. Performance parameters are more related to the process and include the following:

- stand-off distance (x);
- traverse rate (v_T);
- traverse increment (y);
- impact angle (ϕ);
- nozzle guidance.

The traverse rate covers additional parameters, such as the number of cleaning steps, n_S, and the exposure time t_E.

2.4 Concrete parameter effects on material removal

2.4.1 Material failure types

Compressive strength is the standard strength parameter of concrete that can be evaluated under site conditions as well. The most common method is to use cylinder cores drilled off the structure. Momber (1998b) was probably the first to suggest to use the way how a cylinder fails during the compression test as a criterion of the material behaviour during hydrodemolition. The studies showed that two general types of failure, type I and type II as listed in Table 2.5, can be distinguished during the compression testing (see Momber (2000f) for more information about the testing of testing of concrete cores). If the failure type I is observed, the following features can be expected for the hydrodemolition process (compare Fig. 2.30a):

- the predominant material removal mode is intergranular erosion of the cement matrix; the aggregates are completely exposed;
- the eroded surface is cleaved and uneven;
- a comparatively large number of small, regular debris is generated;
- the generated kerfs are deep but small;

Table 2.5 Failure types during cylinder core compression testing of concrete (Momber, 1998b, 2000f)

Feature	Type I	Type II
Failure mode	Slow crumbling	Rapid crushing
Primary debris	Two; symmetric	More than two; asymmetric
Secondary debris	Many small debris; round	Several larger debris; irregular
Debris surface	Aggregates exposed, undamaged	Aggregates fractured

If the failure type II is observed, the following features can be expected for the hydrodemolition process (compare Fig. 2.30b):

- the predominant material removal mode is transgranular spalling of the concrete structure;
- the eroded surface is even; just a few gaps appear;
- the eroded surface preferably contains broken aggregate grains;
- a comparatively low number of large, irregular debris is generated;
- the generated kerfs are shallow but wide.

Type II-response could be observed usually with concrete mixtures containing large, irregular (broken) aggregate. It is assumed that the transition criterion, derived in Section 7.1.2 is partly responsible for this behaviour. Rather hard materials, such as many aggregates, respond elastic, whereas softer materials, such as cement matrix, show plastic response (see Table 1.3 for typical hardness values).

2.4.2 Compressive strength effects

The influence of the compressive strength on the relative erosion rate is already illustrated in Fig. 2.16. There is no general trend between both parameters and it seems that standard compressive strength is not a useful parameter to evaluate concrete resistance. An equal trend was reported by Kauw (1996). However, the figure changes if maximum aggregate diameter is considered as done in the graph in Fig. 2.35. Under this circumstances compressive strength can characterise the efficiency of hydrodemolition processes. For large aggregate diameters (16 mm) removal rate increases if compressive strength increases, whereas the opposite trend can be observed for small aggregate diameters (4 mm). For a given compressive strength, removal rate is always higher for a concrete made with coarse aggregates. The reasons for this behaviour are already outlined in Section 2.2.2. A strong and coarse concrete enables the formation of rather large radial fractures in the structure. Based on fracture mechanics arguments, Momber (2003b) introduced the following semi-empirical relationship:

$$\dot{V}_M \propto \frac{d_A}{\sigma_C^{0.3}} \tag{2.48}$$

Note the agreement with the trends in Fig. 2.35 at least for the medium-grained and fine-grained concrete mixtures.

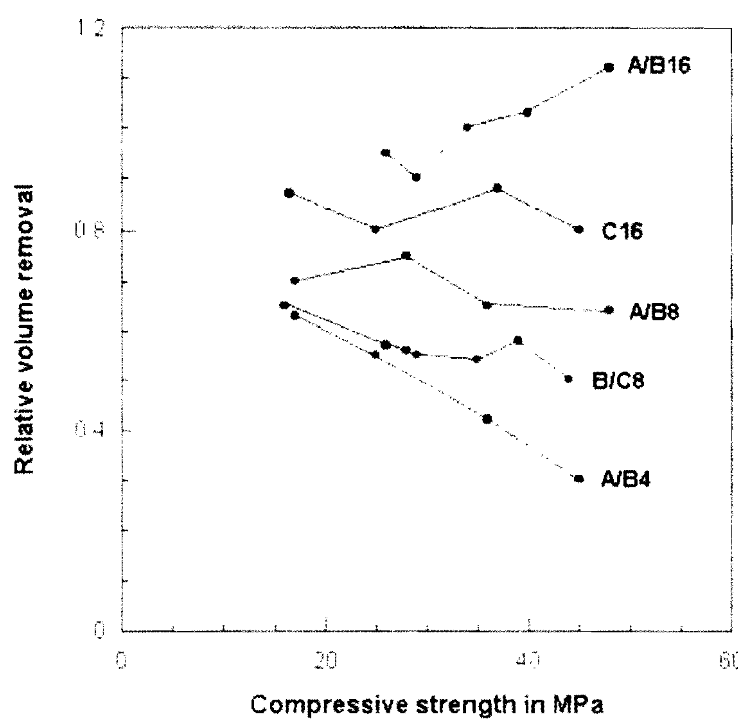


Figure 2.35 Effects of compressive strength and aggregate size on removal rate (Werner, 1991a)

2.4.3 Aggregate fineness effects

Aspects of aggregate fineness are already illustrated in Fig. 2.35 showing that concrete mixtures with fine aggregates are more resistant against water jet erosion. A design parameter that characterises aggregate fineness is the k-number (graining number). The larger the k-number the higher the amount of coarse particles. In Fig. 2.36, the relative removal rate is plotted against the k-numbers for certain sieve lines. A linear relationship, that proves the results obtained in Section 2.4.2, can be noted between both parameters. The coarser the mixture the higher the hydro-demolition efficiency. Another design parameter for concrete manufacture is the flour particle content which is the sum of cement and very fine aggregate particles. It was proven by Werner (1991a) that this parameter did not affect removal rate notably.

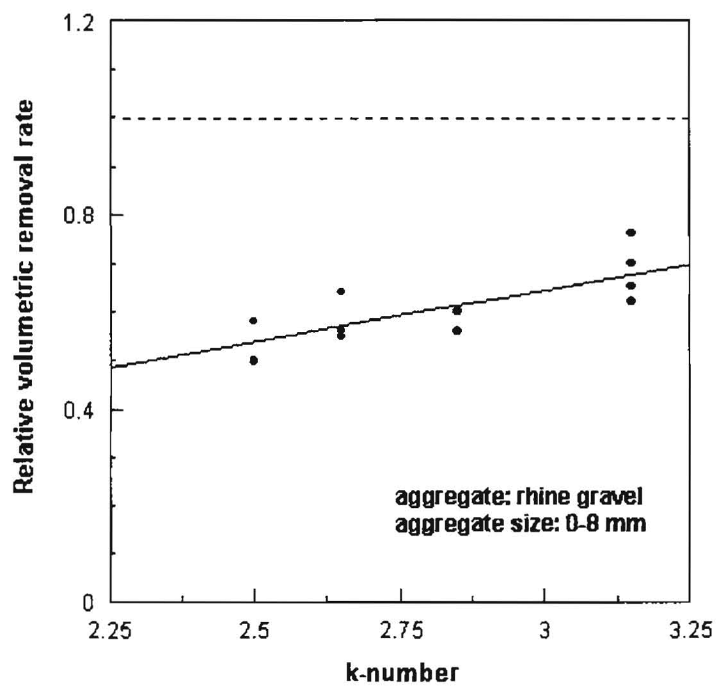


Figure 2.36 Effect of aggregate fineness on removal rate (Werner, 1991a)

2.4.4 Aggregate sort effects

Two basic types of aggregates can be distinguished in concrete. The first type is fine aggregate which is often referred to as 'sand' only. In fact, fine aggregate consists usually of rounded river (quartz) sand. The second type is coarse aggregate which is often referred to as 'gravel'. Coarse aggregate material include in fact gravel (rounded river gravel), but also broken rocks, namely basalt or limestone. It is known that the sort of coarse aggregate affects the response of concrete to hydrodemolition. This includes not only the resistance of the material but also its failure behaviour. Figure 2.37 shows the effects of coarse aggregate sort and sieve line on the relative removal rate. Notably effects can be noted. The difference between maximum and minimum removal rate is about 470%. Removal rate is maximum for a limestone-based concrete under all circumstances, followed by the gravel-based concrete. The basalt-based concrete has the highest hydrodemolition resistance. It also seems that the effects of aggregate sort become more important for the coarser mixtures. The corresponding eroded surfaces are rather even and characterised by always broken aggregates in case of limestone, whereas they are very uneven and characterised by mainly (but not exclusively) broken aggregates in case of basalt. In case of river gravel the amount of broken aggregate was less than 30% (Werner, 1991a). Numerous aspects cause the different behaviours of the materials, among them morphology and surface energy of the aggregates and the compositions and properties of the aggregate-matrix interfaces.

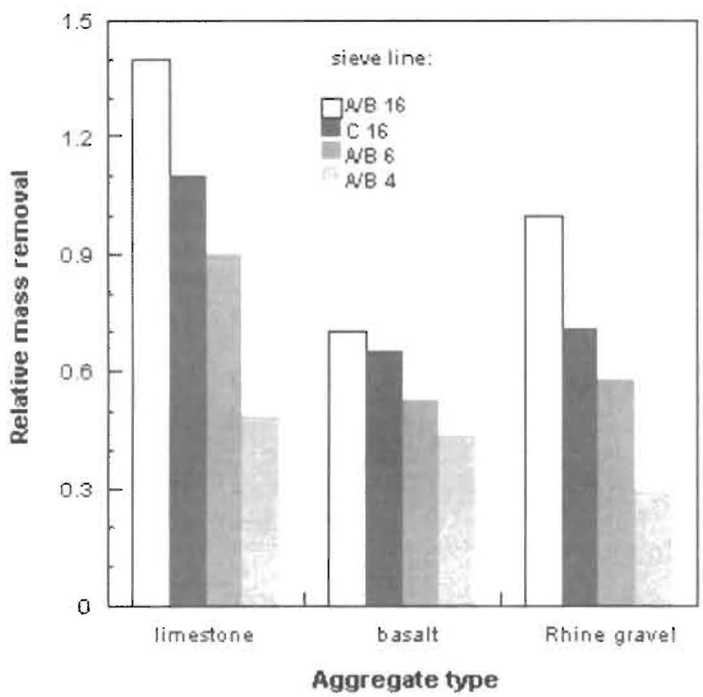


Figure 2.37 Effect of aggregate type on removal rate (Werner, 1991a)

2.4.5 Porosity effects

The influence of cement paste porosity on the relative removal rate are illustrated in Fig. 2.38. The tendencies visible in that graph also apply to the relationship between capillary porosity and removal rate (Werner, 1991a). Porosity parameters alone can not characterise the response of concrete but only if they are combined with another parameter, namely the aggregate size. For coarser concrete mixtures removal rates decrease slightly if porosity increases. The opposite trend is valid for fine concrete mixtures. However, the effects of aggregate size are much more pronounced than those of porosity.

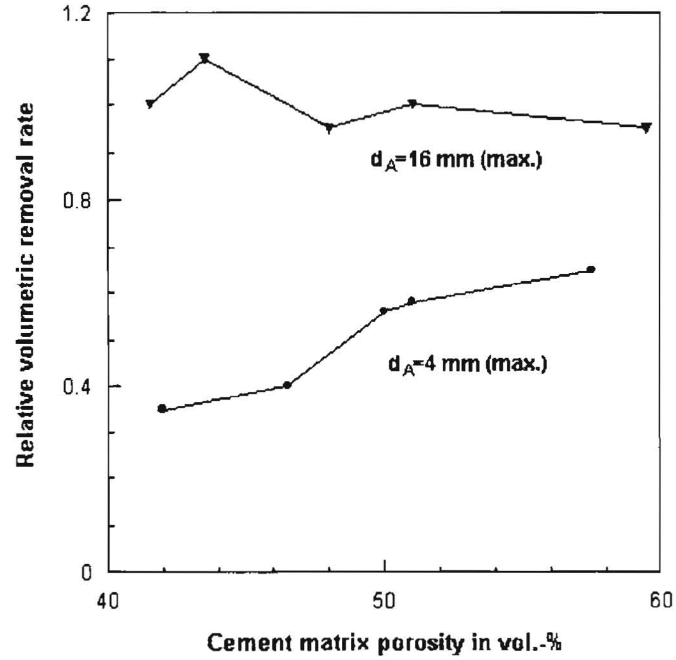


Figure 2.38 Effect of cement paste porosity on removal rate (Werner, 1991a)

2.4.6 Steel bar reinforcement effects

Many practical hydrodemolition applications include reinforced concrete structures. Effects of steel bar reinforcement on volumetric removal rate are shown in Fig. 2.39a where the influence of the depth of reinforcement is illustrated as well. A distinct drop in efficiency can be noted if the depth of reinforcement exceeds a value of 100 mm, which applies to a plain non-reinforced concrete. Thus, reinforcement supports the removal process. The thickness of the concrete layer that covers the reinforcement does, however, not play any role. The same relationship is valid for the removal depth (Kauw, 1996). The reason for the increase in hydrodemolition efficiency due to reinforcement is the installation of weak zones in the interface between concrete and reinforcement bars (Balaguru and Shah, 1992). If a single steel bar is replaced by a bar bundle, as shown in Fig. 2.39b, removal rate drops slightly. However, if a dense reinforcement bar net exists, the concrete removal process is disturbed and 'shadows zones' form at the lower surface of the steel bars. A typical practical example is shown in Fig. 2.40. These 'shadow zones' can be avoided by using complex nozzle guiding systems that include angled jets. The effect of reinforcement becomes stronger if the structure is damaged through chlorides. In that case rust grows at the corroded steel and the stresses generated due to volume expansion form cracks in the surrounding concrete. These cracks are exploited by the water jet. An increase in efficiency of about 15% was estimated if chloride-corroded reinforced concrete was treated instead of non-corroded reinforced concrete (Kauw, 1996).

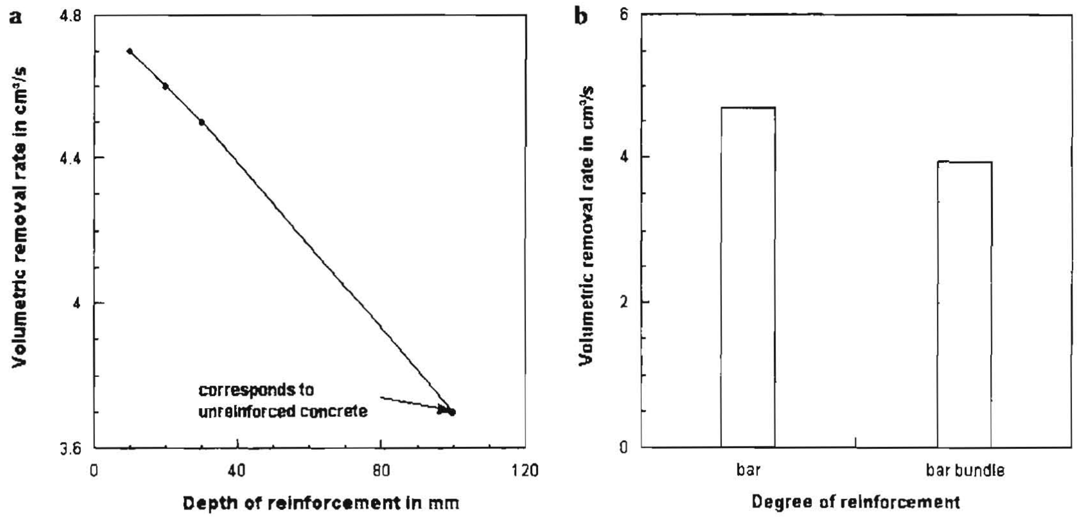


Figure 2.39 Effect of steel bar reinforcement on removal rate (Kauw, 1996)
a – single reinforcement bar
b – bar bundle

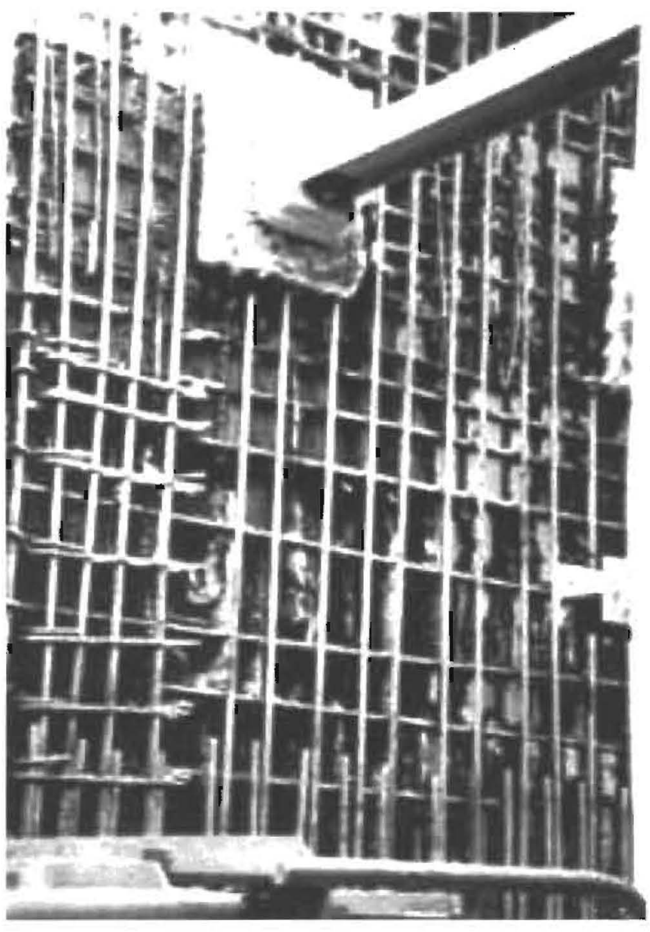


Figure 2.40 Shadow zones, formed under reinforcement bars during hydrodemolition (Rosa, 1991)

2.4.7 Steel fibre reinforcement effects

Hydrodemolition of steel fibre reinforced concrete plays a role if industrial floors or, respectively, hydraulic structures are maintained. Effects of steel fibre reinforcement on hydrodemolition processes are investigated by Hu et al. (2004) who pointed out that impact angle determines the influence of reinforcing fibres. At low impact angles (15°) the fibres form 'shadow zones', as illustrated in Fig. 2.41, that prevent the concrete behind the fibres from being eroded. For this reason, removal rate drops. However, the addition of fibres to a concrete also adds weak interfacial zones between matrix and fibres (Balaguru and Shah, 1992). The pressurised water penetrates these zones and causes a separation of the fibres. Therefore, these zones deteriorate the erosion resistance especially if the material is impinged by jets at vertical angles. Under such conditions, a steel fibre reinforced concrete can even more efficiently be removed by water jets than a corresponding plain concrete; this conclusion is proven in Fig. 2.42.

Figure 2.41 Shadow zones, formed behind reinforcement fibres during water jet erosion (Hu et al., 2004)

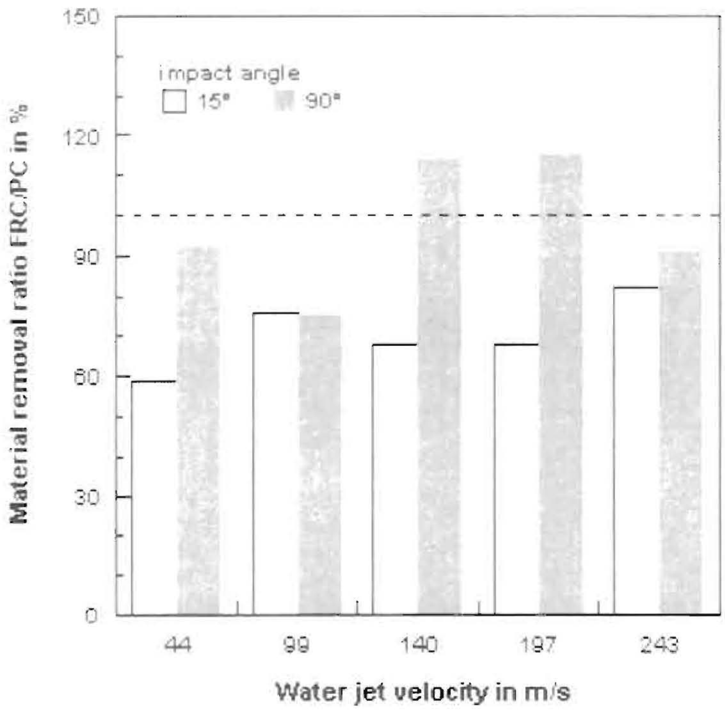


Figure 2.42 Effect of steel fibre reinforcement on mass removal (Hu et al., 2004)

2.5 Hydrodemolition model

Labus (1984) developed models for estimating depth of cut as well as material removal rate for hydrodemolition applications. The model for calculating depth of cut is based on non-dimensional terms; it has the following structure:

$$\frac{h_M}{d_N} = K_0 \cdot \left[\left(\frac{p}{\sigma_C}\right) \cdot \left(\frac{d_N}{x}\right) \cdot \left(\frac{v_J}{v_T}\right)^{0.5}\right]^{K_1} \tag{2.49}$$

Figure 2.43 shows a plot of this relationship as applied to experimentally estimated data. The correlation fits the data quite well with a correlation coefficient of 0.88, and the constants in Eq. (2.49) can be estimated to K_0=9.515, and K_1=0.355. The model was expanded to a rotating nozzle carrier; structure and geometry are illustrated in Fig. 2.44. The final model reads as follows:

$$\dot{V}_M = 2 \cdot v_T \cdot K_0 \cdot \cos\gamma \cdot (b + s \cdot \tan\gamma) \cdot \left[\left(\frac{p}{\sigma_C}\right) \cdot \left(\frac{d_N \cdot \cos\gamma}{s}\right) \cdot \left(\frac{v_J}{[(v_T - \omega_T \cdot b_1 \cdot \sin\omega_T t_E)^2 + (\omega_T \cdot b_1 \cdot \cos\omega_T t_E)^2]^{0.5}}\right)^{0.5}\right]^{K_1} \tag{2.50}$$

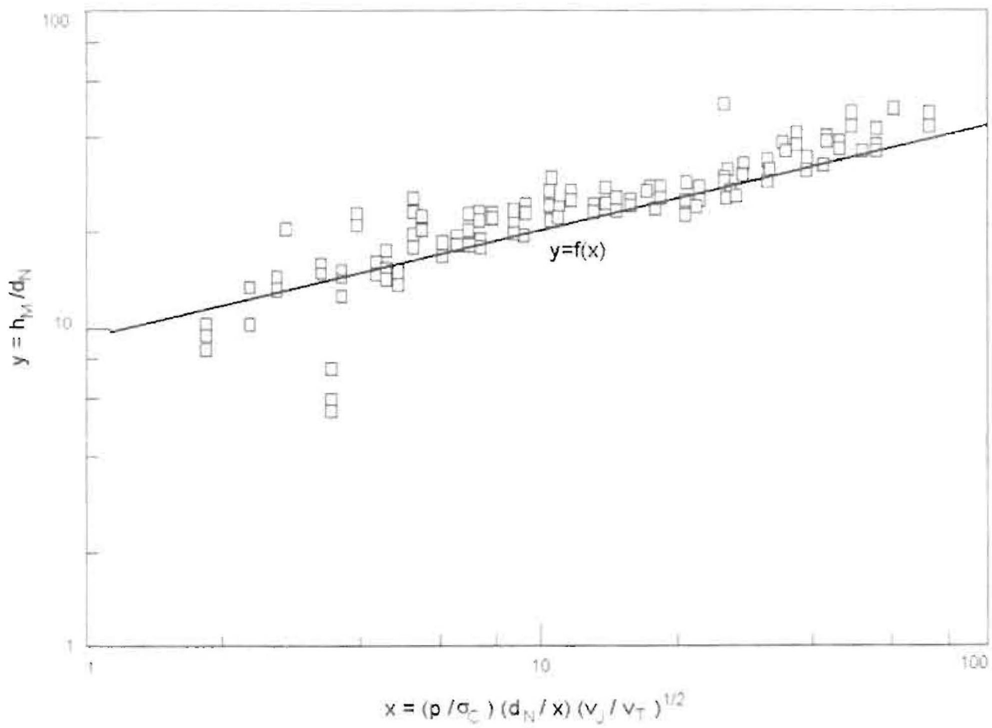


Figure 2.43 Verification of Labus' (1984) hydrodemolition model

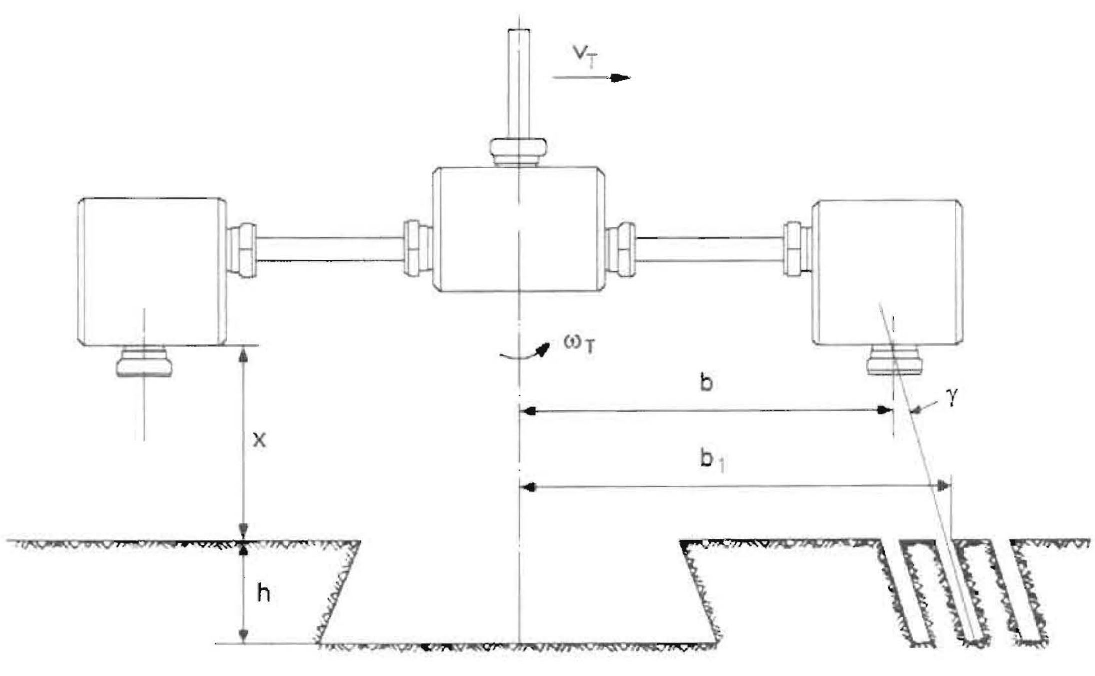


Figure 2.44 Parameters used in Eq. (2.50)

In contrast to the depth of cut, volumetric material removal rate is a function of time, since the relationship is an instantaneous rate. By integrating over the time, it takes for one revolution of the nozzle carrier head, the average volumetric material removal rate can be estimated. Calculations based on the model showed some good agreements with trends from experimental results; this applies in particular to nozzle diameter, pump pressure, and stand-off distance. However, from the results discussed in the previous Sections, it is clear that the model simplifies the effects of material parameters. Compressive strength alone can not determine the resistance of concrete to hydrodemolition. Aggregate type and size are more important, and at least one of these parameters must be included into Eq. (2.50).

Exhibit 24: Hydrodemolition

Exhibit 24

Hydrodemolition for Removing Concrete

by James Warner

Hydrodemolition is a relatively new technology for the removal of concrete. First used in Italy in 1979 — with prototype equipment — to remove concrete on the Viadotto del Lago, its development was slow until 1984, when it was introduced in Canada and Sweden.

Only within the last decade has it been commonly used. Hydrodemolition involves impingement of a discrete blast of water under very high pressure in a controlled manner. The concrete is disintegrated into generally small rubble as a result of both the impact of the water on to the surface and the pressurization of internal pores, cracks, and voids.

Advantages of hydrodemolition

The procedure offers many significant advantages. It is much faster than traditional chipping with pneumatic tools. It is fairly quiet, is free from dust and vibration, and results in small size rubble that can be easily handled and reused for surface cover or base material, often without additional processing. Hydrodemolition is selective in that it will generally find and disintegrate all concrete below a given porosity or strength level.

Bond surface

The greatest advantage, however, is the provision of a superior quality surface upon which to bond new concrete. Hydrodemolition does not "bruise" the resulting surface — it does not cause a network of microcracks in the parent concrete near the bond line, as do other removal methods that involve direct impact on the surface.[1-4] It results in a very clean, rough, undulating, high amplitude surface profile that provides the maximum obtainable bond surface area. It generally will not damage existing reinforcing steel and will, in fact, clean the reinforcing of all corrosion products and other contaminates.

Selective removal

In addition to removing all porous or low-strength concrete, it will tend to open and rout out remaining cracks that extend into competent material. And when properly controlled in concrete with uniform porosity and strength, it will not damage or remove excessive amounts of good quality concrete that will be left with a well-prepared surface for immediate placement of the new overlay.

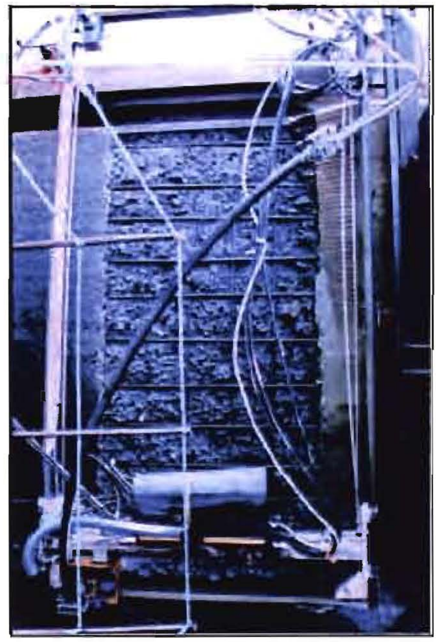
With all these advantages, one might quickly conclude that the introduction of hydrodemolition is truly a blessing. And in most instances it is, but like all good things, there are disadvantages and limitations that sometimes confound its use.

Disadvantages

Large amounts of water are required that not only must be obtained, but handled and disposed of as well. The water must be directed to appropriate areas for collection and controlled so that it does not flow into areas that must remain dry. The spent water is usually very alkaline and contains a large amount of suspended solids that will often require neutralization and separation before it can be run into sanitary or storm sewers. This can be readily handled by the use of settling and neutralization tanks.

Recycling

At least one hydrodemolition contractor is recycling the water on some projects,[5] and because it must be very clean and completely free of even minute suspended solids, an extensive reclamation plant is required. Even with the use of a polishing filter that removes the smallest particles (1 to 5 microns in size), the service life of the nozzle and some pump components is shortened due to the high abrasiveness of even very small particles when traveling at very high velocities that are inherent with the procedure.



Frame assembly for hydrodemolition on inclined/vertical surfaces.

Removal

The demolished rubble is usually pushed into piles on the adjacent undemolished surface, where it can be handled with a small loader. This is generally accomplished with the water pressure of a fire hose or pressure washer. A fine cementitious slurry will remain long after the larger particles have been removed. If allowed to dry, a very fine powder residue will result. If the residue is not removed, it will have a deleterious effect on the bond of the repair material. It is very hard to remove once it has been allowed to dry, but its drying can be precluded by extended washing of the wet surface immediately after hydrodemolition with clean water until the wastewater runs clear.

The process will find and pulverize all low strength or deteriorated concrete, and unfortunately, even competent concrete that is very porous, regardless of depth. There are many projects where the removal of all questionable concrete is not acceptable due to economic or other considerations, and the removal of significant amounts of competent concrete is seldom desired. This is a frequent problem on structural slabs in which reinforcing is not uniformly distributed, or where porosity and/or strength are highly variable.

This can be a particular problem where a minimum thickness of removal below the reinforcing is required. To obtain the required reinforcing clearance in those areas of relatively high competency or strength, those areas of lower strength or lesser quality might be cut excessively deep and in extreme situations, disintegrated for their full depth.

The importance of the strength of the concrete is widely recognized in the hydrodemolition industry; however, the significant influence of porosity or microcracking is not. Terms widely used in the hydrodemolition industry to describe concrete are "sound" and "unsound." Virtually all concrete that is disintegrated in the process is described as low strength or unsound, which often is not accurate because concrete that is porous or contains microcracks can still be competent and of adequate strength.

Application

Typical water pressures used in hydrodemolition will generally be within a range of 55 to 350 MPa (8000 to 50,000 psi) or more, with water flow rates on the order of 19 to 300 L (5 to 80 gal.) per minute. The amount of work accomplished is dependent on the hydrodemolition energy that is basically the product of the pressure and the flow rate. Also of some influence are the nozzle design, trajectory, and distance from the impingement surface. Low flow rates generally require relatively high pressures, whereas a similar amount of removal can be accomplished at lower pressures combined with higher flow rates. There are minimum pressures required, however, that are dependent on the concrete strength and condition.

Rock mechanics research has shown that where a minimum threshold pressure is required to cause erosion, the rate, and thus depth of removal, is dependent on rock permeability, which is a cross-sectional area of the pores.[6] As a general rule, the strength of concrete is directly related to its porosity. Lower strength concrete is usually more porous than high-strength concrete, and because the pressurization of pore space has a great influence on the removal operation, hydrodemolition efficiency is thus directly related to the volume of pore space and thus, concrete strength. Rules of thumb regarding the required water pressure to demolish good quality concrete have been offered by various hydrodemolition equipment manufacturers and contractors. These are generally within a range of 3 to 3-1/2 times the compressive strength of the concrete.

Almost from the original development of the process, there have been two different philosophies as to optimal pressure and flow rates. Equipment commonly used in the United States operates either at a pressure of about 117 MPa (17,000 psi) and a flow rate of 190 to 300 L (50 to 80 gpm) per minute, or so-called ultra-high pressure, about 230 MPa (34,000 psi) and flow rate of about 120 L (32 gal.) per minute. Concrete can be disintegrated equally well with either set of pressure/flow parameters, although the lower flow rates that are inherent with the ultra-high pressure are more gentle on the concrete. The author suggests that whereas use of the higher flow rate equipment is practical on open structures such as bridge decks, applications in parking structures and other enclosed areas (where both water control and debris removal are difficult), might best be accomplished with the ultra-high pressure/lower flow rate equipment.

Primary use

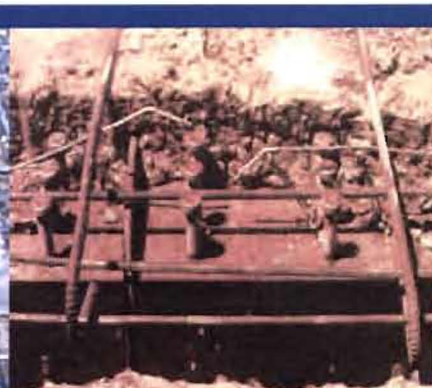
Considerably less energy is required to disintegrate concrete that contains large and extensive voiding, such as cracking and microcracking caused by sulfate attack, alkali silica reaction, and similar mechanisms, or delamination or spalling as a result of corrosion of embedded ferrous metals or reinforcing steel. For this reason, hydrodemolition is particularly suited for removal of such deteriorated concrete, and this is its primary use.

Rate of removal

Concrete removal is dependent on not only the water volume, pressure, and nozzle factors, but also on the rate of movement of the nozzle over the surface.

To maintain control of the concrete removal, all of these parameters must be set and consistently maintained. Although a hand-held lance can be used for small areas, hydrodemolition of large planar areas is generally facilitated


Hydrodemolition cleans reinforcing


Removal of concrete for full depth


Blow through of water jet

by the use of a controlled "robot" or frame assembly onto which one or more oscillating or rotating nozzles are mounted.

Typical robots are self-propelled, hydraulically powered vehicles. The nozzles are mounted on a transverse bar that is commonly about 6 ft (2 m) long. They traverse back and forth on the bar at a uniform rate that can be adjusted from about 1 to 60 seconds. In operation, the robot moves away from the demolished area in discrete "steps," often referred to as "indexing," which is variable from about 0.25 to 6 in. (6 to 150 mm). The traverse speed, number of traverses, and length of the step can be set into the controller, which then uniformly manages the operation.

Uniformity of removal can be enhanced when several rapid traverses are made at a given step, instead of a single, slow traverse. Computers are used to control the parameters on some machines, while manual settings are used on others. To maximize the efficiency of the diesel engines that power the high-pressure pumps, hydrodemolition equipment is generally run at a constant pressure and flow rate.

When hydrodemolition is executed in good quality concrete of uniform strength and free of cracks or microcracks, removal will generally be to a reasonably uniform depth. Depending on the strength of the concrete and the operating parameters that have been entered into the on-board controller, removal depths can be set from a few millimeters to about 20 cm (8 in.) in a single pass. Obviously, under similar operating parameters, much greater depths will be removed if the concrete is deteriorated or contains extensive cracks, microcracks, or other internal imperfections.

The operating parameters will thus vary from one project to another, and often vary between different areas of a given structure. Where hydrodemolition is used on a repair project, typical practice is to initially set the operating parameters by trial and error prior to the production work.

A small test area 3 to 5 m^2 (30 to 50 ft^2) of apparently undamaged concrete will be disintegrated, with the equipment controller set to remove to a depth of about 12 to 25 mm (0.5 to 1 in.). If the expected results are not achieved, the settings will be adjusted and further trials completed until satisfactory results are obtained. The equipment will then be relocated to a typical area of damaged concrete and a similar small area removed. Any further adjustments required will be made until the desired result is obtained. Ideally, this will completely remove all damaged concrete. However, in some cases, this may not be desired or practical.

Concrete of varying strength or a nonuniform distribution of internal defects will result in removal to varying depths, and in some cases, the entire thickness of the section. Where damaged concrete exists only on the surface and is underlain by uniformly good quality concrete, the parameters can be set to remove only defective materials. However, if the thickness of the defective concrete or the porosity or strength of the underlying undamaged concrete varies, periodic adjustment of the parameters will be required and considerable variation of the depth of cut will result. In extreme cases, hydrodemolition may prove to be an inappropriate removal method.

Case studies

In one instance, hydrodemolition was performed to remove the deteriorated surface concrete of a steel girder concrete deck bridge. Following the typical robot trial and error period, an initial production pass was made between the girders. Removal was as desired and hydrodemolition appeared to be a wise choice. However, when the next pass was made, which was over a beam, the concrete was blasted away for the full depth of the deck.

The excessive removal was due to the fact that the concrete contained many cracks for its full depth over the steel girders as a result of corrosion of the closely spaced shear pins, which were attached to the top of the steel sections. It would have been desirable to remove the faulty concrete to full depth, but practicality did not allow that. If all concrete bearing on the girders had been removed there would not be any support for the remaining deck sections. Furthermore, environmental restrictions prohibited the discharge of demolition debris under the bridge. Hydrodemoliton was not an appropriate removal method for this project.

In another case, hydrodemolition was specified as the removal method for defective concrete on the decks of a parking structure. On commencement of the work, it was found that the depth of deterioration varied greatly, and in many areas the concrete was demolished full depth (blow through). While it was advantageous to find and remove all of the deteriorated concrete, a considerable amount of competent concrete was removed as well. The contractor's shoring method had to be changed, and extensive formwork was required,

Robot typically used on horizontal surfaces

Original shoring method changed due to blow through of blast

greatly increasing the cost of the work.

It is believed by many in the industry that the removal depth is related strictly to concrete strength, and that all concrete disintegrated by the water jet is thus either low strength or deteriorated. While that is sometimes the case, it is not true in all instances. Very competent and even high-strength concrete can have a system of microcracks, the distribution of which is not necessarily uniform. Also, significant variation of strength or content of entrained or entrapped air can exist even in very competent concrete. The influence of such porosity is far greater from the standpoint of the concrete's susceptibility to disintegration by high pressure water than it is by its strength. Thus, the effect of high pressure water can be quite variable even on concrete of high and generally uniform strength. To compensate for these factors, the settings of the robot may require frequent adjustment. Accordingly, the quality of the finished cut relies heavily on the ability and attentiveness of the hydrodemolition operator.

Due to these factors, cost overruns and claims have occurred on several hydrodemolition projects due to excessive removal quantities. It is thus important that designers and specifiers be aware of possible variability of the concrete porosity and/or strength when considering hydrodemolition. Budget and contract documents should allow for appropriate additions and adjustments as applicable.

Conclusions

The advantages of hydrodemolition are many, and the ability to remove all concrete that has been subjected to deterioration, or that which is below an approximate strength level is certainly beneficial. However, when considering hydrodemolition, keep in mind that the water will pressurize all voids it is able to access, even those that might exist at a greater depth than the planned removal. Likewise, where the compressive strength, porosity, or defect level of the concrete is variable, the depth of removal will likewise vary.

Any areas that have cracking or excessive porosity for full depth will likely be completely penetrated. This should be considered prior to hydrodemolition. Consideration must be directed to the likelihood of such occurrences, so that both the budget and timely remedial action can be taken if necessary.

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Selected for reader interest by the editors.

The topic covered by this article was presented at the ACI Seattle, Wash., convention in April 1997.

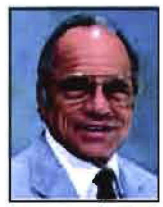

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Exhibit 25: MoDot Hydrodemolition and Repair of Bridge Decks

Exhibit 25

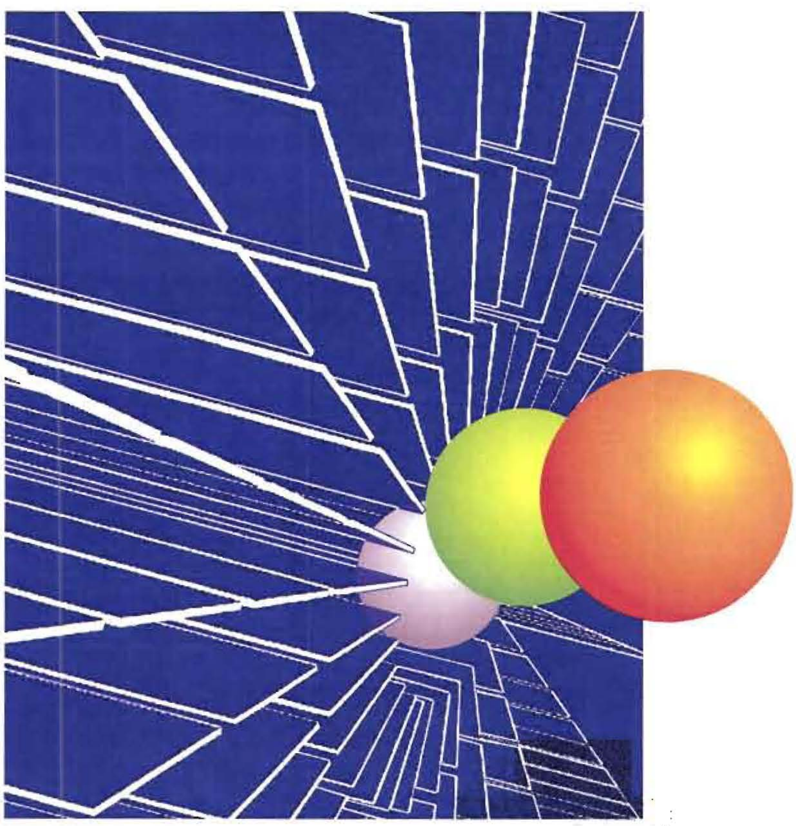
MoDOT

Research, Development and Technology

RDT 02-002

Hydrodemolition and Repair of Bridge Decks

RI 97-025



December, 2002

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Hydrodemolition and Repair of Bridge Decks

Final Report

MISSOURI DEPARTMENT OF TRANSPORTATION
RESEARCH, DEVELOPMENT AND TECHNOLOGY

BY: John D. Wenzlick, P.E.

Acknowledgment to:
Pat Martens, Dave O'Connor

JEFFERSON CITY, MISSOURI
DATE SUBMITTED: December 16, 2002

EXECUTIVE SUMMARY

It was the intent of this study to prove that hydrodemolition is a better alternative to removing deteriorated concrete form bridge decks than conventional mechanical methods such as jackhammers. Jackhammers can cause microfracturing of the concrete left in place. Microfractures in the remaining deck can cause premature loss of bond in the patches or the overlaid surface to which a large investment has been applied in hopes of getting a rehabilitated bridge deck that will last another twenty to thirty years. MoDOT over the past ten years has experienced extensive cracking and debonding of its dense concrete bridge overlays leading to premature deterioration of the rehabilitated decks, well before the end of their design life. Hydrodemolition could help solve these problems in future bridge rehabilitation projects. Additionally, after the hydro-blasted material is removed, hydrodemolition leaves the substrate deck clean, it removes all corrosion from the rebar, and the deck is ready for new concrete to be poured. Additional mechanical cleaning and sandblasting of the concrete surface and rebar is needed with mechanical removal methods. Hydrodemolition has generally been bid cheaper than conventional mechanical methods but is overall more expensive because of mobilization costs and limited availability of hydroblasting equipment and hydrodemolition contractors close to Missouri. Other items like traffic control and staged construction can be an extra cost because it is necessary to have larger areas of bridge deck closed to do hydrodemolition than it is for mechanical methods.

The practices of Missouri's adjoining states were surveyed pertaining to the use of hydrodemolition. Most state specifications use it as an equal alternative to mechanical methods. This study looked at hydrodemolition projects done in Missouri, first by maintenance starting in 1996 in the St. Louis area and continuing on maintenance projects there through 1999. It also looked at the first, and so far only, project designating hydrodemolition as the only method of concrete removal let by construction contract on Route I-44 near Springfield in Green County in 1998. Costs for all the projects done by MoDOT are presented. Costs for hydrodemolition ranged from $ 1.25 to $ 3.50 per square foot ($3.50 bid on the I-44 construction project mentioned above) compared to $ 28.79 to $ 32.99 per square foot for conventional removal. A study of the relative damage done to the concrete left in place was done using direct tension or pull off tests. Generally the testing showed pulloff strengths around 150 psi (pounds per square inch) versus 125 psi for the mechanically prepared concrete. This was not as high as expected since a Swedish study had shown strengths up to 300 psi.

The limited bond testing done did not show large gains in strength over conventional removal but it is believed further testing would show better results. However, hydrodemolition does provide less damage to the remaining concrete and a cleaner surface ready for the patching or overlay concrete to stick to in a third of the time as conventional removal. It is recommended that more maintenance and construction contracts be advertised designating hydrodemolition as the only option for removing deteriorated concrete. It is proposed that, on all bridges that meet the criteria for ease of hydrodemolition, all contracts in 2003 be let specifying hydrodemolition exclusively. (It is estimated this would be about twenty five percent (25%) of bridges contracted to be rehabilitated or widened.) This will foster more availability of this equipment and contractors using it. A report on costs savings and life-cycle costs would be prepared from these 2003 jobs to verify how superior and cost effective hydrodemolition is compared to mechanical methods in ensuring long lasting concrete repairs.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Introduction and Objectives

Hydrodemolition is a faster, cleaner and better way to remove deteriorated concrete from bridge decks in order to patch or rehabilitate the driving surface. The basic steps in the hydrodemolition of concrete bridge decks are as follows. First scarifying of the original bridge deck is required before hydrodemolition of the surface. Hydrodemolition is done with a computerized, self-propelled robotic machine utilizing a high pressure water jet stream in the range of 15,000 to 20,000 PSI and usually removes all unsound concrete in one pass. If required, hand held high pressure wands or 35 lb. maximum jackhammers shall be used in areas inaccessible to the hydrodemolition equipment. The contractor removes the hydrodemolition debris with vacuum equipment before the debris and water is allowed to dry on the deck surface. The contractor will take steps to prevent damage to existing reinforcing steel and not place wheels from heavy equipment, such as vacuum trucks, on areas where the top layer of slab reinforcement has been left unsupported by the hydrodemolition process. After debris is removed the deck surface and patches and the exposed reinforcing steel is usually clean and ready for concrete placement. MoDOT's Job Special Provision (JSP) allows in areas where removal of unsound concrete does not expose the bottom mat of reinforcing in the deck to be patched with latex modified concrete and placed monolithically with the concrete wearing surface.

The hydrodemolition process allows several steps needed in conventional removal to be eliminated. Sounding and marking of delaminated areas is not necessary because after the hydrodemolition equipment is correctly calibrated it will automatically remove any delaminated or deteriorated concrete. This eliminates the need to saw cut around patching areas as needed with conventional jackhammer methods. Sandblasting of rusty or dirty reinforcing steel is not needed because it is cleaned at the time of hydrodemolition. Because of the very good bonding surface left by the hydrodemolition patches are allowed to be placed, if the bottom reinforcement hasn't been exposed, at the same time as the wearing surface concrete. This step alone eliminates the time and labor needed for a separate patching operation and the time to wait for patches to cure before being able to place the wearing surface. The only additional needs for hydrodemolition are a large water supply and the control of runoff water.

It was intended to prove that hydrodemolition is a more efficient and less destructive method than using jackhammers for removing deteriorated concrete from reinforced concrete bridge decks. In hydrodemolition all the deteriorated concrete is removed, the reinforcing steel is cleaned, and the remaining concrete is not left with micro-fractures as it is when jackhammers are used. MoDOT has had a problem over the last ten years or so with premature failures of rehabilitated bridge decks using dense concrete overlays. There have been problems with excessive cracking and with debonding of the overlay from the original deck concrete. These problems have occurred with all types of overlays, latex modified concrete, low slump concrete and silica fume concrete. Curing of the concrete and other factors are causing the cracking problem, but loss of bond could be alleviated by using hydrodemolition instead of conventional mechanical methods of removing deteriorated concrete. Hydrodemolition is more expensive at this time because of the expense of the equipment and the short supply of contractors doing this kind of work. For this reason mobilization costs are high, however, these costs have come down recently with more equipment being manufactured and more contractors now getting into this type of work.

A review of concrete removal practices of the adjacent states was made. Table 1 below lists the states contacted All the states that specify hydrodemolition, allow either it or conventional mechanical methods except Kansas. If a bridge deck is to receive a concrete overlay, Kansas DOT only allows hydrodemolition. Two of the states don't specify hydrodemolition at all.

Table 1: Hydrodemolition Specifications of Other States

State	Hydrodemolition Specifications
Missouri	*Special Provision if an overlay is involved (includes hydrodemolition as alternate for conventional)*
Kansas	*Specification 724 (hydrodemolition only for bridge overlays)*
Illinois	*In Deck Slab Repair Specification (includes hydrodemolition and conventional both)*
Iowa	*Specification 2413 (includes hydrodemolition and conventional both)*
Arkansas	*Nothing found*
Nebraska	*Does not indicate use of hydrodemolition*

Technical Approach

This study was set up to observe the hydrodemolition process and become more familiar with the equipment and its operation. Through pull-off or direct tension testing before and after removal of the deteriorated concrete and after patching, this study was designed to determine the effectiveness of hydrodemolition over conventional jackhammer methods in leaving a better substrate on which to apply new concrete. Hydrodemolition can reduce micro-fracturing while removing all of the deteriorated concrete. Also price comparisons between the two methods were made using costs from several maintenance projects and also one bridge rehabilitation construction project and previous maintenance and contract work.

Results and Discussion (Evaluation)

Hydrodemolition, Fall 1996

The Missouri Department of Transportation, MoDOT, first tried hydrodemolition for the repair and concrete overlay of a bridge by maintenance contract on St. Mary's Way over I-44 in Franklin County just southwest of St. Louis. (Figure 1) The cost was $ 12,000 for one pass of the hydroblast machine over the whole bridge, 5,800 square feet, or $ 2.06/sf. This price included vacuuming up the debris and dumped on site. MoDOT maintenance forces were used to haul the rubble away. MoDOT also had to set up straw bail dams to catch the solids in the water before it was allowed to enter the roadway ditch. The effluent was checked by MoDOT to supply information on turbidity to the Missouri Department of Natural Resources to make sure it passed clean water standards.

Figure 1: Hydro machine in action. (Note the rubble in Front, compared to the milled deck behind.)

Figure 2: Note the straw bails covered with burlap at the end of the bridge to filter waste water.

The biggest concern from this first project was about the vacuum truck backing onto the rebar mat and bending it down where concrete was removed below the top mat. The heavy truck (Figure 3) worked all right here. However, if a lot of reinforcing steel is showing, plywood would need to be placed under the truck tires to distribute the load better. Alternately, a hand guided vacuum, or one with a boom (Figure 4), which didn't have to travel over the rebar could be substituted for the truck.

Figure 3: Heavy, self contained vacuum truck .
Note: vacuum nozzle located in front of the rear wheel works very well to pick up debris.

Figure 4: Vacuum truck with hose on boom; can stay off rebar mat but is slower picking up debris.

Hydrodemolition, 1997

A second hydrodemolition project was done again by maintenance contract in the summer of 1997 on Bridge L-896, Franklin County, Rt. 100/I-44 only about a mile from the first bridge. Hydroblasting of 5,800 sf. was done for $ 1.25/sf. or a total price of $ 7250. This bridge received a full concrete overlay like Bridge L-868. It demonstrated that MoDOT could extend the life of a second bridge deck by relatively low cost hydrodemolition and repair with a dense concrete overlay. On both the 1996 bridge and this one, one step in the repair process, pouring concrete patches before overlaying, was also eliminated by pouring the patches and overlay at the same time (a monolithic concrete placement).

Under the same bid, hydrodemolition of unsound concrete and patching of 6 other bridges decks on the aging I-70 corridor in St. Louis was completed in a third of the time of conventional jack hammer repair done by MoDOT maintenance crews. (Figure 5) Prices were bid lump sum for each bridge and depended on the amount of square feet patched, they ranged from $ 1.33 sf. ($ 12/sy.) to $ 8.33/sf. ($ 75/sy.).

Repairing these 7 bridge decks (the complete overlay of Br. L-868 plus patching of 6 others) was done in 20 working days using hydrodemolition. It would have taken 60 days by normal hand methods.

Figure 5: Shows traffic control and containment of blast water while hydroblasting for patches in two center lanes of a four lane bridge.

Hydrodemolition, 1998

In 1998 the first construction contract specifying use of hydrodemolition was let for bridges A01741 E and A01741 W on Route I-44, Greene County near Springfield. This was also MoDOT's first contract allowing a monolithic pour after removal of deteriorated deck with a Latex Modified Concrete overlay. This eliminated the usual patching step in between by filling of excavated areas and overlaying with new concrete at the same time.

Because of staged construction, this project required two mobilizations of the hydrodemolition equipment. The westbound bridge was closed to traffic in 1998. The whole deck of the westbound structure, 7,100 sf., was completed. The hydrodemolition contractor returned in early 1999 to do the 7,100 sf. of the eastbound bridge (Figure 6). Even though two trips were required, it is believed the bid was lower than expected due to being able to hydroblast a fairly large amount of surface each trip. Also, no traffic control was needed since the bridges were shut down to traffic.

Figure 6: Finished hydrodemolition of half (background) of Bridge A0174 E. In the foreground, new latex modified concrete overlay. Note: 2" core holes in the overlay are where pull-off tests for direct tension were taken.

Hydrodemolition 1999

Bridge A-185R Ramp on Route I-70, St. Louis City was shut down due to construction in the area. MoDOT maintenance forces took this opportunity to again use hydrodemolition work to repair this bridge deck. A maintenance contract was let for hydrodemolition. The cost was \$29.09/sy (\$ 3.23/sf) which compared well with the only construction project MoDOT had let with hydrodemolition, discussed above, which bid at \$ 31.50/sy (\$ 3.50/sf). Poor concrete and a thin 6 1/2" upper deck on this type box girder bridge made it necessary to make two passes with the hydrodemolition machine set at 13,000 psi. (see Figure 7) One pass at the normal setting of 17,000 – 18,000 psi would have blown through the poor quality concrete. Hydrodemolition makes it easier to regulate than conventional methods with regards to how much concrete is removed when it's necessary to patch and keep open a badly deteriorated deck.

Figure 7: Poor concrete and a thin 61/2" upper deck made it necessary to do hydrodemolition at 13,000 psi . This is after the first pass (Note how clean the re-steel is in the foreground on the left). A second pass with the hydro-blaster was necessary to remove the island of unsound concrete left over the rebar in the center of the photo.

Bid prices for Hydrodemolition

On maintenance contracts the bid prices have stayed consistently low $ 1.25/sf. to $3.23/sf. Only one construction project has been let and the price was $ 3.50/sf, this is almost an order of ten times less than mechanical removal, which bid for $ 28.79/sf for partial depth and $ 32.99/sf. for full depth removal. The limiting factor in getting hydrodemolition bid in Missouri has been the lack of contractors and hydro machinery and the need for numerous mobilizations of the equipment on most projects. It should be noted that allowing larger areas of deck to be opened up for hydrodemolition may cause additional traffic control costs. A summary of the costs of hydrodemolition for the study projects is listed in Table2 below.

Table 2: Bid prices for Hydrodemolition

Location	Date	Total Area	Bid Price	Total Cost
Bridge L-868, St. Marys/I-44, Franklin Co.	Fall 1996	5,800 sf.	$ 2.06/sf.*	$12,000
Bridge L-896, Rt.100/I-44, Franklin Co	Summer 1997	5,800 sf.	$ 1.25/sf.*	$7,250
Patching of several bridges on I-70, St. Louis	Summer 1997		$ 12/sy to $ 75/sy ** ($ 1.33/sf to $ 8.33 /sf)	
Bridge A01741 E&W, I-44, Greene County	1998	14,220sf. (1580sy.)	$ 3.50/sf ($ 31.50/sy.)	$49,770
1st construction contract specifying use of hydrodemolition.				
Bridge A-185R Ramp I-70, St. Louis City	1999		$ 3.23/sf. ($ 29.09/sy.)	

* One pass over whole bridge , vacuumed up and dumped on site. Maintenance hauled away rubble.
** Prices ranged from $ 12/sy to $ 75/sy depending on the amount of area contracted

TESTING PROCEDURES

Hydrodemolition does not cause damage to the good concrete left in place. Milling and jack hammering leave micro-fractures in the surface of the concrete, which can cause poor bond to patching or overlay material. Note: during surface preparation the milling step cannot be excluded if specifying hydrodemolition because the hydroblasting requires a rough concrete surface to initiate the removal process. Milling is a separate bid item and no savings with regard to milling are realized by using hydrodemolition over jack hammering. However, all micro-fracturing caused from milling is later removed by hydrodemolition leaving a more sound substrate.

Direct tension or pull off strength testing was done on each project using the ACI-503R, Field Test for Surface Soundness and Adhesion, method. Testing was performed on the original concrete after milling and either hydrodemolition or jackhammer removal. Additionally, direct tension tests were taken through the overlay and patch material into the original deck after the new concrete reached required strength.

The limited testing performed on these bridges showed hydrodemolition resulting in average pull off strengths of the bond between the overlay and the hydrodemolition prepared deck to be 151 psi and 166 psi on maintenance work on bridges L-868 and A-135 Ramp respectively. (These pull off tests were taken after milling, hydrodemolition and the deck overlay was placed - see Notes: on Table 3 and Table 5) On the one construction contract using hydrodemolition, the average pull off strength was 121 psi. on Br. A-174. (see Notes: on Table 4) This compares to 80 psi pull off strength on Br. A-241 using jackhammer removal for patching and 140 psi pull off strength on a milled only area. (no jackhammer or hydrodemolition done in this area - see Notes: on Table 6) It was expected to get higher pull off strengths using hydrodemolition as the literature said strengths were up to twice as strong as surfaces using mechanical methods. It is believed that with a larger number of tests and with a more agile testing machine better results wouldl be obtained. The base plate of the tester used is very large (1 ft. x 1 ft) and testing on rough surfaces and around rebar made it hard to always ensure it was normal to the surface. Sweden has obtained pull off strengths up to 300 psi on testing of over 300 hydro blasted decks. (*Improving Concrete Bond in Repaired Decks.* Concrete International, September 1990)

Values for MoDOT testing are included in the tables below.

Table 3: Pull-Off Strength - Hydrodemolition

Bridge L-868, St. Mary's Way/I-44, Franklin Co. Tested: 10/2/97

Core No.	Tension, #	Pull Off, psi	Location of Failure
1	745	237	1/4" into overlay
2	**805**	**256**	**Interface, 50% old deck, 50% in overlay**
3	1080	344	1/4" into overlay
4	**230**	**73**	**Interface, only small part of overlay attached**
5	500	159	Middle of overlay, 1 3/4' down into overlay
6	**390**	**124**	**Interface about 75% old concrete**
Avg. Pull Off Strength = 199 psi			

Note: ACI calls for a minimum PO strength of 100psi. Only cores that break off at the interface give a true bond strength; Average of cores 2, 4 & 6 = **151 psi**.

Bridge A-174, I-44 EBL, Greene Co **Tested**: 07/16/1999
(Constructon hydrodemolitrion contract with 1.75 in. latex modified concrete overlay.

Location No.	Core No.	Tension, #	Pull Off, psi	Location of Failure
1	1	180	57	100% in base
1	2	1020	325	100% in base
1	**3**	**420**	**134**	**100% interface**
Avg. Pull Off Strength = 172 psi				
2	**1**	**340**	**108**	**100% interface**
2	2	520	166	Not recorded
2	3	120	38	50% old patch/50% interface
Avg. Pull Off Strength = 104 psi				
3	1	320	102	100% in base
3	2	420	134	100% in base
3	3	960	306	100% in base
Avg. Pull Off Strength = 180 psi				

Note: ACI calls for a minimum PO strength of 100psi. Only cores that break off at the interface give a true bond strength; location 1, core 3 and location 2 core 1: average **121 psi**.

Table 5: Pull-Off Strength – Hydrodemolition
Pull-Off Strength

Bridge A-135RP, I-70 WBL, ST. Louis Co **Tested: 3/01/00**

Location No.	Core No.	Tension, #	Pull Off, psi	Location of Failure
1	1	760	242	100% in epoxy
1	2	640	**204**	**Interface, 50% in base**
1	3	400	**127**	**100% interface**
1	4	980	312	100% in epoxy
Avg. Pull Off Strength = 191 psi				

Note: Only cores that break off at the interface give a true bond strength; location 1, core 2 and location 1, core 3 average **166 psi**

Table 6: Pull-Off Strength – Mechanical Methods

Stage 3 - Silica Fume Overlay poured March 22, 2000,
Control: Mechanical equipment used for concrete removal

Location: Bridge A-241, I-270 wbl, St. Louis Co. **Tested: 05/24/2000**

Location No.	Core No.	Tension, #	Pull Off, psi	Avg. Pull Off, psi	Location of Failure
(Silicafume overlay on top of patch)					
1 (sf/patch)	**3**	120	38		broke in orig. concrete-1 7/8" thick, sf patch 2 1/4" thick
1 (sf/patch)	4	380	121	80	broke at epoxy, 2" sf & 2 1/4" limestone patch
1 (sf/patch)	1	100	**32**		**broke @ interface of overlay & orig. deck, no patch-2" thick sf**
1 (sf/patch)	2	400	**127**		**broke at interface-2"sf,no patch**
(Silicafume overlay on top of milled surface only)					
2 (sound sf)	5	360	**115**		**broke @ interface w/deck, very smooth-2 1/16" sf overlay**
2 (sound sf)	6	400	**127**	**140**	**broke @ interface w/deck, Interface rough 2 1/2" thick sf**
2 (sound sf)	7	560	**178**		**broke 100% @ interface w/orig. deck, interface smooth surface-**

sf = silica fume overlay

Note: Only cores that break off at the interface give a true bond strength;
For the cores over patches, core1and core 2 average **80 psi**,

Conclusions

The follow findings were made from monitoring of various maintenance and construction contracts using hydrodemolition:

1. Cost can range from $ 12/sy ($ 1.33/sf) to $ 75/sy ($ 8.33/sf) depending on the amount of area contracted.
2. Hydrodemolition does not cause damage to the good concrete left in place. Milling and jack hammering leave micro-fractures in the surface of the concrete, which can cause poor bond to patching or overlay material.
3. In direct tension or pull off testing, limited field data has shown pulloff strengths between overlays or patches and surfaces prepared by hydrodemolition of (121-161 psi) slightly higher than a jack hammered surface (80 psi) or a milled only surface (140 psi).
4. Pulloff strengths for hydrodemolition prepared surfaces averaged 150 psi, which was not as high as expected. It is believed a bigger sample is needed and that with more testing the average would rise. Also there were problems keeping the pulloff tester at a perfect right angle to surface, which would cause lower readings. Sweden claims of pull off strengths for hydrodemolition prepared decks at least twice as strong as those od decks prepared using conventional methods. Sweeden has used hydrodemolition on over 300 bridges.[1]
5. Hydrodemolition leaves the rebar and deck ready in one operation.

Recommendations

1.) Results of this study show that hydrodemolition should be used on all construction projects where the cost of mobilization isn't prohibitive. Costs can become prohibitive because of many small spread out work zones caused by zoned repairs on structures with concrete superstructures integral with the deck. Costs also go up because of staged construction and difficult traffic control plans, or because hydrodemolition equipment isn't available in the area. However, the advantages gained from not damaging the remaining concrete as well as the speed of preparation of the existing reinforced concrete will far outweigh any additional costs and can save MoDOT and the contractors money. It is estimated that at least one-quarter of the bridge decks contracted by MoDOT for rehabilitation each year meet the criteria that could use hydro demolition and even be more economical than conventional jackhammer methods. It is believed that equipment and the number of contractors available to do hydrodemolition should increase and the bid prices go down as this new technology establishes itself.

2.) Maintenance bridge repair crews statewide should try to employ hydrodemolition whenever possible on bridge decks with good service ratings that are expected to remain in use for a long time. A video recording of the process was made on the first bridge using hydrodemolition in the St. Louis district and has been distributed to all district maintenance units to let them familiarize themselves with the process.

Principal Investigator and Project Members

John Wenzlick was the principal investigator for RDT with help in reporting by Anika Careaga and field testing by Steven Clark. Hydrodemolition work in the St. Louis area was initiated and coordinated by Pat Martens, District Bridge Inspection Engineer. Testing done on I-44, Greene County project was coordinated through Jim Blackburn, Resident Engineer in Buffalo, Mo. Testing done on the I-270, St. Louis County was coordinated through Lucy Smith, Senior Construction Inspector.

Affected Business Units and Principal Contact

All district maintenance and design personnel as well as Bridge Design should consider the use of new hydrodemolition techniques for repair of bridge decks. John 'JD' Wenzlick in Research, Development and Technology or Pat Martens of District 6 Maintenance can be contacted for further information.

Technology Transfer

Designers should use this report to promote the use of hydrodemolition in areas where it can be expected that the bid price will be close to that of conventional mechanical repair methods. Contractors should be more receptive when they find how much quicker it is and also the little or no preparatory work needed before pouring new concrete. Reduced time and preparation costs should outweigh the higher capital equipment costs as more subcontractors get into the hydrodemolition business.

Districts wanting to do hydrodemolition with their own maintenance forces already have an excellent videotape describing the process that was distributed statewide back in 1997. The video covers all steps in the hydrodemolition process, just as they were done on the St. Marys/I-44 bridge in Franklin County.

Bibliography

1. Silfwerbrand, Johan *Improving Concrete Bond in Repaired Decks.* Concrete International, September 1990

2. American Concrete Institute, P.O. Box 9094, Farmington Hills, MI 48333, *ACI Manual of Concrete Practice, 1997*

Exhibit 26: Test Report from WJE

WJE

January 30, 2012
WJE No. 2012.0222

Prepared for:

Prepared by:

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Petrographic studies of two concrete cores, reportedly from the Davis-Besse Nuclear Generating Station in Ohio, were requested by Dr. Yunping Xi, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, on behalf of Performance Improvement International, in Oceanside, California. According to Dr. Xi, the cores had been taken from the wall of a nuclear containment structure at the power station. Discovery of a crack in the structure raised concern for its cause(s) and also for the general properties of the concrete mixture. Neither of the cores had been taken through the crack of concern. At Dr. Xi's request, one of the cores was chosen for petrographic examination, ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*, and one of the cores was chosen for point-count analysis, ASTM C 457, *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete.*

According to Dr. Xi, the nuclear containment structure is reportedly approximately 40 years old, and it is not directly exposed to water from the exterior. The crack, age unknown, is reportedly located parallel to and 3 to 4 inches from the exterior surface of the wall. We do not know the locations that the cores had been taken. The required water-cement ratio of the concrete is reportedly 0.51. Dr. Xi has reportedly conducted physical tests of cores from the structure, including compressive strength (approximately 7,600 psi), and splitting tension (approximately 900 psi). In addition, accelerated creep is reportedly underway.

The purpose of the petrographic studies was to determine the general properties of the concrete. The purpose of the air-void system analysis was to determine the air content and the parameters of the air-void system, as well as to determine the volumes of the paste and the aggregate. Dr. Xi also requested determination if the properties of the concrete mixture would contribute to the formation of the crack. This subject was briefly addressed.

[illegible]

The two cores received for the studies had been labeled F2-7923-4.5 and F4-791-2.5. The cores each had a diameter of 2-5/8 inches. Photographs of the cores in as-received condition are given in Figure 1.

Core F2 was 4-3/4 inches long and had a transverse separation that fit together exactly at 2-1/2 to 3 inches from its top end. Portions of the separated surface appear tear-like and formed-over, and do not go through coarse aggregate particles, suggesting that the separation may be a partial cold joint. The top end of Core F2 exhibited a moderately rough surface texture with exposed sand particles, as shown in Figure 2. Additional examination of the near-surface zone was conducted as part of the more detailed petrographic examination. The bottom end of Core F4 was a fractured surface that went through nearly all coarse aggregate particles in its path.

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Core F4 was 4 inches long and both of its ends had been saw cut. The core does not appear to have been compression tested. We do not know which end of the core had been closer to the exposed surface, nor the depth from the exposed surface that the core represents. One end of the core exhibited minor amounts of larger entrapped voids and the other end exhibited relatively abundant amounts of medium-size voids, as shown in Figures 3 and 4. This variation in consolidation is one indication of less-than-optimal consolidation.

Brief visual comparison of the cores showed that the concretes appear to be similar, although Core F2 appears to have overall gray paste with some variation in color and Core F4 has more uniform, light tan paste, as shown in Figure 1.

[illegible]

Point-count analysis of one of the cores was requested. This analysis enables determination of the volumes of the components of the concrete, including air content, more closely than is possible by petrographic examination alone. Precisely knowing the volumes of air, paste, and aggregate is helpful to determine if the concrete has a proper air-void system for frost durability and also if the concrete corresponds to its mix design.

[illegible]

The specimen was analyzed utilizing the modified point-count method of ASTM C 457. In a point-count analysis, a lapped (polished) specimen is placed on a movable stage under a stereomicroscope. The stage traverses back and forth in regular, automated increments over the entire surface of the sample. Each time the stage stops, the component of the concrete (aggregate, paste, or air void) that falls directly under the microscope cross-hairs is recorded by the operator in a specialized computer program. Each air void that the cross hairs pass over is also recorded. Nearly 2000 points over a minimum area of approximately 12 square inches are counted to obtain a statistically representative sampling of the concrete.

[illegible]

Core F4 was chosen for the point-count analysis because it did not contain an internal separation. Two longitudinal slices were prepared from the core to obtain sufficient area to analyze in accordance with the method. Figure 5 shows the specimens of Core F4 that were analyzed. Differentiation between coarse and fine aggregate in this concrete was not readily apparent at the magnification of the analysis; therefore, the total aggregate volume is reported. The results of the point-count analysis are given in the following table, and are discussed in more detail in the remainder of the report.

[illegible]

[illegible]	[illegible]	[illegible]
Paste	28.7 %	NA
Total Aggregate	66.3 %	NA
Air	5.0 %	4-1/2 to 6 %

[illegible]	[illegible]	[illegible]
Specific Surface	207 in.^{2}/in.^{3}	$\geq$ 600 in.^{2}/in.^{3}
Spacing Factor	0.024 in.	$\leq$ 0.008 in.

[illegible]

Core F2 was chosen for detailed petrographic examination. Significant features of the core are described first, followed by characteristics of the concrete mixture. Limited examination of Core F4 was also conducted to briefly address differences in the concrete mixture represented by the cores. This discussion is given in the last part of the petrographic examination section. A photograph of the lapped longitudinal cross-section of Core F2 is given in Figure 6.

[illegible]

In a typical petrographic examination, a sample is examined both visually and microscopically in accordance with the applicable procedures of ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*. A petrographic examination consists of a progressive series of qualitative observations that are interpreted to draw conclusions about the composition, quality, and probable cause(s) of a number of problems associated with concrete.

A thorough visual examination of a sample is the first step in a petrographic examination. The sample is then generally sliced in half lengthwise to prepare it for more in-depth examination. One of the resultant halves is lapped (polished) to accentuate the appearance of the components of the concrete (cement paste, air-void system, and aggregates). Lapped specimens, as well as freshly fractured surfaces, are examined visually and using a stereomicroscope at magnifications up to about 100X to further characterize any deterioration and to evaluate the concrete's individual components.

A typical full petrographic examination also involves observation of powder mounts of the paste, thin sections of the concrete, or both, using a petrographic, transmitting-light microscope at magnifications up to 500X. Examination of these specimens allows the composition of the cement paste, mineral components of the aggregate, and other significant characteristics of the concrete to be studied in detail.

[illegible]

Petrographic examination of Core F2 revealed a number of significant features about the concrete, including its near-surface zone, and the presence of relatively abundant amounts of secondary deposits.

Near-Surface Zone

Fine aggregate particles are exposed slightly in relief along the entire exposed surface of Core F2. The surface exhibits a light tan portion and a gray portion; the gray portion is thin and appears to have been applied over the tan section. Closer observation revealed that the tan section, about 1/16 to 1/8 inch thick, is a cement-based coating, and is not water-repellant. The gray portion of the coating is also cement based, contains moderate amounts of polymeric fibers, and has a pronounced water repellant nature. The coating is well bonded to the substrate. Figure 7 illustrates the near-surface zone of Core F2.

The outer approximately 1/4 to 1/2 inch of Core F2 beneath the coating is variable, mainly with respect to the paste and also with consolidation. The paste varies widely in color, suggesting a significantly variable water-cement ratio. A number of unconsolidated voids are present in the near-surface zone. These voids range from microscopic up to 3/4 inch in diameter and are irregularly shaped, suggesting a harsh mixture, poor consolidation, or both.

Drying-shrinkage microcracks oriented perpendicular to the exposed surface are present. A few microcracks extend to a depth of about 1 inch, the rest are generally 1/8 inch deep. Paste carbonation is typically about 1/4 inch; although it is up to 3/4 inch along the deeper drying-shrinkage microcrack and also in a poorly consolidated zone. Discoloration of the paste due to carbonation of the cement paste and an elevated water-cement ratio zone are visible in Figure 7.

Based on the characteristics listed above, the general quality of the near-surface zone is considered to be relatively poor. The variations in the near-surface zone continue in to the main body of the concrete, but to a slightly lesser degree. These characteristics are discussed in more detail in the next section.

Secondary Deposits

Secondary deposits thinly line virtually all of the air voids throughout the concrete in Core F2. The deposits have an approximately equal thickness throughout and appear to consist of ettringite and calcium hydroxide. The presence of these deposits in air voids typically suggests long term exposure to moisture migrating through the concrete. However, the occurrence of deposits lining essentially every void in an approximately equal thickness layer is a somewhat unusual pattern that may indicate an internal reaction.

□□□□r□□□□M□□□□r□□

The paste, air-void system, and aggregate in the concrete mixture represented by Core F2 were examined in detail microscopically to determine their characteristics. The results of the point-count analysis of Core F4 are also discussed in this section because the concretes appear to have a similar characteristics.

Paste

The volume of the paste represented by Core F4 was measured by point-count analysis to be 28.7 percent, and the volume of the paste represented by Core F2 appears to be about the same as Core F4.

Significant color variability of the paste, some of which is visible without a microscope on the lapped cross-section, is present in the concrete. Moderate amounts of small to moderately elongated (1/16 to 3/8 inch), soft zones of paste with an elevated to significantly elevated water-cement ratio are present throughout the concrete. Small unconsolidated air voids are commonly associated with these zones, which are generally located adjacent to coarse aggregate particles. Figure 8 illustrates one of these zones, which represent areas of weakness and increased potential for fluid penetration. In addition, minor amounts of zones of dark paste that have an especially low water-cement ratio are present in aggregate indentations. Trace, insignificant, amounts of microcracks were detected in the main body of the paste.

The paste appears to be well hydrated, and moderate amounts of relatively coarsely crystalline unhydrated portland cement clinker particles are present. No supplementary cementitious materials were detected in the paste. Optical and physical properties of the paste indicate that the majority of the concrete represented by Core F2 has a low water-cement ratio, estimated to be 0.38 to 0.42. Moderate amounts of areas with an even lower water-cement ratio and an elevated water-cement ratio are also commonly present. Based on the estimated water-cement ratio and the measured volume of the paste in Core F4, the cement content of the concrete is estimated to be 6 +/- 1/2 bags per cubic yard.

Air-Void System

The measured air content of Core F4 was 5.0 percent. The calculated parameters of its air-void system, namely specific surface and spacing factor, indicate that according to industry standards the concrete does not have an air-void system that is adequate for protection against cyclic freezing and thawing damage in moist conditions.

The air-void system of Core F2 appears to be similar to that of Core F4. The concrete is poorly air-entrained and less-than-optimally consolidated. In addition, the entrained air voids are not uniformly distributed. No large honeycombed zones were detected in the concrete; however, moderate amounts of zones concentrated with larger entrapped voids are present, as shown in Figure 5. Other areas that contain few, if any, air voids are also present. These characteristics were also noted along the circumference of the core

A relatively small proportion of the air-void system consists of small, spherically-shaped, air voids that are consistent with entrained air. These voids are sometimes concentrated adjacent to sub-round entrapped voids. The majority of the air-void system consists of small to medium size (up to about 1/4 inch), sub-spherically to irregularly-shaped voids that are consistent with entrapped bleed water voids or unconsolidated voids.

According to ACI 318, *Building Code Requirements for Structural Concrete*, for concrete subjected to severe and moderate exposure with a 1 inch nominal maximum size aggregate, air contents of 6 and 4-1/2 percent are recommended, respectively. These air contents are normally considered to be -1 and +2 percent by project specifications. We do not know exactly what the exposure environment of the structure is. ACI 318 allows a one percent decrease in air content for concrete that has a compressive strength over 5,000 psi. Therefore, based on air content alone, the 5.0 percent air content of the concrete would appear to be acceptable.

However, neither of the parameters of the air-void system meets the requirements for a durable air-void system, as indicated by ACI 201, ACI 211, and Design and Control of Concrete Mixtures (PCA publication). The recommended specific surface for adequate freeze-thaw resistance of concrete is considered to be a minimum of 600 in.^{2}/in.^{3}. The recommended spacing factor of concrete that has adequate freeze-thaw resistance is 0.008 inch or less. The concrete represented by Core F4 does not meet these requirements.

Aggregate

The coarse and fine aggregate both consist of the same crushed limestone. The use of crushed limestone as the entire fine aggregate is somewhat unusual and likely gave the plastic concrete a tendency to be harsh. Point-count measured the total volume of the aggregate (coarse and fine) to be 66.3 percent.

The coarse aggregate has a maximum diameter of 1 inch. Particles are generally firm to moderately firm for a limestone, dense to somewhat vuggy (containing numerous voids), fresh (un-weathered), gray to light tan, equi-dimensional to slightly elongated, minimally absorptive, angular to subrounded, fine-grained, and relatively pure (minimal amounts non-calcareous minerals). Minor amounts of particles that are moderately soft, absorptive, or both also compose the coarse aggregate. The fine fraction of the aggregate has the same general properties as the coarse aggregate. Overall grading, distribution, volume, and physical and chemical soundness of the coarse and fine aggregates appear to be adequate.

Very occasionally, partial rims of pale-orange, carbonated paste surrounding aggregate particles are present. The exact cause(s) of this occurrence is not known, although it could be related to deep penetration of atmosphere into the concrete.

[illegible]

Core F4 was briefly examined petrographically as part of the point-count analysis and also for general comparison with Core F2. Figure 9 is a photograph of the lapped cross-sections of Cores F2 and F4 for comparative purposes. A full-depth, tight, meandering microcrack extends the full thickness of Core F4. The crack goes through a few aggregate particles in its path, indicating that it most likely did not occur early in the life of the concrete. Air voids in Core F4 contain secondary deposits linings in the same abundance and pattern as those of Core F2. Variability of the water-cement ratio and air-void system are evident in Core F4, although the variations do not appear to be as common as in Core F2. Additional petrographic examination of Core F4 would be necessary to determine its water-cement ratio and other properties more closely.

[illegible]

The concrete, especially Core F2, is moderately variable with respect to its consolidation, air-void system, and water-cement ratio. Less-than-optimal initial mixing or less-than-optimal remixing after re-tempering are the two most common causes of this type of variability. Localized zones of weakness and higher absorption are expected from the variability. The overall estimated water-cement ratio of the concrete was 0.38 to 0.42, which is significantly less than the reported required water-cement ratio of 0.51.

The parameters of the air-void system of the concrete represented by Core F4 did not meet industry requirements for freeze/thaw durability. However, the higher apparent compressive strength of the concrete provides resistance to water absorption, which is a key component to freeze-thaw deterioration. The widespread nature of secondary deposits in air voids throughout the length of the concrete given the reported lack of exposure to moisture from outside the structure is usual. Long term exposure to moisture is clearly indicated. Perhaps the environment inside the structure has a high humidity, is warm, or both.
No evidence of freeze-thaw deterioration was detected in either core.

Dr. Xi reported that the crack of concern was located 3 to 4 inches below the surface and that no cracks are present between the crack and the exposed surface. This pattern is not consistent with cyclic freeze-thaw deterioration. Additional cracks would be expected closer to the surface in typical freeze-thaw damage.

General properties of a concrete mixture that may promote cracking, include but are not limited to, a high volume of paste, an elevated water-cement ratio, and unsound aggregate. The overall volume of paste represented by this concrete is not considered to be high. A paste volume greater than approximately 30 to 32 percent would be considered to be relatively high. An elevated water-cement ratio may promote cracking due to high shrinkage. The overall estimated water-cement ratio of the concrete is considered to be low. The crushed limestone aggregate appears to be chemically and physically sound. In general, variability in concrete may lead indirectly to cracking if areas of poor consolidation, elevated water-cement ratio, or concentration of air voids are extensive; however, this does not appear to be the case in the concrete represented by the cores.

Many other factors may contribute to the formation of a crack, such as reinforcement configuration and structural causes. If more insight into the cause(s) of the crack or to the extent of variation in the concrete is desired, petrographic examination of additional cores is recommended, including cores taken through the crack.

[illegible]

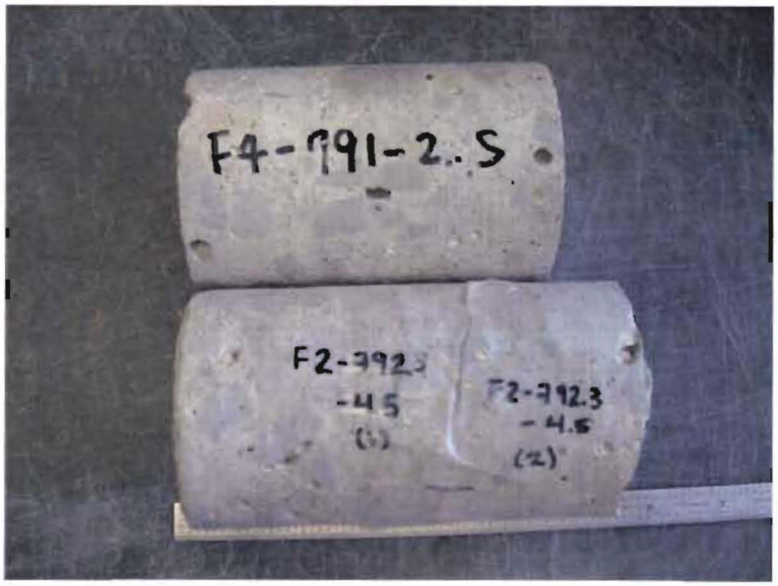

Figure 1. Cores in as-received condition. Core F4 (top) is overall light tan and Core F2 is more gray.

Figure 2. Exposed end of Core F2, showing sand particles elevated slightly in relief above the coating. The gray and tan portions of the surface are readily apparent.

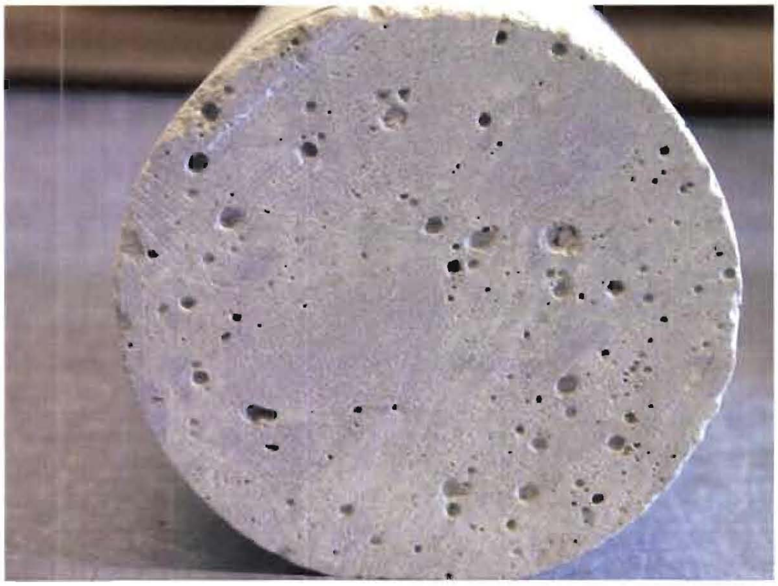

Figure 3. One of the cut ends of Core F4. Relatively abundant amounts of entrapped air voids are visible. The other saw cut end of the core is shown in the next figure.

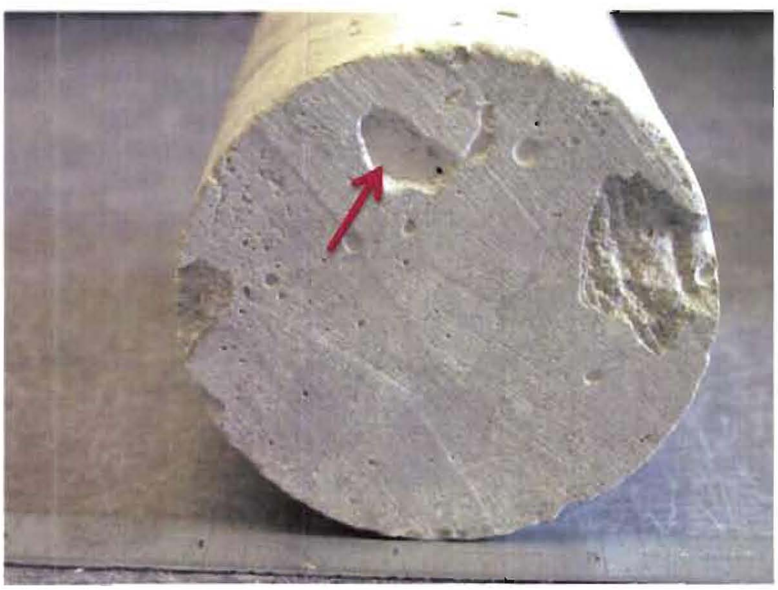

Figure 4. Other cut end of Core F4 showing one relatively large entrapped void (arrow) and two fresh-appearing chips off the side of the core that were likely produced by the coring or cutting operation.

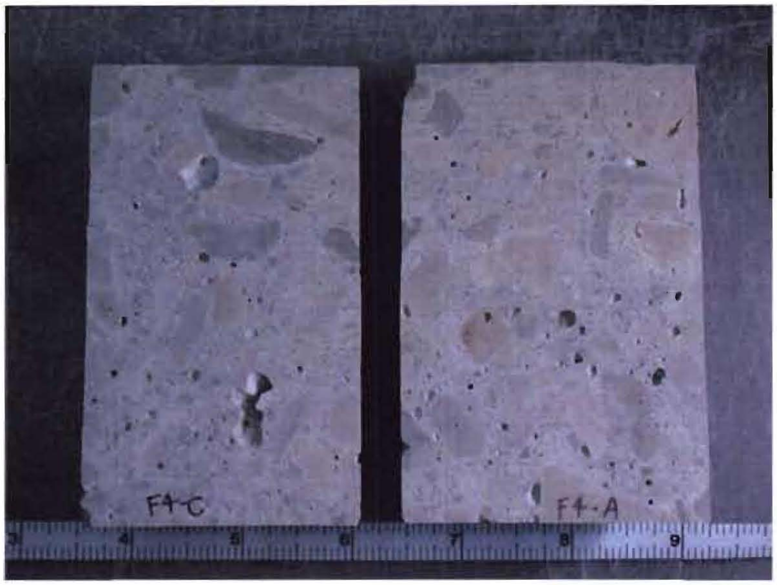

Figure 5. Lapped cross-sections of Core F4 that were analyzed by point-count analysis. The bottom approximate half of the concrete shows significant amounts of entrapped voids, including one larger unconsolidated void in the specimen on the left, while the upper halves show less air voids.

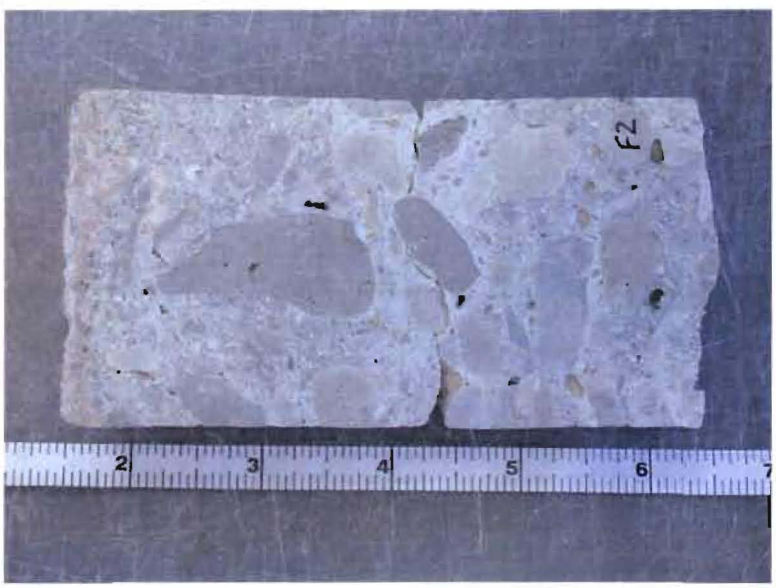

Figure 6. Lapped cross-section of Core F2. The exposed surface of the core is on the left side of the photograph.

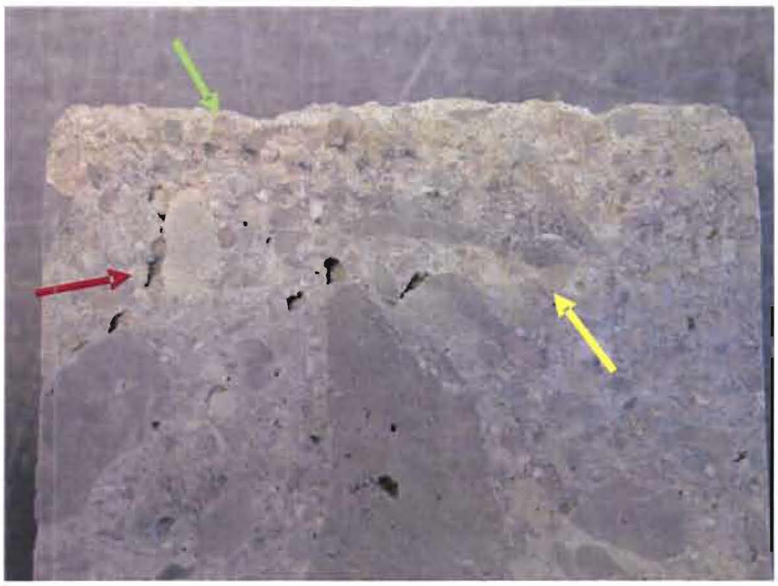

Figure 7. Near-surface zone of Core F2. For scale, the specimen is 2-1/2 inches wide. The most prominent feature is the pale-orange discoloration of the paste caused by carbonation. The deepest depth of carbonation is indicated by the yellow arrow. A number of unconsolidated air voids are present up to a depth of 3/4 inch on the left half of the near-surface zone. The red arrow indicates one discontinuous string of unconsolidated voids. One section of the coating is shown in by the green arrow.

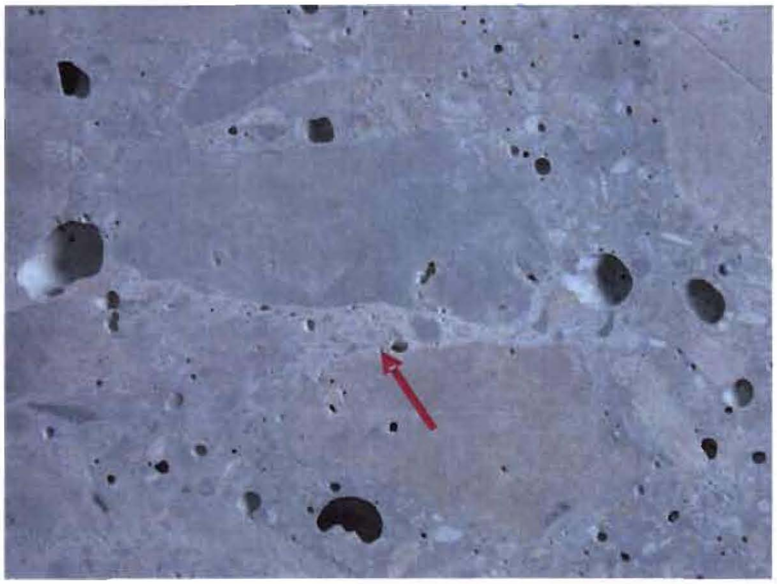

Figure 8. Closer view of Core F2 showing water-cement ratio variation in the paste. As indicated by the arrow, an elongated zone (about 1 inch long) of light-colored elevated water-cement ratio paste is located below an aggregate particle.

Figure 9. Lapped cross-sections of Cores F2 (top) and F4. Slight differences in paste color are evident. Core F2, on top, exhibits overall more gray and light-colored paste, while Core F4 exhibits more uniformly-colored tan paste.

Exhibit 27: 347R-63 Guide to Form Work

TITLE NO. 64-33

Proposed Revision of ACI 347-63:

Recommended Practice for Concrete Formwork

Reported by ACI Committee 347

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Presents brief introductory statement on the need for formwork standards based on the fact that 35 to 60 percent of the total cost of the concrete work in a project in the United States is in the formwork. A section is given on engineer-architect specifications noting the kind and amount of specification the engineer or architect should provide the contractor. Since the committee concludes that formwork design and engineering, as well as construction, must be the responsibility of the contractor, the recommendations contained in the report are directed to that group. However, an understanding of these recommendations by engineers and architects will aid these groups in their specification functions.

The report is divided into five chapters: 1. Design, 2. Construction, 3. Materials for Formwork, 4. Forms for Special Structures, and 5. Formwork for Special Methods of Construction.

Keywords: aggregates; aluminum; anchors; architectural concrete; bridges (structures); buildings; canal linings; civil defense; coatings; composite construction; concretes; construction; construction materials; culverts; drawings; falsework; fiberboard; folded plates; form removal; formwork (construction); glass fibers; hangers; inserts; insulating board; loads (forces); lumber; mass concrete; paperboard; parting agents; plastics; plywood; precast concrete; preplaced aggregate concrete; pressure; prestressed concrete; reinforced concrete; roofs; safety; safety factor; shells (structural forms); shelters; shoring; slipforms; specifications; steels; structural design; supports; ties; tolerances; tunnels (transportation); underground construction; underwater construction; viaducts.

CONTENTS

NEED FOR STANDARD

Since the cost of the formwork for a concrete structure may be 35 to 60 percent of the total cost of concrete work in the project, its design and construction demand sound judgment and planning to achieve adequate forms that are both economical and safe. The engineer or architect responsible for the successful completion of any concrete structure usually will include in his specifications provisons for stripping time, reshoring of concrete in place, inspection, and approval of formwork procedure which could affect the strength and appearance of the completed structure.

Neat, well-built, heavily braced forms may still fail due to inadequately tied corners or insufficient provision against uplift. Form failures have occurred when shoring has been improperly spliced, inadequately cross braced, or was otherwise inadequate to resist all possible stresses. Shores supported on previously completed floors usually can be assumed to have equal unit bearing. However, shores supporting forms for the first level above ground often are supported on "mudsills" and may not have uniform bearing. This condition may occur when mudsills rest on soft ground, on backfill recently placed and perhaps softened by surface water, or on frozen ground which may thaw out in numerous ways. Unequal settlement of mudsills seriously changes shore reactions, and may cause serious overloading of shores which do not settle as much as others.

Proper engineering design of formwork often saves contractors more than the saving from the use of poorly designed forms. Formwork is generally more economically constructed on the ground and under the contractor's yard inspection than in the air under field conditions. A carpenter not experienced in formwork may find it difficult to conceive of forms as pressure vessels which need to be adequately tied together, braced, and anchored to resist uplift and to be capable of resisting forces in several directions. Proper detailed instruction is therefore necessary.

If working drawings are made for formwork, the necessary detailed study of the contract drawings may uncover omissions and dimensional errors. Field work is expedited and the structural engineer can see how his design is being interpreted by the contractor. Other benefits may be shorter job duration, avoidance of delays in field operations, more efficient re-use of forms, and better utilization of material and men. There should be "notes to the erector" on such drawings to eliminate need for referring to specific field customs.

TABLE 3.2 (cont.)—FORM MATERIALS AND STRENGTH DATA FOR DESIGN

Item	Principal use	Specifications and design data sources
Plastics: Polystyrene Polyethylene Polyvinyl chloride	Form liners for decorative concrete	Manufacturers' data
Rubber	Form lining and void forms	Manufacturers' data
Form ties, anchors, and hangers	For securing formwork against placing loads and pressures	Manufacturers' specifications; see Section 1.3 of this standard for recommended safety factors ASTM
Plaster	Waste molds for architectural concrete	Manufacturers' recommendations
Coatings	Facilitate form removal	Manufacturers' specifications
Steel joists	Formwork support	"Standard Specifications and Load Tables for Open Web Steel Joists," Steel Joist Institute
Steel frame shoring	Formwork support	"Recommended Steel Frame Shoring Erection Procedure," Steel Scaffolding and Shoring Institute; also manufacturers' data
Form insulation	Cold weather protection of concrete	ACI 306-66 and manufacturers' specifications

3.3.2—*Recommendations*

3.3.2.1—Recommended factors of safety for ties, anchors, and hangers are given in Section 1.3.1. Yield point of the material should not be exceeded.

3.3.2.2—The rod or band type form tie, with supplemental provision for spreading the forms and a holding device engaging the exterior of the form, is the common type used for light construction.

The threaded internal disconnecting type is more often used for formwork on heavy construction such as heavy foundations, bridges, power houses, locks, dams, and architectural concrete.

Removable portions should be of a type which can be readily removed without damage to the concrete and which leave the smallest practicable holes to be filled.

Where ties are permitted in construction of water-retaining structures, they should be designed to prevent seepage or flow of water along the embedded tie.

Although there is no general uniformity at the present time, a minimum specification for form ties should require that the bearing area of external holding devices be adequate to prevent severe crushing of form lumber.

3.3.2.3—Form hangers must support the dead load of forms, weight of concrete, and construction and impact loads. Form hangers should be symmetrically arranged on the supporting member to minimize twisting or rotation of supporting members.

3.3.2.4—Where the concrete surface is to be exposed and appearance is important, the proper type of form tie or hanger which will not leave exposed metal at the surface is essential.

3.4—Form coatings or release agents

3.4.1—*Form coating*—Form coatings or sealers are usually applied in liquid form to contact surfaces either during manufacture or in the field to serve one or more of the following purposes:

(a) Alter the texture of the contact surface

(b) Improve the durability of the contact surface

(c) In addition to (b) above, to facilitate release from concrete during stripping

(d) Seal the contact surface from intrusion of moisture.

3.4.2—*Release agents*—Form release agents are applied to the form contact surfaces to prevent bond and thus facilitate stripping. They may be applied permanently to form materials in manufacture or applied to the form before each use. When applied in the field before each use, care must be exercised to prevent coating adjacent construction joint surfaces or reinforcing steel.

3.4.3—*Manufacturers' recommendations*—Manufacturers' recommendations should be followed in the use of coatings, sealers, and release agents, but independent investigation of their performance is recommended before use. Where surface treatments such as paint, tile adhesive, or other coatings are to be applied to formed concrete surfaces, be sure that adhesion of such surface treatments will not be impaired or prevented by use of the coating, sealer, or release agent.

dling the form during erection or installation for concrete placement plus the method of bracing and anchorage during normal operation.

4.7.2.2.4—In the case of the tunnel arch form, whether it is intended for use with the unit or bulkhead system of concrete placement or is restricted to use with the continuously advancing slope method (see Section 4.7.2.3).

4.7.2.2.5—When placement of concrete by pumping or pneumatic methods is anticipated, the capacity and working pressure of the prime mover and the size, length, and maximum embedment of the discharge line should be as assumed in the design. Also, when the design provides for a method of placement other than by sustained pumping via a buried slick line, it should be clearly stated that the design pressures would be exceeded if sustained pumping were adopted.

4.7.2.3—Construction—The two basic methods of placing a tunnel arch entail problems in the construction of the formwork that require special provisions to permit proper re-use. These two basic methods are commonly known as the "bulkhead method" and the "continuously advancing slope" method.

The former is used exclusively where poor ground conditions exist, requiring the lining to be placed concurrently with tunnel driving operations. It is also used when some factor, such as the size of the tunnel, the introduction of reinforcing steel, or the location of construction joints precludes the advancing slope method. The advancing slope method, a continuous method of placement, usually is preferred for tunnel driven through competent rock, ranging between 10 and 25 ft in diameter and at least 1 mile in length.

The arch form for the bulkhead method is usually fabricated into a single unit between 50 and 150 ft long which is stripped, moved ahead, and re-erected using screw jacks or hydraulic rams. These are permanently attached to the form and supporting traveling gantry. The arch form for the continuously advancing slope method usually consists of eight or more sections that range between 15 and 30 ft in length. These are successively stripped or collapsed, telescoped through the other sections and re-erected using a form traveler.

Although the minimum stripping time for tunnel arch forms usually is established on the basis of experience, it can be safely predetermined by tests in the laboratory. It is recommended that at the start of a tunnel arch concreting operation, the minimum stripping time be 12 hr for exposed surfaces and 8 hr for construction joints. If the specifications provide for a reduced minimum stripping time based on site experience, such reductions should be in time increments of 30 min or less and should be established by laboratory tests and visual inspection and surface scratching of sample areas exposed by opening the form access covers. Arch forms should not be stripped prematurely when unvented ground water seepage could become trapped between the rock surface and the concrete lining.

4.7.2.4—Materials — The choice of materials for underground formwork usually is predicated on the shape, degree of re-use and mobility of the form, and the magnitude of pump or pneumatic pressures to which it is subjected. Usually, tunnel and shaft forms are made of steel, or a composite of wood and steel. Experience is of paramount importance in the design and fabrication of a satisfactory tunnel form, due to the nature of the pressures developed by the concrete, placing techniques, and the high degree of mobility usually required.

When re-use is not a factor, plywood and tongue-and-groove lumber sometimes are used for exposed surface finishes, but more consideration may be given to wood sheathing because the high humidity often precludes the normal shrinkage and warping.

CHAPTER 5—FORMWORK FOR SPECIAL METHODS OF CONSTRUCTION

5.1—Recommendations

The applicable provisions of Chapters 1, 2, and 3 also apply to the work covered in this chapter.

5.2—Preplaced aggregate concrete

5.2.1—*Discussion*—Preplaced aggregate concrete is made by injecting (intruding) mortar into the voids of a preplaced mass of clean, graded aggregate. For normal construction the preplaced aggregates are wetted and kept wet until the injection of mortar into the voids is completed. In underwater construction, the mortar displaces the water and fills the voids. In both types of construction this process can create a dense concrete having a high content of coarse aggregate.

The injected mortar contains water, fine sand, portland cement, pozzolanic filler, and an additive designed to increase the penetration and pumpability of the mortar. The coarse aggregate is similar to coarse aggregate for conventional concrete. It is well washed and graded from ½ in. to the largest size practicable. After compaction in the forms, it usually has a void content ranging from 35 to 45 percent.

5.2.2—*Recommendations*

5.2.2.1—Design considerations

5.2.2.1.1—Lateral pressure of concrete—Due to the method of placement, the lateral pressures on formwork are considerably higher than those developed for conventional concrete as given in Section 1.2.2.

Forms, ties, and bracing should be designed for the sum of

(a) The lateral pressure of the coarse aggregate as determined from the equivalent fluid lateral pressure of the dry aggregate using the Rankine or Coulomb theories for granular materials; or a reliable bin action theory; and

(b) The lateral pressure of the injected mortar as an equivalent fluid weighing 130 lb per cu ft. The time required for the initial set of the mortar (from 6 to 24 hr) and the rate of rise (1 to 2 ft per hr) should be ascertained. The maximum height of fluid to be assumed in determining the lateral pressure of the mortar is the product of the rate of rise (ft per hr) and the time of initial set in hours.

The lateral pressure for the design of formwork at any point is the sum of the pressures determined from Steps (a) and (b) for the given height.

5.2.2.2—Construction—In addition to the provisions of Chapter 2, the forms must be literally "mortar-tight" because preplaced aggregate concrete entails forcing mortar into the voids of the coarse aggregate. The increased lateral pressure usually requires that the workmanship and details of formwork be of better quality than formwork for conventional concrete.

5.2.2.3 — Materials for formwork — Tongue-and-groove lumber is preferred for exposed surfaces; the joints between boards permit the escape of traces of mortar. For unexposed surfaces, mortar-tight forms of steel or plywood are acceptable. Prefabricated panel-type forms usually are not suitable because of the difficulty in making mortar-tight seals between panels. Absorptive form linings are not recommended because they permit the coarse aggregate to indent the lining and form an irregular surface. Form linings, such as hardboard on common sheathing, are not successful because they do not withstand the external form vibration normally required.

5.3—Slipforms

5.3.1—*Discussion*—Placing of concrete by use of slipforms is similar to an extrusion process. Plastic concrete is placed or pumped into the forms, and the forms act as moving dies to shape the concrete. The rate of movement of the forms is regulated so that the forms leave the formed concrete only after it is strong enough to retain its shape while supporting its own weight. Formwork of this type can be used for vertical structures such as silo and storage bins, bridge piers, shaft type buildings, water tanks, and missile launchers or for horizontal structures such as tunnel inverts, water conduits, drainage channels, canal linings, and paving. Sometimes there may be fixed forms on one side (such as sheathing, rock, earth, or existing masonry) and a sliding form on the other. For other types of work, there are sliding forms on both sides.

Vertical slipforms are usually moved by jacks which ride on smooth steel rods or pipe embedded in or attached to the hardened concrete, whereas horizontal slipforms generally move on a rail system or on a shaped berm. Working decks, concrete supply hoppers, and worker's or finisher's scaffolding, where required, are attached to and carried by the moving formwork.

The vertical or horizontal movement of forms may be a continuous process carried on 24 hr of the day until the structure is completed, or in a planned sequence of finite placements.

This section is divided into two parts: *vertical slipforms*; and *horizontal slipforms* such as used on drainage channels and canal linings.

Slipforms used on such structures as tunnels and mine shafts should comply with the applicable provisions of Section 4.7. Slipforms used on mass concrete structures such as dams should comply with the applicable provisions of Section 4.6.

5.3.2—*Recommendations*

5.3.2.1—Vertical slipforms

5.3.2.1.1—Design considerations—Slipforms should be designed and constructed and the sliding operation should be carried out under the immediate supervision of a person or persons experienced in slipform work.

In the design of the forms in which jacks on vertical rods are used, care must be taken to place jacks in such a manner that the vertical loads are as nearly equal as possible and do not exceed the safe capacity of the jacks. The steel rods or pipe on which the jacks climb or by which the forms are lifted should be especially designed for this purpose. These rods must be properly braced where not encased in concrete. Jacking rods or pipes may be left in concrete or with-drawn as conditions permit but splices and low bond value must be given special consideration if they are to be used as reinforcement.

The design of the yokes must provide for adequate clearance to install horizontal reinforcing bars and embedments in their correct locations prior to their submergence in the rising concrete.

The hydraulic-electric jacking system, which provides for the precise simultaneous movement of the entire form in small preselected increments of approximately 1 in. at 5 to 10-min intervals, is recommended for large structures, especially when single units are involved.

Lateral and diagonal bracing of forms must be provided to insure that the shape of the structure will not be distorted beyond allowable tolerances during the sliding operation.

When slipforms are used for single unit structures in excess of 50 ft diameter, they are usually segmental, or are provided with a substantial center guide made of steel or concrete, to overcome the latent uncertainty of maintaining correct alignment of an otherwise unguided form.

Drawings should be prepared by a competent and experienced engineer employed by the contractor, showing the jack layout, formwork, working decks, and scaffolds.

5.3.2.1.2—*Loads*

(a) *Vertical loads*

1. In addition to the dead loads, live loads assumed for design of decks should not be less than the following:

Sheathing and joists	75 psf or concentrated buggy wheel loads, whichever is the greater
Beams, trusses, and wales	40 psf

2. Where working decks are used as a bottom form for cast-in-place construction, the deck must be designed for the dead load of the concrete construction plus any superimposed loads, and in no case less than the design loads given in Section 1.2. The deflection of the working deck should not exceed ⅛ in. or 1/360 of span, whichever is greater.

3. Vertical loads and possible torsional forces resulting from deck loads and friction of concrete on the forms must also be considered since the forms must act as trusses for the vertical loads between jacks. Knee braces should be provided for top wales where span between jacks exceeds 6 ft or where vertical loads are unusually heavy.

(b) *Lateral pressure of concrete*—The lateral pressure of fresh concrete to be used in designing forms, ties, bracing, and wales may be calculated as follows:

$$p = c_1 + \frac{6000R}{T}$$

where

$c_1 = 100$*

p = lateral pressure, psf

R = rate of concrete placement in ft per hr

T = temperature of concrete in the forms, deg F

Wales must be adequately nailed or bolted together to transmit shear due to lateral pressure of concrete and vertical posts should be placed between wales at lift points.

5.3.2.1.3—Construction and materials—Forms should be a minimum of 3 ft 6 in. high† and should be constructed of at least 1-in. board, ⅝-in. plywood, 10-gage minimum steel sheets, or other approved material. The 1-in. boards should be straight grained and center-matched and placed with the grain running downward and boards spaced 1/16 to ⅛ in. apart to allow for expansion when they become wet. Forms should be erected with slight draft, particularly for the inside faces so that the form is wider at the bottom than at the top.

Timber wales should be of 2- or 3-ply lumber at least one ply of which will be 2-in. material. The minimum depth of segmental wales for curved walls should be 4½ in. at the center after cutting.

Special care must be taken in building the forms and arranging the jacks so that the forms will draw straight and true without strain or twist. To avoid unplanned cold joints, especially when such an occurrence would adversely affect the integrity of the structure, it is essential that reserve jacking and placing equipment and standby con-

*It is felt that $c_1 = 100$ is justified because vibration is slight in slipform work, since the concrete is placed in shallow layers of 6 to 10 in. and because there is no revibration. However, for some applications such as for gastight or containment structures, additional vibration may be required to achieve maximum density of the concrete. In such cases, the value of c_1 should be increased to 150.

†The minimum height is a function of the rate of slipping (ft per hour) and the time required for the concrete to gain sufficient strength to support itself without sagging after leaving the slipform. A slightly higher form will provide some working space in the top of the form for placing of concrete and reinforcement. Forms less than 3½ ft high are believed to be dangerously shallow. Forms as high as 6 ft may be required when low temperature or slow setting concrete is specified.

struction service equipment is immediately available to maintain a continuous operation.

5.3.2.1.4—Tolerances—Maximum variation in wall thickness should not exceed ±⅜ in. for walls up to 8 in. thick nor ±½ in. for walls thicker than 8 in. The maximum deviation of any point on the slipform with respect to a vertical projection of a corresponding reference point at the base of the structure should not exceed 1 in. per 50 ft of height. This is the total deviation which may be composed of translational and rotational components.

5.3.2.1.5—Sliding operation—Maximum rate of slide should be limited by the rate for which the forms are designed. In addition, both maximum and minimum rates of slide must be determined by an experienced slipform supervisor to meet changes in weather, concrete slump, and workability, and the many exigencies which arise during a slide and which cannot be predicted accurately beforehand. A man experienced in slipform construction must be present on the deck at all times during the slide operation.

Forms must be leveled before and after they are filled and must be maintained level throughout the slide. Care must be taken to prevent drifting of the forms from alignment or designed dimensions and to prevent torsional movement.

Experience has shown that a plumb line or optical plummet used in conjunction with a water level system serviced by a central reservoir is effective in maintaining the form on line and grade and for positioning openings and embedded items.

Alignment and plumbness of structure should be checked at least once during every 8 hr that the slide is in operation and preferably every 4 hr. In work that is done in separate, intermittent slipping operations, a check on alignment and plumbness should be made at the beginning of each slipping operation.

5.3.2.2—Horizontal slipforms for tunnel inverts, drainage channels, canal linings, and highways.*

5.3.2.2.1—Tunnel inverts—Linings for tunnel inverts often are constructed in a continuous longitudinally operating method of placement. The transverse section of the invert usually is curved to a prescribed shape. The best way to hold such a shape and at the same time obtain good vibratory consolidation of the higher areas along the side forms is to use a heavy weighted slipform supported on the fixed side forms and having a length equal to or greater than the width of the invert. The slipform is moved forward by winches. The concrete is delivered by pump and pipeline or conveyor belt and is placed and vibrated immediately ahead of the slipform. If arch form anchors are inserted in the invert concrete immediately behind the slipform, care should be taken to insure that they are properly embedded in the wet concrete without despoiling the newly formed surface.

5.3.2.2.2—Drainage channels—Linings for drainage channels and canals may be constructed in either a planned sequence of finite placements or a continuous longitudinally operating method of placement. In a planned sequence of finite placements the procedures may range from hand operations for small laterals where the concrete may be dumped and spread on the sides and bottom, to the larger channels where the lining may be placed in alternate sections. In the latter, the bottom slab is placed first to provide support at the toes of the side panels.

An efficient placement of concrete on slopes is accomplished by use of a weighted, unvibrated steel-faced slipform screed about 27 in. wide in the direction of movement. The screed may be pulled up the slope by equipment located on the berm or by air hoists mounted on the slipform. The concrete vibrators should be manually operated just ahead of the slipform rather than mounted on the form. If the form is vibrated, this procedure will cause a swell in the finished surface emerging from the trailing edge.

(a) Small channels and canals—A simplified type of slipform machine has been used with good results. This machine is held to grade and line by a steel pan, shaped to fit the previously prepared excavation section, and is pulled forward by an external source of power. Behind the pan and immediately preceding the slipform is a transverse, compartmented trough for uniformly distributing the mix. This type of form which depends on the subgrade for its support is applicable for placing only unreinforced concrete lining.

(b) For reinforced linings and also for medium-sized canal linings, more elaborate slipform machines are required. A framework, traveling on rails, or a tractor crawler assembly on the berm of the drainage channel or canal, supports the working platform, the distributor plate or drop

*The simplified type of slipform is a relatively minor structure; its design is straight-forward and is not discussed here. The material in this subsection deals principally with the design and construction of the more elaborate slipform structures.

chutes, the compartmented supply trough, vibrator tube in the bottom of the trough, and the slipform. The slipform is a steel plate, curved up at the leading edge, extending across the bottom and up the slopes of the canal and shaped to conform to the finished surface of the lining. When a distributor plate is used, it is fastened to the leading edge of the slipform and extends upward on a steep incline to the working platform. On some of the machines, a continuous row of hoppers in the working platform feed into drop chutes, each supplying one compartment of the trough below. Concrete is dumped, usually from a shuttle car on the working platform, and is guided to the trough below by the distributor plate or the drop chutes.

As the concrete passes out at the bottom of the trough and under the slipform, it is consolidated by a vibrating tube parallel to and a few inches ahead of the leading edge of the form. Consolidation must be accomplished as the concrete passes under the slipform. Proper consolidation cannot be obtained by vibrating the slipform of a lining machine, apparently due to lack of means to supply additional concrete needed to fill the voids. The trailing edge of the slipform is usually adjustable to positions somewhat lower than that of the leading edge. This improves consolidation and tends to mold the concrete more closely to the subgrade. Too low a setting of the trailing edge causes tearing, rather than smoothing, of the surface. On some machines, the slipform is followed within a few feet by an "ironer" plate 18 x 20 in. wide, which, under favorable conditions, leaves a surface that requires little or no hand treatment.

(c) For large channels (bottom widths of 50 to 110 ft) it is impractical to build machines to span the entire waterway prism. The slope paver is a crawler-mounted slipform which places the concrete lining on one side slope and the adjacent 8-10 ft of the invert. After the opposite side slope is similarly completed, the invert is finished by horizontal pavers. All three operations are kept on line and grade electronically through sensors probing guide wires.

(d) The slipform used for highways is similar in principle to the slope form pavers. No fixed side forms are required as the side forms of the machine slide forward with the paver leaving the slab edges unsupported. The concrete is deposited either on the subgrade ahead of the paver or into a hopper box. Following spreading by a dozer-type strike-off, the concrete is consolidated by vibration and shaped by an extrusion plate or meter. Flat, parabolic, or hip roof crowns can be provided with a quick change device for transitions in and out of horizontal curves. Surface elevations can be maintained by electronic controls.

5.3.2.2.3—Design considerations—This specialized formwork should be designed by experienced, competent structural engineers employed or engaged by the contractor. A complete structural analysis, including stress diagrams of the structural members must be made to insure satisfactory performance. Due regard should be given to unsymmetrical and eccentric loadings and the fact that the machine must be regularly disassembled as it encounters siphons, bridges, chutes, etc., along the waterway. The large machines are usually hinged so that sections may be passed through or beneath structures. The vertical or lateral deflections, particularly of long-span machines, must be investigated, and sufficient rigidity provided to insure that concrete tolerances will be met. The stability of the machine under the aforementioned loading conditions must be carefully investigated to insure satisfactory performance.

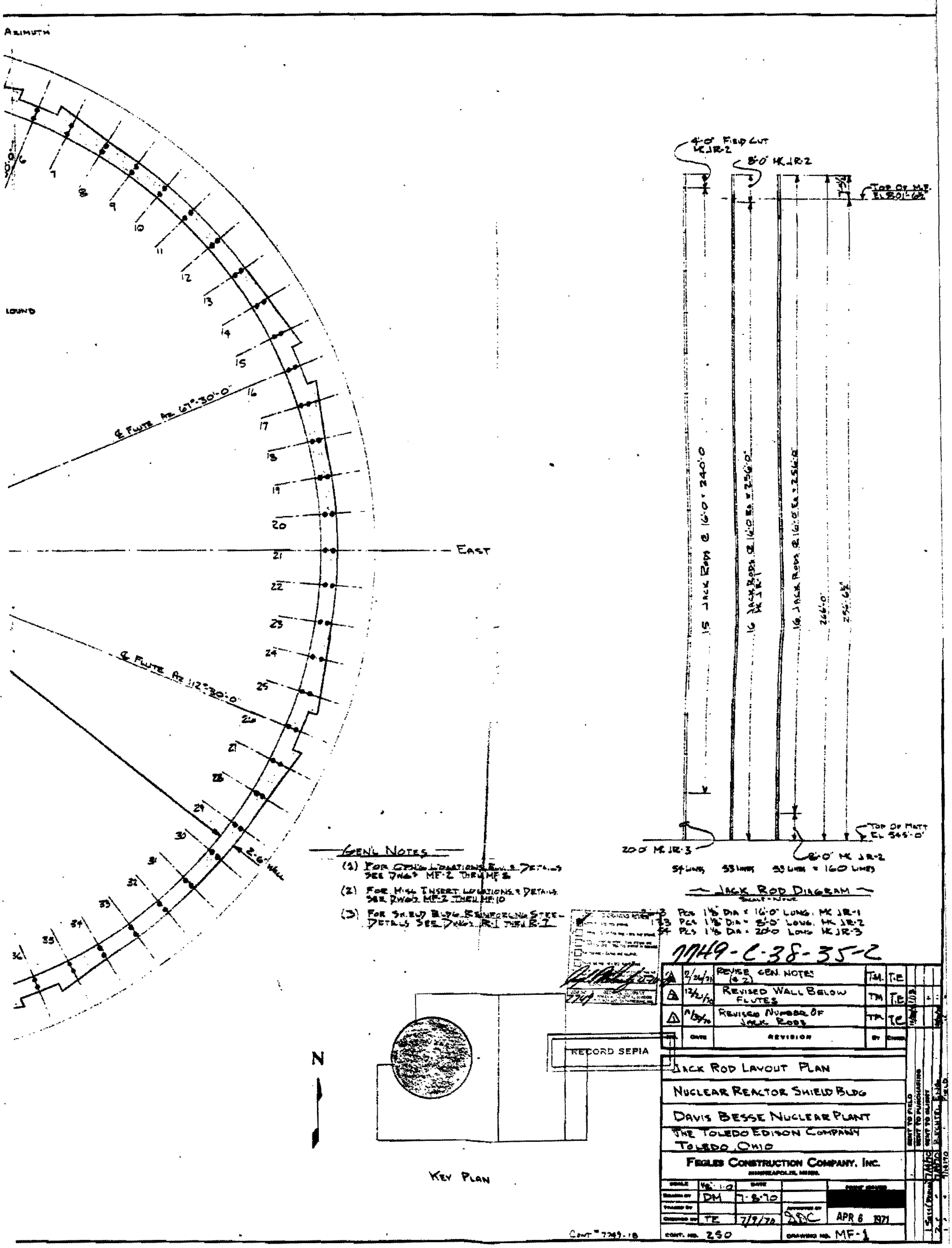
5.3.2.2.4 — Drawings — The general provisions of "Drawings" in Section 1.4 should be met and the contractor should submit drawings of the slipform for review and approval by the engineer-architect. These drawings should show the handling diagrams, the placing procedure, and the provisions for insuring attainment of the required concrete surfaces.

5.4—Permanent forms

5.4.1 — *Discussion* — Permanent forms, as the name implies, are forms left in place that may or may not become an integral part of the structural frame. These forms may be the rigid type such as metal deck, precast concrete, wood, plastics, and various types of fiberboard; or the flexible type such as reinforced water-repellent corrugated paper, or wire mesh with waterproof paper backing.

Where the permanent form is used as a deck form it is generally supported from the main structural frame with or without an intermediate system of temporary supports.

Exhibit 28: Fegles Drawing of Jack Bar Layout Plan



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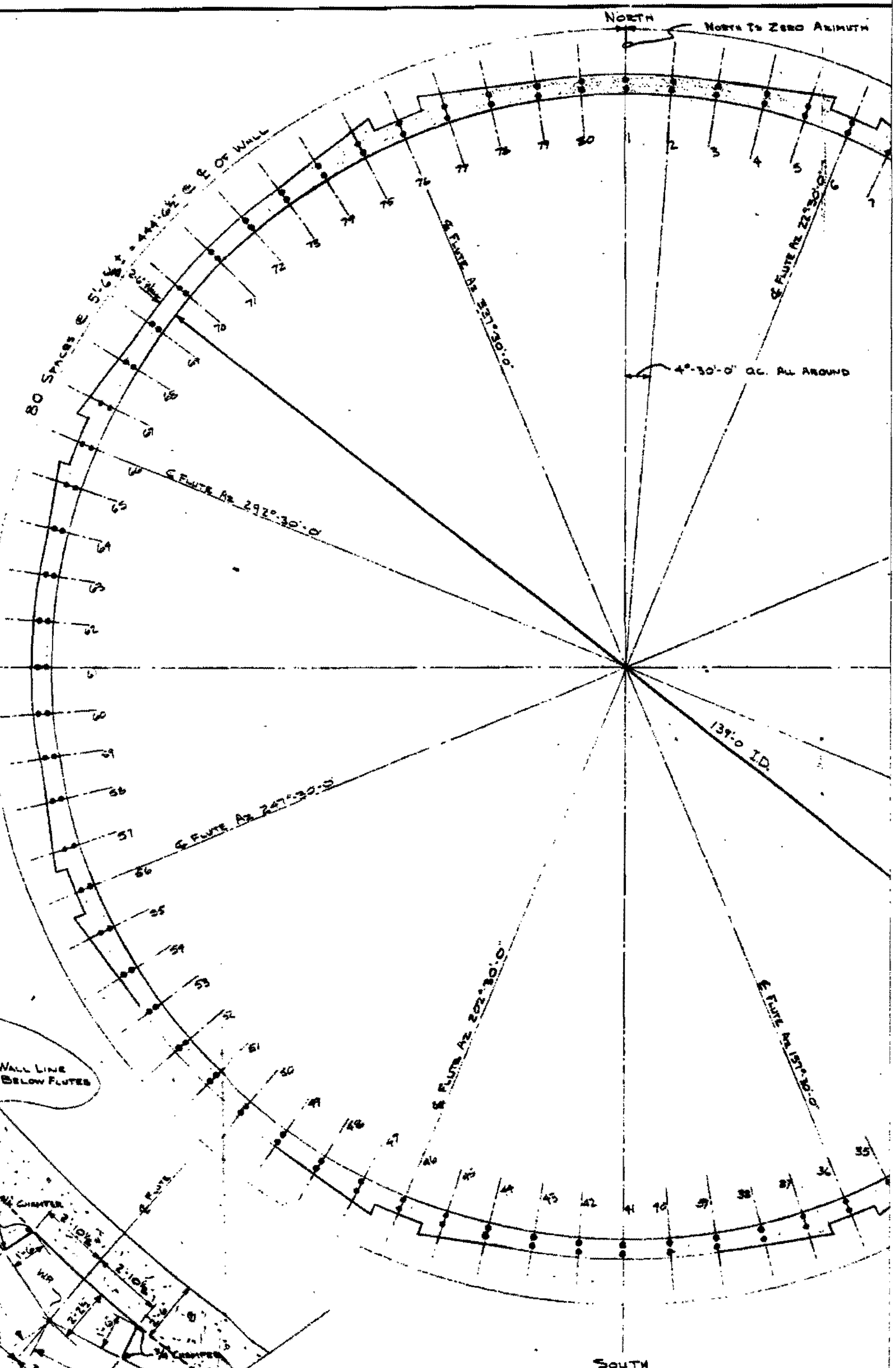
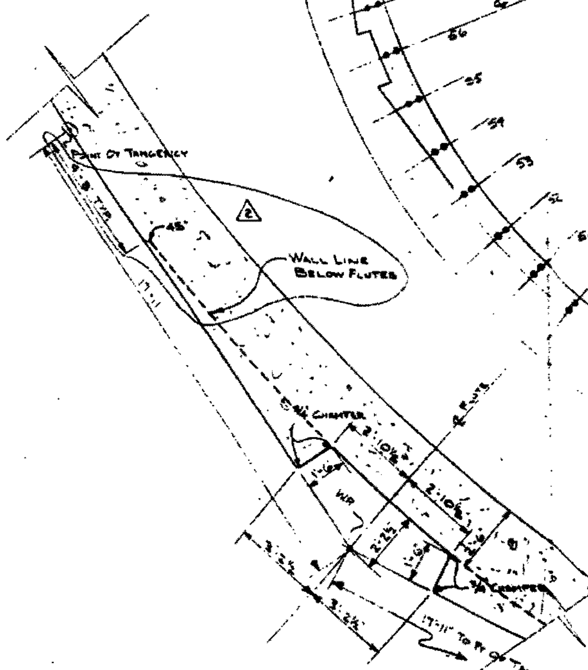
Page 1 of 2

Exhibit 28



PLAN VIEW MOVING FORM WALLS
JACK ROD LAYOUT
SCALE 1/8" = 1'-0"



DETAIL TYPICAL ARCH FLUTE
SCALE 3/8" = 1'-0"

I certify that the image contained on this frame was made in the normal and regular course of business, on the date stated below and that it is an accurate reproduction of the document(s) submitted to Micrographics.
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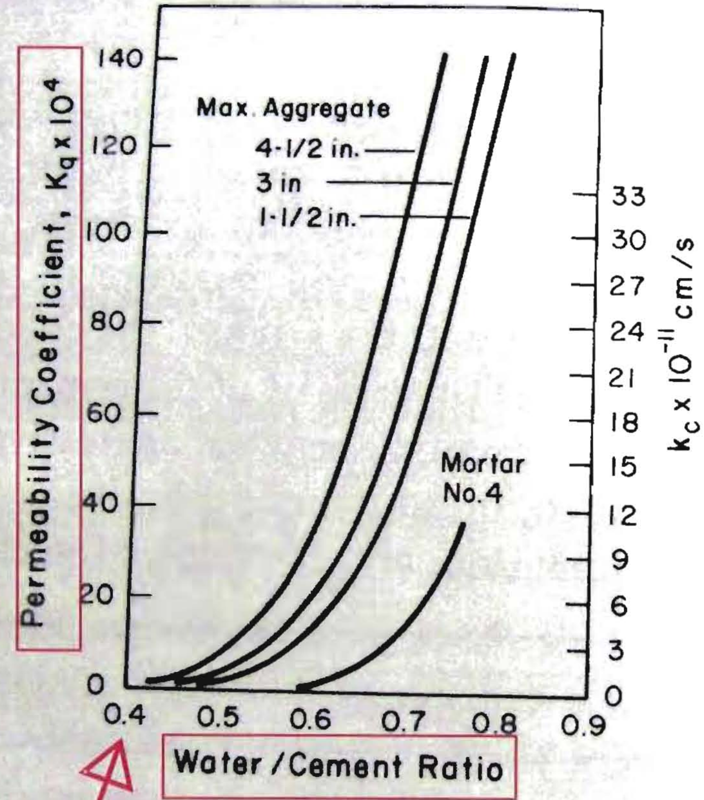
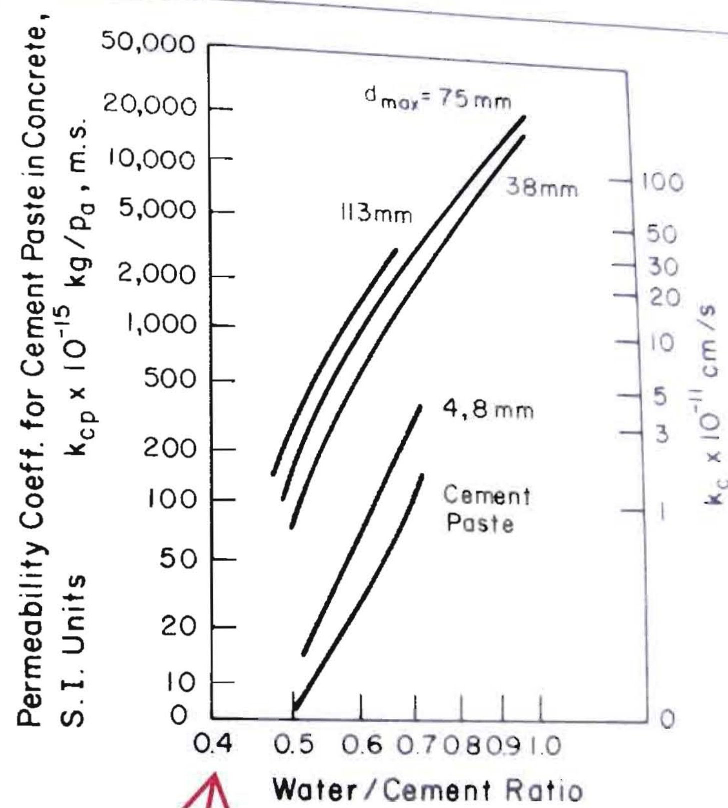
Exhibit 28

Exhibit 29: Permeability versus Water Cement Ratio

Exhibit 29



(a)



(b)

Figure 5-2 Influence of water/cement ratio and maximum aggregate size on concrete permeability: (a) K_q is a relative measure of the flow of water through concrete in cubic feet per year per square foot of area for a unit hydraulic gradient. [(a), From *Concrete Manual, 8th Edition,* U.S. Bureau of Reclamation, 1975, p. 37, (b), adapted from *Beton-Bogen,* Aalborg Cement Co., Aalborg, Denmark, 1979.]

The permeability of concrete to water depends mainly on the water/cement ratio (which determines the size, volume, and continuity of capillary voids) and maximum aggregate size (which influences the microcracks in the transition zone between the coarse aggregate and the cement paste).

Exhibit 30: Irradiation Effect

NUREG/CR-6927
ORNL/TM-2006/529

Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures - A Review of Pertinent Factors

Oak Ridge National Laboratory

U.S. Nuclear Regulatory Commission
Office of Nuclear Regulatory Research
Washington, DC 20555-0001

Thermal cycling, even at relatively low temperatures (i.e., 65°C), can have deleterious effects on concrete's mechanical properties (i.e., compressive, tensile and bond strengths, and modulus of elasticity are reduced).[86] Most reinforced concrete structures are subjected to thermal cycling due to daily temperature fluctuations and are designed accordingly (i.e., inclusion of steel reinforcement). At higher temperatures (200 to 300°C), the first thermal cycle causes the largest percentage of damage, with the extent of damage markedly dependent on aggregate type and is associated with loss of bond between the aggregate and matrix.[87] Thermal cycles also can become important if the deformation of the structure resulting from the temperature variations is constrained.

Additional information on the effects of elevated temperature on concrete materials and structures is available.[88,89]

Irradiation Irradiation in the form of either fast and thermal neutrons emitted by the reactor core or gamma rays produced as a result of capture of neutrons by members (particularly steel) in contact with concrete can affect the concrete. Changes in the properties of concrete appear to depend primarily on the behavior of the concrete aggregate that can undergo a volume change when exposed to radiation.[90] The fast neutrons are mainly responsible for the considerable growth, caused by atomic displacements, that has been measured in certain aggregate (e.g., flint). Quartz aggregates that contain crystals with covalent bonding should be more affected by radiation than calcareous aggregates that contain crystals with ionic bonding.[91] Furthermore, when nuclear radiation is attenuated or absorbed in the concrete almost all the absorbed radiation is converted into heat. Nuclear heating occurs as a result of energy introduced into the concrete as the neutrons or gamma radiation interact with the molecules within the concrete material. The heat generated may have detrimental effects on the physical, mechanical, and nuclear properties of the concrete. Reference 92 indicates that nuclear heating is negligible for incident energy fluxes less than 10^{10} MeV/cm^2 per s. Determination of whether any deterioration that may occur in concrete properties is due to radiation damage or thermal effects can be difficult.

Prolonged exposure of concrete to irradiation can result in decreases in tensile and compressive strengths and modulus of elasticity. Figure 4.7 presents a summary of the effects of neutron radiation on the compressive strength and modulus of elasticity of several concretes.[90] Results in the literature[90] indicate that: (1) for some concretes, neutron radiation of more than1 x 10^{19} neutrons/cm^2 or 10^{10} rads of dose for

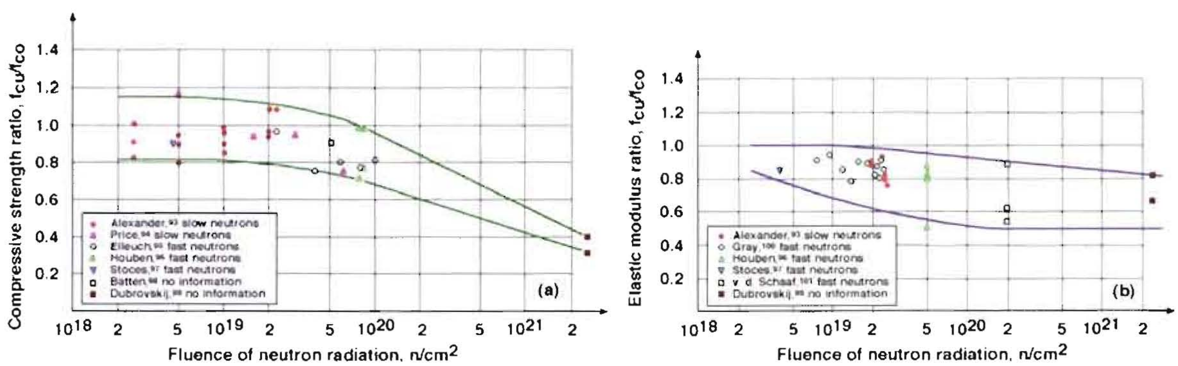


Figure 4.7 Effect of neutron radiation on concrete compressive strength and modulus of elasticity relative to unirradiated and unheated control specimen results.

Source: H. K. Hilsdorf et al., *The Effects of Nuclear Radiation on the Mechanical Properties of Concrete*, ACI SP-55, Douglas McHenry International Symposium on Concrete and Concrete Structures, American Concrete Institute, Farmington Hills, Michigan, 1978.

gamma radiation may cause a reduction in compressive strength; (2) tensile strength of concrete is significantly reduced at neutron fluences exceeding 10^{19} n/cm^2 with the decrease of tensile strength caused by neutron radiation more pronounced than the decrease of compressive strength; (3) resistance of concrete to neutron radiation apparently depends on the type of neutrons (slow or fast) involved, but the effect is not clarified; (4) resistance of concrete to neutron radiation depends on mix proportions, type of cement, and type of aggregate; (5) the effect of gamma radiation on concrete's mechanical properties requires clarification; (6) the deterioration of concrete properties associated with a temperature rise resulting from irradiation is relatively minor; (7) coefficients of thermal expansion and conductivity of irradiated concrete differ little from those of temperature-exposed concrete; (8) when exposed to neutron irradiation, the modulus of elasticity of concrete decreases with increasing neutron fluence; (9) creep of concrete is not affected by low-level radiation exposure, but for high levels of exposure creep probably would increase with exposure because of the effects of irradiation on the concrete's tensile and compressive strengths;[+] (10) for some concretes, neutron radiation with a fluence of more than 1×10^{19} neutrons/cm^2 can cause a marked increase in volume; (11) generally, concrete's irradiation resistance increases as the irradiation resistance of the aggregate increases; and (12) irradiation has little effect on shielding properties of concrete beyond moisture loss caused by a temperature increase. Furthermore, there is an indication that nuclear radiation can significantly increase the reactivity of silica-rich aggregates to alkali (i.e., alkali-silica reaction).[102] Results from an investigation of the effect of γ-irradiation on the strength of a nuclear power plant concrete indicate that for a dose up to 6×10^5 Gy the compressive, splitting-tensile, and flexural strength of concrete decreased with dose, reaching a reduction of about 10%, 5%, and 5%, respectively, at the maximum dose.[103] It was noted in the reference that interaction of concrete with irradiation generated a succession of chemical reactions starting with radiolysis of water and terminating in formation of calcite crystals that decrease both the size of pore space and the strength of the concrete.

Section III, Division 2 of the American Society of Mechanical Engineers Pressure Vessel and Piping Code gives an allowable radiation exposure level of 10×10^{20} nvt.[104] The British Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors[105] states that the maximum permissible neutron dose is controlled by the effects of irradiation on concrete properties, and the effects are considered to be insignificant for doses up to 0.5×10^{18} neutrons/cm^2. Table 2.7 from Ref. 106 provides data for estimated radiation environments at the outside surface of light-water reactor pressure vessels for a 1000 MW(e) plant operating at a capacity factor of 80%. These results indicate that radiation levels may approach the limits provided above in a concrete primary shield wall after 40 years of operation (32 equivalent full-power years). However, these values are upper limits and probably higher than would be experienced because of the attenuating effects that would occur due to the presence of air gaps, insulation, etc., that could be positioned between the pressure vessel and concrete structures.

More detailed information on the interaction of radiation and concrete is available in Ref. 91.

Fatigue/Vibration Concrete structures subjected to fluctuations in loading, temperature, or moisture content (that are not large enough to cause failure in a single application) can be damaged by fatigue. Fatigue damage initiates as microcracks in the cement paste, proximate to the large aggregate particles, reinforcing steel, or stress risers (e.g., defects). Upon continued or reversed load application, these microcracks may propagate to form structurally significant cracks that can expose the concrete and reinforcing steel to hostile environments or produce increased deflections. Ultimate failure of a concrete structure in fatigue will occur as a result of excessive cracking, excessive deflections, or brittle fracture. As concrete ages and gains strength, for a given stress level the cycles to failure will increase. If the concrete is reinforced or prestressed, properties of the steel tend to control structural performance since

[+] Gamma rays produce radiolysis of water in cement paste that can affect concrete's creep and shrinkage behavior to a limited extent and also result in evolution of gas.

to that of the yield stress. Other data[186] confirm the effects of temperatures above 200°C on the mild steel reinforcing as well as providing a threshold temperature of about 300°C for loss of bond properties with the concrete. Figure 4.33 presents stress-strain relationships, Young's modulus/elongation, and yield/ultimate strength data as a function of temperature for a 3,500 kgf/cm^2 specified minimum yield strength 51-mm diameter steel bar.[187] Additional information on the effect of elevated temperature on the stress-strain behavior of 12- and 25-mm diameter quenched and tempered steel bars as well as a comparison of results with recommendations provided in the European Code for structural fire design[188] is available.[189]

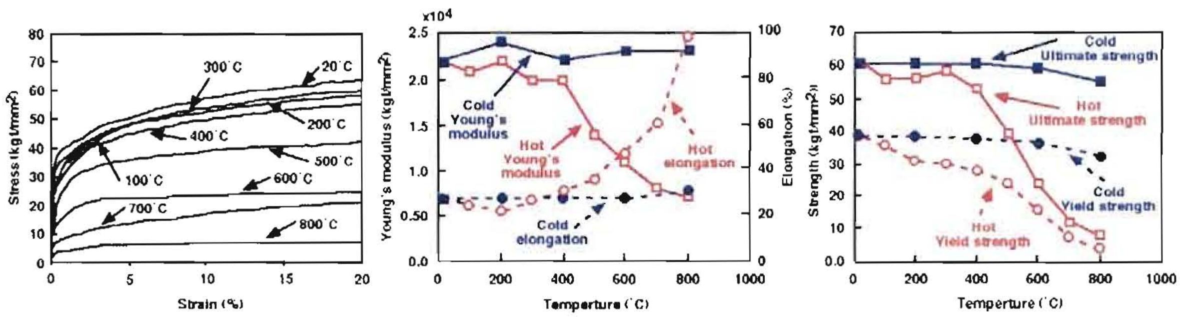


Figure 4.33 Effect of temperature on properties of a 3,500 kgf/cm^2 minimum specified yield strength steel bar.

Source: M. Takeuchi et al., "Material Properties of Concrete and Steel Bars at Elevated Temperatures," *12th International Conference on Structural Mechanics in Reactor Technology*, Paper H04/4, pp. 13-138, Elsevier Science, North-Holland, Netherlands, 1993.

4.3.2.3 Irradiation

Neutron irradiation produces changes in the mechanical properties of carbon steels (e.g., increased yield strength and rise in the ductile-to-brittle transition temperature). The changes result from the displacement of atoms from their normal sites by high-energy neutrons, causing the formation of interstitials and vacancies. A threshold level of neutron fluence of 1×10^{18} neutrons per square centimeter has been cited for alteration of reinforcing steel mechanical properties.[190] Fluence levels of this magnitude are not likely to be experienced by the safety-related concrete structures in nuclear power plants, except possibly in the concrete primary biological shield wall over an extended operating period.[106]

4.3.2.4 Fatigue

Fatigue of the mild steel reinforcing system would be coupled with that of the surrounding concrete. The result of applied repeated loadings, or vibrations, is generally a loss of bond between the steel reinforcement and concrete. For extreme conditions, the strength of the mild steel reinforcing system may be reduced or failures may occur at applied stress levels less than yield. However, there have been few documented cases of fatigue failures of reinforcing steel in concrete structures and those published occurred at relatively high stress/cycle combinations.[191] Because of the typically low normal stress levels in reinforcing steel elements in nuclear power plant safety-related concrete structures, fatigue failure is not likely to occur.

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97. B. Stôces, P. Otopal, V. Juricka, and J. Gabriel, "The Effect of Irradiation on the Mechanical Properties of Concrete," Ceskoslovenska Akademie, Translated from the Czech, Oak Ridge National Laboratory Purchase Order 34B-83481, Letter Release No.: T81, STS No.: 14087, Oak Ridge, Tennessee.
98. A. W. Chisholm-Batten, "Effect of Irradiation on Strength of Concrete," Atomic Energy Research Establishment Research Report AERE R 3332, 13 pp., United Kingdom Atomic Energy Authority, Harwell, 1960.
99. V. B. Dubrovskij, Sh.Sh. Ibragimov, A. Ya. Ladygin, and B. K. Pergamenshckik, "The Effect of Neutron Irradiation on Certain Properties of Refractory Concretes," Atomnaya Energiya 21, pp. 108-112, 1966.
100. B. S. Gray, "The Effect of Reactor Radiation on Cements and Concrete," *Conference on Prestressed Concrete Pressure Vessels*, pp. 17-39, Commission of European Communities, Luxembourg, Belgium, 1972.
101. C. F. Van der Schaaf, "Effect of Irradiation and Heating on the Strength of Mortar and Concrete," *2nd Conference on Prestressed Concrete Pressure Vessels and Their Thermal Insulation*, Commission of European Communities, Brussels, Belgium, 1969.
102. T. Ichikawa and H. Koizumi, "Possibility of Radiation-Induced Degradation of Concrete by Alkali-Silica Reaction of Aggregates," *Journal on Nuclear Science and Technology* **39**(8), pp. 880-884, Atomic Energy Society of Japan, Tokyo, August 2002.
103. F. Vodák, K. Trtík, V. Sopko, O. Kapicková, and P. Demo, "Effect of γ-Irradiation on Strength of Concrete for Nuclear Safety-Related Structures," *Cement and Concrete Research* **35**, pp. 1447-1551, Elsevier Ltd., 2005.
104. American Society of Mechanical Engineers, "Code for Concrete Reactor Vessels and Containments," Section III, Division 2 of *ASME Boiler and Pressure Vessel Code*, ACI Standard 359, New York, New York, 2005.
105. British Standards Institution, *Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors*, BS 4975, London, United Kingdom, 1973.
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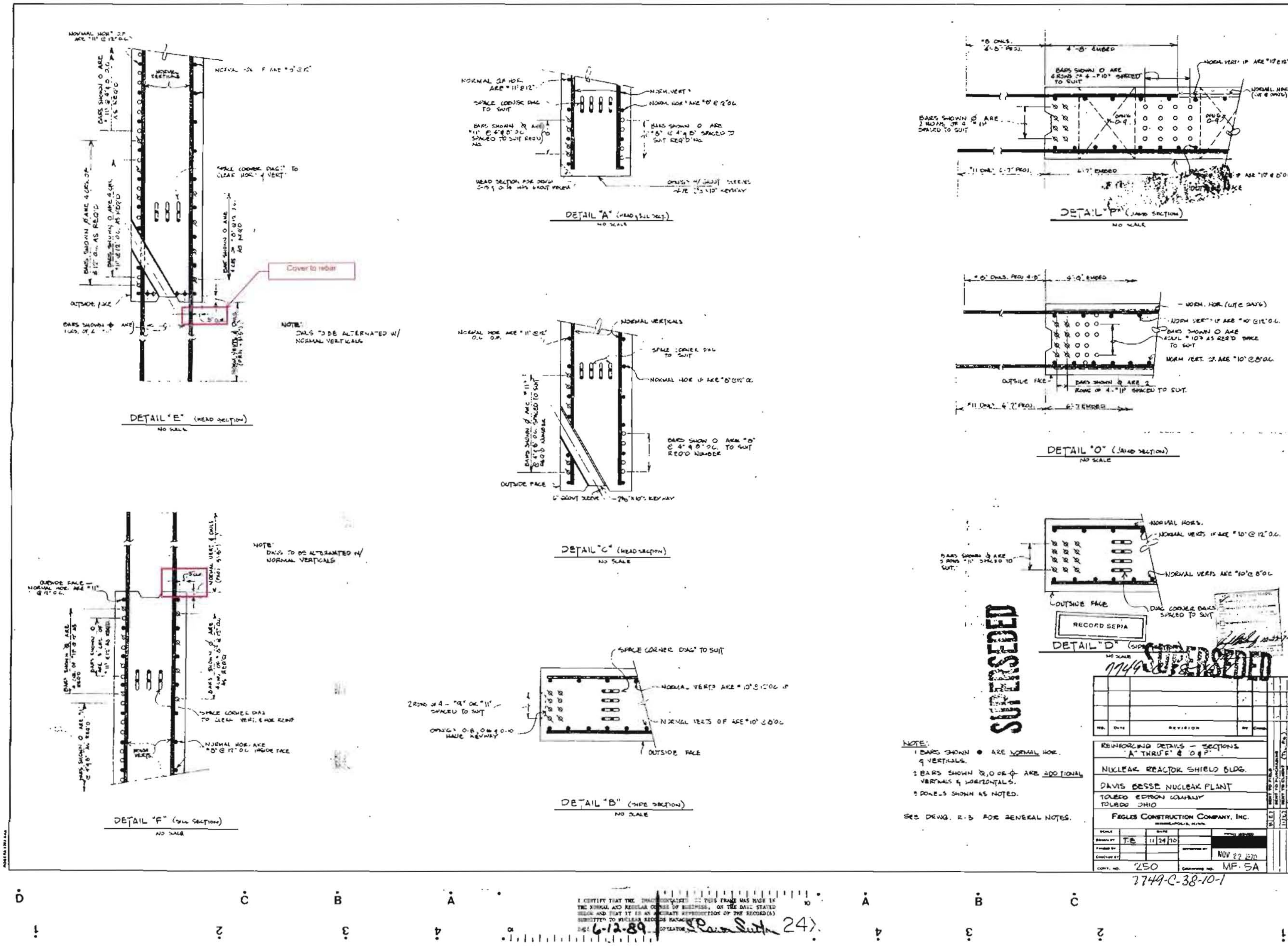
107. ACI Committee 215, *Considerations for Design of Concrete Structures Subjected to Fatigue*, ACI 215R-92, American Concrete Institute, Farmington Hills, Michigan, 1992.

Exhibit 31: MF-6, Rebar Detail by Fegles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SUPERSEDED
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SUPERSEDED
NOTE:
1 BARS SHOWN ● ARE NORMAL HOR. & VERTICALS.
2 BARS SHOWN ⊗, O OR ⌀ ARE ADDITIONAL VERTICALS & HORIZONTALS.
3 DOWELS SHOWN AS NOTED.
SEE DRWG. R-3 FOR GENERAL NOTES.
REINFORCING DETAILS — SECTIONS "A" THRU "F" & "O & P"
NUCLEAR REACTOR SHIELD BLDG.
DAVIS BESSE NUCLEAR PLANT
TOLEDO EDISON COMPANY
TOLEDO OHIO
FEGLES CONSTRUCTION COMPANY, INC.
T.E. 11/24/70
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I CERTIFY THAT THE IMAGE CONTAINED IN THIS FRAME WAS MADE IN THE NORMAL AND REGULAR COURSE OF BUSINESS, ON THE DATE STATED BELOW AND THAT IT IS AN ACCURATE REPRODUCTION OF THE RECORD(S) SUBMITTED TO NUCLEAR RECORDS MANAGEMENT
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Exhibit 32: ACI 515 Protective Systems

ACI 515.1 R-79

(Revised 1985)

A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete

Reported by ACI Committee 515

Byron I. Zolin, Chairman

Warner K. Babcock
Arthur E. Blackman, Sr.
Donald E. Brotherson
Robert W. Gaul
Clark R. Gunness
Kenneth A. Heffner
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Charles O. Pratt
Andrew Rossi, Jr.
Donald L. Schlegel
Lawrence E. Schwietz

The revising committee is listed at the end of the document.

This Guide updates and expands the scope of the committee report "Guide for the Protection of Concrete Against Chemical Attack by Means of Coatings and Other Corrosion Resistant Materials," which appeared in the December 1996 ACI JOURNAL. The previous Guide has been revised and is found in Chapter 6 of this Guide entitled "Protective Barrier Systems." In addition, there are new chapters on "Waterproofing Barrier Systems," "Dampproofing Barrier Systems," and "Decorative Barrier Systems." A separate chapter on conditioning and surface preparation of concrete is included because it is relevant to all the other chapters.

This Guide is not to be referenced as a complete unit.

Keywords: abrasive blasting; acid treatment (concrete); acid resistance; adhesion; asphalts; chemical attack; chemical cleaning; **coatings;** concrete bricks; concretes; detergents; emulsifying agents; epoxy resins; **finishes;** furan resins; glass fibers; inspection; joint sealers; latex (rubber); mortars [materials); paints; phenolic resins; plastics, polymers, and resins; polyester resins; polyurethane resins; **protective coatings;** repairs; sealers; silicates; sulfur; surfactants; temperature; tests; vaporbarriers; **waterproofing.**

Foreword

ACI Committee 515 was organized in 1936 and published a report "Guide for the Protection of Concrete Against Chemical Attack by Means of Coatings and Other Corrosion Resistant Materials," in the December 1966 ACI JOURNAL. William H. Kuenning was chairman when this Guide was published. Albert M. Levy was chairman from 1974 to 1977 when some of the information, found in the chapters on "Waterproofing Barrier Systems" and "Dampproofing Barrier Systems," was developed.

CONTENTS

Table 2.5.2-Effect of chemicals on concrete (see end of Table 2.5.2 for special notations)

Material	Effect
*Acetic acid, all concentrations	Disintegrates slowly
Acetone	Liquid loss by penetration. May contain acetic acid as impurity (which see)
Acid waters (pH of 6.5 or less) (a)	Disintegrates slowly. In porous or cracked concrete, attacks steel
*Alcohol	See ethyl alcohol, methyl alcohol
Alizarin	Not harmful
*Almond oil	Disintegrates slowly
*Alum	See potassium aluminum sulfate
Aluminum chloride	Disintegrates rapidly. In porous or cracked concrete, attacks steel
*Aluminum sulfate	Disintegrates. In porous or cracked concrete, attacks steel
*Ammonia, liquid	Harmful only if it contains harmful ammonium salts (see below)
Ammonia vapors	May disintegrate moist concrete slowly or attack steel in porous or cracked moist concrete
Ammonium bisulfate	Disintegrates. In porous or cracked concrete, attacks steel
Ammonium carbonate	Not harmful
*Ammonium chloride	Disintegrates slowly. In porous or cracked concrete, attacks steel
Ammonium cyanide	Disintegrates slowly
Ammonium fluoride	Disintegrates slowly
Ammonium hydroxide	Not harmful
Ammonium nitrate	Disintegrates. In porous or cracked concrete, attacks steel
Ammonium oxalate	Not harmful
*Ammonium sulfate	Disintegrates. In porous or cracked concrete, attacks steel
Ammonium sulfide	Disintegrates
Ammonium sulfite	Disintegrates
Ammonium superphosphate	Disintegrates. In porous or cracked concrete, attacks steel
Ammonium thiosulfate	Disintegrates
Animal wastes	See slaughter house wastes
Anthracene	Not harmful
Arsenious acid	Not harmful
Ashes	Harmful if wet, when sulfides and sulfates leach out (see sodium sulfate)
Ashes, hot	Cause thermal expansion
Automobile and diesel exhaust gases (n)	May disintegrate moist concrete by action of carbonic, nitric, or sulfurous acid
*Baking soda	See sodium bicarbonate
Barium hydroxide	Not harmful
Bark	See tanning bark
*Beef fat	Solid fat disintegrates slowly, melted fat more rapidly
*Beer	May contain, as fermentation products, acetic, carbonic, lactic, or tannic acids (which see)
Benzol (benzene]	Liquid loss by penetration
Bleaching solution	See specific chemical, such as hypochlorous acid, sodium hypochlorite, sulfurous acid, etc.
*Borax	Not harmful
*Boric acid	Negligible effect
*Brine	See sodium chloride or other salt
Bromine	Gaseous bromine disintegrates. Liquid bromine disintegrates if it contains hydrobromic acid and moisture
*Buttermilk	Disintegrates slowly
Butyl stearate	Disintegrates slowly
Calcium bisulfite	Disintegrates rapidly
*Calcium chloride	In porous or cracked concrete, attacks steel. (b) Steel corrosion may cause concrete to spall
*Calcium hydroxide	Not harmful
Calcium nitrate	Not harmful
*Calcium sulfate	Disintegrates concrete of inadequate sulfate resistance
Carbazole	Not harmful
Carbolic acid	See phenol
*Carbon dioxide	Gas may cause permanent shrinkage (see also carbonic acid)
*Carbon disulfide	May disintegrate slowly
*Carbon tetrachloride	Liquid loss by penetration of concrete
*Carbonic acid	Disintegrates slowly (c)

Table 2.5.2-(Continued)

Material	Effect
Castor oil	Disintegrates, especially in presence of air
Chile saltpeter	See sodium nitrate
China wood oil	Liquid disintegrates slowly.
Chlorine gas	Slowly disintegrates moist concrete
Chrome plating solutions (o)	Disintegrates slowly
Chromic acid, all concentrations	Attacks steel in porous or cracked concrete
Chrysen	Not harmful
*Cider	Disintegrates slowly (see acetic acid]
Cinders	Harmful if wet, when sulfides and sulfates leach out (see, for example, sodium sulfate)
Cinders, hot	Cause thermal expansion
Coal	Sulfides leaching from damp coal may oxidize to sulfurous or sulfuric acid, or ferrous sulfate (which see)
Coal tar oils	See anthracene, benzol, carbazole, chrysen, creosote, cresol, cumol, paraffin, phenanthrene, phenol, toluol, xylol
Cobalt sulfate	Disintegrates concrete of inadequate sulfate resistance
*Cocoa bean oil	Disintegrates, especially in presence of air
*Cocoa butter	Disintegrates, especially in presence of air
Coconut oil	Disintegrates, especially in presence of air
*Cod liver oil	Disintegrates slowly
Coke	Sulfides leaching from damp coke may oxidize to sulfurous or sulfuric acid (which see]
Copper chloride	Disintegrates slowly
Copper plating solutions (p)	Not harmful
Copper sulfate	Disintegrates concrete of inadequate sulfate resistance
Copper sulfide	Harmful if it contains copper sulfate (which see)
*Corn syrup	Disintegrates slowly
Corrosive sublimate	See mercuric chloride
*Cottonseed oil	Disintegrates, especially in presence of air
Creosote	Phenol present disintegrates slowly
Cresol	Phenol present disintegrates slowly
Cumol	Liquid loss by penetration
Deicing salts	Scaling of non-air-entrained or insufficiently aged concrete (b)
Diesel gases	See automobile and diesel exhaust gases
Dinitrophenol	Disintegrates slowly
Distiller's slop	Lactic acid causes slow disintegration
Epsom salt	See magnesium sulfate
*Ethyl alcohol	Liquid loss by penetration
*Ethyl ether	Liquid loss by penetration
*Ethylene glycol	Disintegrates slowly (d)
Feces	See manure
*Fermenting fruits, grains, vegetables, or extracts	Industrial fermentation processes produce lactic acid. (e) Disintegrates slowly (see lactic acid)
Ferric chloride	Disintegrates slowly
Ferric nitrate	Not harmful
Ferric sulfate	Disintegrates concrete of inadequate quality
Ferric sulfide	Harmful if it contains ferric sulfate (which see)
Ferrous chloride	Disintegrates slowly
Ferrous sulfate	Disintegrates concrete of inadequate sulfate resistance
Fertilizer	See ammonium sulfate, ammonium superphosphate, manure, potassium, nitrate, sodium nitrate
Fish liquor	Disintegrates (f)
*Fish oil	Disintegrates slowly
Flue gases	Hot gases (400-1100 F) cause thermal stresses. Cooled, condensed sulfurous, hydrochloric acids disintegrate slowly
Foot oil	Disintegrates slowly
*Formaldehyde, 37 percent	Formic acid, formed in solution, disintegrates slowly
Formalin	See formaldehyde

Table 2.5.2-(Continued)

Material	Effect
*Formic acid, 10 percent	Disintegrates slowly
*Formic acid, 30 percent	Disintegrates slowly
*Formic acid, 90 percent	Disintegrates slowly
*Fruit juices	Hydrofluoric, other acids, and sugar cause disintegration (see also fermenting fruits, grains, vegetables, extracts)
Gas water (g)	Ammonium salts seldom present in sufficient quantity to disintegrate
Gasoline	Liquid loss by penetration
*Glucose	Disintegrates slowly
*Glycerine	Disintegrates slowly
*Grain	See fermenting fruits, grains, vegetables, extracts
*Honey	Not harmful
Horse fat	Solid fat disintegrates slowly, melted fat more rapidly
Humic acid	Disintegrates slowly
*Hydrochloric acid, all concentrations	Disintegrates rapidly, including steel
Hydrofluoric acid, all concentrations	Disintegrates rapidly, including steel
Hydrogen sulfide	Not harmful dry. In moist, oxidizing environments converts to sulfurous acid and disintegrates slowly
Hypochlorous acid, 10 percent	Disintegrates slowly
Iodine	Disintegrates slowly
Kerosene	Liquid loss by penetration of concrete
*Lactic acid, 5-25 percent	Disintegrates slowly
*Lamb fat	Solid fat disintegrates slowly, melted fat more rapidly
*Lard and lard oil	Lard disintegrates slowly, lard oil more rapidly
Lead nitrate	Disintegrates slowly
Lead refining solutions (q)	Disintegrates slowly
Leuna saltpeter	See ammonium nitrate and ammonium sulfate
Lignite oils	If fatty oils are present, disintegrates slowly
*Linseed oils	Liquid disintegrates slowly. Dried or drying films are harmless
Locomotive gases (r)	May disintegrate moist concrete by action of carbonic, nitric or sulfurous acids (see also automobile and diesel exhaust gases)
Lubricating oil	Fatty oils, if present, disintegrate slowly
Lye	See sodium hydroxide
Machine oil	Fatty oils, if present, disintegrate slowly
*Magnesium chloride	Disintegrates slowly. In porous or cracked concrete, attacks steel
Magnesium nitrate	Disintegrates slowly
*Magnesium sulfate	Disintegrates concrete of inadequate sulfate resistance
Manganese sulfate	Disintegrates concrete of inadequate sulfate resistance
Manure	Disintegrates slowly
*Margarine	Solid margarine disintegrates slowly, melted margarine more rapidly
Mash, fermenting	Acetic and lactic acids, and sugar disintegrate slowly
Mercuric chloride	Disintegrates slowly
Mercurous chloride	Disintegrates slowly
Methyl alcohol	Liquid loss by penetration
Methyl ethyl ketone	Liquid loss by penetration
Methyl isobutyl ketone	Liquid loss by penetration
*Milk	Not harmful. However, see sour milk
Mine water, waste	Sulfides, sulfates, or acids present disintegrate concrete and attack steel in porous-or cracked concrete
*Mineral oil	Fatty oils, if present, disintegrate slowly
Mineral spirits	Liquid loss by penetration
*Molasses	At temperatures $\geqslant$ 120 F, disintegrates slowly
Muriatic acid	See hydrochloric acid
*Mustard oil	Disintegrates, especially in presence of air
Nickel plating solutions (v)	Nickel ammonium sulfate disintegrates slowly

Table 2.5.2-(Continued)

Material	Effect
Nickel sulfate	Disintegrates concrete of inadequate sulfate resistance
Niter	See potassium nitrate
Nitric acid, all concentrations	Disintegrates rapidly
*Oleic acid, 100 percent	Not harmful
Oleum	See sulfuric acid, 110 percent
*Olive oil	Disintegrates slowly
Ores	Sulfides leaching from damp ores may oxidize to sulfuric acid or ferrous sulfate (which see)
Oxalic acid	Not harmful. Protects tanks against acetic acid, carbon dioxide, salt water. Poisonous. Do not use with food or drinking water
Paraffin	Shallow penetration not harmful, but should not be used on highly porous surfaces like concrete masonry (u)
*Peanut oil	Disintegrates slowly
Perchloric acid, 10 percent	Disintegrates
Perchloroethylene	Liquid loss by penetration
Petroleum oils	Liquid loss by penetration. Fatty oils, if present, disintegrate slowly
Phenanthrene	Liquid loss by penetration
Phenol, 5-25 percent	Disintegrates slowly
*Phosphoric acid, 10-85 percent	Disintegrates slowly
*Pickling brine	Attacks steel in porous or cracked concrete
Pitch	Not harmful
*Poppy seed oil	Disintegrates slowly
*Potassium aluminum sulfate	Disintegrates concrete of inadequate sulfate resistance
*Potassium carbonate	Harmless unless potassium sulfate present (which see)
*Potassium chloride	Magnesium chloride, if present, attacks steel in porous or cracked concrete
Potassium cyanide	Disintegrates slowly
Potassium dichromate	Disintegrates
Potassium hydroxide, 15 percent	Not harmful (h)
Potassium hydroxide, 25 percent or over	Disintegrates concrete
*Potassium nitrate	Disintegrates slowly
Potassium permanganate	Harmless unless potassium sulfate present (which see)
Potassium persulfate	Disintegrates concrete of inadequate sulfate resistance
Potassium sulfate	Disintegrates concrete of inadequate sulfate resistance
Potassium sulfide	Harmless unless potassium sulfate present (which see)
Pyrites	See ferric sulfide, copper sulfide
*Rapeseed oil	Disintegrates, especially in presence of air
Rock salt	See sodium chloride
Rosin	Not harmful
Rosin oil	Not harmful
Sal ammoniac	See ammonium chloride
Sal soda	See sodium carbonate
Salt for deicing roads	See text. Also calcium chloride, magnesium chloride, sodium chloride
Saltpeter	See potassium nitrate
*Sauerkraut	Flavor impaired by concrete. Lactic acid may disintegrate slowly
Sea water	Disintegrates concrete of inadequate sulfate resistance. Attacks steel in porous or cracked concrete
Sewage	Usually not harmful (see hydrogen sulfide]
Silage	Acetic, butyric, lactic acids (and sometimes fermenting agents of hydrochloric or sulfuric acids) disintegrate slowly
Slaughter house wastes (w)	Organic acids disintegrate
Sludge	See sewage, hydrogen sulfide
Soda water	See carbonic acid
*Sodium bicarbonate	Not harmful
Sodium bisulfate	Disintegrates
Sodium bisulfite	Disintegrates
Sodium bromide	Disintegrates slowly

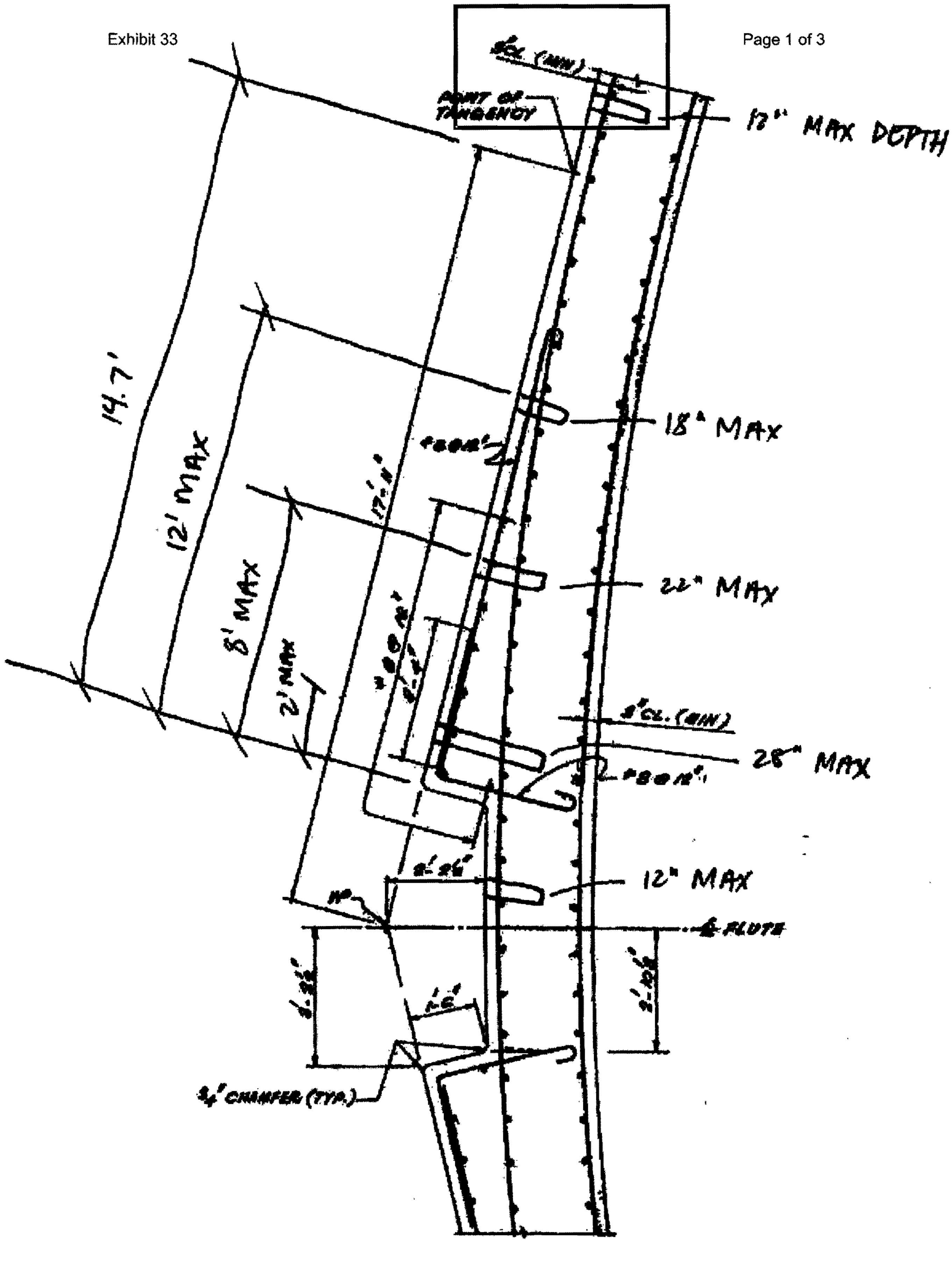
Table 2.5.2-(Continued)

Material	Effect
Sodium carbonate	Not harmful, except to calcium aluminate cement
*Sodium chloride	Magnesium chloride, if present, attacks steel in porous or cracked concrete. (b) Steel corrosion may cause concrete to spall
Sodium cyanide	Disintegrates slowly
Sodium dichromate	Dilute solutions disintegrate slowly
*Sodium hydroxide, 1-10 percent	Not harmful (h)
*Sodium hydroxide, 20 percent or over	Disintegrates concrete
Sodium hypochlorite	Disintegrates slowly
*Sodium nitrate	Disintegrates slowly
Sodium nitrite	Disintegrates slowly
Sodium phosphate (monobasic)	Disintegrates slowly
Sodium sulfate	Disintegrates concrete of inadequate sulfate resistance
Sodium sulfide	Disintegrates slowly
*Sodium sulfite	Sodium sulfate, if present, disintegrates concrete of inadequate sulfate resistance
Sodium thiosulfate	Slowly disintegrates concrete of inadequate sulfate resistance
*Sour milk	Lactic acid disintegrates slowly
*Soybean oil	Liquid disintegrates slowly. Dried or drying films harmless
Strontium chloride	Not harmful
*Sugar	Disintegrates slowly
Sulfite liquor	Disintegrates
Sulfite solution	See calcium bisulfite
*Sulfur dioxide	With moisture forms sulfurous acid (which see)
*Sulfuric acid, 10-80 percent	Disintegrates rapidly
*Sulfuric acid, 80 percent oleum	Disintegrates
Sulfurous acid	Disintegrates rapidly
Tallow and tallow oil	Disintegrates slowly
Tannic acid	Disintegrates slowly
Tanning bark	May disintegrate slowly if damp (see tanning liquor)
Tanning liquor	Disintegrates, if acid
*Tartaric acid solution	Not harmful
Tobacco	Organic acids, if present, disintegrate slowly
Toluol (toluene)	Liquid loss by penetration
*Trichloroethylene	Liquid loss by penetration
*Trisodium phosphate	Not harmful
Tung oil	Liquid disintegrates slowly. Dried or drying films are harmless
Turpentine	Mild attack. Liquid loss by penetration
*Urea	Not harmful
Urine	Attacks steel in porous or cracked concrete
Vegetables	See fermenting fruits, grains, vegetables, extracts
Vinegar	Disintegrates slowly (see acetic acid)
Walnut oil	Disintegrates slowly
*Whey	Disintegrates slowly (see lactic acid)
*Wine	Not harmful. Necessary to prevent flavor contamination
Wood pulp	Not harmful
Xylol (xylene)	Liquid loss by penetration
*Zinc chloride	Disintegrates slowly
Zinc nitrate	Not harmful
Zinc refining solutions (x)	Hydrochloric or sulfuric acids, if present, disintegrate concrete
Zinc slag	Zinc sulfate (which see) sometimes formed by oxidation
Zinc sulfate	Disintegrates slowly

Key to special notations-Table 2.5.2

*	Sometimes used in food processing or as food or beverage ingredient. Ask for advisory opinion of Food and Drug Administration regarding coatings for use with food ingredients.
a	Waters of pH higher than 6.5 may be aggressive if they also contain bicarbonates. (Natural waters are usually of pH higher than 7.0 and seldom lower than 6.0, though pH values as low as 0.4 have been reported. For pH values below 3, protect as for dilute acid.)
b	Frequently used as a deicer for concrete pavements. If the concrete contains too little entrained air or has not been aged more than one month, repeated application may cause surface scaling. For protection under these conditions, see "deicing salts."
c	Carbon dioxide dissolves in natural waters to form carbonic acid solutions. When it dissolves to extent of 0.9 to 3 parts per million it is destructive to concrete.
d	Frequently used as deicer for airplanes. Heavy spillage on runway pavements containing too little entrained air may cause surface scaling.
e	In addition to the intentional fermentation of many raw materials, much unwanted fermentation occurs in the spoiling of foods and food wastes, also producing lactic acid.
f	Contains carbonic acid, fish oils, hydrogen sulfide, methyl amine, brine, other potentially reactive materials.
g	Water used for cleaning coal gas.
h	However, in those limited areas of the United States where concrete is made with reactive aggregates, disruptive expansion may be produced.
n	Composed mostly of nitrogen, oxygen, carbon dioxide, carbon monoxide, and water vapor. Also contains unburned hydrocarbons, partially burned hydrocarbons, oxides of nitrogen, and oxides of sulfur. Nitrogen dioxide and oxygen in sunlight may produce ozone, which reacts with some of the organics to produce formaldehyde, peracylnitrates, and other products.
o	These either contain chromium trioxide and a small amount of sulfate, or ammonium chromic sulfate [nearly saturated) and sodium sulfate.
p	Many types of solutions are used, including (a) Sulfate-Contain copper sulfate and sulfuric acid. (b) Cyanide-Contain copper and sodium cyanides and sodium carbonate. (c) Rochelle-Contain these cyanides, sodium carbonate, and potassium sodium tartrate. (d) Others such as fluoborate, pyrophosphate, amine, or potassium cyanide.
q	Contains lead fluosilicates and fluosilicic acid.
r	Reference here is to combustion of coal, which produces carbon dioxide, water vapor, nitrogen, hydrogen, carbon monoxide, carbohydrates, ammonia, nitric acid, sulfur dioxide, hydrogen sulfide, soot, and ashes.
u	Porous concrete which has absorbed considerable molten paraffin and then been immersed in water after the paraffin has solidified has been known to disintegrate from sorptive forces.
v	Contains nickelous chloride, nickelous sulfate, boric acid, and ammonium ion.
w	May contain various mixtures of blood, fats and oils, bile and other digestive juices, partially digested vegetable matter, urine, and manure, with varying amounts of water.
x	Usually contains zinc sulfate in sulfuric acid. Sulfuric acid concentration may be low (about 6 percent in "low current density" process) or higher (about 22-28 percent in "high current density" process).

Exhibit 33: Corrosion Related Photos


12" MAX DEPTH
POINT OF TANGENCY
14.7'
12' MAX
8' MAX
2' MAX
18" MAX
22" MAX
28" MAX
12" MAX
FLUTE
3/4" CHAMFER (TYP.)

DETAIL 1

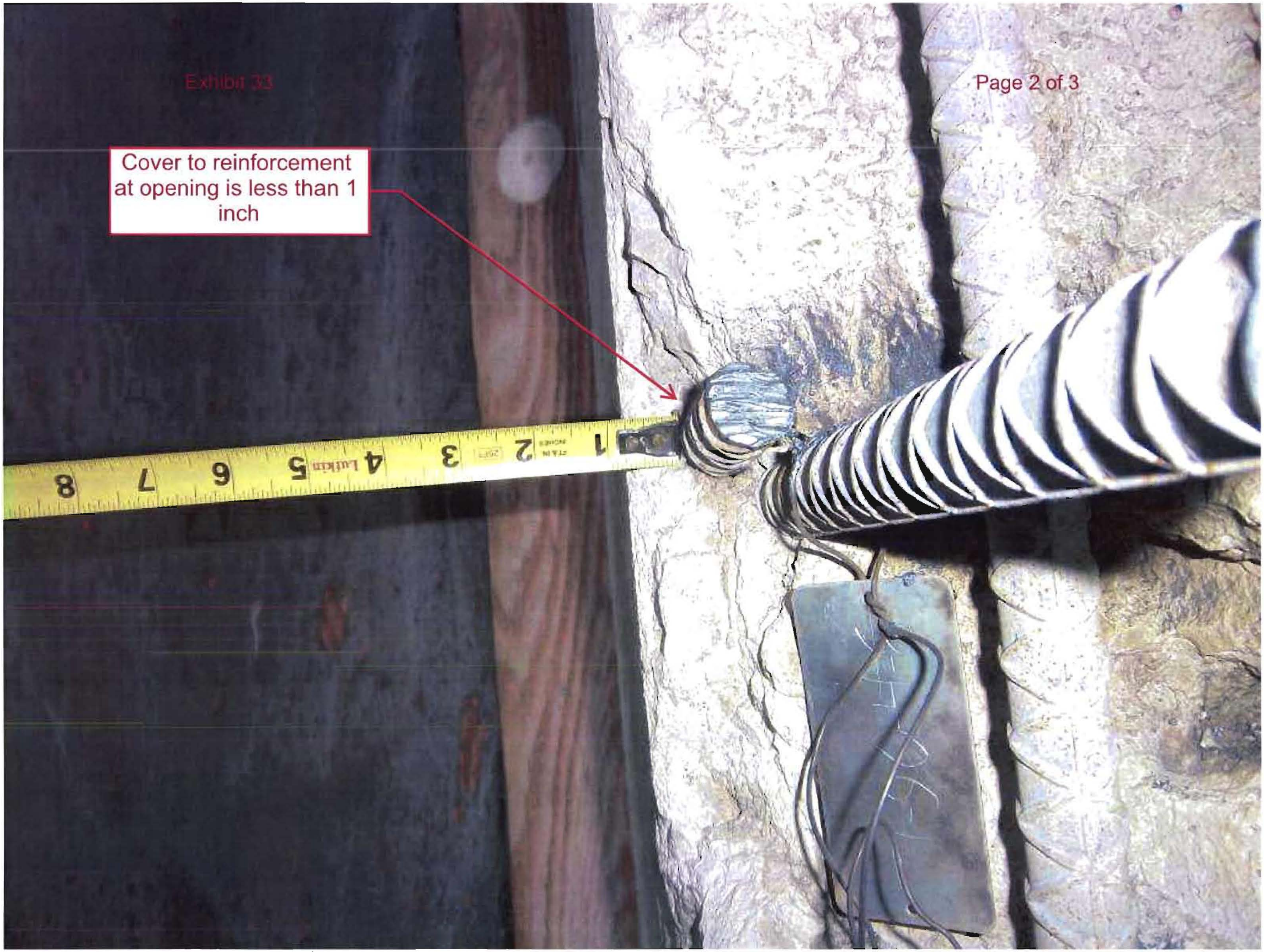

Cover to reinforcement at opening is less than 1 inch

Exhibit 33
Lightly corroded rebar embedded in the opening
Page 3 of 3

Exhibit 34: Concrete Mix Summary for Below Grade

Date	Time	Cement	Water Gal/bag	Cement [lbs.]	FA [lbs.]	CA #4 [lbs]	CA #62 [lbs.]	Water [Gals.]	Water [lbs.]	W/C	Air %	Slump	Air Temp.
1/25/1971	8:53	II	5.65	3384	9120	3522	5556	207.6	1727.232	0.51	5.5%	5.50	31
1/25/1971	16:22	II	5.15	3384	9030	3642	5400	185.4	1542.528	0.46	4.5%	6.00	31
1/25/1971		II	5.32	3348	9000	3570	5646	190.2	1582.464	0.47	5.0%	3.75	37
1/25/1971		II	Field cast										
1/26/1971	3:45	II	5.46	3384	8940	3678	5340	196.2	1632.384	0.48	4.5%	4.00	38
1/26/1971		II	6.34	3102	8700	4080	5970	184.8	1537.536	0.50	4.5%	4.00	24
2/1/1971		II	5.77	3384	9108	3480	5640	207.6	1727.232	0.51	3.5%	4.50	5
2/1/1971	15:32	II	5.95	3384	9270	3300	5496	214.2	1782.144	0.53	6.0%	5.75	9
2/1/1971	21:38	II	5.93	3384	9270	3480	5292	213.6	1777.152	0.53	5.2%	5.75	0
2/2/1971	3:42	II	5.63	3384	9156	3480	5460	207	1722.24	0.51	4.5%	5.50	-2
2/2/1971	12:30	II	5.90	3384	9000	3690	5460	212.4	1767.168	0.52	5.5%	6.00	12
2/2/1971	21:12	II	5.96	3384	8940	3540	5610	213.6	1777.152	0.53	4.5%	6.50	16
2/3/1971	2:48	II	5.96	3384	8940	3564	5610	213.6	1777.152	0.53	4.8%	6.00	19
2/3/1971	9:13	II	5.86	3384	8940	3660	5610	210.6	1752.192	0.52	6.0%	6.00	22
2/3/1971	14:29	II	5.64	3384	9000	3876	5280	202.8	1687.296	0.50	5.5%	5.00	29
2/3/1971	21:02	II	5.72	3384	8940	3570	5010	205.8	1712.256	0.51	5.5%	5.50	28
2/4/1971	3:41	II	5.57	3384	9000	3540	5010	200.4	1667.328	0.49	6.0%	4.50	32
2/4/1971	11:17	II	6.02	3384	8550	3690	5436	217.2	1807.104	0.53	6.5%	5.00	34
2/4/1971	18:33	II	5.78	3384	8724	3684	5496	207.6	1727.232	0.51	4.5%	5.50	36
									Average:	**0.51**	**5.1%**	**5.26**	
									StD:	**0.02**	**0.8%**	**0.81**	
									Count:	**18**	**18**	**18**	

Concrete mix summary for below-grade concrete – Mix C-2-SF-2 with Type II cement

Exhibit 35: Concrete Strength Summary for Below Grade

Date	Measured Strength [psi]								Average Strength		
	2 days	3 days	7 days	7 days	28 days	28 days	90 days	90 days	7 days	28 days	90 days
1/25/1971		2299		2989	5022	5005	5974	5871	2989	5014	5923
1/25/1971			3661	3643	6083	6066	7498	7342	3652	6075	7420
1/25/1971			3236	3272	5959	5889	6402	7074	3254	5924	6738
1/25/1971			3395	3289	5730	5765	6967	6950	3342	5748	6959
1/26/1971	1734			3608	6437	6348	7286	7321	3608	6393	7304
1/26/1971			3059	3059	5730	5730			3059	5730	
2/1/1971			3625	3537	6083	6172	6808	6791	3581	6128	6800
2/1/1971			3059	3042	5517	5482	6770	6791	3051	5500	6781
2/1/1971			3059	3234	5517	5411	6826	6684	3147	5464	6755
2/2/1971			3908	3837	6225	6295	7569	7622	3873	6260	7596
2/2/1971			3342	3183	5376	5252	6631	6755	3263	5314	6693
2/2/1971			3059	3042	5376	5287	6437	6437	3051	5332	6437
2/3/1971			3360	3307	5765	5730	6720	6773	3334	5748	6747
2/3/1971			3289	3236	5730	5712	6348	6402	3263	5721	6375
2/3/1971			3378	3431	6172	6119	6755	6773	3405	6146	6764
2/3/1971			3148	3148	5712	5464	6402	6348	3148	5588	6375
2/4/1971			3272	3342	5942	5641	6879	6932	3307	5792	6906
2/4/1971			3218	3183	5482	5424	6437	6455	3201	5453	6446
2/4/1971			3059	3095	5287	5394	6791	6614	3077	5341	6703
								Average:	3295	5719	6762
								StD:	239	367	399

Concrete Strength summary for below-grade concrete – Mix C-2-SF-2 with Type II cement

Exhibit 36: Concrete Strength Summary for Above Grade

Date	Measured Strength [psi]						Average Strength [psi]		
	7 days	7 days	28 days	28 days	90 days	90 days	7 days	28 days	90 days
5/4/1971	4881	4881	6402	5588	6967	6861	4881	5995	6914
5/4/1971	4615	4587	6136	6172	7491	7352	4601	6154	7422
5/4/1971	4442	4562	6085	6083	7003	7576	4502	6084	7290
5/4/1971	4032	4386	5871	5853	6844	6861	4209	5862	6853
5/4/1971	4739	4828	6649	6207	7463	7109	4784	6428	7286
5/4/1971	4669	4880	5924	6136	7427	7356	4775	6030	7392
5/5/1971	4810	4775	6070	5942	7074	7162	4793	6006	7118
5/5/1971	4810	4825	5517	5606	7250	6791	4818	5562	7021
5/5/1971	5040	4916	5553	5234	6285	6720	4978	5394	6503
5/5/1971	4067	4191	5126	4881	5896	5871	4129	5004	5884
5/5/1971	4474	4315	5234	5553	6826	6879	4395	5394	6853
5/5/1971	4297	4330	5483	5853	6667	7144	4314	5668	6906
5/6/1971	4633	4669	6154	6366	7374	7639	4651	6260	7507
5/6/1971	5146	5181	5641	5942	7639	7392	5164	5792	7516
5/6/1971	5199	5111	6702	5765	7445	7728	5155	6234	7587
5/6/1971	4916	4916	6225	6313	7498	7958	4916	6269	7728
5/6/1971	4651	4580	6154	6260	7887	7421	4616	6207	7654
5/6/1971	4315	4421	6154	5959	6720	6861	4368	6057	6791
5/7/1971	4757	4434	5234	5270	7356	7162	4596	5252	7259
5/7/1971	3837	3926	4704	4757	7162	6738	3882	4731	6950
5/7/1971	4633	4704	5712	5747	7816	7993	4669	5730	7905
5/7/1971	4562	4669	6348	5836	7816	7710	4616	6092	7763
5/7/1971	4492	4403	5500	5836	6867	7639	4448	5668	7253
5/7/1971	4315	4262	5765	5730	7162	6649	4289	5748	6906
5/7/1971	4279	4563	5447	5959	7569	7463	4421	5703	7516
5/8/1971	4244	4069	5730	5482	7056	7215	4157	5606	7136
5/10/1971	4456	4368	5464	5553	6967	7003	4412	5509	6985
5/10/1971	4969	4645	5977	5485	7374	7604	4807	5731	7489
5/10/1971	5252	5093	6543	6366	7816	8099	5173	6455	7958
5/10/1971	4633	4474	5818	6030	7250	7569	4554	5924	7410
5/10/1971	4050	3802	5199	5164	6508	6684	3926	5182	6596
5/10/1971	4403	4244	5058	5252	6832	7180	4324	5155	7006
5/11/1971	4633	4633	5800	5659	7233	7569	4633	5730	7401
5/11/1971	4545	4633	6083	6030	7463	7710	4589	6057	7587
5/11/1971	4368	4323	5659	6295	6967	7003	4346	5977	6985
5/11/1971	4226		5641	5341	7710	7321	4226	5491	7516
5/11/1971	4526	4244	5588	5439	7250	7091	4385	5514	7171
5/11/1971	4156	4209	5508	5500	7063	6614	4183	5504	6839
5/12/1971	4103	4138	5270	5694	6561	6578	4121	5482	6570
5/12/1971	3714	3754	5376	5120	6225	6295	3734	5248	6260
5/12/1971	4709	4612	6172	6437	7533	7427	4661	6305	7480
5/12/1971	4562	4598	6541	6360	7215	7003	4580	6451	7109
5/12/1971	4244	4244	5863	5059	6738	7109	4244	5461	6924
5/12/1971	4350	4350	4225	5800	6355	6508	4350	5013	6432
5/12/1971	3873	3969	4810	5199	6245	6295	3921	5005	6270
5/13/1971	4491	4403	5411	5022	6619	6295	4447	5217	6457
5/13/1971	4032	4191	5677	5677	6667	6614	4112	5677	6641
5/13/1971	4315	4598	5889	6278	7922	7710	4457	6084	7816
5/13/1971	4297	4226	6860	5800	7675	8135	4262	6330	7905
5/13/1971	3908	3714	5906	5818	7533	7374	3811	5862	7454
5/13/1971	4180	4156	6043	5617	7034	6985	4168	5830	7010
5/14/1971	4478	4598	6109	5836	7356	6890	4538	5973	7123
5/14/1971	4375	4173	5487	5588	7567	7394	4274	5538	7481
5/14/1971	3696	3714	5906	6119	7763	7874	3705	6013	7819
5/14/1971	3376	3466	5401	5783	7409	7576	3421	5592	7493
5/14/1971	4562	4492	6824	6242	8001	8192	4527	6533	8097
5/14/1971	4334	4278	5694	5765	7410	7657	4306	5730	7534
5/14/1971	3943	3908	6066	5800	7269	6718	3926	5933	6994
5/14/1971	3784	3655	5553	5482	7749	7927	3720	5518	7838
5/15/1971	4527	4527	6331	6500	7985	7995	4527	6416	7990

5/15/1971	4474	4492	5820	6154	7120	6980	4483	5987	7050
5/17/1971	3714	4226	5765	5783	6739	7180	3970	5774	6960
5/17/1971	4810	4589	6525	6172	7675	8028	4700	6349	7852
5/17/1971	4315	4297	6419	6030	7692	7675	4306	6225	7684
5/17/1971	3760	3749	5906	5698	6423	6808	3755	5802	6616
5/18/1971	4032	4050	5500	5517	6702	6861	4041	5509	6782
5/18/1971	3979	3678	5747	5765	6455	6684	3829	5756	6570
5/18/1971	3767	3661	5783	5219	6596	6773	3714	5501	6685
5/18/1971	3466	3943	5623	5600	6826	6932	3705	5612	6879
5/18/1971	3289	3590	5959	5765	7569	7889	3440	5862	7729
5/19/1971	3431	3448	4987	5447	6437	6331	3440	5217	6384
5/19/1971	2812	2812	5111	4845	6455	6561	2812	4978	6508
5/19/1971	3979	3784	5181	5500	6260	6189	3882	5341	6225
						Average:	4100	5749	7079
						StD:	582	400	505
						Count:	92	92	91
						Max:	5173	6533	8097
						Min:	2812	4731	5884

Concrete Strength summary for above-grade concrete – Mix C-2-SF-2 (5/4/71 to 5/6/71) and C-2-SF-4 with Type I cement

Exhibit 37: Earthquake Event

Davis-Besse Classifiable Events

A total of 34 declared emergencies/classifiable events have occurred at Davis-Besse. Of these events there have been **4** Alerts and **30** Unusual Events. Several of the pre-1993 events are not currently classifiable under our current EALs due to the NRC allowing the elimination of the Technical Specification 3.0.3 Unusual Event EAL and several others EALs that were administrative situations and not actual emergency events in 1993.

Event classifications are linked to the NRC Daily Event Report, where applicable.

DATE	CLASSIFICATION	EVENT
11/16/11	**Alert**	Alert Due to Fire in Electrical Bus Affecting Safety Related Equipment
01/19/11	Unusual Event	Unusual Event Due to a Fire and Explosions in the Protected Area
06/25/09	**Alert**	Transitory Alert due to catastrophic failure of CCPD in Switchyard
08/14/03	Unusual Event	Loss of offsite power caused by grid disturbance/blackout
04/23/00	Unusual Event	Loss of offsite power caused by outage electrical system testing
06/26/98	Unusual Event	Misc. - Downgrade from Alert
06/24/98	**Alert**	Tornado striking the facility
12/22/93	Unusual Event	Both Control Room ventilation systems were declared inoperable due to a small refrigerant leak
10/08/90	Unusual Event	Chlorine gas release in Water Treatment Building
05/18/90	Unusual Event	Unplanned ECCS actuation during outage testing
02/09/89	Unusual Event	Low Lake Level
04/06/88	Unusual Event	Transportation to an offsite medical facility of a potentially contaminated injured individual
03/04/88	**Alert**	Loss of Decay Heat Removal System (Cooler inlet valves declared inoperable)

01/01/88	Unusual Event	Low Lake Level
12/15/87	Unusual Event	Low Lake Level
03/30/87	Unusual Event	SFAS Sequencer out of tolerance (T.S. 3.0.3)
01/12/87	Unusual Event	Auxiliary Feedwater pressure switches out of ~~tolerance (T.S. 3.0.3)~~
03/05/86	Unusual Event	Seismic Activity
~~12/16/85~~	~~Unusual Event~~	~~Low Lake Level~~
06/09/85	Unusual Event	Loss of Main Feedwater/Auxiliary Feedwater Malfunction
05/16/85	Unusual Event	RCS Leak (Pressurizer Spray Value Packing Leak)
05/06/85	Unusual Event	Loss of Meteorological Indications
05/02/84	Unusual Event	Loss of Meteorological Indications
03/02/84	Unusual Event	Main Steam Line Safety Valve Stuck Open
02/21/84	Unusual Event	Loss of Meteorological Indications
01/17/84	Unusual Event	Loss of Meteorological Indications
12/17/83	Unusual Event	RCS Leak (Letdown System Packing Leak)
01/18/83	Unusual Event	Loss of Containment Integrity (No. 2 Main Steam Safety Leaking)
12/16/82	Unusual Event	Overturned Gasoline Truck east of DBAB on SR 2 under transmission lines
02/19/81	Unusual Event	Loss of Meteorological Indications
02/04/81	Unusual Event	Loss of Meteorological Indications
07/29/80	Unusual Event	Eye Injury Non-Nuclear Related
06/25/80	Unusual Event	Minor Fire in Control Room
04/13/80	Unusual Event	Flood Watch

Return to Other Licensing Documents

POWER REACTOR	EVENT NUMBER: 3837

FACILITY: DAVIS BESSE REGION: 3 UNIT: [1] [ ] [ ] STATE: OH RX TYPE: [1] B&W-R-LP	NOTIFICATION DATE: NOTIFICATION TIME: [ET] EVENT DATE: 03/05/86 EVENT TIME: 08:07[EDT]
NRC NOTIFIED BY: HQ OPS OFFICER:	LAST UPDATE DATE:
	NOTIFICATIONS
EMERGENCY CLASS: 10 CFR SECTION:	CREED RIII ALLISON EO GREINER FEMA

UNIT	SCRAM CODE	RX CRIT	INIT PWR	INIT RX MODE	CURR PWR	CURR RX MODE
1	N	N	0	COLD SHUTDOWN	0	COLD SHUTDOWN

EVENT TEXT

UNIT IN COLD SHUTDOWN - AT 0807 EST STATION SEISMIC MONITOR #ZT2957 INDICATED A SEISMIC EVENT ON SITE. AT 0845 EST LICENSEE DECLARED AN UE. NO PHYSICAL MOVEMENT AT THE SITE WAS OBSERVED. LICENSEE CONTACTED FERMI AND PERRY SITES AND THERE WAS NO SEISMIC INDICATION AT EITHER SITE. LICENSEE SUSPECTED THAT THE INDICATION WAS SPURIOUS. RI WILL BE INFORMED.

UPDATE AT 1034 LICENSEE TERMINATED UE. NOTIFIED R3 CREED, EO ALLISON, FEMA GREINER.

[redacted] BB 91-0066 [redacted] Page 1 of 6

DAVIS-BESSE NUCLEAR POWER ST[redacted]ON
DEVIATION REPORT
EO 6731 2

Attach additional pages as appropriate

DVR NO.	PAGE
86-060	1 of 2

FOUND BY	DATE	TIME	UNIT(S)	MODE	POWER (MWt)	LOAD (MWE)
W. D. Siferd	3/5/86	0807 A.M. / P.M.	1	5	0	0

SYSTEM(S) AFFECTED (Include System Number(s))	COMPONENT(S) AFFECTED (Include Equipment Number(s))

NPRDS REPORTABLE	TECHNICAL SPECIFICATION SECTION(S) REFERENCE
☐ Yes ☒ No [redacted]	3.3.3.3

PART 1

DESCRIPTION OF EVENT: This DVR is being initiated to track and record findings relating to the Seismic Trigger Actuation of 3/5/86. The actuation occurred at 0807 EST.

CAUSE OF EVENT:

INITIAL ACTION TAKEN: Declared Unusual Event. Contacted Fermi and Perry but they observed no seismic event. Notified NRC via ENS of unusual event. Contacted Geology Dept Univ. of Toledo. They didn't detect any seismic ~~event~~ activity. Completed walkdown of Safety Systems per [illegible].

EVENT REPORTABILITY (See AD 1804.00)	REFERENCE RRF NO'S (See AD 1804.00)	REPORTING REQUIREMENTS SUMMARY (From AD 1804.00, Enclosure 6)
☐ Non Periodic ☐ Periodic ☒ Not Reportable	N/A	IMMED. / 24 HR. / 5 DAY / 7 DAY / OTHER/NOTES: MAR 12 1986 TECHNICAL SUPPORT; 10 DAY / 14 DAY / 30 DAY / 60 DAY / 90 DAY

IMMEDIATE AND 24 HR. REPORTS

REPORT DESTINATION (See AD 1804.00)	TELEPHONE Date	TELEPHONE Time	TELEPHONE REPORT(S) MADE BY	TELEGRAPH/TELECOPY Date	TELEGRAPH/TELECOPY Time	CONFIRMING TELEGRAPH(S) MADE BY
USNRC REGION III						

REMARKS/RESTRICTIONS/SPECIAL INSTRUCTIONS

EVENT/CONDITION INVESTIGATION/DETERMINATION OF CORRECTIVE ACTION/REPORT PREPARATION

RESPONSIBLE SECTION	RESPONSIBLE PERSON(S)	ACTION DEADLINE	TECHNICAL ENGINEER	DATE
Tech Support	Dean Siferd	3-24-86	[signature]	3/12/86

PART 2

RESOLUTION AND CORRECTIVE ACTION SUMMARY

The attached correspondence from the University of Toledo identifies that no seismic event occurred during the period of time that the seismic trigger actuated. There is therefore no reason to suspect equipment damage ~~or~~ or any otherwise Significant Condition Adverse to quality resulting from the actuation that prompted initiation of this DVR 86-060. This DVR should be closed. No root cause can be determined.

REF.	[redacted]	MWO NO.	FCR NO.	NCR NO.	CAR NO.	PROCEDURE MODS.	RESPONSIBLE PERSON	DATE
			86-322				[signature]	3 Feb 87

RELIABILITY ENGR.	DATE	TECHNICAL ENGR.	DATE	SRB CHAIRMAN	DATE	STATION	DATE
[signature]	2/5/87	Jack [signature]	2/25/87	D. W. Briden	JUN 2 1987	[signature]	JUN 2 1987

DISTRIBUTION (Part 1)
- ☐ VP, Nuclear
- ☐ Asst. to VP, Nuclear
- ☐ Station Superintendent
- ☒ Quality Assurance Mgr.
- ☐ Gen. Supt., PE&C
- ☒ Central File
- ☐ Assistant Supt.
- ☐ Nuclear Services
- ☐ Reliability Engineer
- ☐ Operations Engineer
- ☐ Training Supervisor

DISTRIBUTION (Part 2)
- ☐ VP, Nuclear
- ☐ Asst. to VP, Nuclear
- ☐ Quality Assurance Mgr.
- ☐ Training Supervisor
- ☐ Central File (Original)
- ☐ Nuclear Services
- ☐
- ☐
- ☐

[illegible] SUPT.(C) [illegible] R. [illegible] [illegible] [illegible]

U.S. NUCLEAR REGULATORY COMMISSION

EVENT NOTIFICATION WORKSHEET

OPERATIONS CENTER

DVR 86-060

PALLIEFF 2 3

NOTIFICATION TIME	FACILITY OR ORGANIZATION UNIT	CALLER'S NAME	TELEPHONE NUMBER (For call back)
0907	DB unit 1 Cren	N. Weh l	1-419-249-5664

EVENT CLASSIFICATION	
	GENERAL EMERGENCY
	SITE AREA EMERGENCY
	ALERT
X	UNUSUAL EVENT
	50.72 NON-EMERGENCY
	SECURITY / SAFEGUARDS
	TRANSPORTATION EVENT
	OTHER

T	N	EVENT CATEGORY
	X	REACTOR TRIP/SCRAM
	X	ESF ACTUATION
	X	ECCS ACTUATION
	X	SAFETY INJECTION FLOW
	X	LCO ACTION STATEMENT
		OTHER

INITIATION SIGNAL: ZT-5945? Station Seismic Instr on Alarm

	CAUSE OF FAILURE
	MECHANICAL
	ELECTRICAL
	PERSONNEL ERROR
	PROCEDURE INADEQUACY
	OTHER

SYSTEM: Unit Seismic Instr

COMPONENT: Seismic Trigger

EVENT TIME	ZONE	EVENT DATE	MONTH	DAY
0807	EST		03	05

EVENT DESCRIPTION

At 0807 the station recieved a Siesmic ~~Alarm~~ Trigger Actuation, on OBE Longitudinal. An Unusual Event emergency classification was declared at 0845 Actions of Seismic Procedures and Emergency Plan performed. There ~~was~~ were no reports from anywhere on ~~or~~ or offsite of noticeable tremmor activity. No Security Fence alarms were recieved. The trigger area was observed, and no activity was discernable. ~~Do~~ Initial de-logging of the Seismic Instrument showed no significant disturbance, although at the highest gain, some noise was seen. Called Fermi + Perry, nothing

POWER PRIOR TO EVENT (%):	0
CURRENT POWER OR MODE:	5

Did all systems function as required ?	YES	If NO, Explain above.
Anything "unusual" or not understood ?	NO	If YES, Explain above.

OUTSIDE AGENCY OR PERSONNEL NOTIFIED BY LICENSEE

STATE(S): ODSA

LOCAL: Ottawa Co Sheriff

RESIDENT: YES | NO | WILL BE

OTHER: State Hwy Patrol

CORRECTIVE ACTION(S)

MODE OF OPERATION TILL CORRECTION:

ESTIMATE TIME TO RESTART:

X PRESS RELEASE

ADDITIONAL INFORMATION ON BACK

4

was Noted. [redacted] walkdown of all [redacted] Safety
systems was completed. Event was
Terminated at 1034 per engineering.
NRC Again notified by red phone at
1038; Resident Inspector notified at 1045.

DVR 86-060 78

992 Attachment 1

5

The University of Toledo

2801 W. Bancroft Street
Toledo, Ohio 43606

College of Arts and Sciences
Department of Geology
(419) 537-2009

March 10, 1986

IA86-0079

Mr. Louis Storz, Plant Manager
Toledo Edison Company
300 Madison Avenue, M.S. #2103
Toledo, Ohio 43652

Dear Mr. Storz:

This letter is to confirm my oral conversation of March 5, 1986, during which I stated that no unusual seismic activity occurred in the Toledo area between 8:00 A.M. and 9:00 A.M., Eastern Standard Time (13:00 through 14:00 Universal Time), of that date. Our instrumentation includes one short-period vertical seismometer which has been and continues to detect moderate to large teleseisms as well as local quarry blasts and cultural noise. The only vibrations detected by this transducer, located in the basement of Bowman-Oddy Laboratories on the University of Toledo campus, during the time in question were the vibrations associated with traffic or other human activities nearby. I am thus certain that the instrument was working properly and that nothing that could be termed an "earthquake" occurred in the Toledo area.

Vibrations from the Chardon, Ohio earthquake of January 31, 1986, on the other hand, were so strong that our record was clipped (the dynamic range of the system was exceeded).

Sincerely,

Donald J. Stierman

Donald J. Stierman
Assistant Professor of
Geology

DJS/b

DATE RECEIVED AT DSNPS / MAR 13 1986				ROUTE		ADD TO M&O
	A	BY	I	INIT	DATE	
LFS			X			
DWB						
WIO						
LMS						
SJS						
RKF	X					
DMM						
ALM						
JLB						
Novak			CC			
SIC				Tickle ( )		
FILE			5114			

COMMENTS:
Attach to DVR on false trip

DVR 86-060
Attachment 2

6

THE TOLEDO EDISON COMPANY
DOCUMENT REVIEW
ED 8800-1

FILE: End

DATE

SHEET ______ OF ______

REVIEW SUBMITTED TO		REVIEW CONDUCTED BY	
ORGANIZATION	INDIVIDUAL	ORGANIZATION	INDIVIDUAL
SRB/494		SRB	D W BRIDEN

☐ NO RESPONSE REQUESTED

☐ A RESPONSE TO EACH COMMENT IS REQUESTED. PLEASE RETURN THIS FORM WITH YOUR RESPONSE IN THE SPACE PROVIDED.

DOCUMENT TITLE: DVR 86-060

COMMENTS	RESPONSE
Need to address why the seismic trigger activated.	The exact cause of the seismic trigger activation has not been determined. Steps to prevent seismic trigger actuation from erroneous input have been taken in the form of FCR 86-322. W. Dean Lofland 4/2/87
4-7-87 What is root cause? If none can be determined.. Say it! EWalsmith 4/7/87	See the last sentence on DVR form identifying that no root cause can be determined. W Dean Lofland 4/20/87

805 / 3755

DAVIS-BESSE NUCLEAR POWER STATION
UNIT LOG

MAR 5 1986

No. 10072

0800-1600 Shift Supervisor R E Mizek	Mode 5, RCS on DH Loop 2 thru MU 4 P
Shift Supervisor	TS Actions: Fire barriers per Station Board
...wed Safety Tagging Log Rem /	(3.7.10 & DOC 50-346); Rooms 123, 124, 304, 345
...wed Jumper and Lifted Wire Log Rem /	601, 602 (3.7.10); Both EVS Trains; Emerg
...wed D-B Daily Status Rem /	Hatch, RE8446, RE8442, CV5034, CV5025,
...wed Unit Log Rem /	Neg Press Area Boot Seals (3.9.12); RE1878A,
...wed Reactor Operator's Log Rem /	RE1878B (3.3.3.9); NI 1 & 2 (3.3.1.1); All 4
...wed Alarms Rem /	SFAS Chans (3.3.2.1); WGST O2 Monitor, FT
...wed Blue Status Lights Rem /	5090, FT5090A (3.3.3.10);
...wed Locked Valve Log Rem /	Equip OOS: RE's per Station Board; CCW
...wed Capped Valve Log Rem /	Pump #1; CCW Pump Room HAV Trains
...wed Passive Valve Log Rem /	functional but not operable; Main Station
Temperature 98 Pressure 40 Flow 3,000	
Technical Specifications for Limits	

Exhaust Fan #1; RW Exhaust Fan #2; DH 11 & DH 12; CCA-4-H7 Snubber on DH Drop Line (Mode 1-4); Equip in Room 500 & 501 non "EQ" (Mode 1-3); YVA; XYB, DBC 1N & 2N Rem, Battery 1N & 1P; EDG #1-1; EDG 1-2 operable on D.A 31 side only

0807 Received computer & annunciator alarms on Station Seismic System.
The alarm on the Seismic Panel was for OBE longitudinal

0815 Informed the Ops Office

0845 Declared an "Unusual Event" due to Seismic Trigger Actuation

0910 Contacted Perry NPS; they did not have any indication of any seismic activity.

0914 Contacted Fermi NPS; they had no indication of any seismic activity

LE 0855 Paged Emergency Response ~~Grp~~ Rem Group

0900 Recorded tape for Edison Operator

0907 Notified the NRC via ENS of the Unusual Event (JOLLIFFE)

0917 Released by the NRC on the ENS

0923 Contacted Dr. Stierman at the University of Toledo Geology Dept.

0927 Received a call back from University of Toledo Geology Dept. (Dr Stierman)
They did not detect any seismic activity

WHITE: To remain in book
YELLOW: Technical Engineer

DAVIS-BESSE NUCLEAR POWER STATION
UNIT LOG

MAR 5 1986

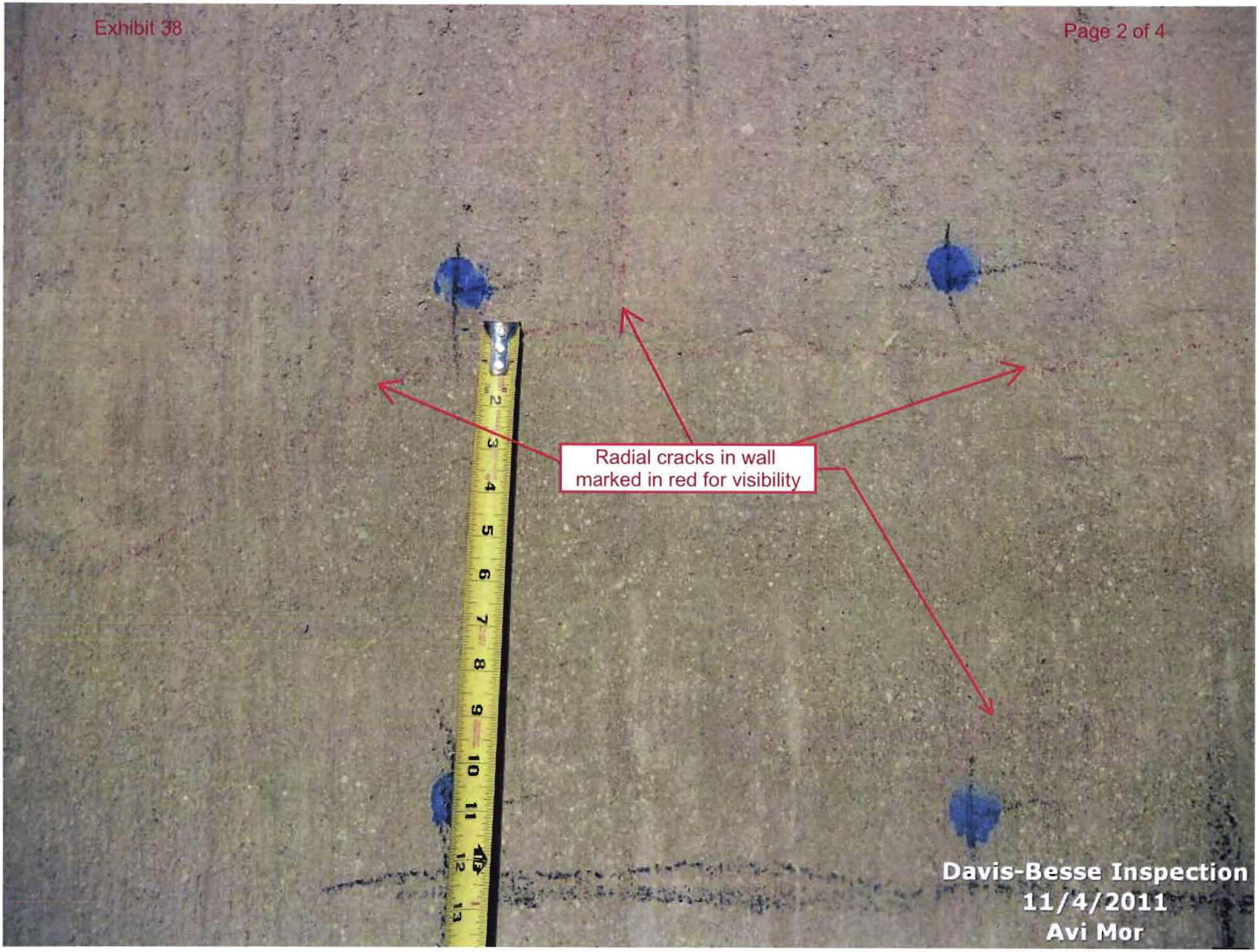
No. 10073

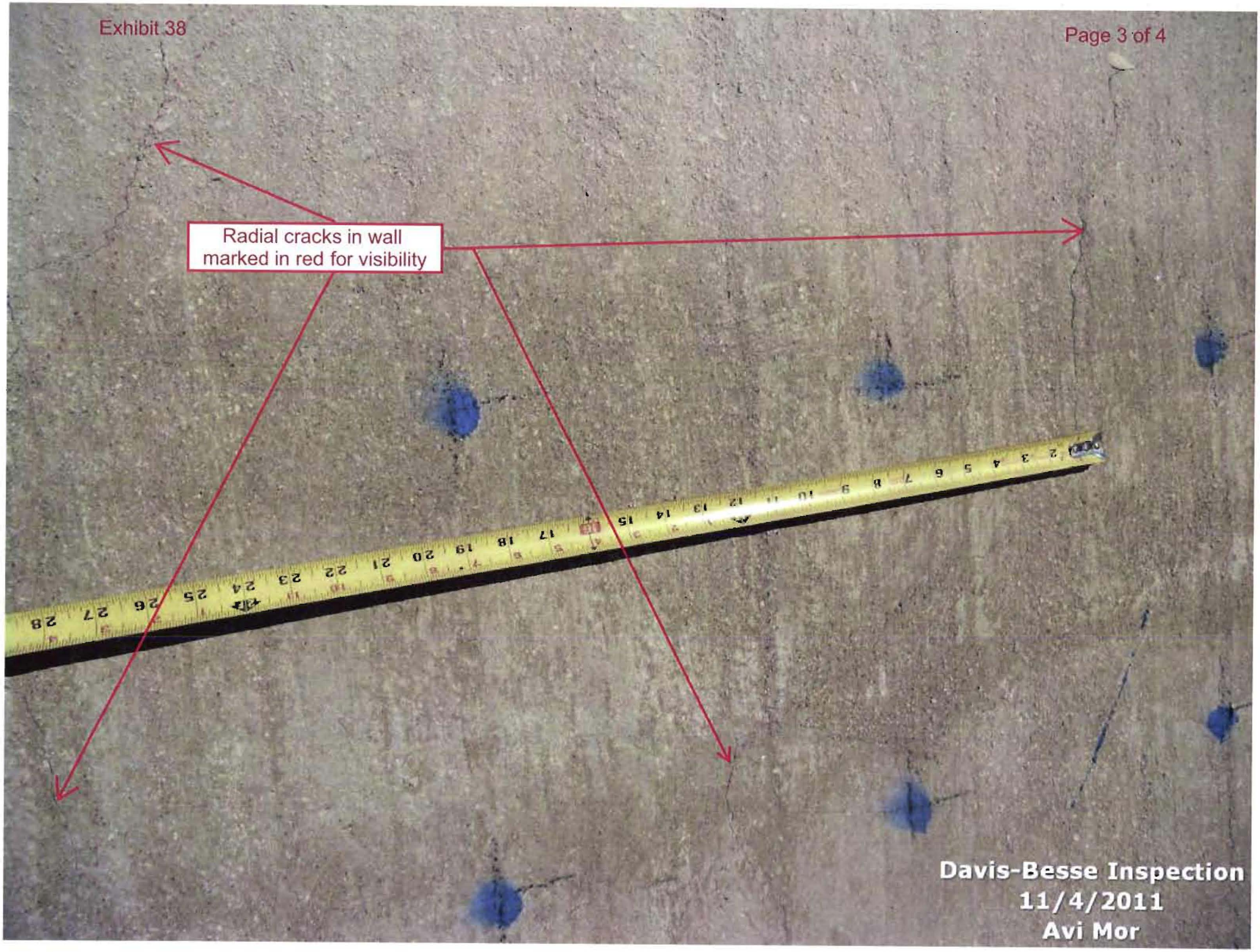
1024 Walkdown of Safety Systems completed per AD 1827.07
1030 Seismic Instrumentation inoperable due to only 3 of 4 recording tapes available for the recorder, Spares at Camp Perry. Seismic System will be operable when the fourth recording tape is inserted.
1031 Per T. Seley; based on analysis of the tapes, there is no evidence of a seismic event. Seismic Insts inoperable. Entered TS 3.3.3.3
1034 Terminated the "Unusual Event"
1038 Notified the NRC
1045 All notifications of terminating the Unusual Event complete
LE 0855 Initiated an hourly fire watch in Room 238. Wackenhut to perform. (Welding & grinding in the room)
LE 0945 Battery Room temperatures: #1 = 84°F, #2 = 78°F
1054 Door 300 opened
1056 Door 300 closed
LE 1012 Reviewed ST 5099.01 & ST 5099.05. Verified SDM ≥ 1% Δk/k
1100 Received a DVR written for #2 Aux Feed Pump Room H&V Train. Prior to energizing the control circuit for MP 1411.09, the starter contacts ~~were~~ appeared to be welded closed.
1143 Received a DVR written due to a fire watch patrol missing the 04:00 fire watch of the Main Steam Line Room on 3-4-86
1232 Received 1 load of fuel oil
LE 1130 Shut down the Auxiliary Boiler due to a bad steam leak on a plug in the first blowdown valve off of the Aux Blr mud drum
1245 Suspended ST 5031.04 Ctmt Rad to SFAS - in SFAS Chan 3
1316 Reviewed ST 5084.01 Station Batterys Weekly. 1P & 1N batteries remain inoperable as the weekly test only checks pilot cells.
1330 I&C began ST 5031.05 BWST Level input to SFAS Chan Calibration on SFAS Ch 1

WHITE: To remain in book
YELLOW: Technical Engineer

Exhibit 38: Shrinkage Crack Photos


Exhibit 38
Page 1 of 4
Radial cracks in Parapet
CTL
Davis-Besse Inspection
11/4/2011
Avi Mor


Radial cracks in wall marked in red for visibility
Davis-Besse Inspection
11/4/2011
Avi Mor


Radial cracks in wall marked in red for visibility
Davis-Besse Inspection
11/4/2011
Avi Mor

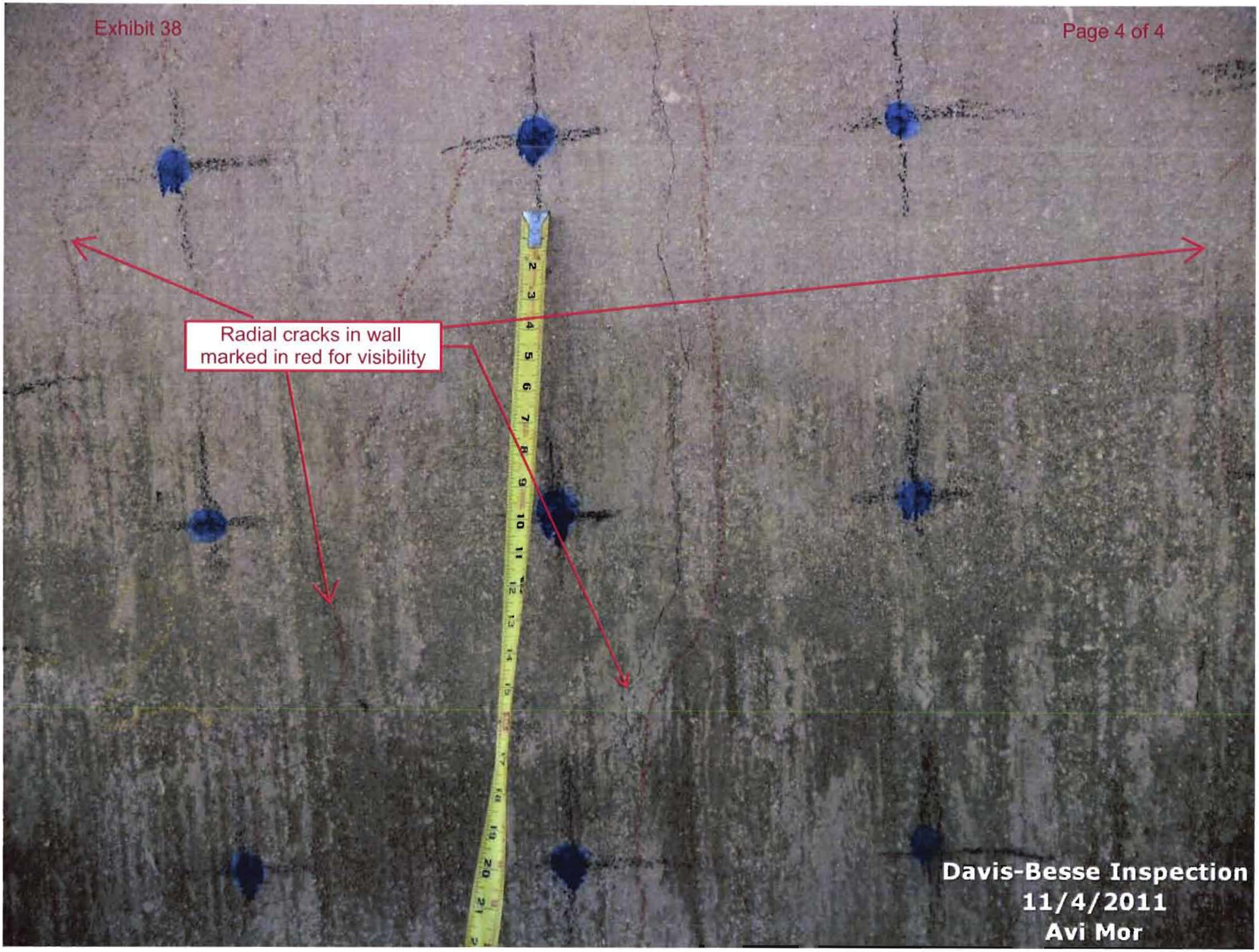

Radial cracks in wall
marked in red for visibility
Davis-Besse Inspection
11/4/2011
Avi Mor

Exhibit 39: C-0112 – Shield Building Details

Exhibit 39

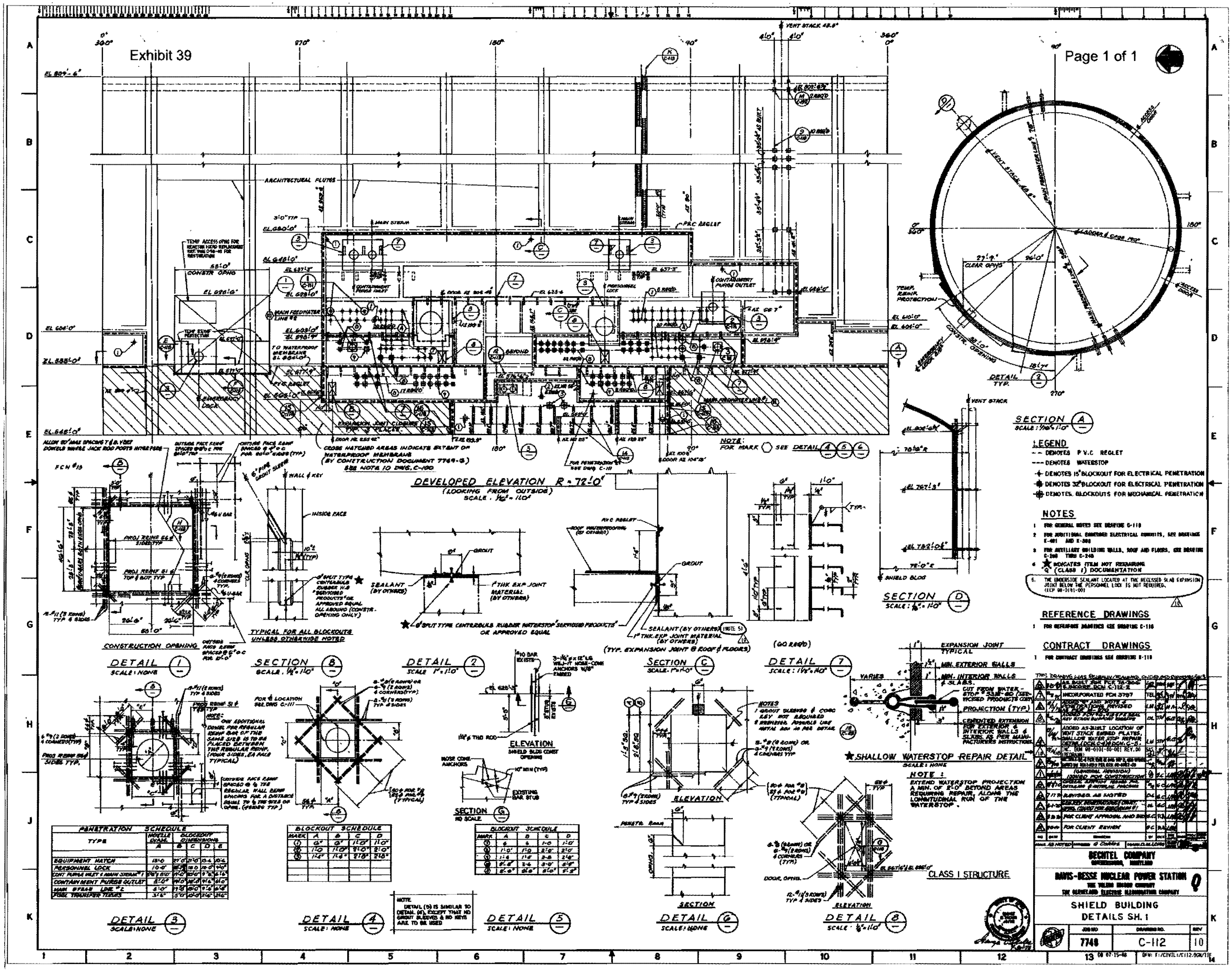

DEVELOPED ELEVATION R = 72'-0"
(LOOKING FROM OUTSIDE)
SCALE: 1/8" = 1'-0"
SECTION A
SCALE: 1/16" = 1'-0"
LEGEND
DENOTES P.V.C. REGLET
DENOTES WATERSTOP
DENOTES 15" BLOCKOUT FOR ELECTRICAL PENETRATION
DENOTES 32" BLOCKOUT FOR ELECTRICAL PENETRATION
DENOTES BLOCKOUTS FOR MECHANICAL PENETRATION
NOTES
REFERENCE DRAWINGS
CONTRACT DRAWINGS
DETAIL 1
CONSTRUCTION OPENING
SCALE: NONE
SECTION B
DETAIL 2
SECTION C
DETAIL 7
SECTION D
SHALLOW WATERSTOP REPAIR DETAIL
SCALE: NONE
ELEVATION
SECTION G
NO SCALE
PENETRATION SCHEDULE
BLOCKOUT SCHEDULE
DETAIL 3
SCALE: NONE
DETAIL 4
SCALE: NONE
DETAIL 5
SCALE: NONE
DETAIL 6
SCALE: NONE
DETAIL 8
CLASS I STRUCTURE
FOR CLIENT APPROVAL AND BIDS
FOR CLIENT REVIEW
SCALE: AS NOTED
BECHTEL COMPANY
DAVIS-BESSE NUCLEAR POWER STATION
THE TOLEDO EDISON COMPANY
THE CLEVELAND ELECTRIC ILLUMINATING COMPANY
Q
SHIELD BUILDING
DETAILS SH. 1
JOB NO.
7749
DRAWING NO.
C-112
REV
10

Exhibit 40: C-0111 – Shield Building Wall Development

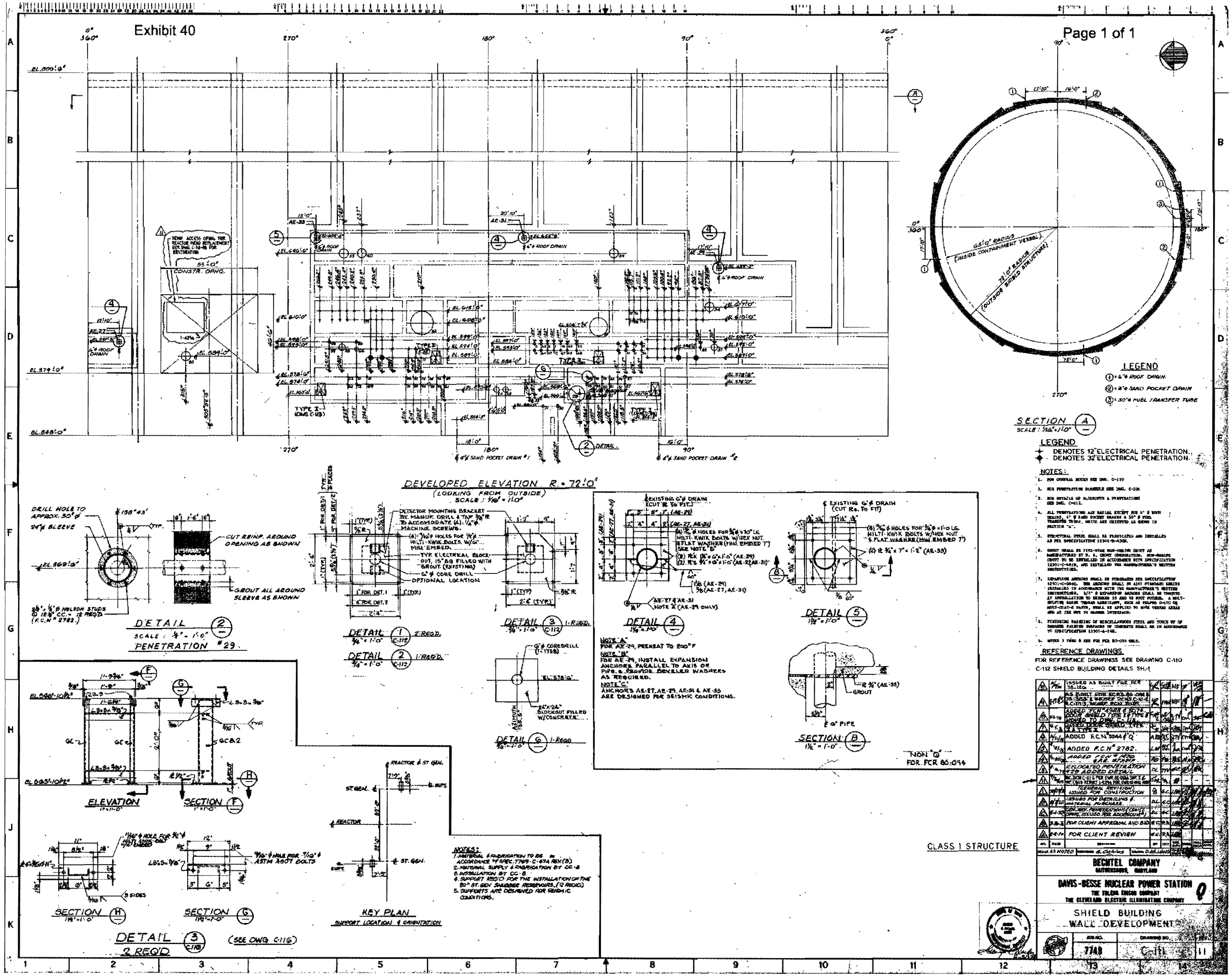
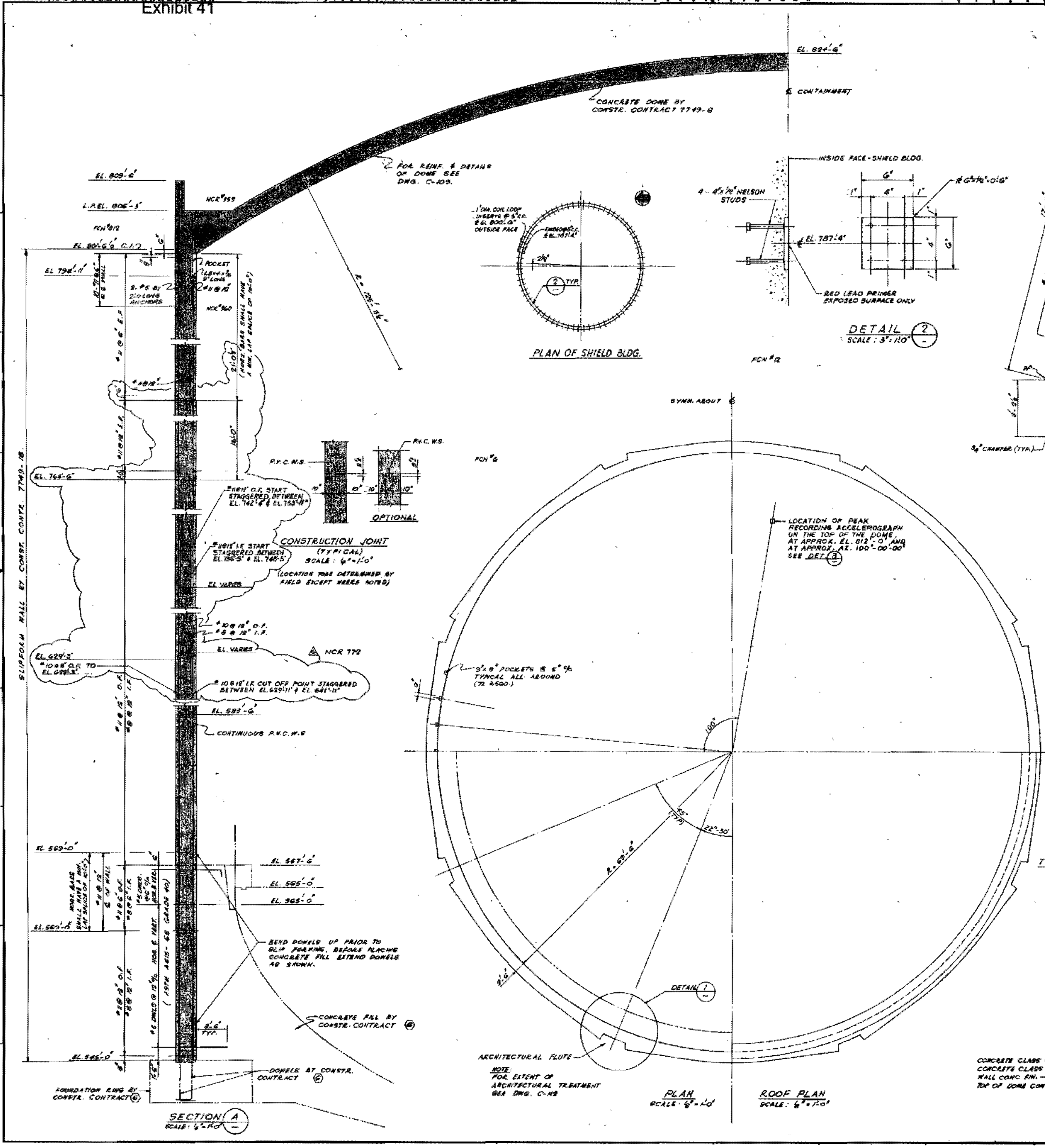
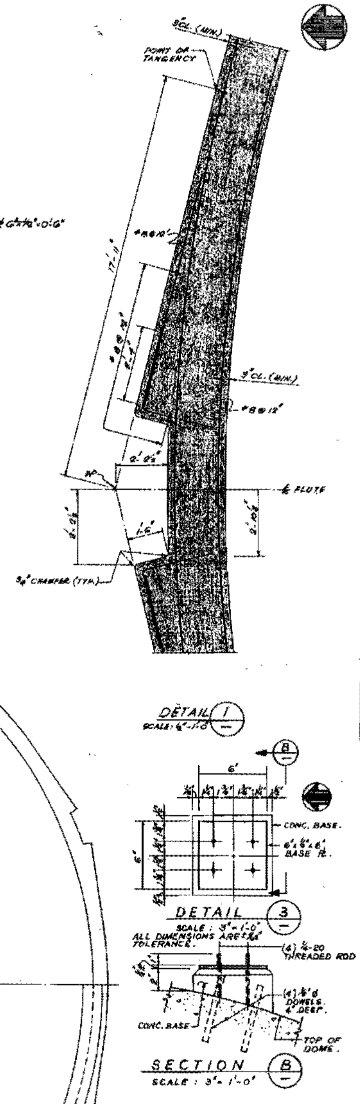

DEVELOPED ELEVATION R = 72'-0"
(LOOKING FROM OUTSIDE)
SECTION A
SCALE: 1/16" = 1'-0"
LEGEND
1 = 6" ⌀ ROOF DRAIN
2 = 8" ⌀ SAND POCKET DRAIN
3 = 30" ⌀ FUEL TRANSFER TUBE
65'-0" RADIUS (INSIDE CONTAINMENT VESSEL)
72'-0" RADIUS (OUTSIDE SHIELD BUILDING)
LEGEND
DENOTES 12" ELECTRICAL PENETRATION.
DENOTES 32" ELECTRICAL PENETRATION.
NOTES:
REFERENCE DRAWINGS
FOR REFERENCE DRAWINGS SEE DRAWING C-110
C-112 SHIELD BUILDING DETAILS SH-1
DETAIL 2
SCALE: 1/2" = 1'-0"
PENETRATION #29.
CUT REINF. AROUND OPENING AS SHOWN
GROUT ALL AROUND SLEEVE AS SHOWN
DETAIL 1
DETAIL 2
DETAIL 3
DETAIL 4
DETAIL 5
DETAIL 6
SECTION B
TYP ELECTRICAL BLOCK-OUT, 15" SQ FILLED WITH GROUT (EXISTING)
6" ⌀ CORE DRILL
OPTIONAL LOCATION
DETECTOR MOUNTING BRACKET
EXISTING 6" ⌀ DRAIN (CUT RE. TO FIT)
NOTE "A"
FOR AE-29, PREHEAT TO 200°F
NOTE "B"
FOR AE-29, INSTALL EXPANSION ANCHORS PARALLEL TO AXIS OF PIPE & PROVIDE BEVELED WASHERS AS REQUIRED.
NOTE "C"
ANCHORS AE-27, AE-29, AE-31 & AE-33 ARE DESIGNED FOR SEISMIC CONDITIONS.
NON "Q"
FOR FCR 86-094
ELEVATION
SECTION F
SECTION H
SECTION G
KEY PLAN
SUPPORT LOCATION & ORIENTATION
DETAIL 3
2 REQ'D
(SEE DWG C-116)
CLASS I STRUCTURE
ISSUED AS BUILT FOR FCR 78-126
ADDED F.C.N. 3544 & "Q"
ADDED F.C.N. 2782.
RELOCATED PENETRATION #29 ADDED DETAIL
(GENERAL REVISION) ISSUED FOR CONSTRUCTION
ISSUED FOR DETAILING & MATERIAL PURCHASE
FOR CLIENT APPROVAL AND BID
FOR CLIENT REVIEW
AS NOTED
BECHTEL COMPANY
GAITHERSBURG, MARYLAND
DAVIS-BESSE NUCLEAR POWER STATION
THE TOLEDO EDISON COMPANY
THE CLEVELAND ELECTRIC ILLUMINATING COMPANY
Q
SHIELD BUILDING
WALL DEVELOPMENT
7749
C-111
11

Exhibit 41: C-0110 – Roof Plan Wall Section Details

Exhibit 41





GENERAL NOTES

1. ORIENTATION 0° AZIMUTH IS EQUAL TO PLANT REFERENCE NORTH.
2. FOR LIGHTNING PROTECTION, ANNULAR SPACE LIGHTING AND OTHER EMBEDDED ITEMS, SEE ELECTRICAL DRAWINGS E-401 AND E-300.
3. CONCRETE FOR SHIELD BUILDING SHALL BE CLASS C-2 WITH A COMPRESSIVE STRENGTH OF 4000 PSI @28 DAYS. MAXIMUM SLUMP SHALL BE 5" 3.5" FOR SLIPFORM WORK. CEMENT SHALL BE ASTM TYPE II BELOW EL.585 AND ASTM TYPE I ABOVE EL.585.
4. GROUT FOR BLOCKOUTS SHALL BE CLASS C-1 WITH A DESIGN COMPRESSIVE STRENGTH OF 4000 PSI @28 DAYS. CEMENT USED WILL BE AS IN GENERAL NOTE 3 ABOVE.
5. ALL REINFORCING STEEL SHALL BE ASTM A-615-68, GRADE 60, DEFORMED BILLET STEEL BARS EXCEPT AS NOTED.
6. ALL SPLICES OF REINFORCEMENT SHALL BE IN ACCORDANCE WITH THE TABLE.
7. ALL REINFORCING SPLICES SHALL BE STAGGERED.
8. NO CONCRETE SHALL BE POURED UNTIL ALL REINFORCING STEEL, EMBEDDED PLATES, PIPES, ELECTRICAL GROUNDS AND CONDUIT AND BLOCKOUTS ARE IN PLACE, INSPECTED AND APPROVED BY THE FIELD ENGINEER.
9. WATERSTOPS AT ALL CLOSED CONSTRUCTION JOINTS (JOINTS WHERE THERE IS NO MOVEMENT) SHALL BE P.V.C., FLAT DUMBELL TYPE SIMILAR TO "GRACE SERVICISED", CODE #5017-C. ALL WATERSTOPS AT EXPANSION JOINTS (WHERE RELATIVE MOVEMENT IS POSSIBLE, e.g. BETWEEN BUILDINGS) SHALL BE RUBBER, CENTER BULB TYPE SIMILAR TO "GRACE SERVICISED", CODE ~~#5005-16~~, EXCEPT THAT ALL RUBBER WATERSTOPS SHALL BE SPLIT TYPE.
10. NO BACKFILL SHALL BE PLACED AGAINST THE STRUCTURE UNTIL CONCRETE HAS OBTAINED SUFFICIENT STRENGTH TO SUPPORT IT.
11. FOR WATERPROOF MEMBRANE SEE CONSTRUCTION DOCUMENT NO. 8.
12. REFERENCE SPECIFICATIONS
    a. ACI 318-63, "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE".
    b. ACI 307-69 "SPECIFICATION FOR DESIGN AND CONSTRUCTION OF REINFORCED CONCRETE CHIMNEYS".
    c. ACI 301, "SPECIFICATION FOR STRUCTURAL CONCRETE FOR BUILDINGS".
    d. ACI MANUAL OF CONCRETE PRACTICE, 1968, PARTS 1, 2, AND 3.
    e. ACI MANUAL OF CONCRETE INSPECTION.
    f. STATE OF OHIO BUILDING CODE.
    g. ASTM DESIGNATION C-150, "STANDARD SPECIFICATION FOR PORTLAND CEMENT".
    h. SPECIFICATION NO. 7749-C-25 CENTRAL CONCRETE MIX PLANT.
    i. SPECIFICATION NO. 7749-C-27, CONCRETE MIXES AND MATERIAL TESTING SERVICES.
    j. SPECIFICATION NO. 7749-C-28, SUPPLY, DELIVERY, AND PLACING OF WATERPROOF MEMBRANES.
13. ALL DOWELS AT CONST. JOINT EL. 801'-6 5/8" SHALL BE GALVANIZED.

SPECIFICATIONS

1. THE WORK DESCRIBED HEREON SHALL BE PERFORMED IN STRICT ACCORDANCE WITH SPECIFICATION NO. 7749-C-38, "SHIELD BUILDING".
2. SPECIFICATION NO. 7749-C-37 "CONTAINMENT VESSEL", SHALL BE REFERRED TO FOR INTERFACE WORK.
3. SPECIFICATION NO. 7749-C-26, "FORMING, PLACING, FINISHING AND CURING OF CONCRETE".
4. SPECIFICATION NO. 7749-C-30, "FURNISHING, DETAILING, FABRICATING AND DELIVERING REINFORCING STEEL".
5. SPECIFICATION NO. 7749-C-47 "FURNISHING, DETAILING, FABRICATING AND DELIVERING OF MISCELLANEOUS METAL".

CONTRACT DRAWINGS (CC.7749-18)

1. C-110 SHIELD BUILDING ROOF PLAN, WALL SECTIONS AND DETAILS.
2. C-111 SHIELD BUILDING WALL DEVELOPMENT.
3. C-112 SHIELD BUILDING DETAILS, SHEET 1.
4. C-113 SHIELD BUILDING DETAILS, SHEET 2.
5. E-401 SHIELD BUILDING LIGHTING AND LIGHTNING PROTECTION.
6. A-20 NORTH ELEVATION
7. A-21 SOUTH ELEVATION.
8. A-22 EAST ELEVATION.
9. A-23 WEST ELEVATION.

REFERENCE DRAWINGS

1. M-101 GENERAL ARRANGEMENT PLAN @ EL 643.
2. M-102 GENERAL ARRANGEMENT PLAN @ EL 623.
3. M-103 GENERAL ARRANGEMENT PLAN @ EL 603.
4. M-104 GENERAL ARRANGEMENT PLAN @ EL 585.
5. M-105 GENERAL ARRANGEMENT PLAN @ EL 565.
6. M-106 GENERAL ARRANGEMENT PLAN @ EL 545.
7. C-100 SHIELD BUILDING FOUNDATION PLAN AND DETAILS, SHEET 1, REV. 2
8. C-101 SHIELD BUILDING FOUNDATION PLAN AND DETAILS, SHEET 2, REV. 2
9. C-104 CONTAINMENT VESSEL PLAN AND SECTION, REV. 1
10. C-105 CONTAINMENT VESSEL WALL DEVELOPMENT, REV. 1
11. C-106 CONTAINMENT VESSEL PENETRATION DETAILS, REV. 1
12. C-107 CONTAINMENT VESSEL CRANEWAY AND PAINTERS TROLLEY, REV. 1
13. C-117 CONTAINMENT VESSEL INTERIOR AND EXTERIOR GROUTING SHEET 1, REV C
14. C-118 CONTAINMENT VESSEL INTERIOR GROUTING, SHEET 2, REV. C
15. C-1 SITE PLAN.

TYPICAL REINFORCING SPLICES

f'_c = 4,000 OR 5,000 psi
f_y = 60,000 psi (U.S.D.)

BAR #	TOP BARS L IN.	TOP BARS L DIA.	OTHERS L IN.	OTHERS L DIA.
3	17	46	17	45
4	22	44	22	44
5	27	43	27	41
6	32	43	32	41
7	43	45	38	45
8	57	57	43	49
9	72	64	51	45
10	92	71	65	51
11	112	79	79	56

CLASS I STRUCTURE

CONCRETE CLASS C-2 (SLIP-FORM) FOR CONSTR. CONT. 7749-18
CONCRETE CLASS C-2 FOR CONSTR. CONTRACT 7749-8
WALL CONC. FIN. — SLIPFORM
TOP OF DOME CONC. FIN. — (LATER)

NO.	DATE	REVISIONS	BY
6	6-3-74	REV'D DWG. PER NCR 772	
5	7-11-73	ADDED PEAK RECORDING ACCELEROGRAPH & DETAILS	LM
4	2-23-73	ADDED FCN #6, #12, NCR #358 & RE-SEAL ADD NCR #360, FCN #12	DL
3	4-4-72	REVISED AS NOTED	
2	12-3-71	REVISED AS NOTED	
1		REVISED AS NOTED ISSUED FOR CONSTRUCTION	
0		REDRAWN, ISSUED FOR DETAILING & MATERIAL PURCHASE	

SCALE AS NOTED DESIGNED N.P. CHAO DRAWN

BECHTEL COMPANY
GAITHERSBURG, MARYLAND

DAVIS-BESSE NUCLEAR POWER STATION Q
THE TOLEDO EDISON COMPANY
THE CLEVELAND ELECTRIC ILLUMINATING COMPANY

SHIELD BUILDING
ROOF PLAN WALL SECTION & DETAILS

JOB NO.	DRAWING NO.	REV.
7749	C-110	6

Exhibit 42: Entire Structure, 11-23

Exhibit 42

Figure 1: Impulse Response Average Mobility Values Through 22 November 2011
Shield Building Exterior Elevation

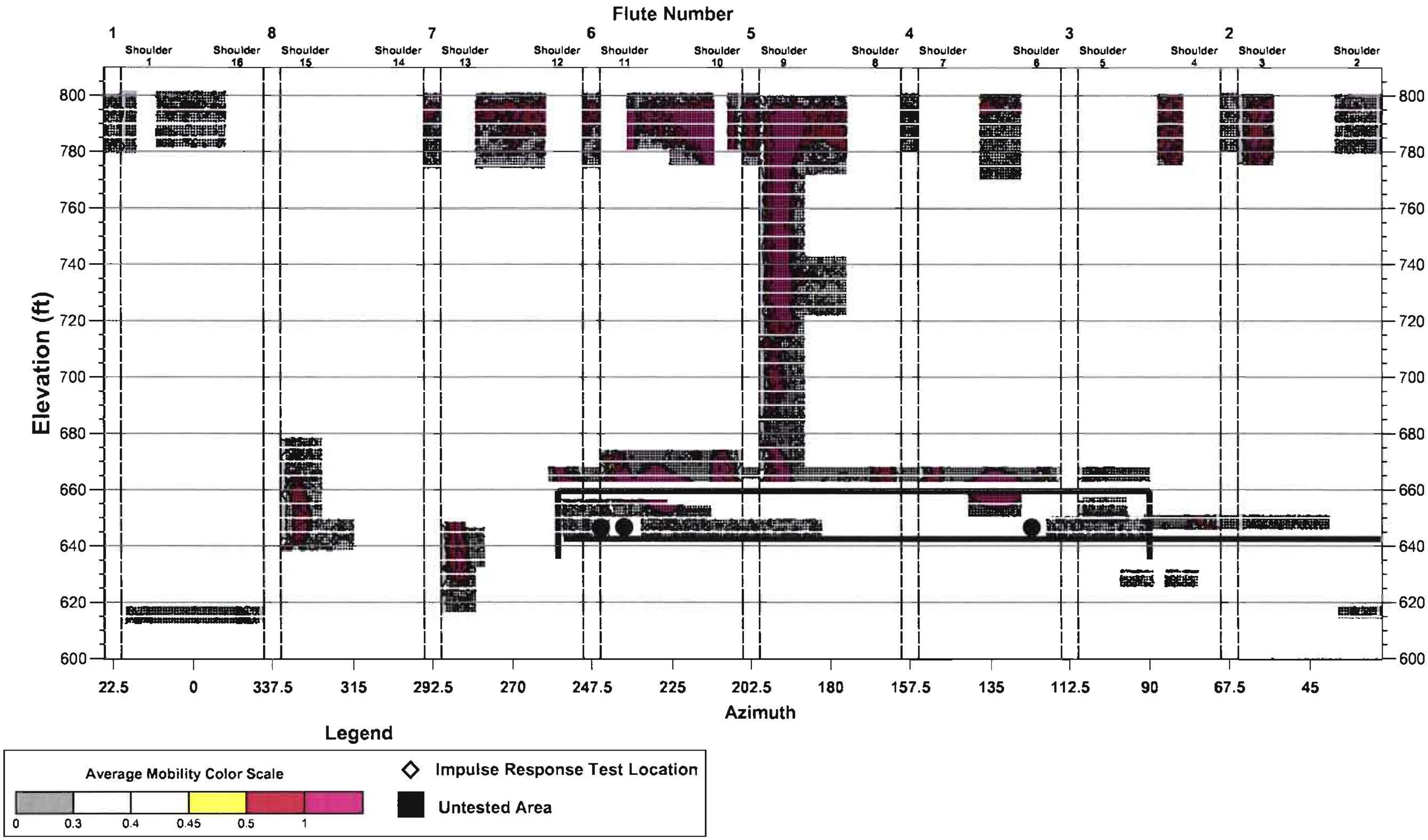


Exhibit 42

CTLGroup
CONSTRUCTION TECHNOLOGY LABORATORIES
ENGINEERS & CONSTRUCTION TECHNOLOGY CONSULTANTS

5400 OLD ORCHARD ROAD
SKOKIE, ILLINOIS 60077-1030
PHONE: 847-965-7500
FAX: 847-965-6541
www.CTLGroup.com

Davis-Besse Nuclear Power Station

Oak Harbor, OH

ISSUE

No.	Description	Date
1	Revised Test Area 28	23NOV11

CTLGroup No.: **262600**
Drawn: **AMS/SEH**
Checked: **CAO/ECD**
Date: **22NOV11**
Scale: **NTS**

Impulse Response Mobility Plot

Figure No.:

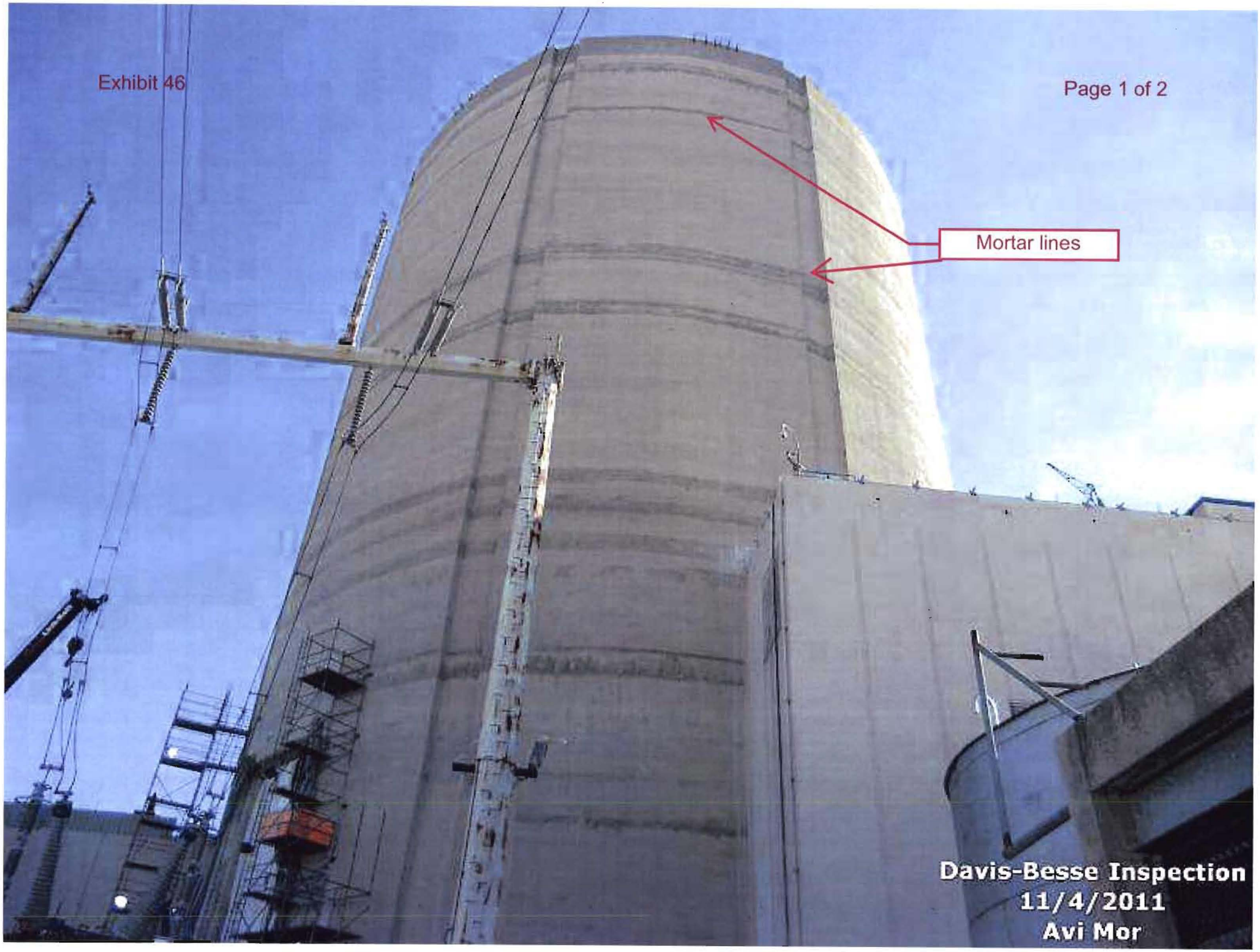
1

Exhibit 43: Calc. C-CSS-099.20-054 R1
(FENOC Document – Not Included as
an Attachment to this Report)

Exhibit 44: Drawing C-0111a
(FENOC Document – Not Included as
an Attachment to this Report)

Exhibit 45: C-0100 Shield Building Foundation
(FENOC Document – Not Included as
an Attachment to this Report)

Exhibit 46: Shield Building Inspection Overview


Mortar lines
Davis-Besse Inspection
11/4/2011
Avi Mor

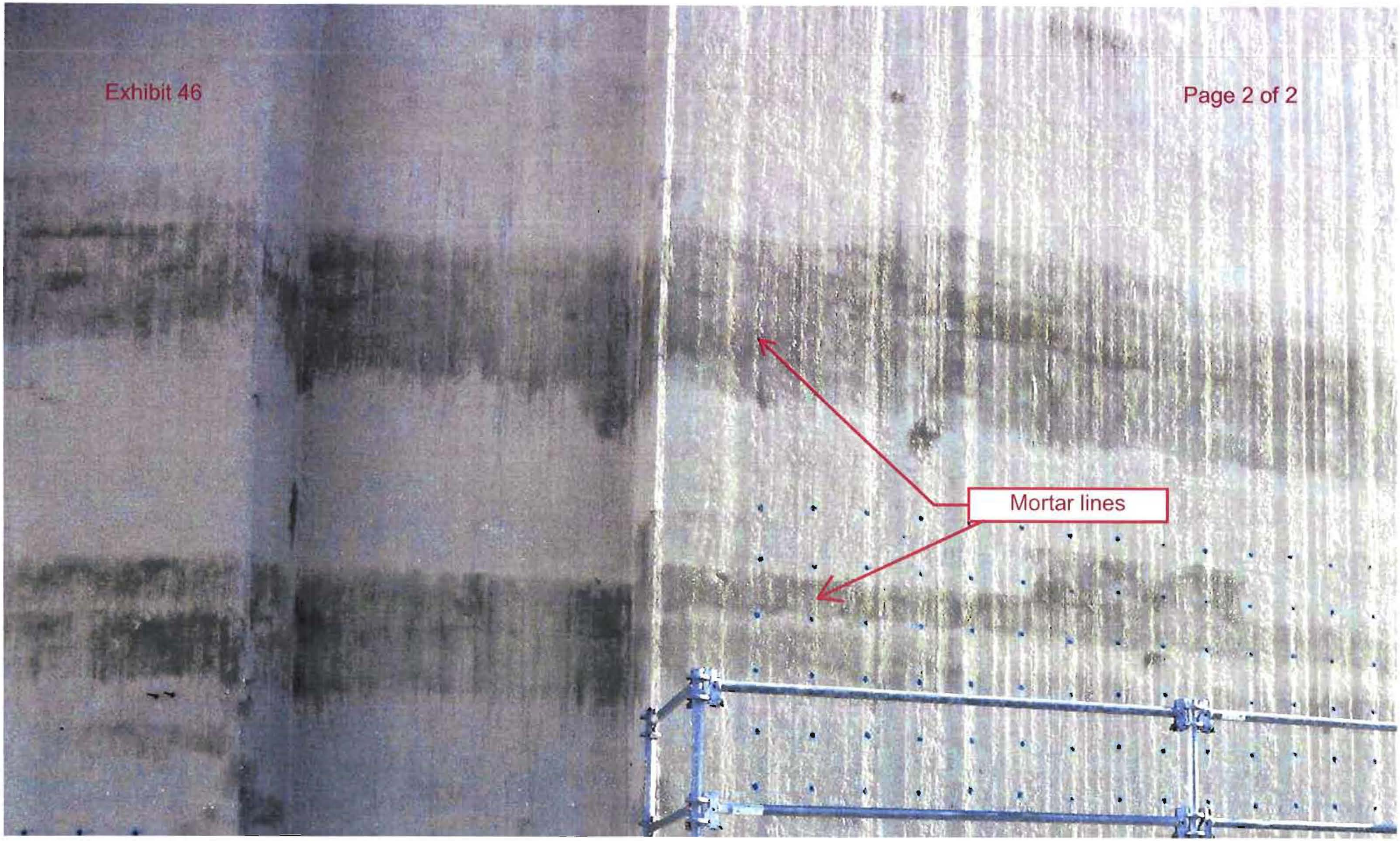


Exhibit 46
Page 2 of 2
Mortar lines

Exhibit 47: E-0401 Lighting & Lightning
(FENOC Document – Not Included as
an Attachment to this Report)

Exhibit 48: 1995 – 0395 Lightning
(FENOC Document – Not Included as
an Attachment to this Report)

Exhibit 49: Corrosion Related Photos (FENOC Document – Not Included as an Attachment to this Report)

Exhibit 50: Calc. CSS 090.022-056 Rev 02
(FENOC Document – Not Included as
an Attachment to this Report)

Exhibit 51: Freezing and Rebar Spacing Study

Freezing Failure Mode & Rebar Density Study Modeling Results (Appendix)

Performance Improvement International LLC

Feb 13, 2012

RESULTS

0.6% VF

RESULTS

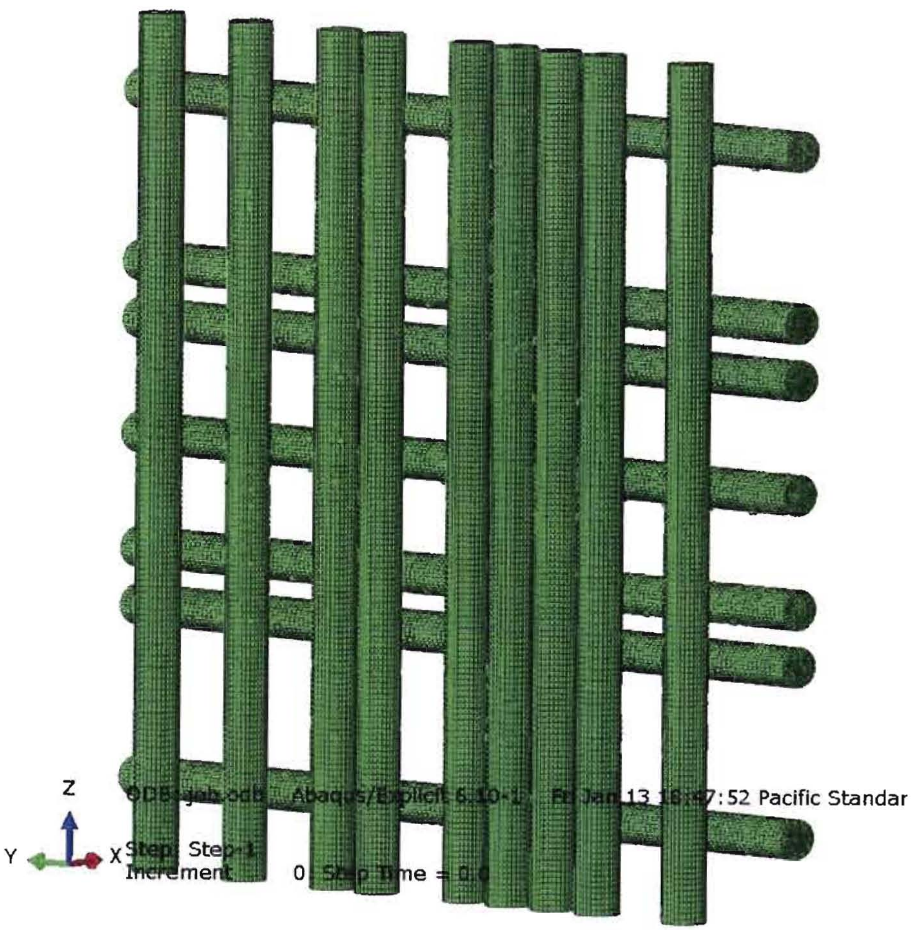
Freezing Results

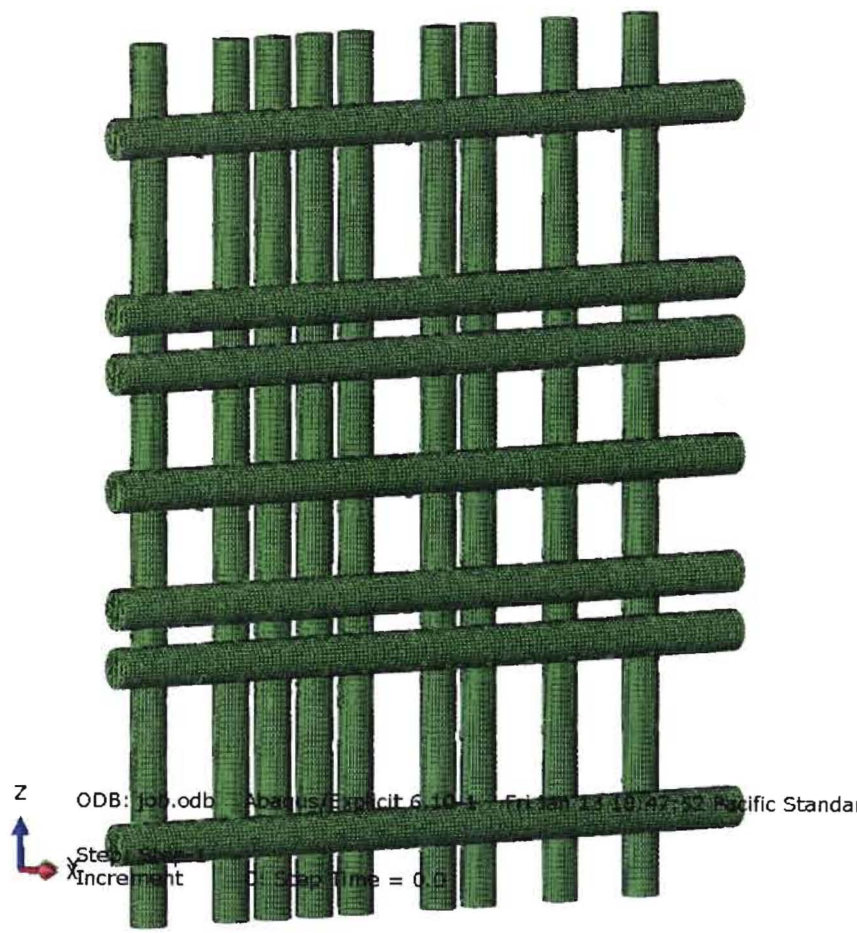
DENSE REBAR, 0.6% VF

Dense Rebar, 0.6% VF

- This section of results are from a model with the following parameters:
- 0.6% VF i.e. 0.6% *of the elements in the first 0.1" under the horizontal rebars (bottom 180°)* are given a 7% expansion to simulate ice freezing.
- Dense rebar
  - Assumes some sub-6" spacing (2" to 6" centers)
  - Based on the photo earlier in these slides

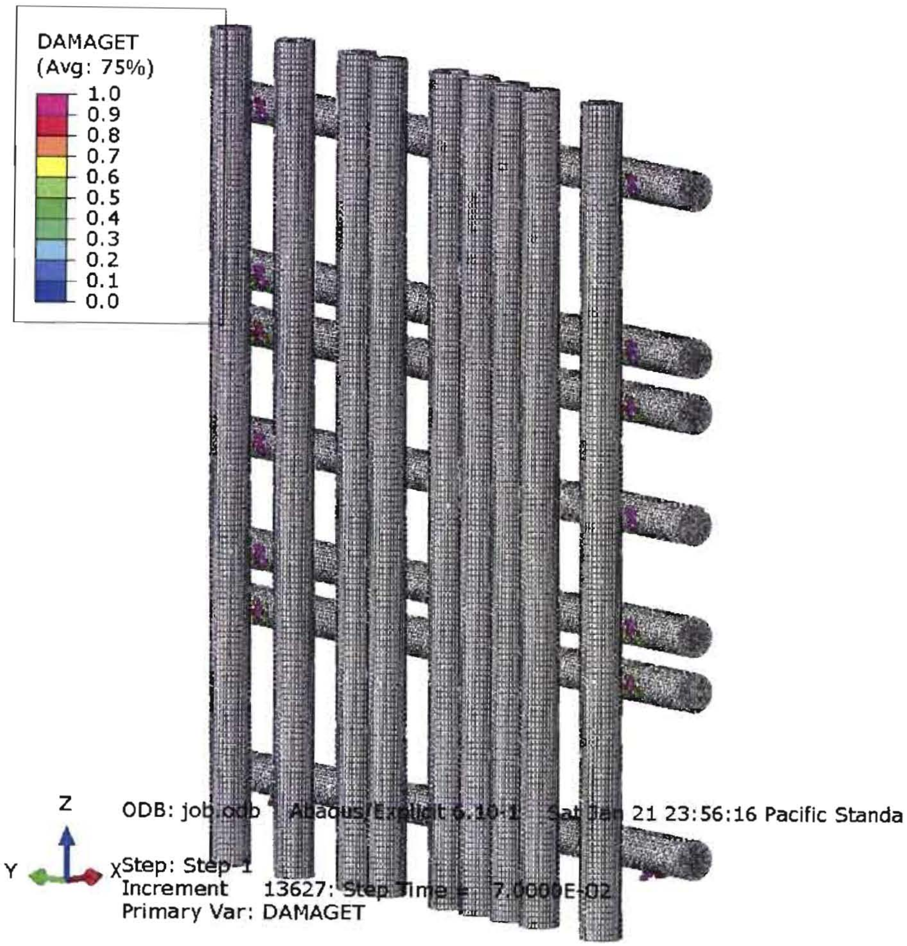
Mesh: Dense Rebar

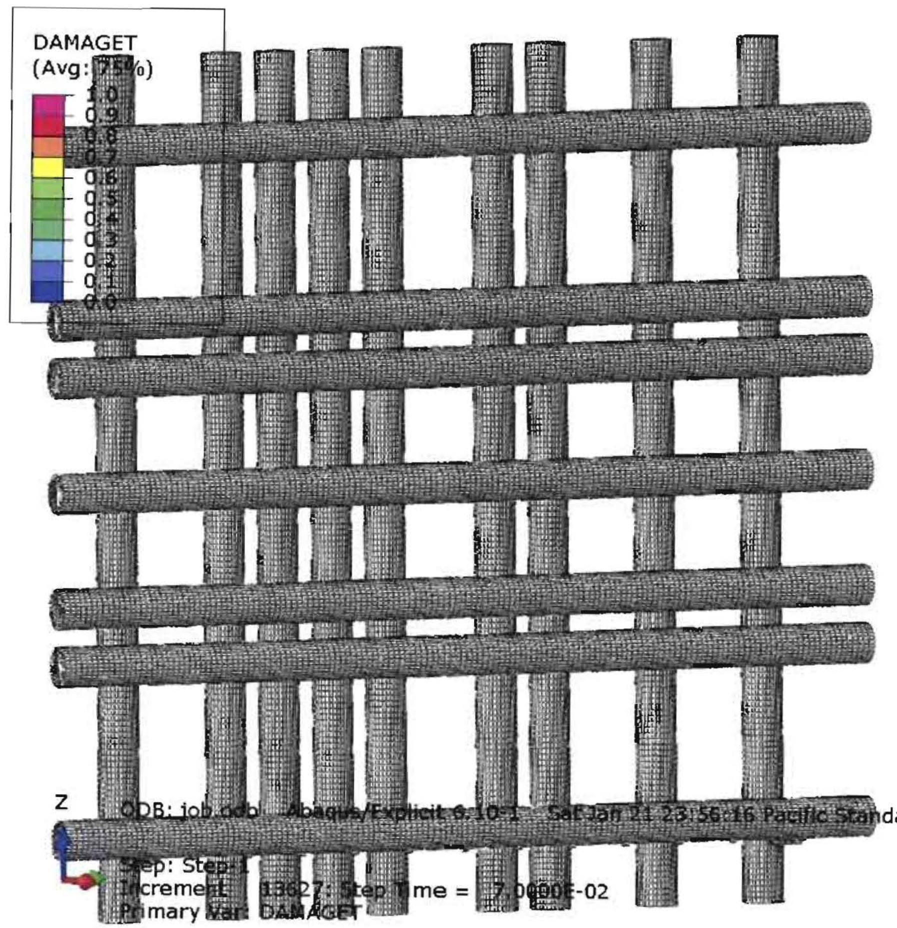




Outer 2” Frozen (7% Expansion)

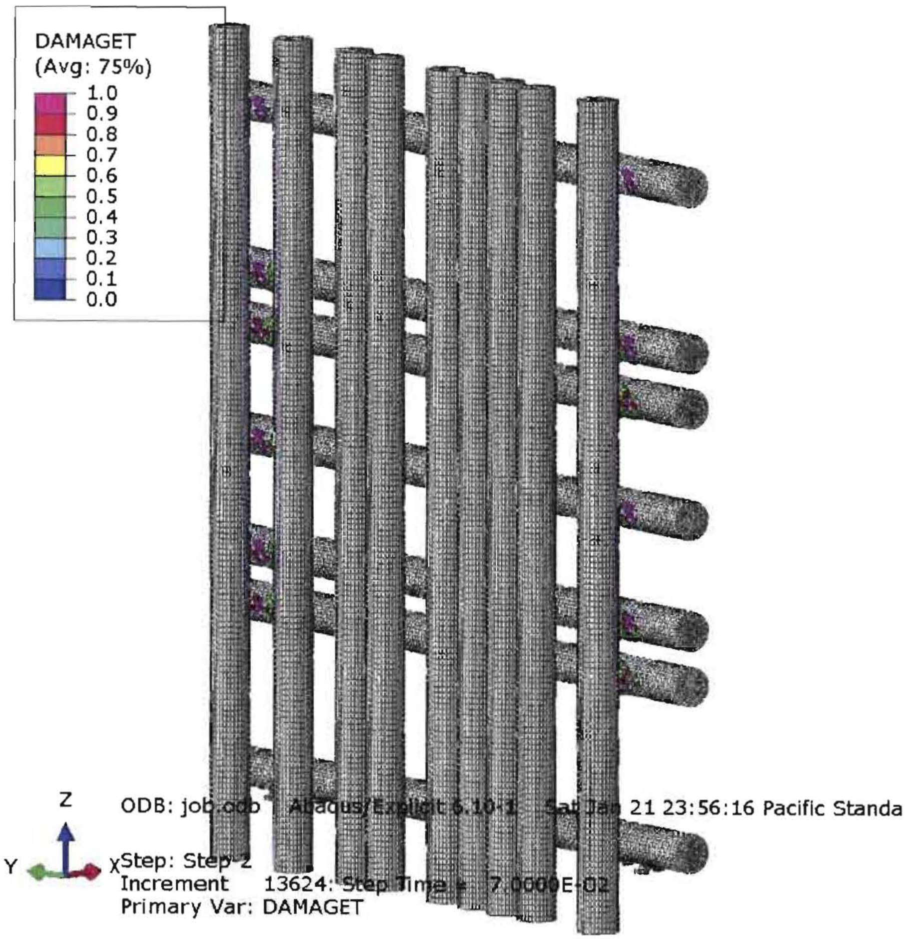
Dense Rebar, 0.6% VF

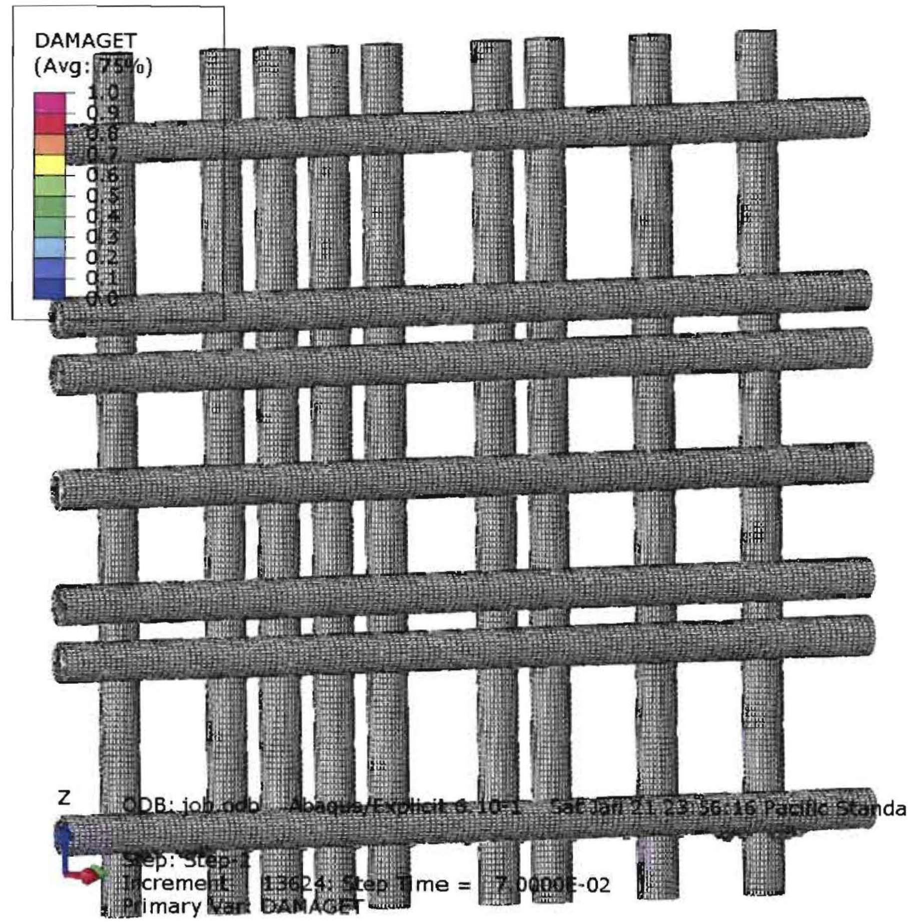




Outer 4” Frozen

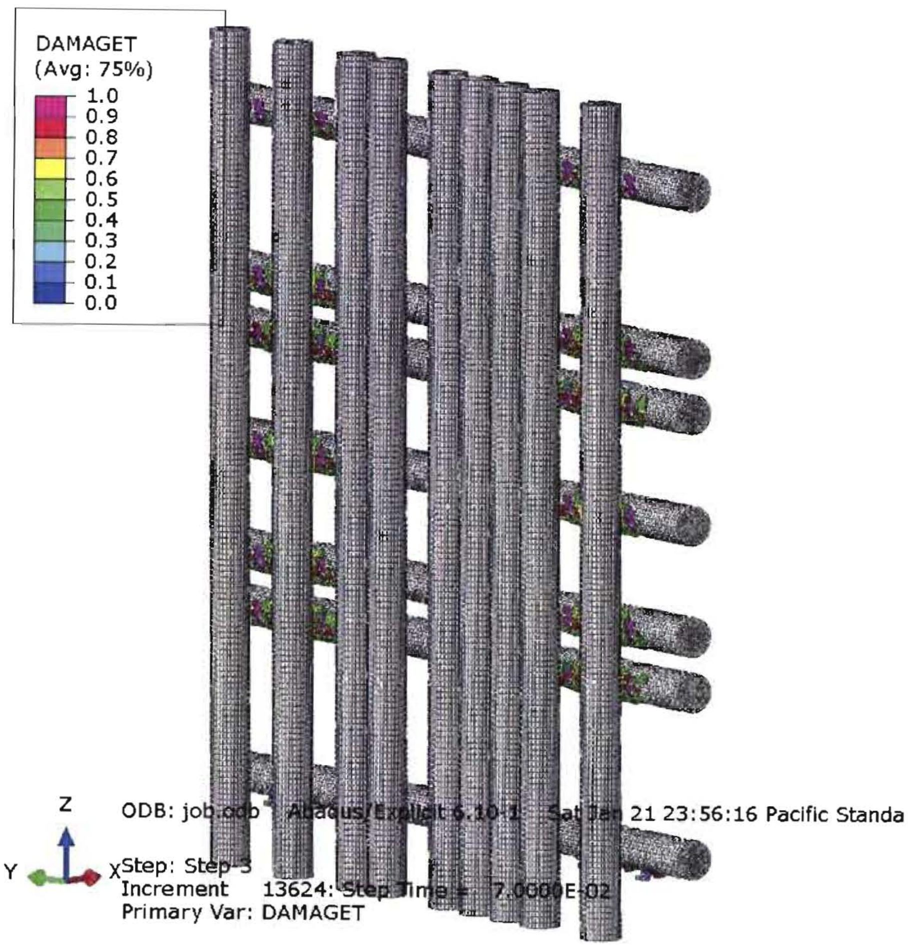
Dense Rebar, 0.6% VF

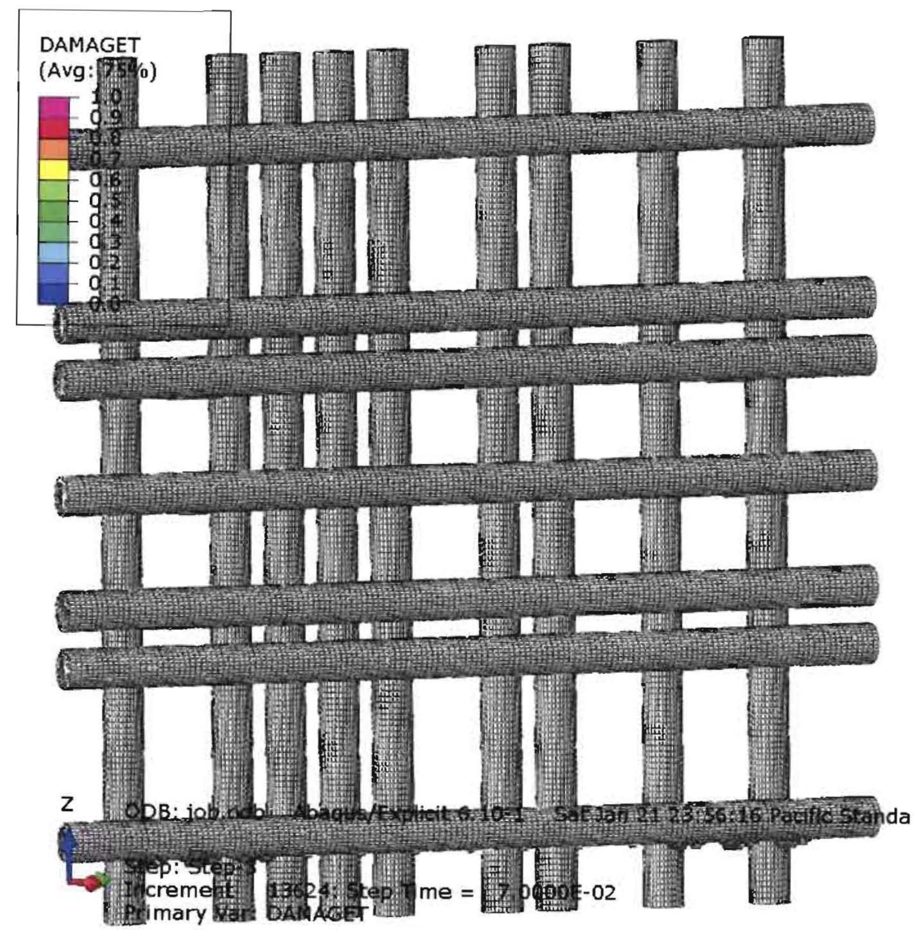




Outer 6" Frozen

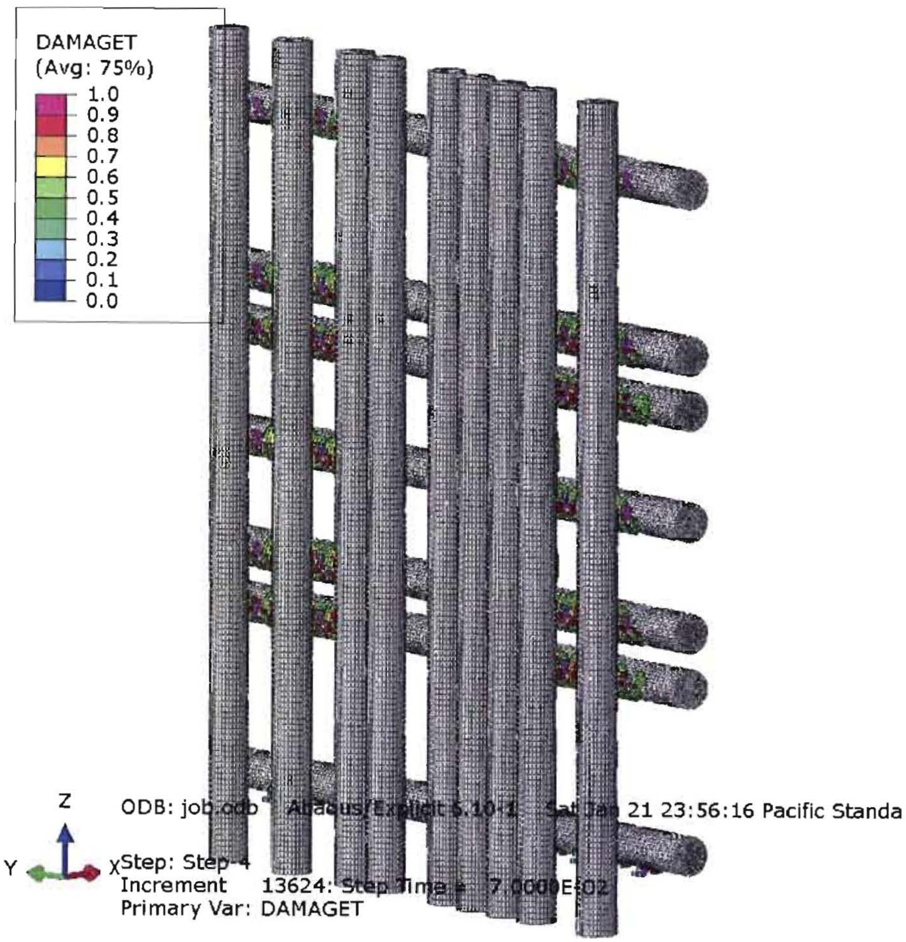
Dense Rebar, 0.6% VF

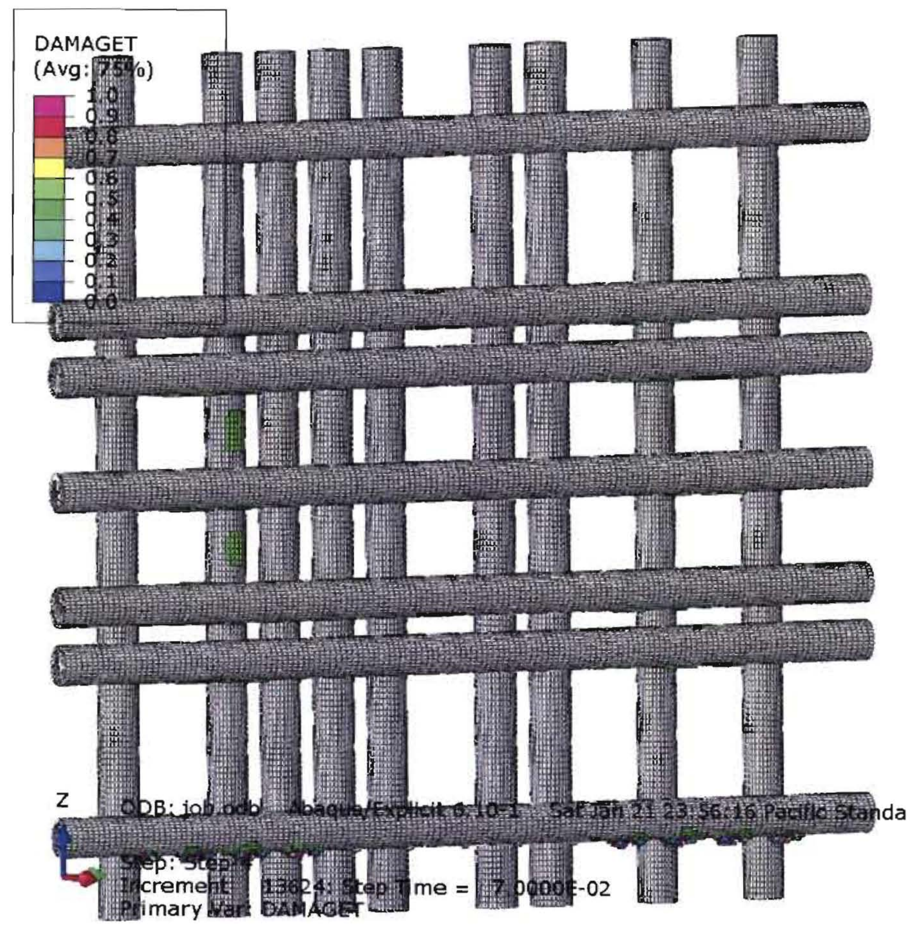




Outer 8" Frozen

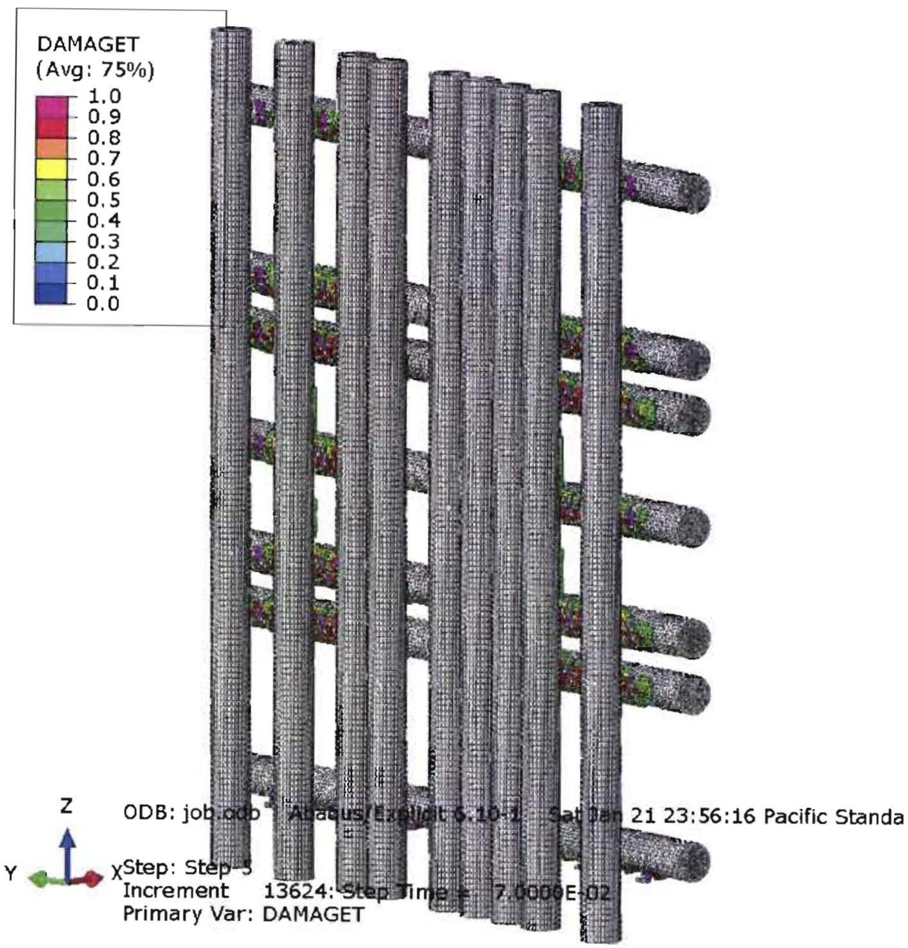
Dense Rebar, 0.6% VF

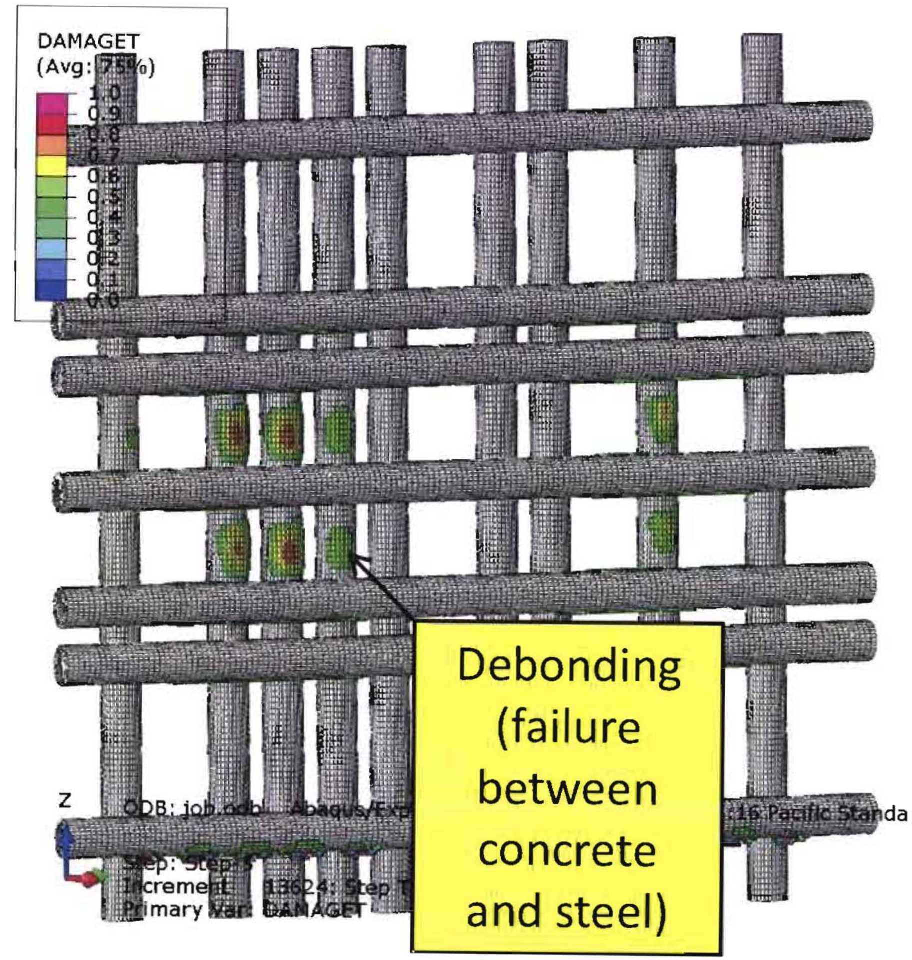




Outer 10" Frozen

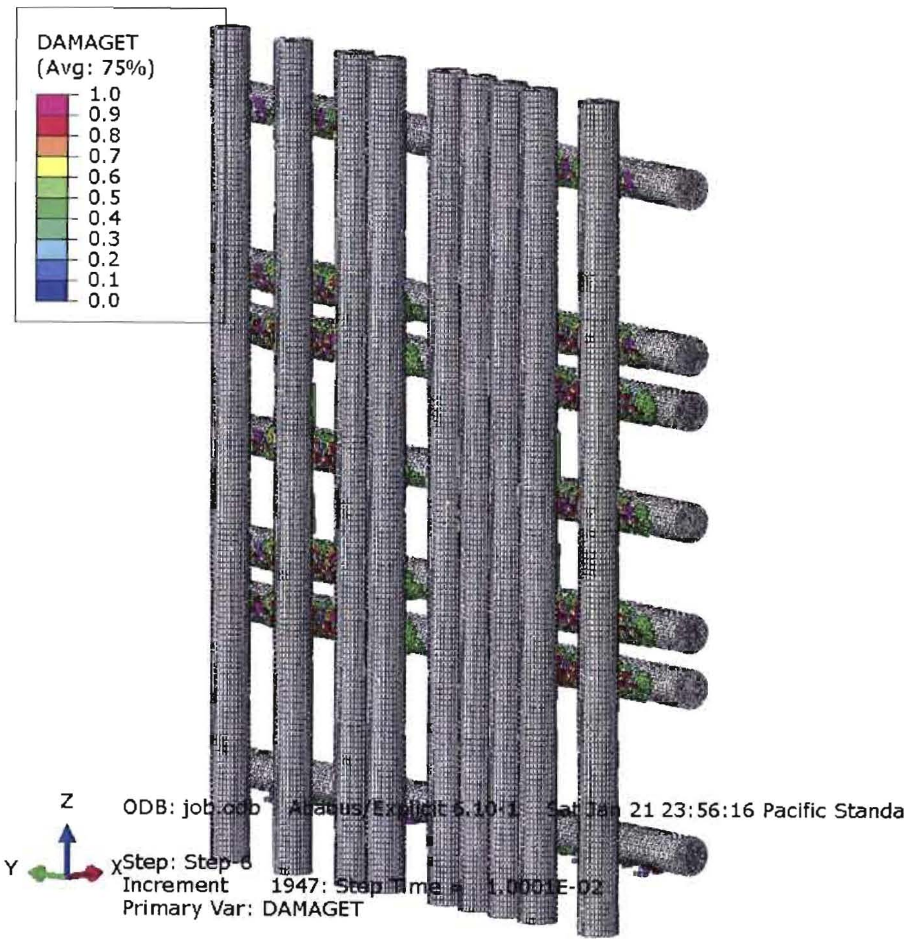
Dense Rebar, 0.6% VF

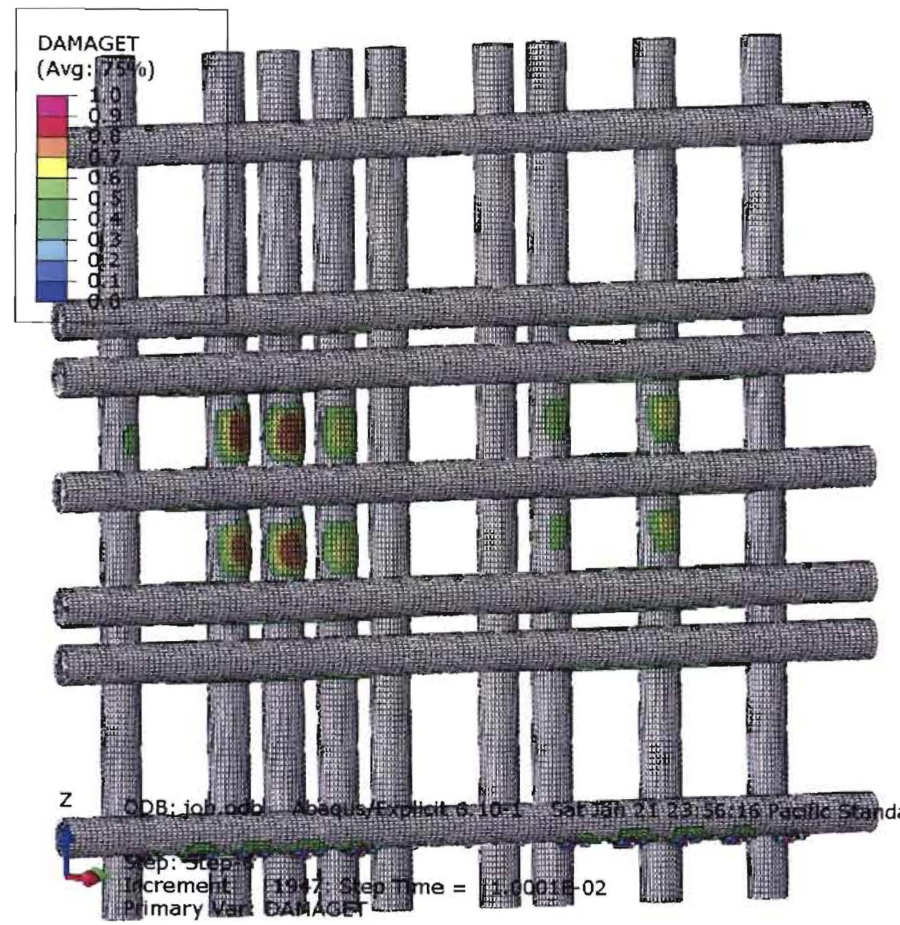




Inner 6" @ 1% Expansion

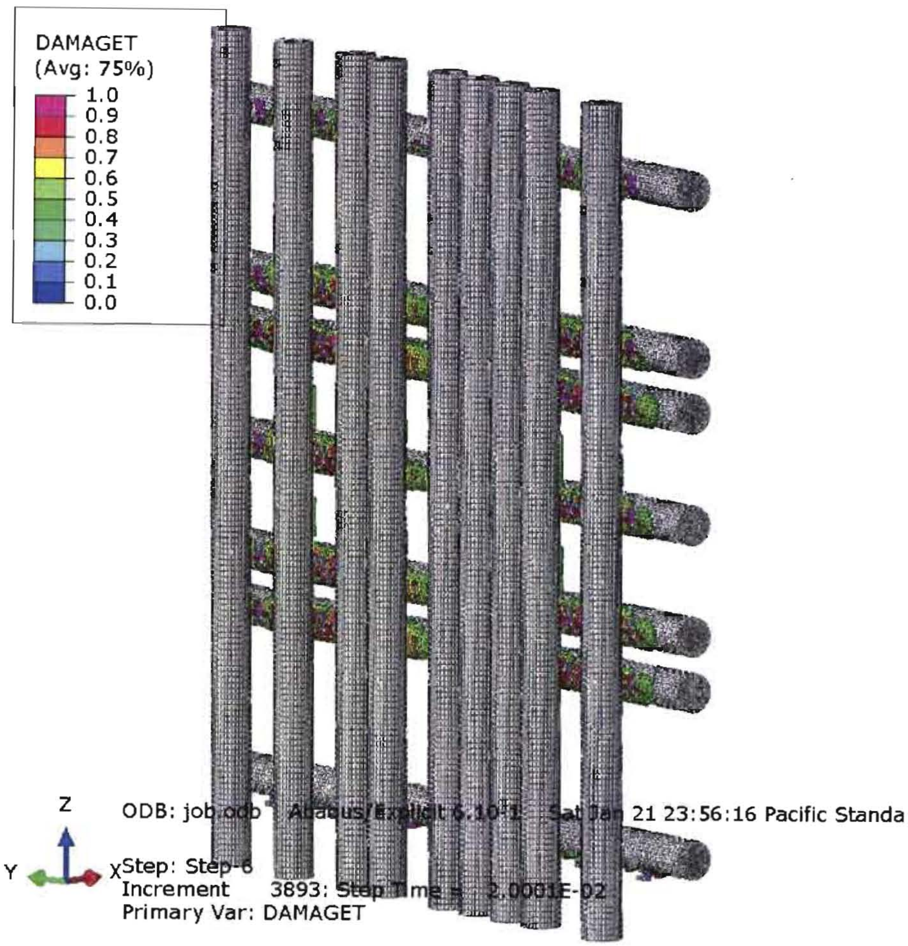
Dense Rebar, 0.6% VF

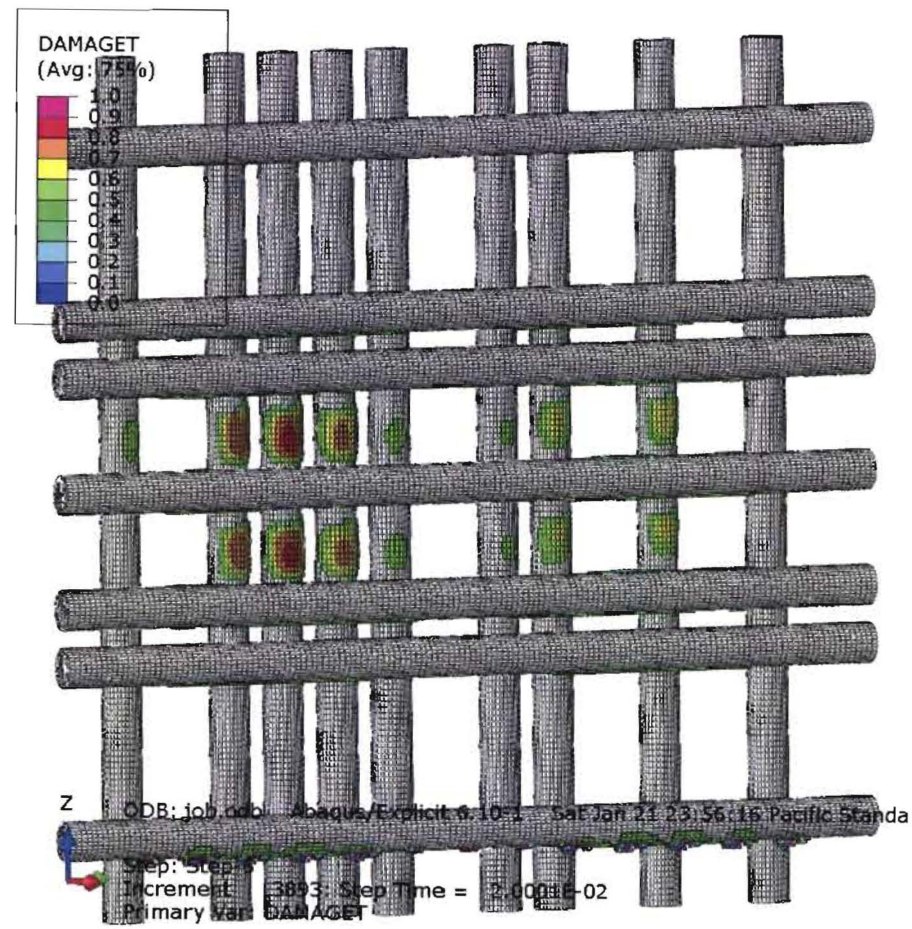




Inner 6” @ 2% Expansion

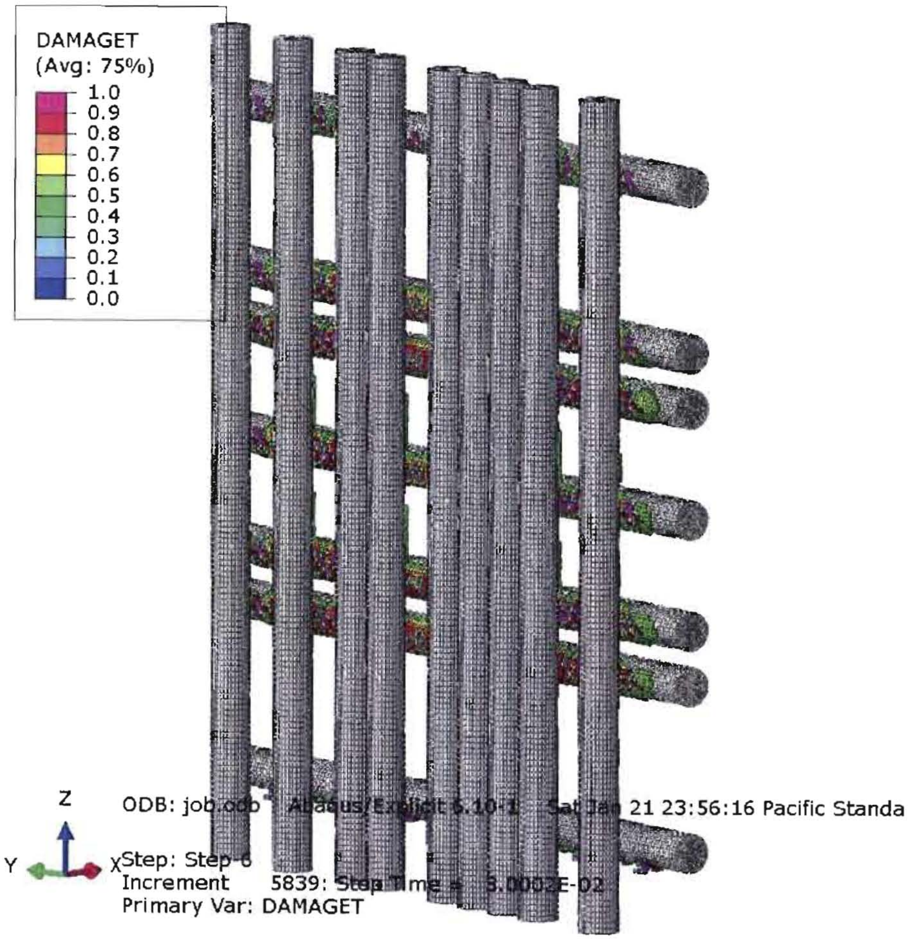
Dense Rebar, 0.6% VF

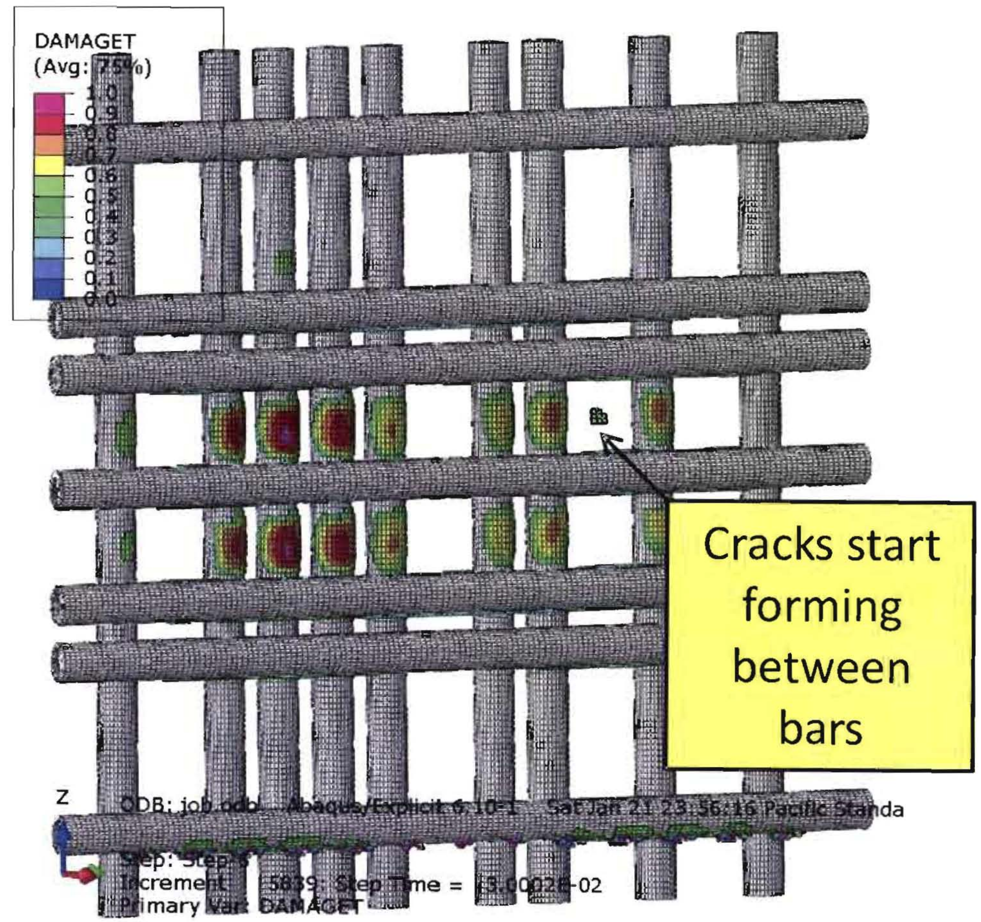




Inner 6" @ 3% Expansion

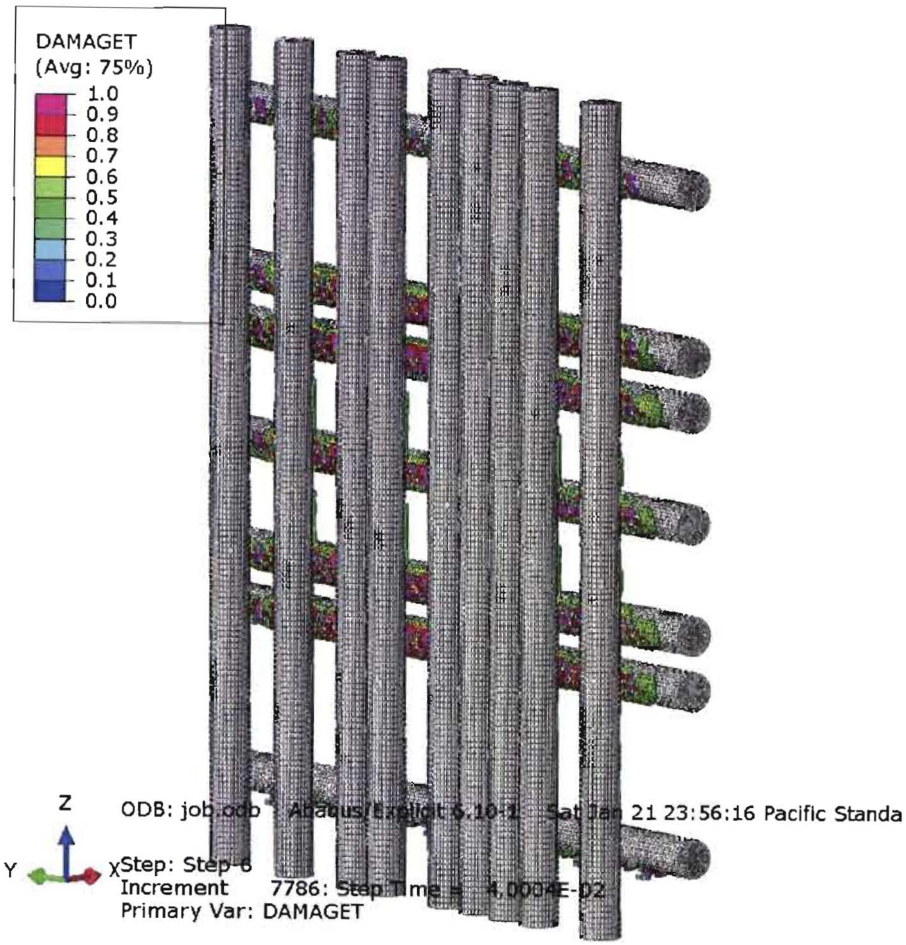
Dense Rebar, 0.6% VF

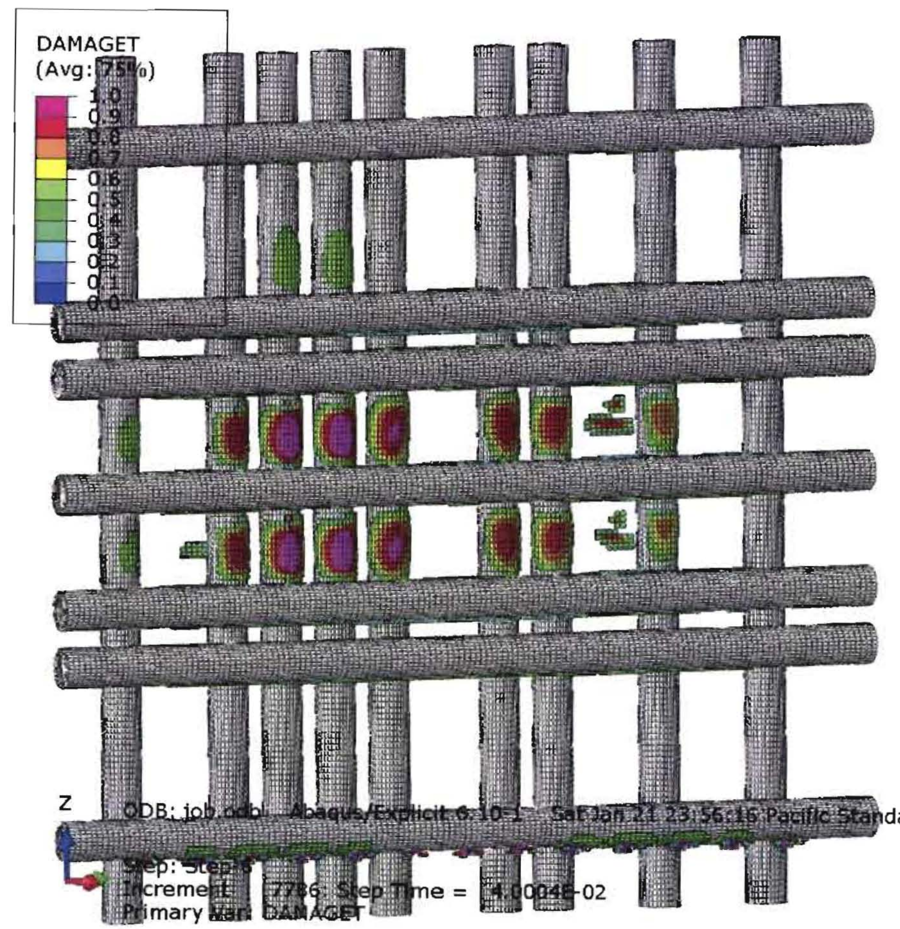




Inner 6” @ 4% Expansion

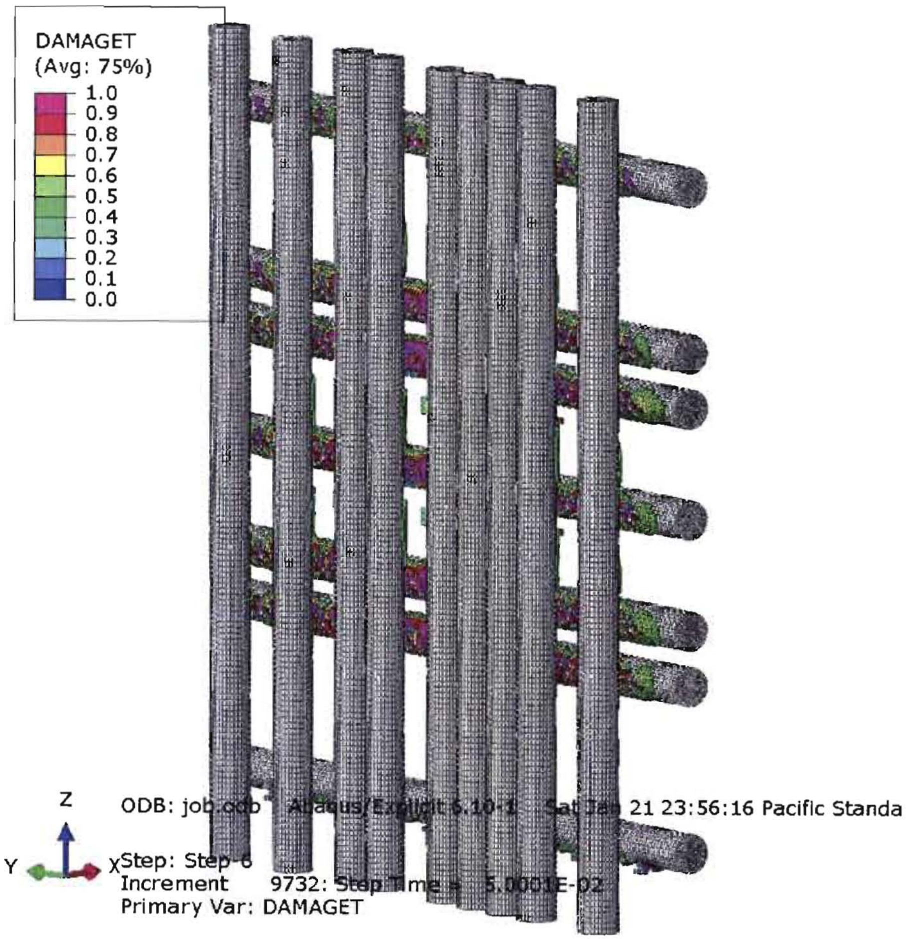
Dense Rebar, 0.6% VF

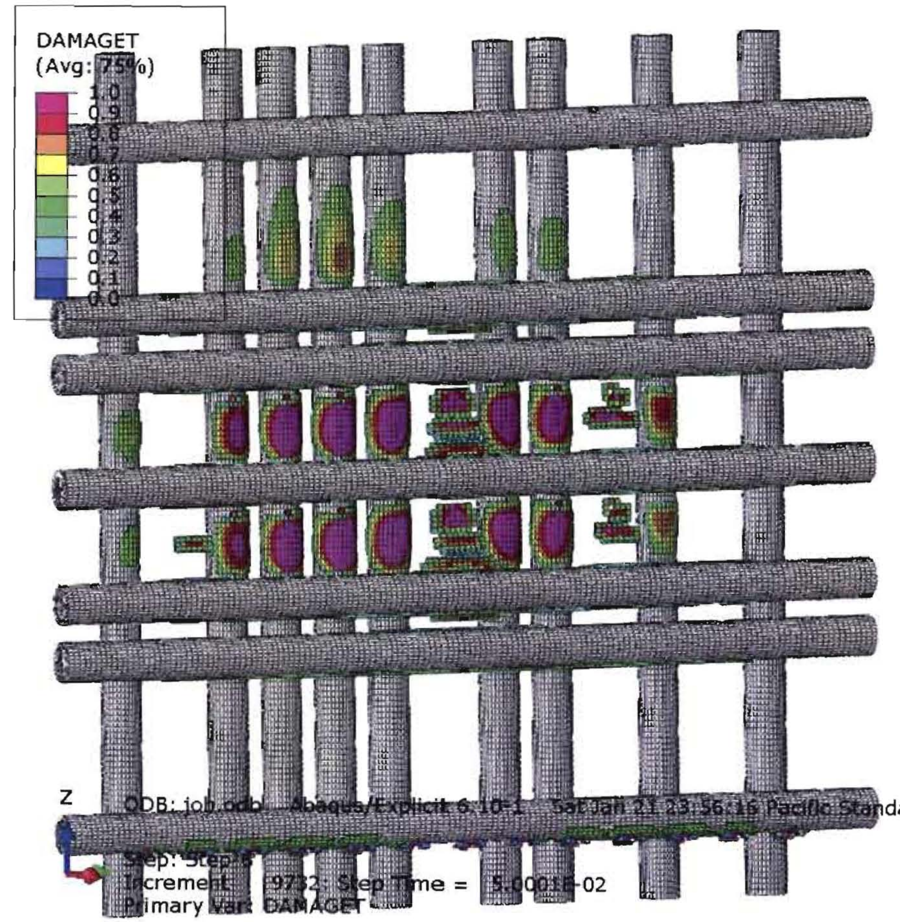




Inner 6” @ 5% Expansion

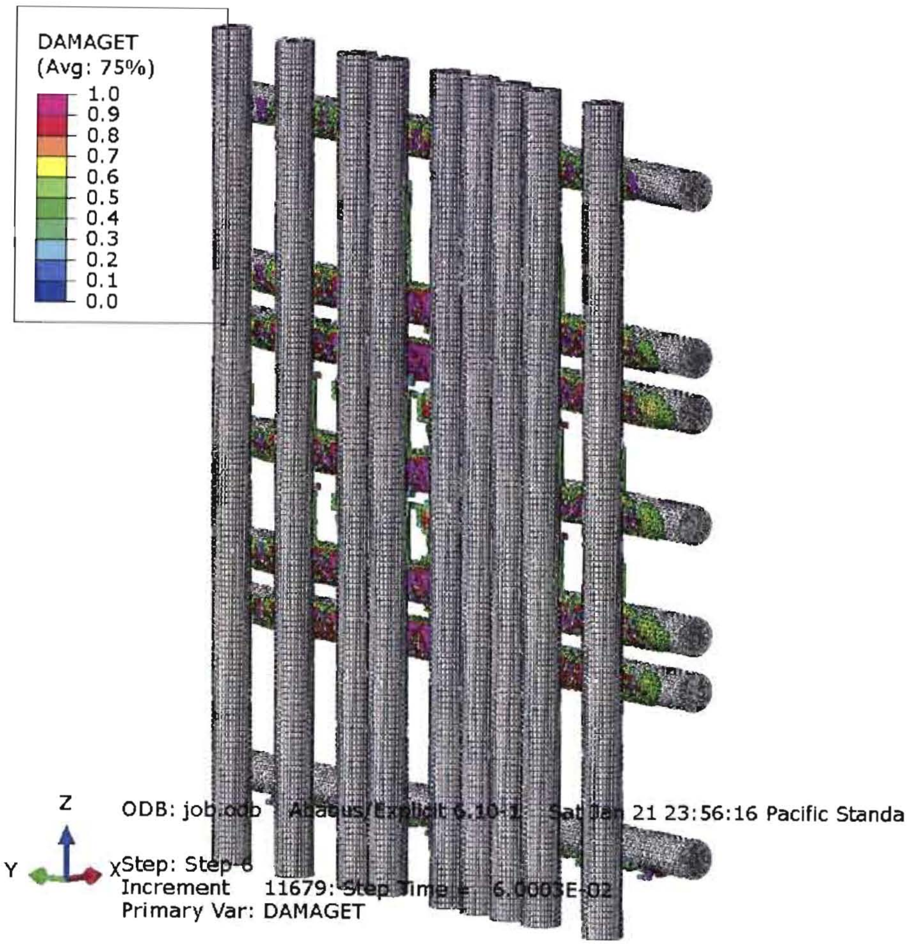
Dense Rebar, 0.6% VF

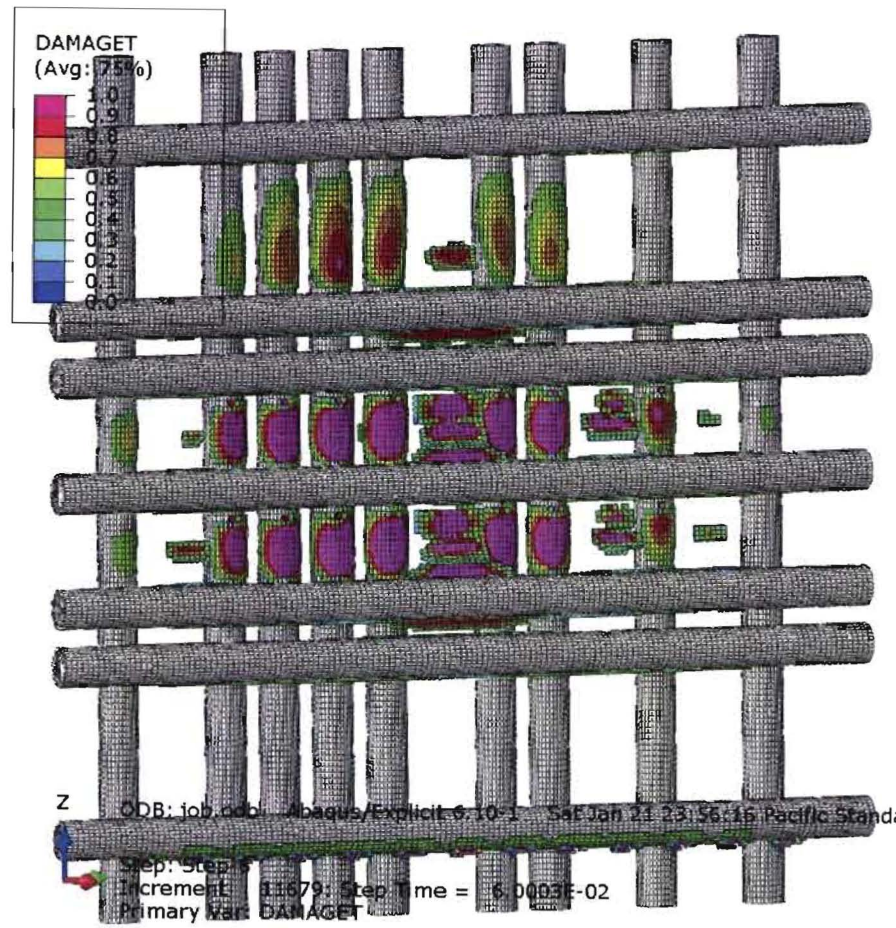




Inner 6” @ 6% Expansion

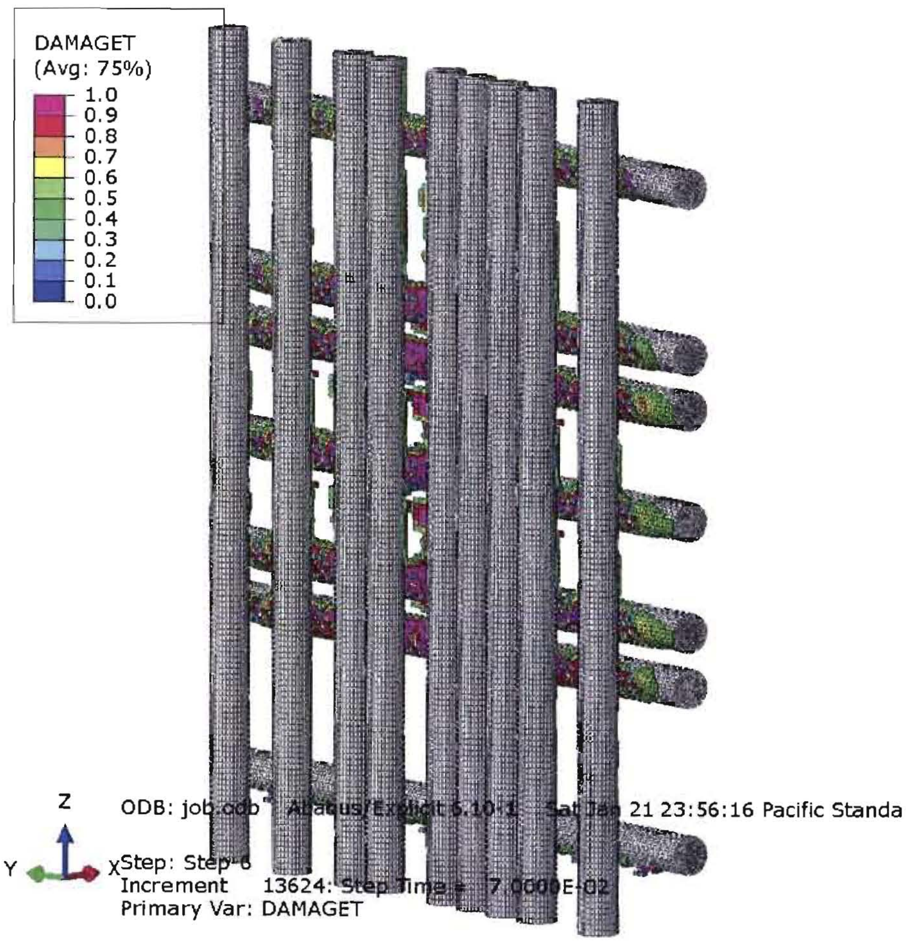
Dense Rebar, 0.6% VF

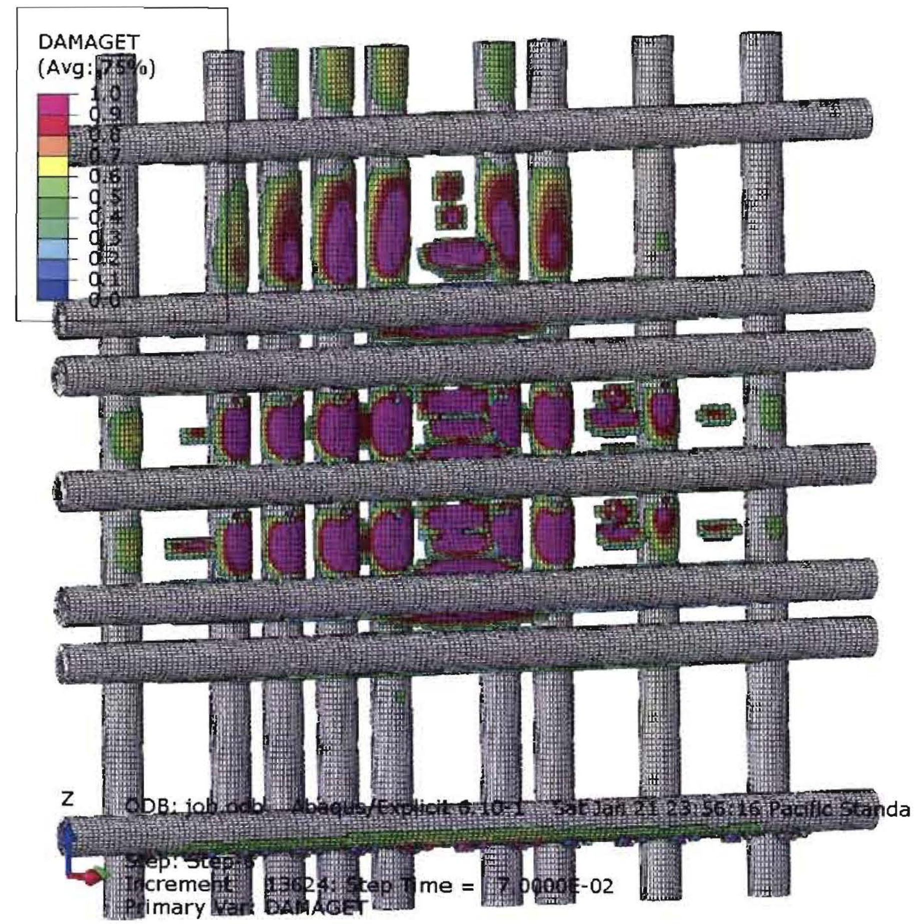




All Frozen (7% Expansion)

Dense Rebar, 0.6% VF





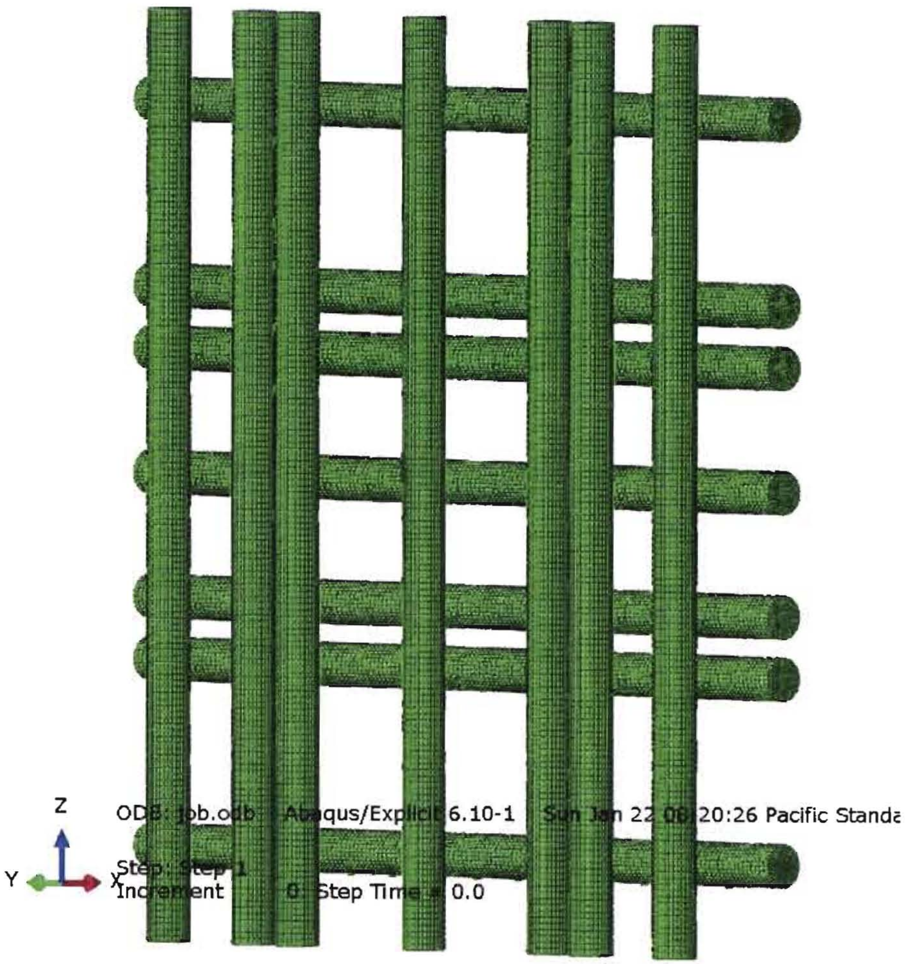
Freezing Results

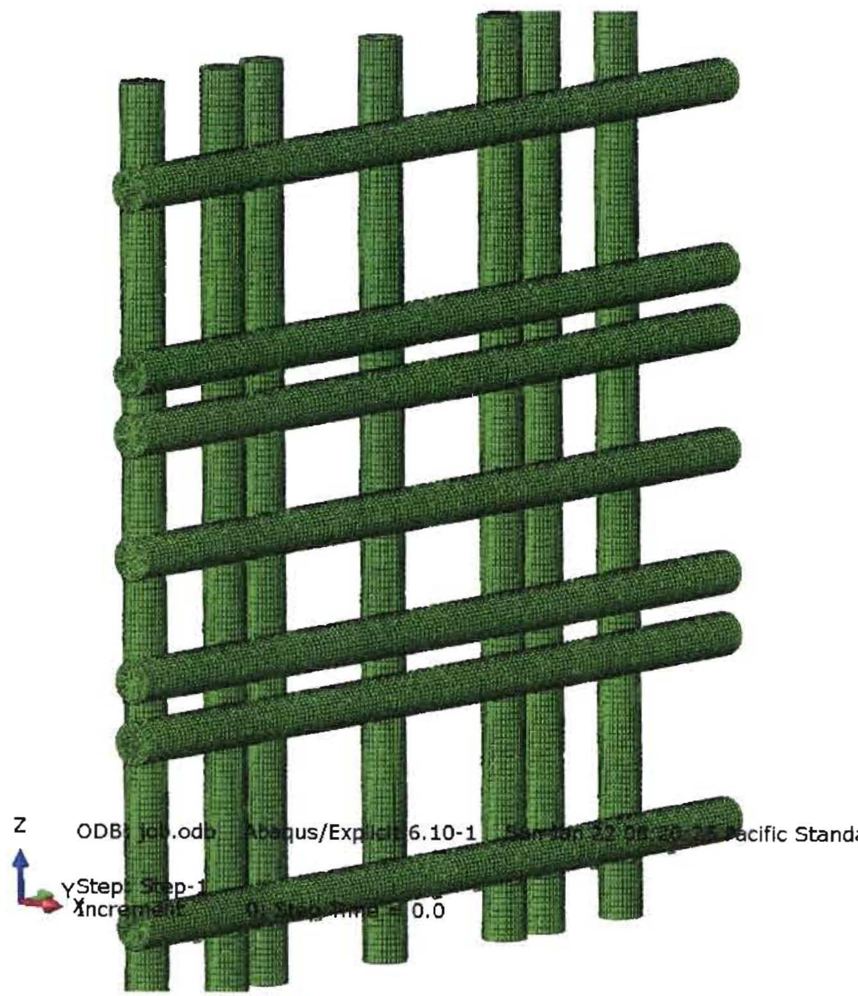
NOMINAL 6” REBAR, 0.6% VF

Nominal 6” Rebar, 0.6% VF

- This section of results are from a model with the following parameters:
- 0.6% VF i.e. 0.6% *of the elements in the first 0.1” under the horizontal rebars (bottom 180°)* are given a 7% expansion to simulate ice freezing.
- Nominal 6” spaced rebar
  - Assumes 6” spacing and includes lap regions

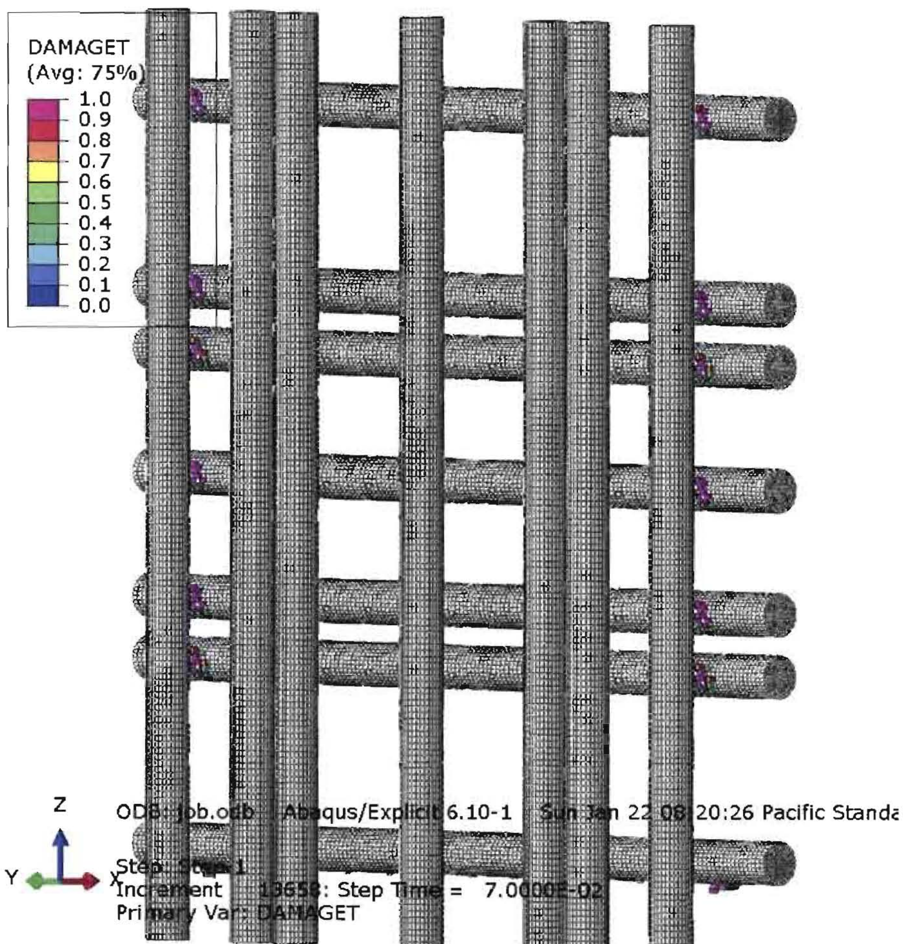
Mesh: Nominal 6" Rebar

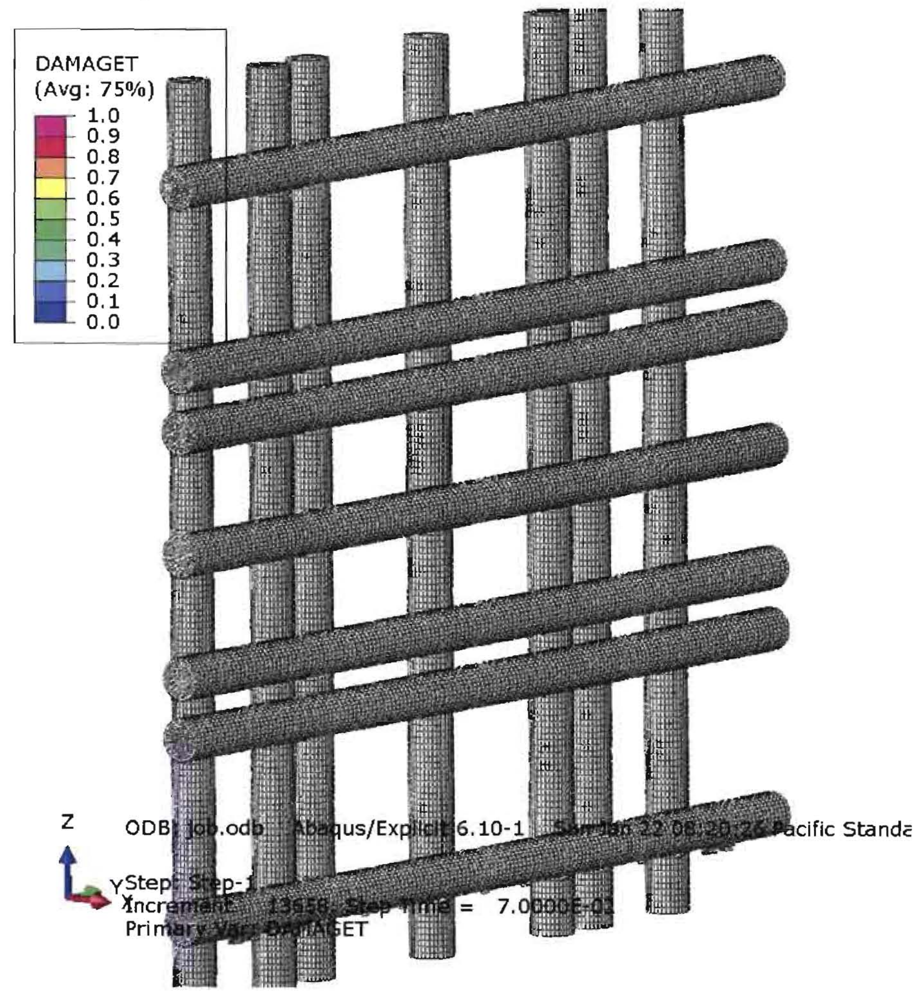




Outer 2" Frozen (7% Expansion)

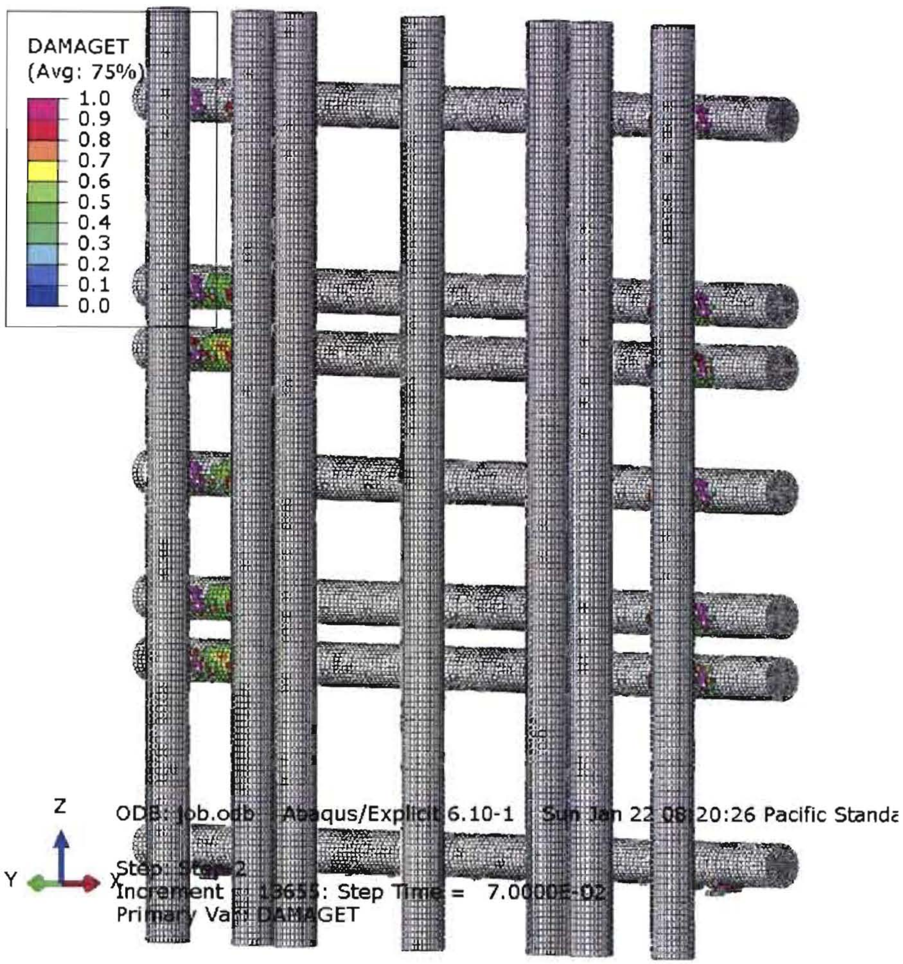
Nominal 6" Rebar, 0.6% VF

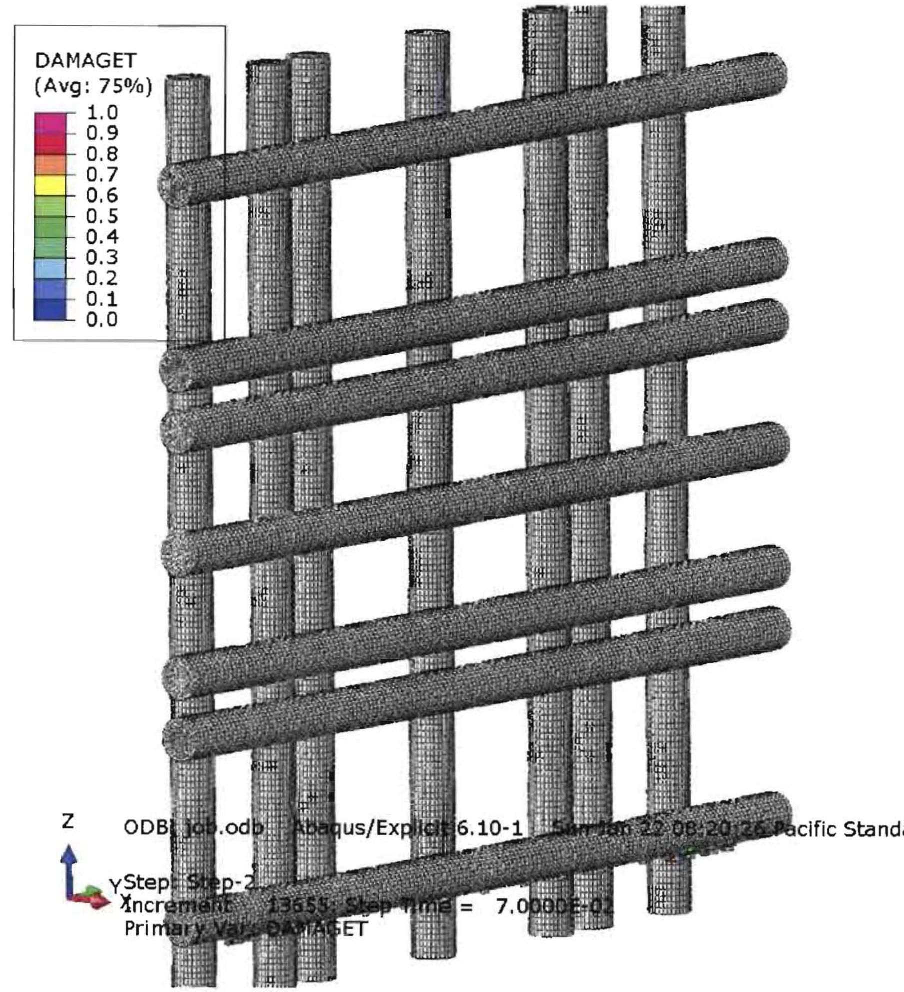




Outer 4" Frozen

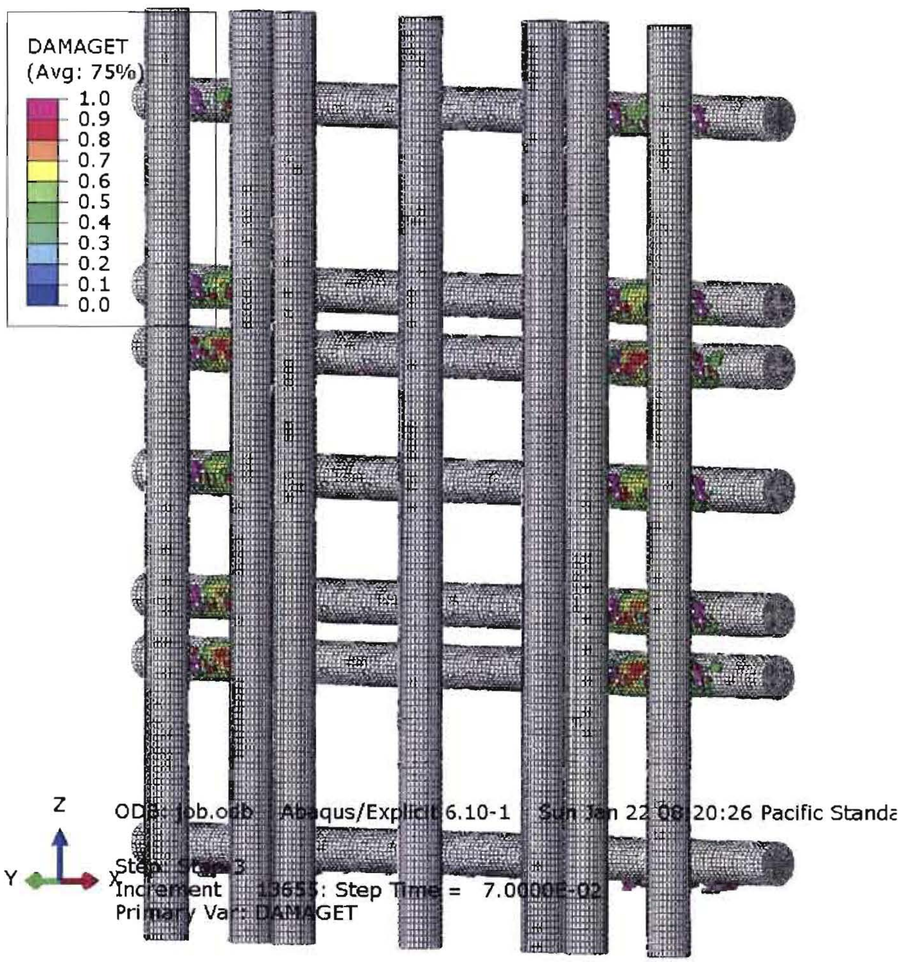
Nominal 6" Rebar, 0.6% VF

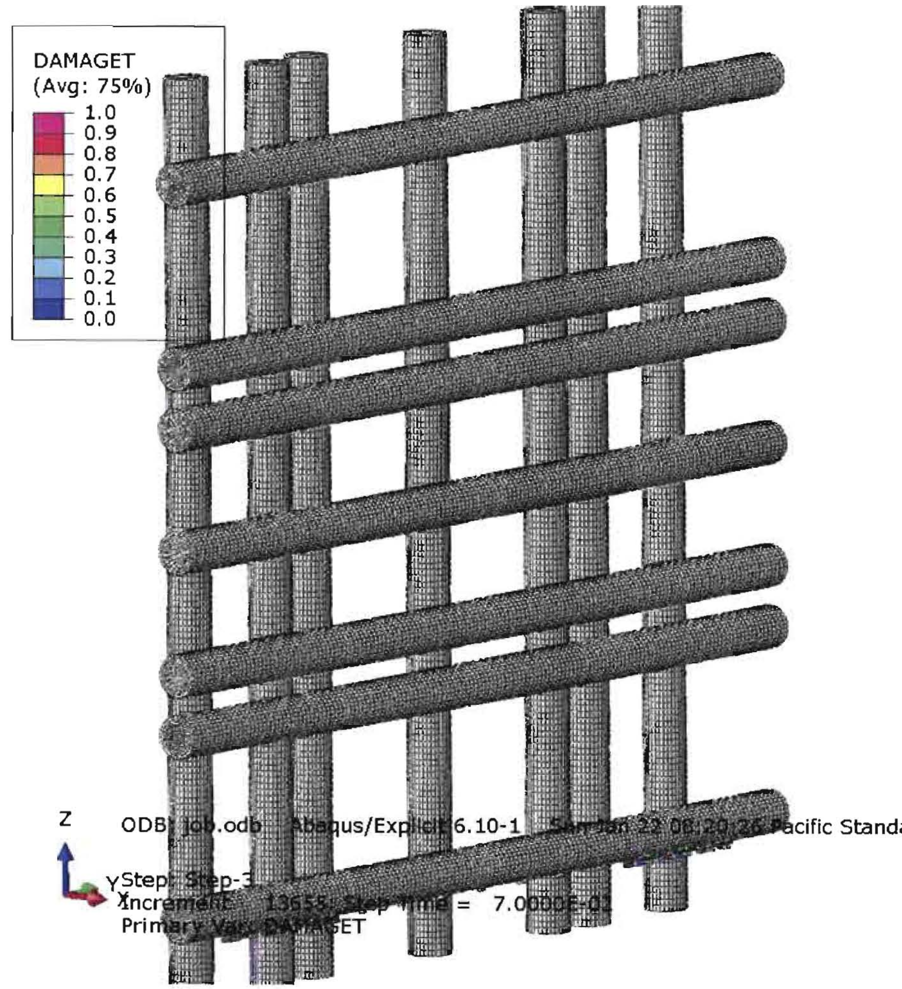




Outer 6" Frozen

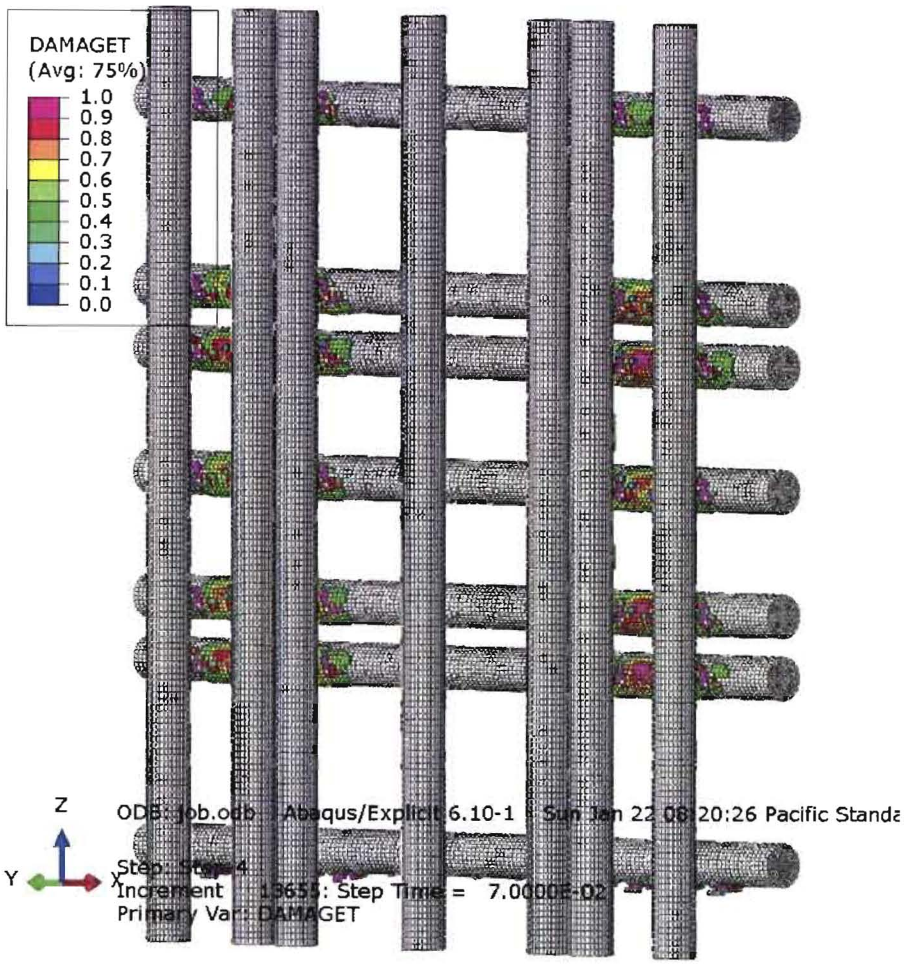
Nominal 6" Rebar, 0.6% VF

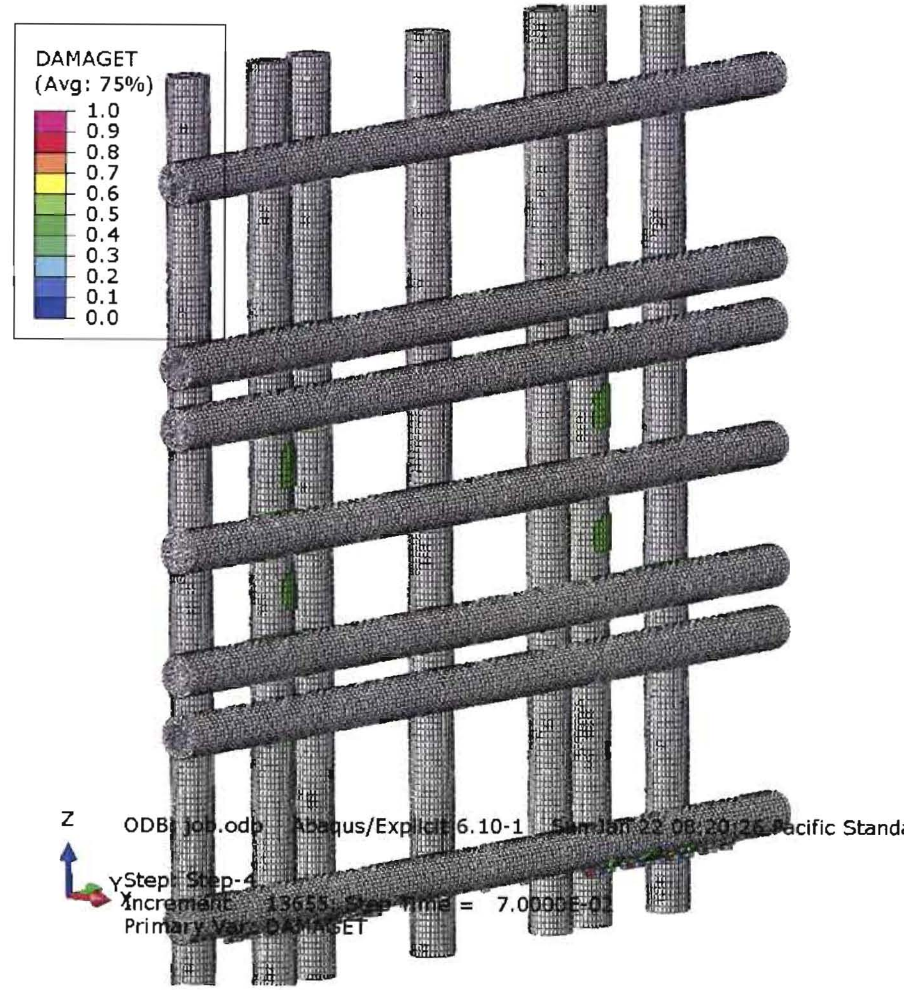




Outer 8” Frozen

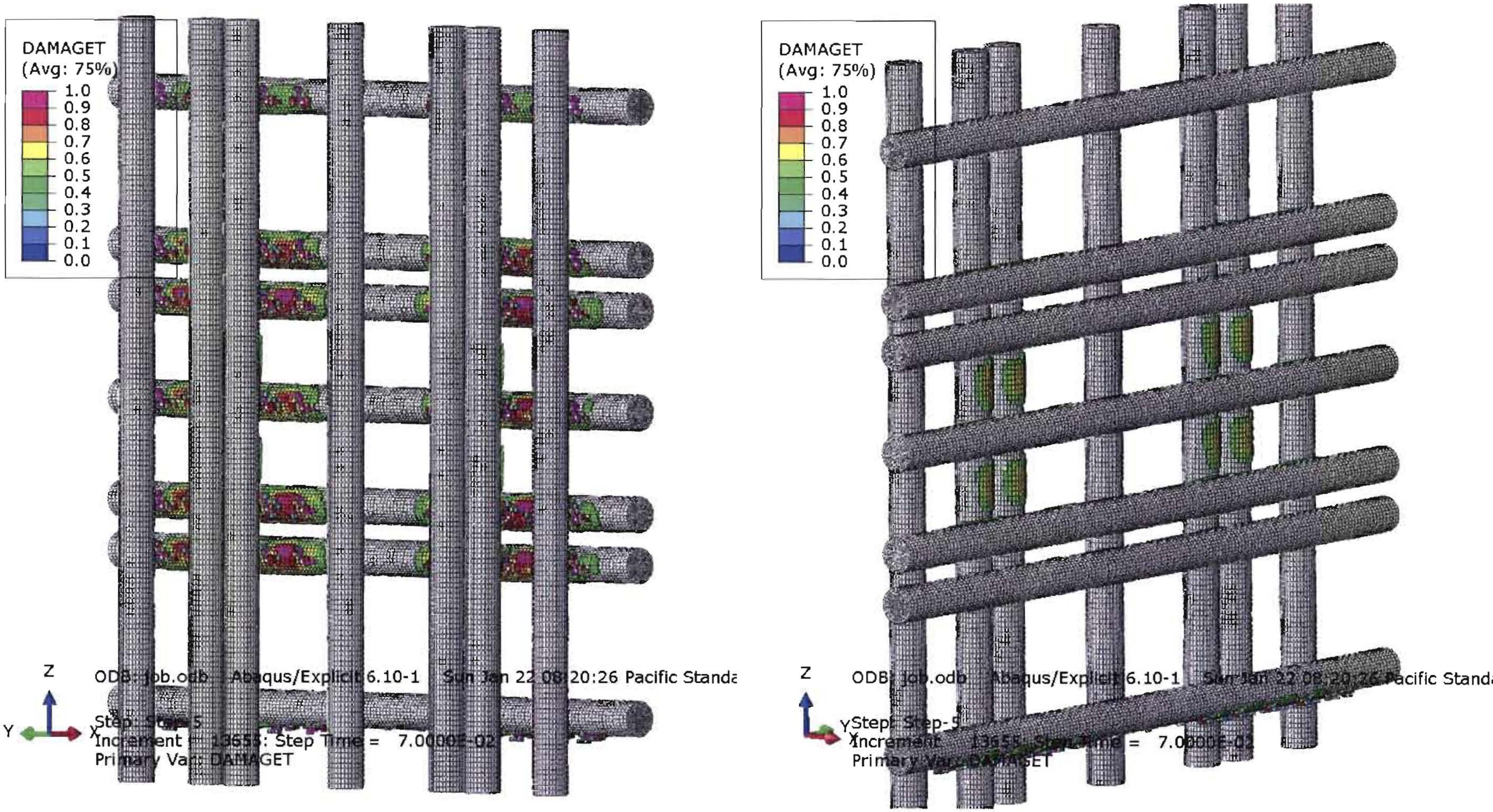
Nominal 6” Rebar, 0.6% VF





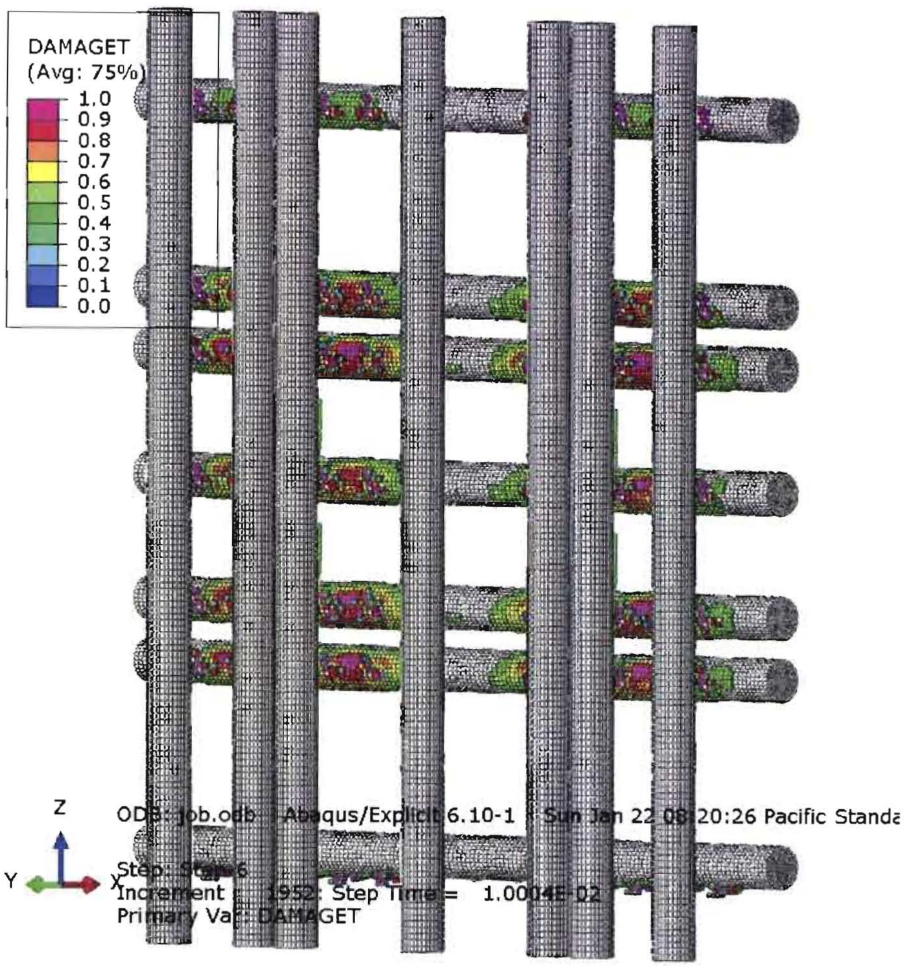
Outer 10" Frozen

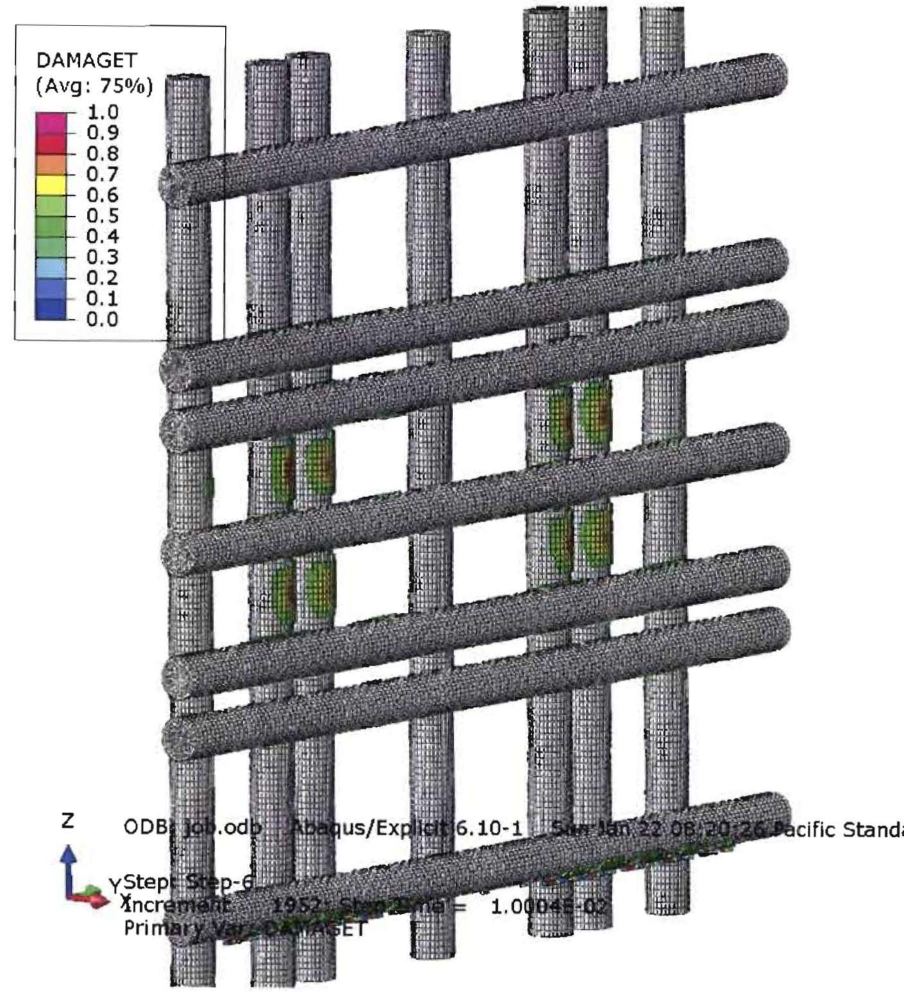
Nominal 6" Rebar, 0.6% VF



Inner 6” @ 1% Expansion

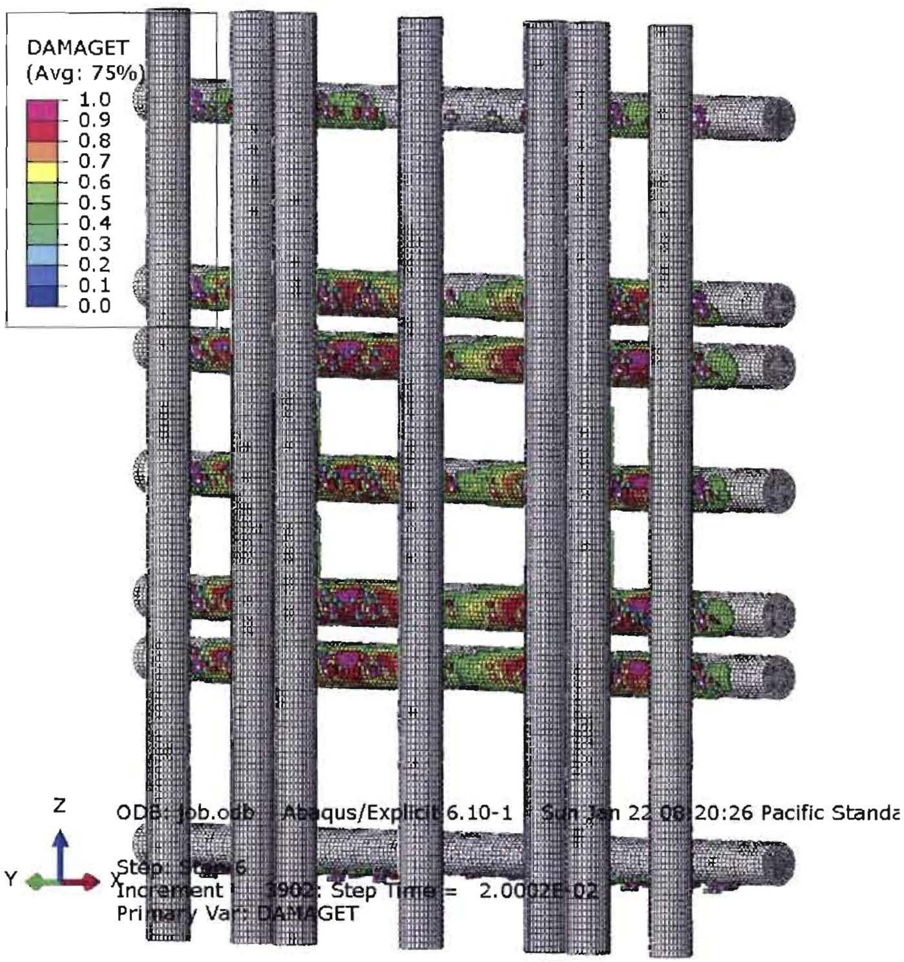
Nominal 6” Rebar, 0.6% VF

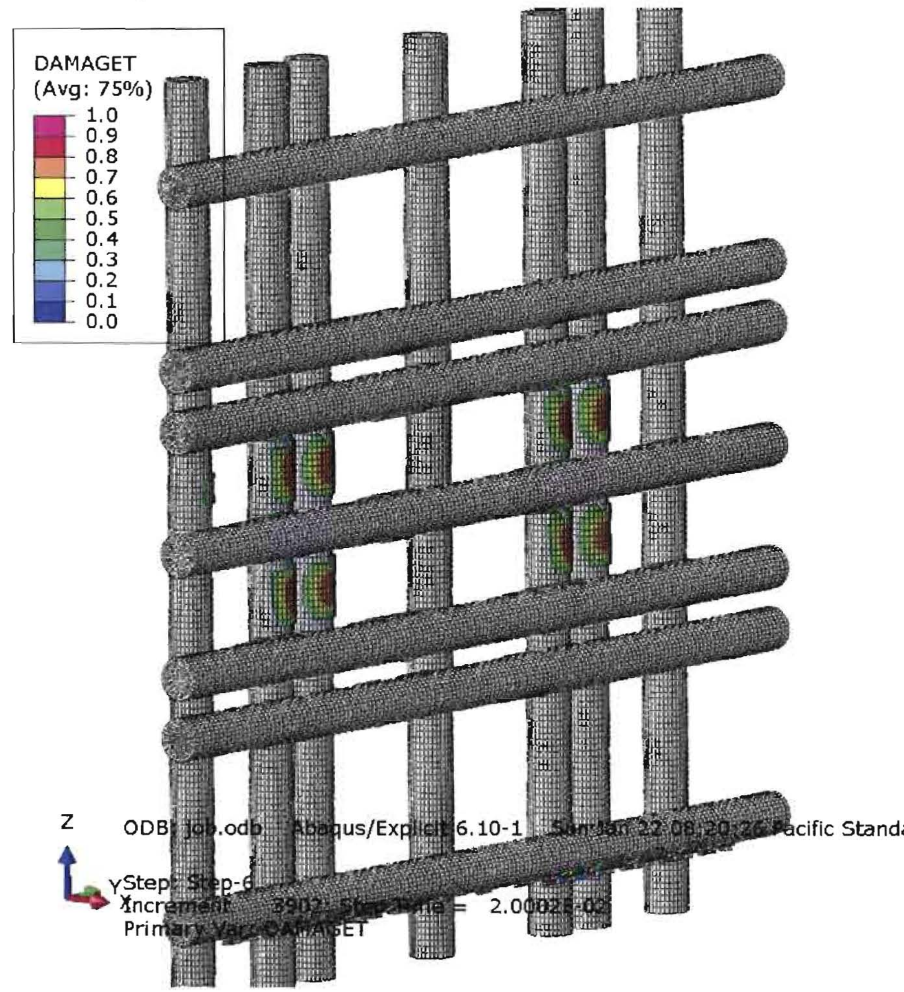




Inner 6" @ 2% Expansion

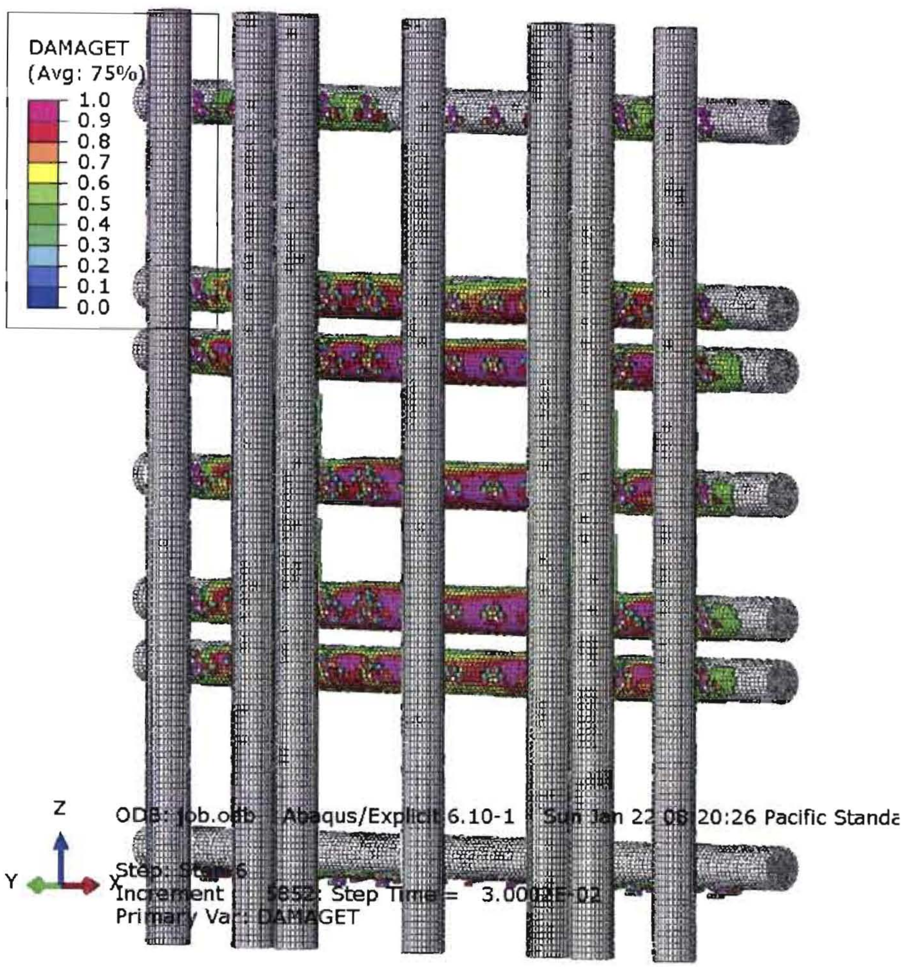
Nominal 6" Rebar, 0.6% VF

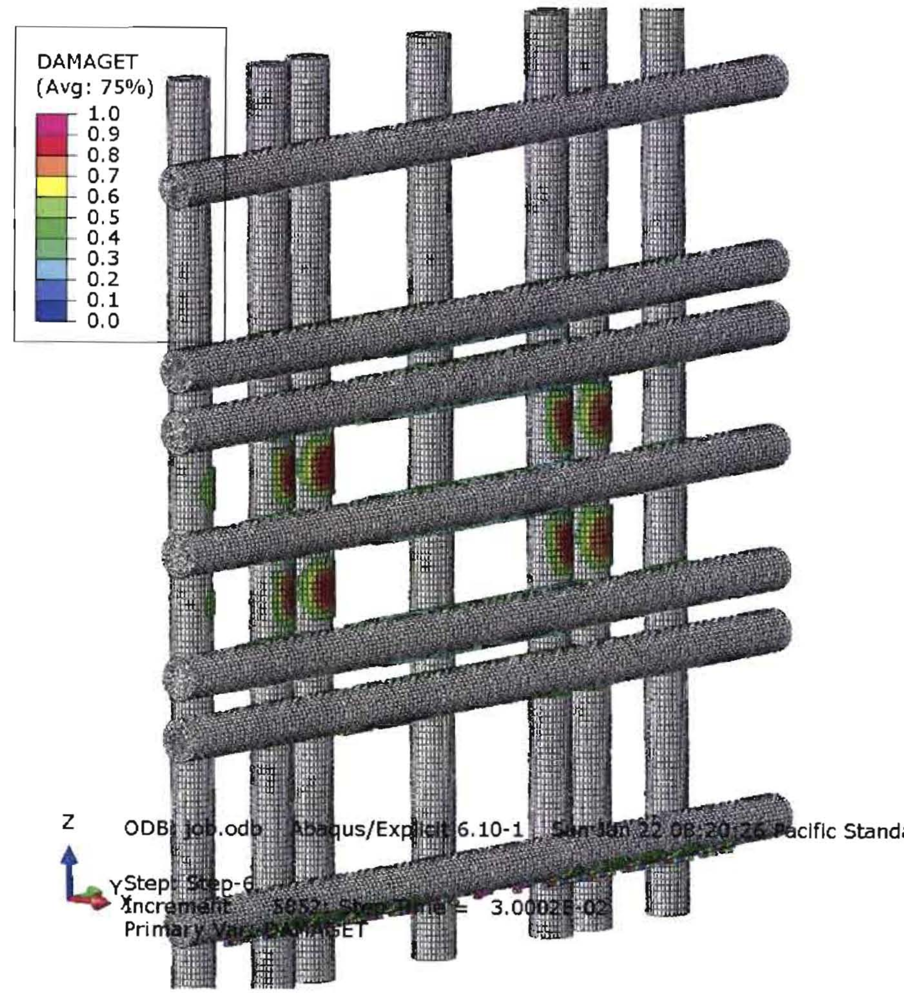




Inner 6" @ 3% Expansion

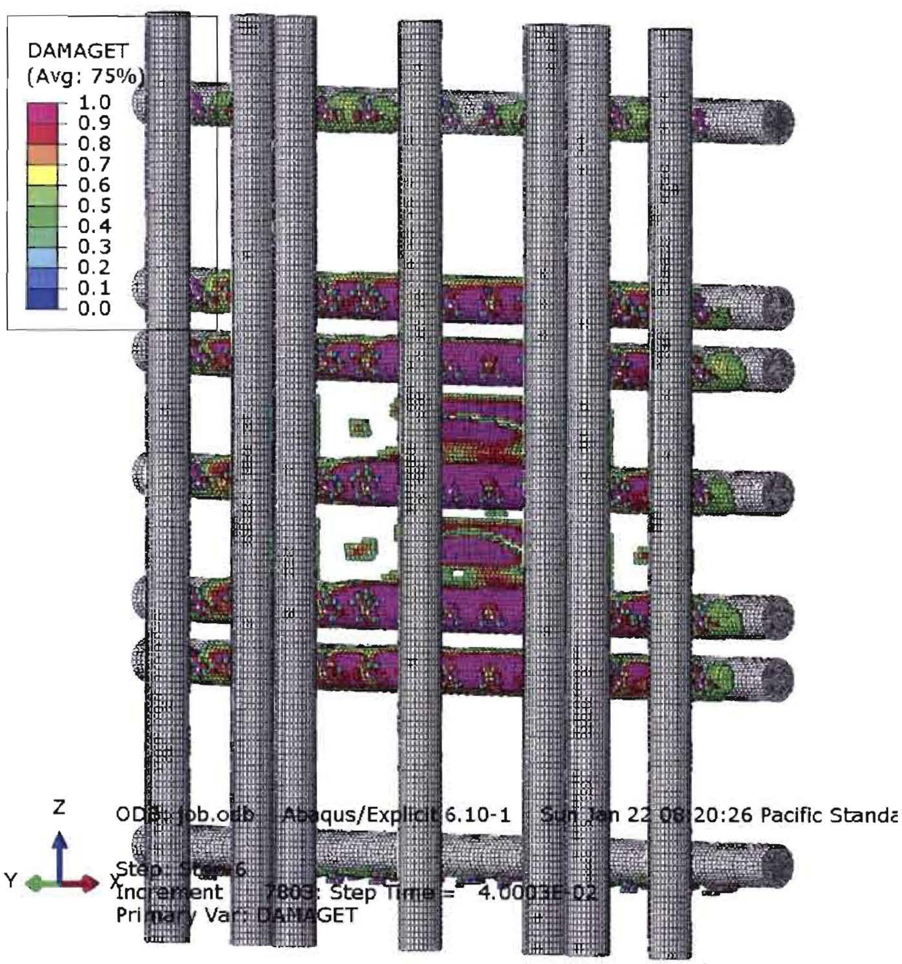
Nominal 6" Rebar, 0.6% VF

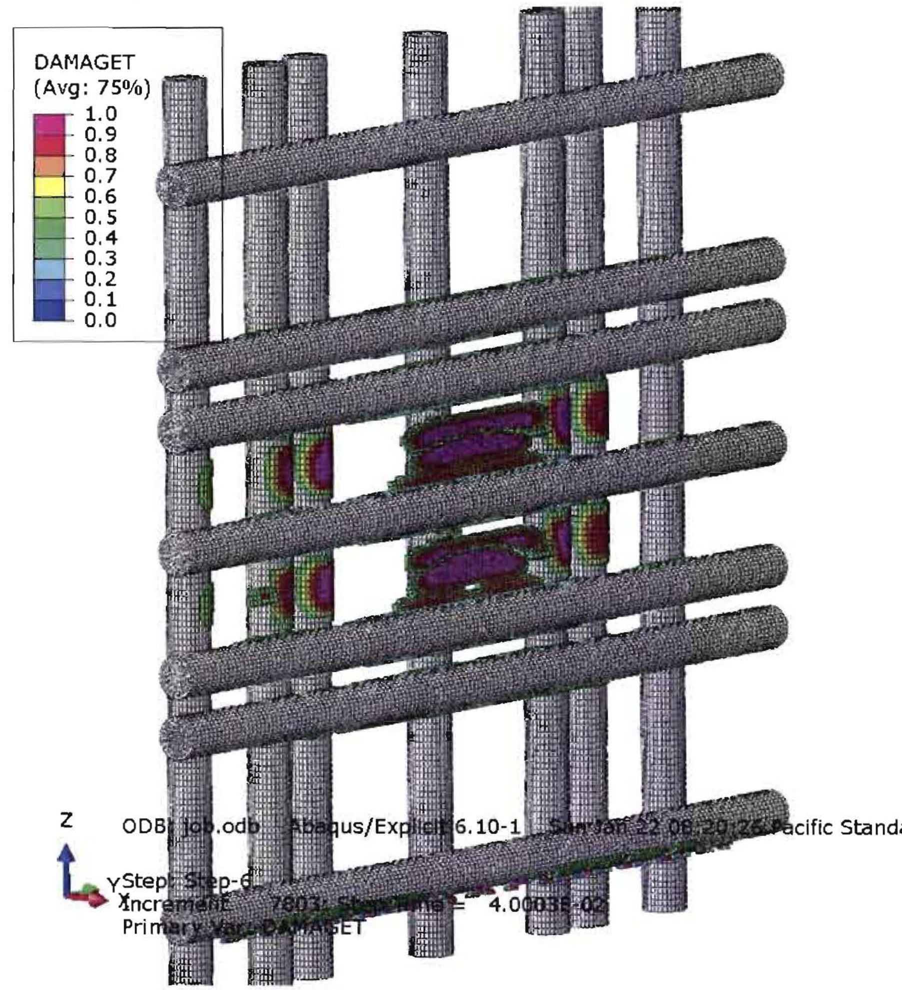




Inner 6" @ 4% Expansion

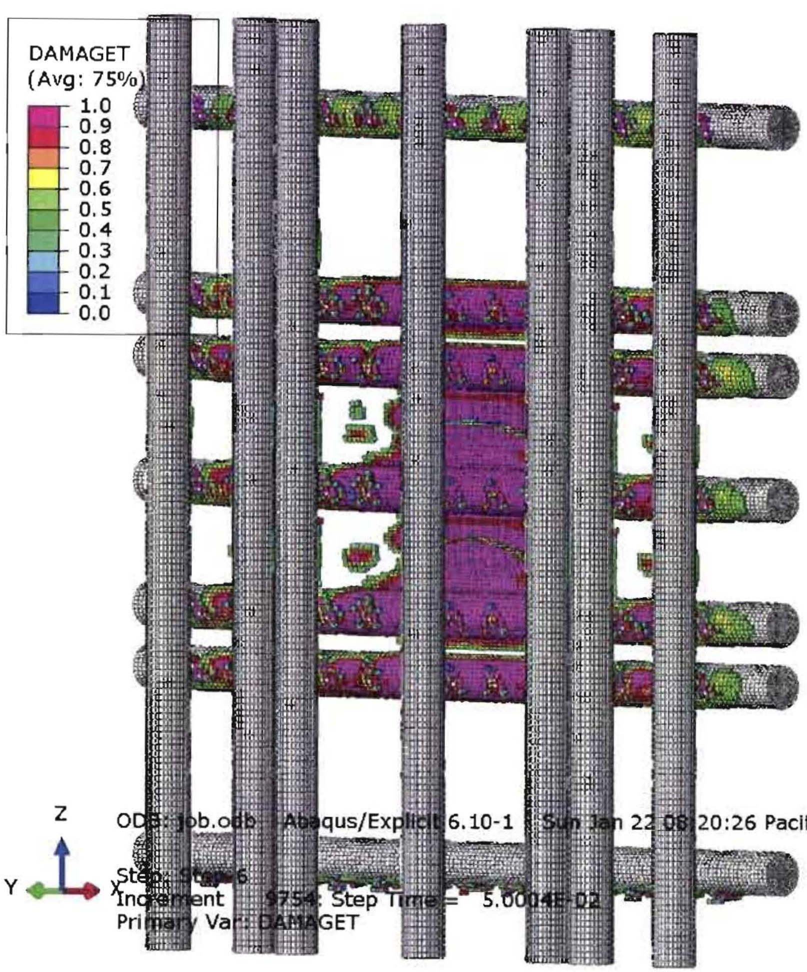
Nominal 6" Rebar, 0.6% VF

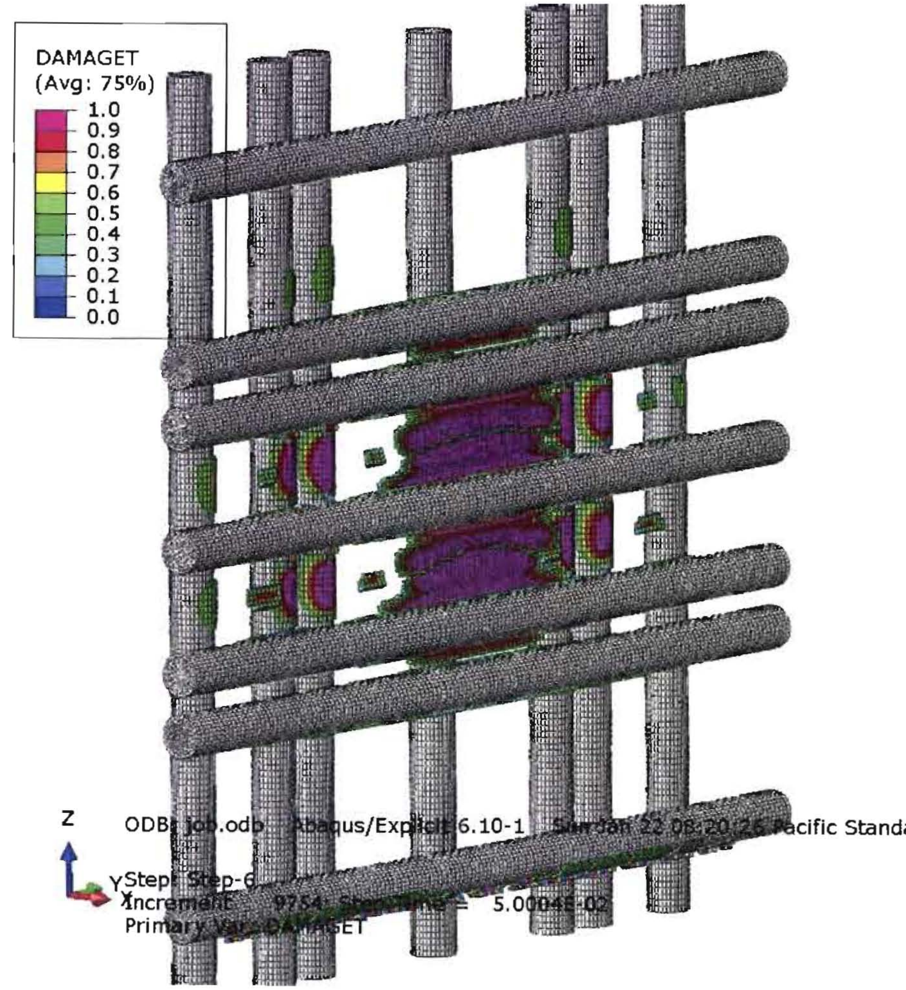




Inner 6” @ 5% Expansion

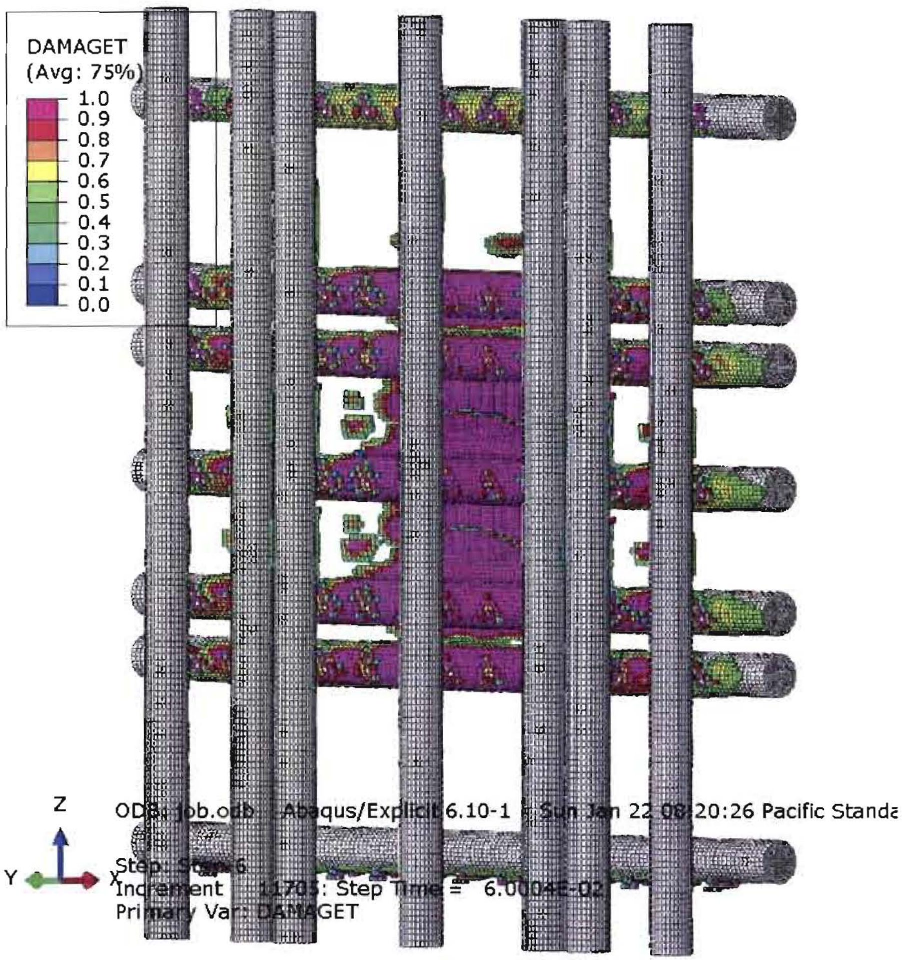
Nominal 6” Rebar, 0.6% VF

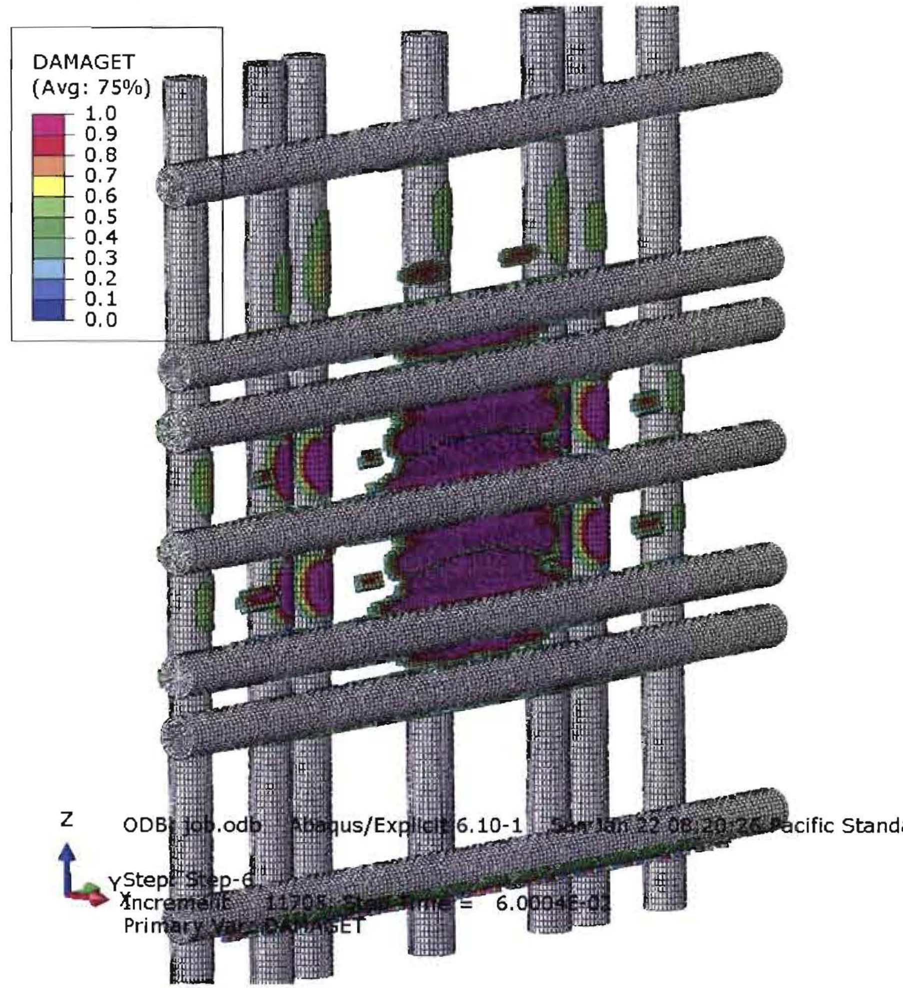




Inner 6” @ 6% Expansion

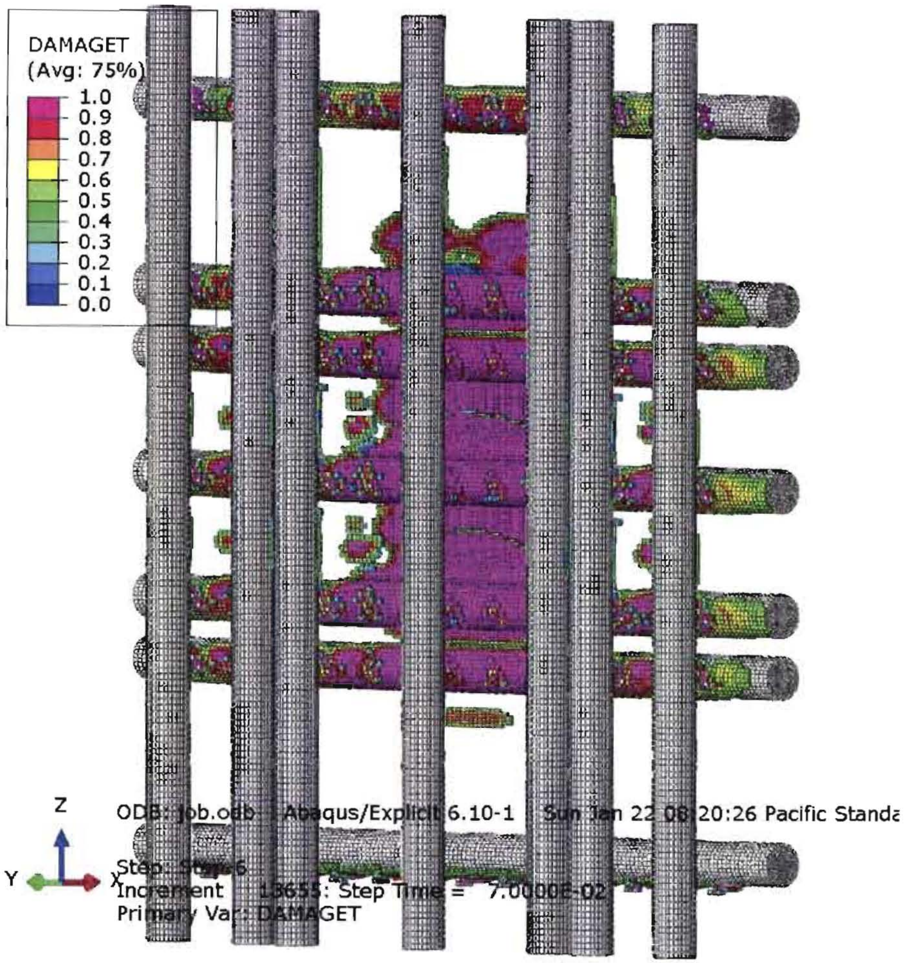
Nominal 6” Rebar, 0.6% VF

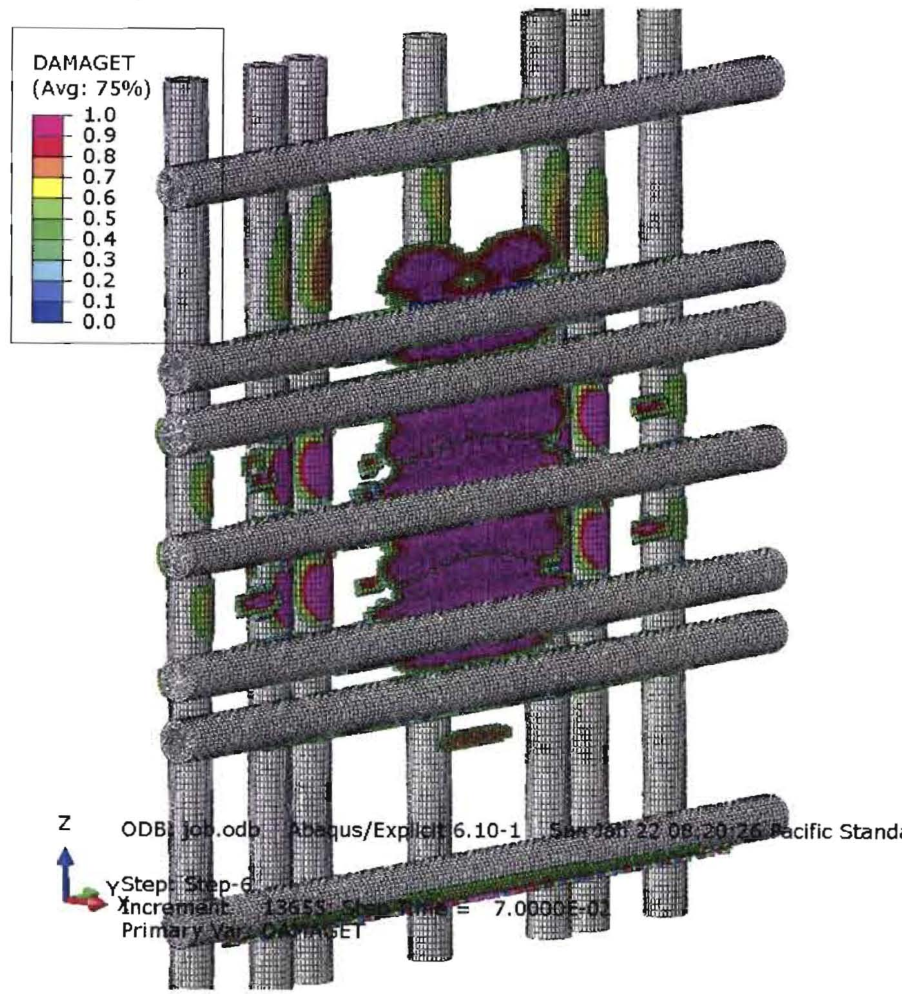




All Frozen (7% Expansion)

Nominal 6" Rebar, 0.6% VF





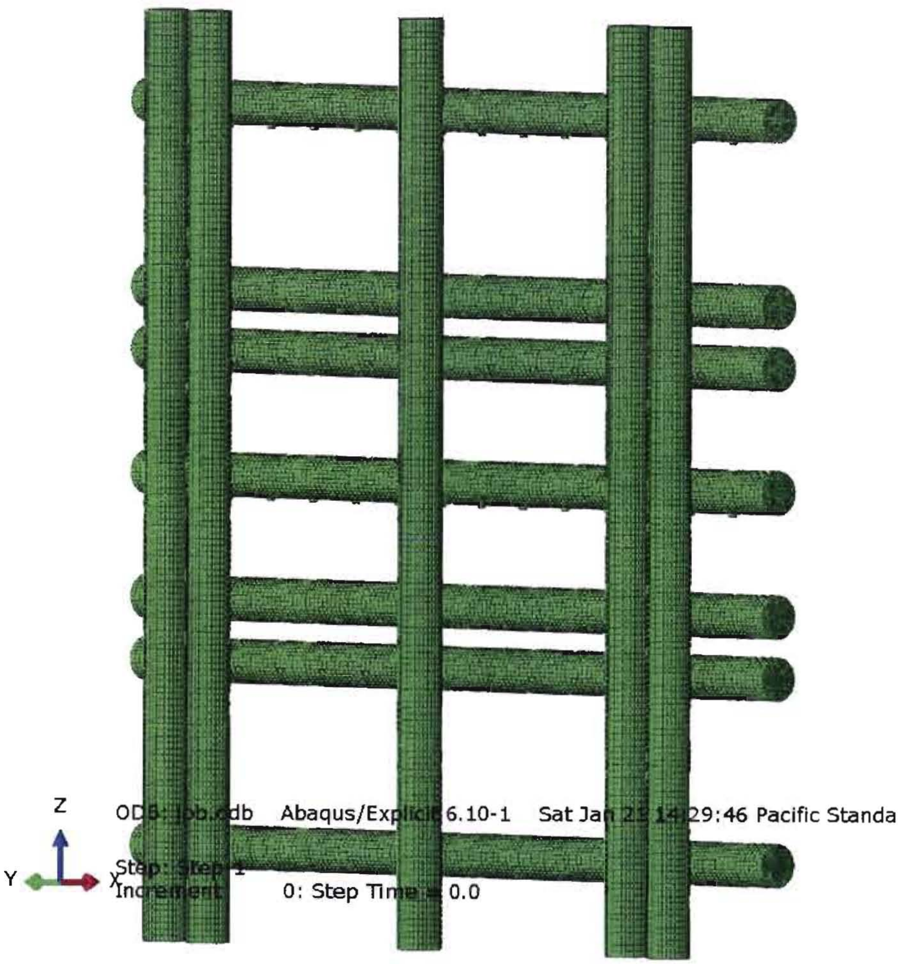
Freezing Results

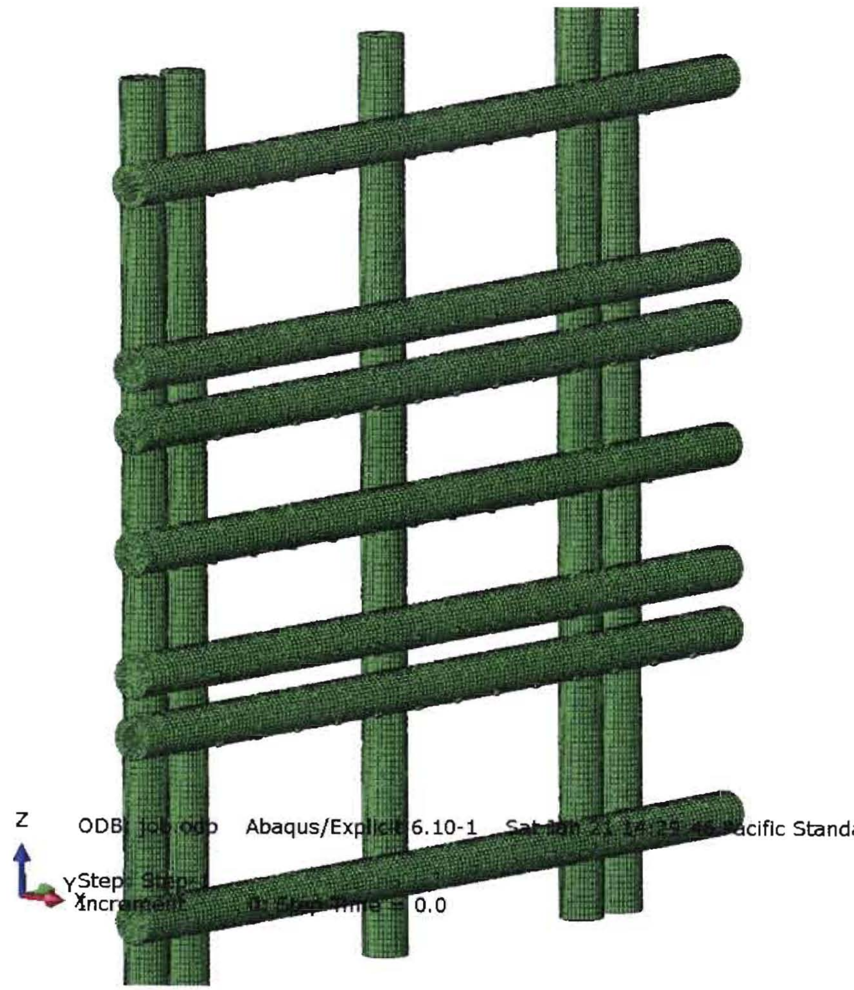
12" VERTICALS ONLY, 0.6% VF

12" Verticals Only, 0.6% VF

- This section of results are from a model with the following parameters:
- 0.6% VF i.e. 0.6% *of the elements in the first 0.1" under the horizontal rebars (bottom 180°)* are given a 7% expansion to simulate ice freezing.
- Nominal 12" spacing *of verticals only*
  - Assumes 12" spacing and includes lap regions
  - Horizontal bars stay at 6" nominal spacing

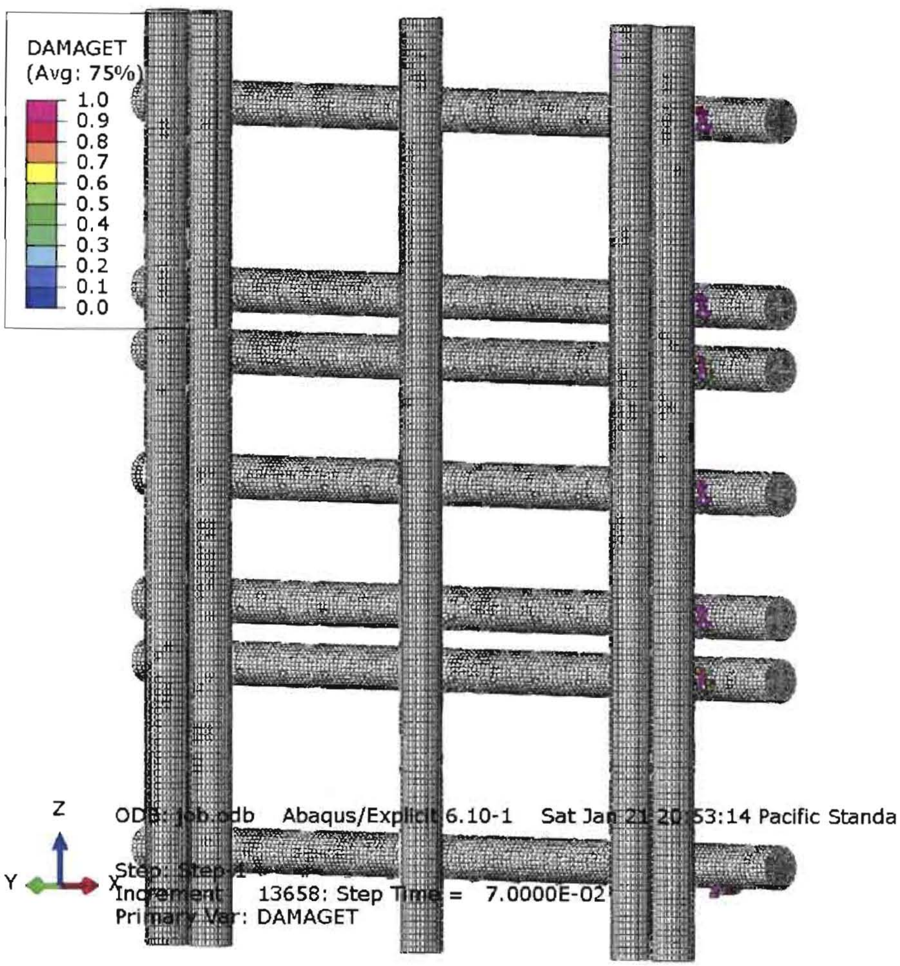
Mesh: 12" Verticals Only

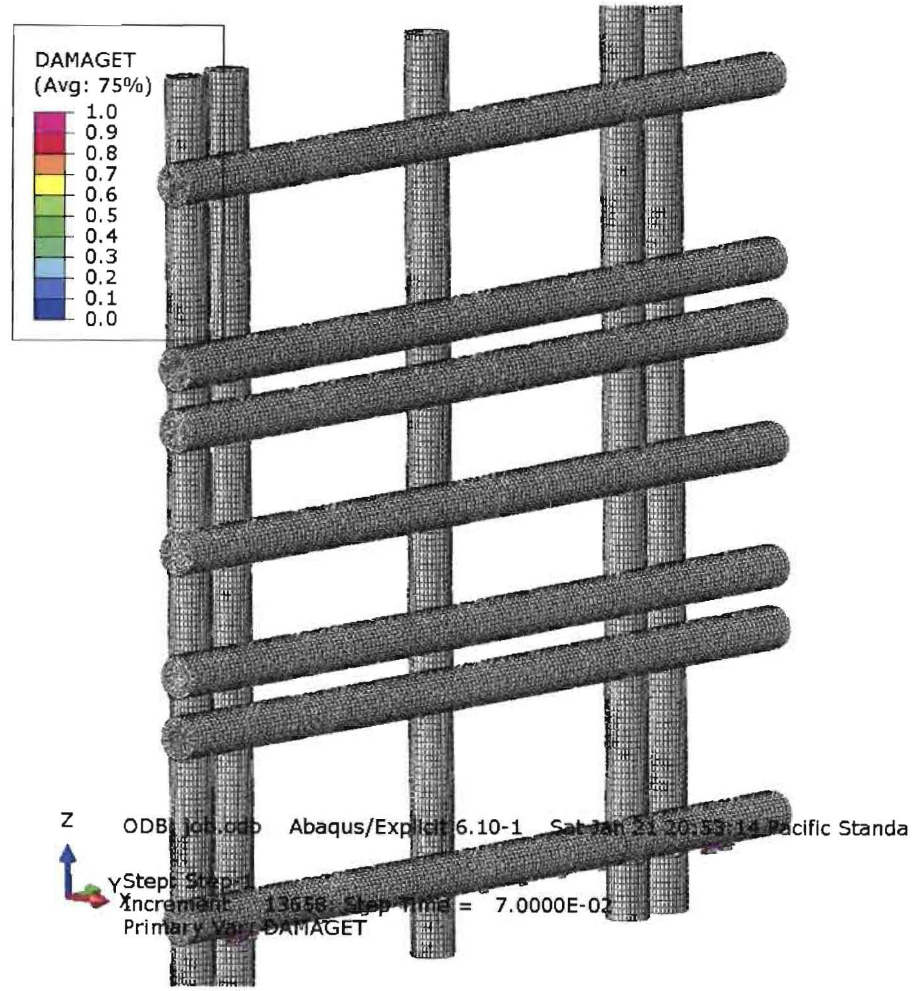




Outer 2" Frozen (7% Expansion)

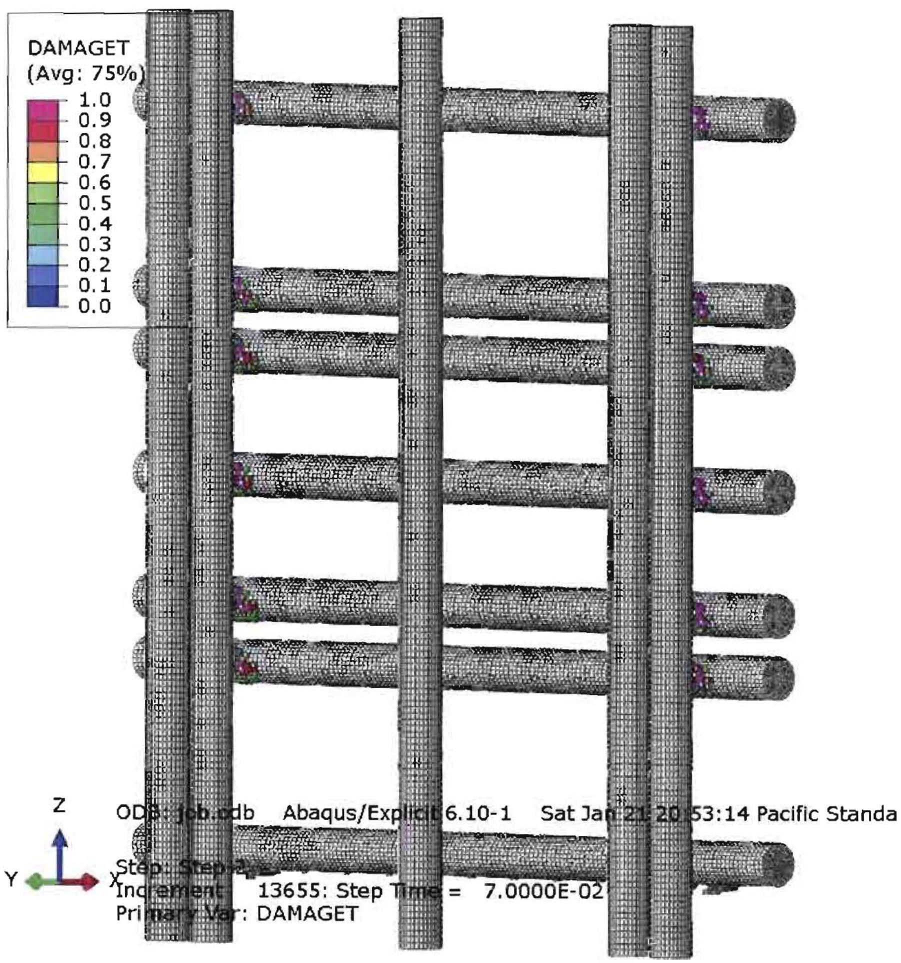
12" Verticals Only, 0.6% VF

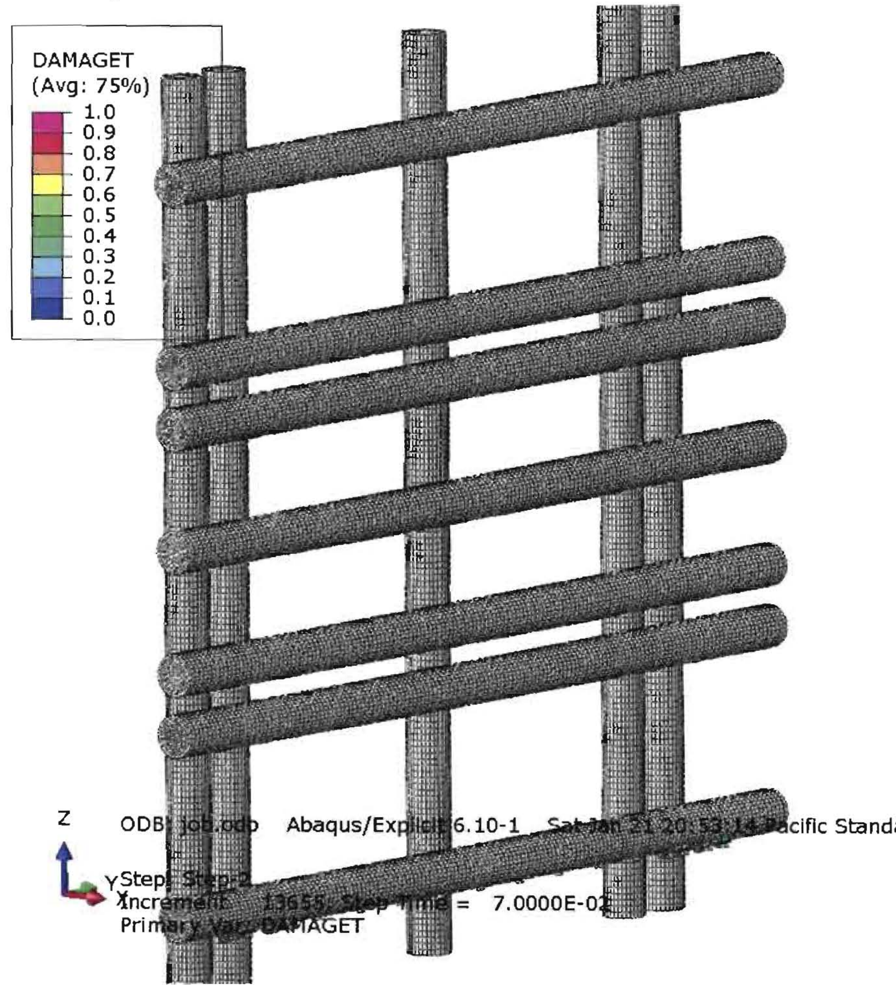




Outer 4" Frozen

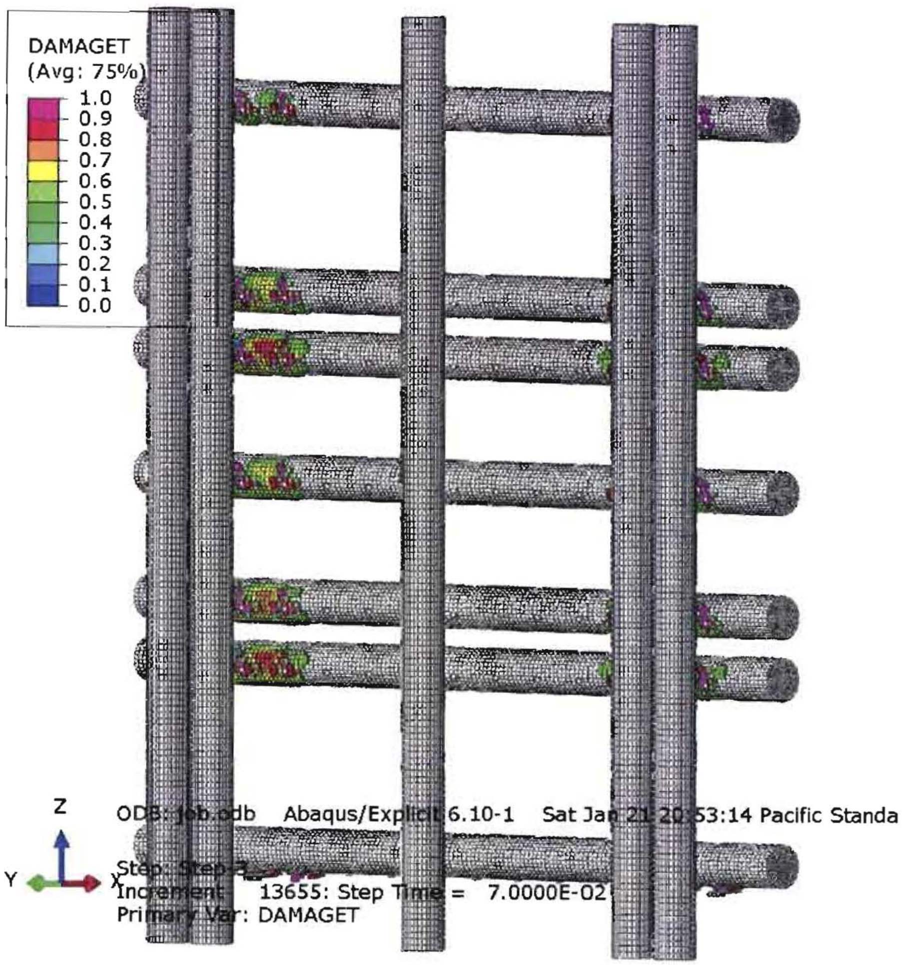
12" Verticals Only, 0.6% VF

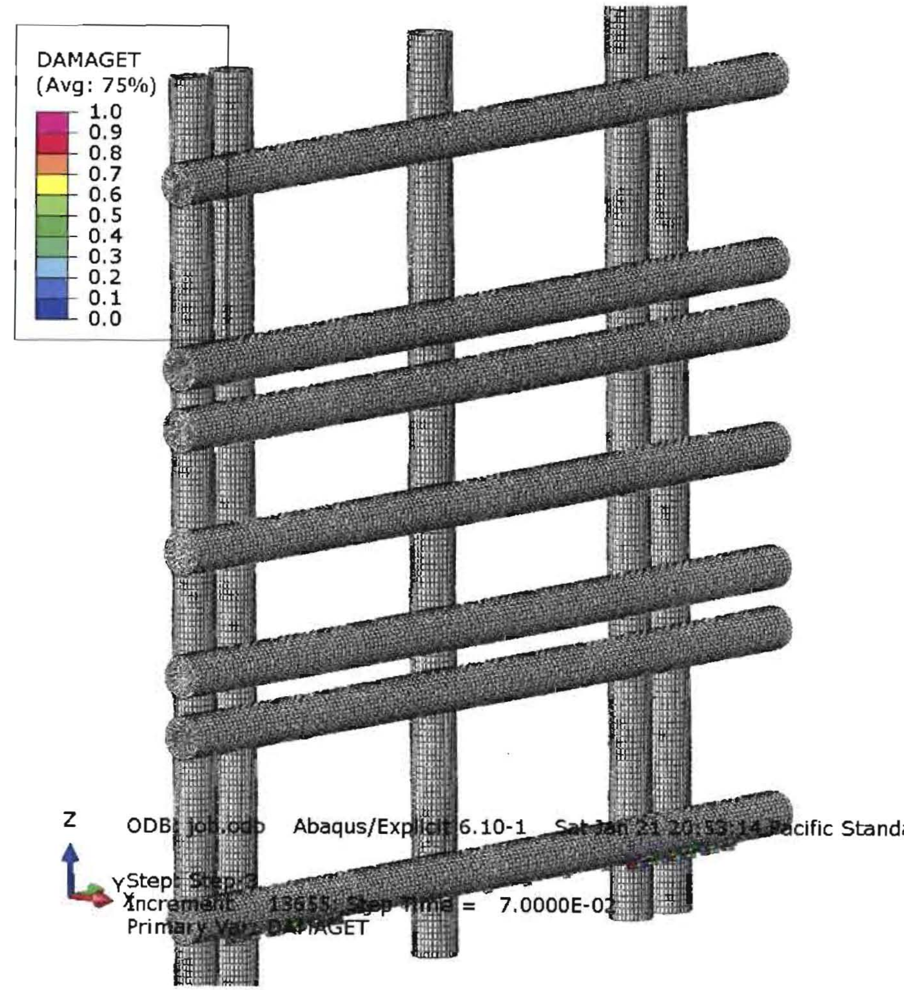




Outer 6" Frozen

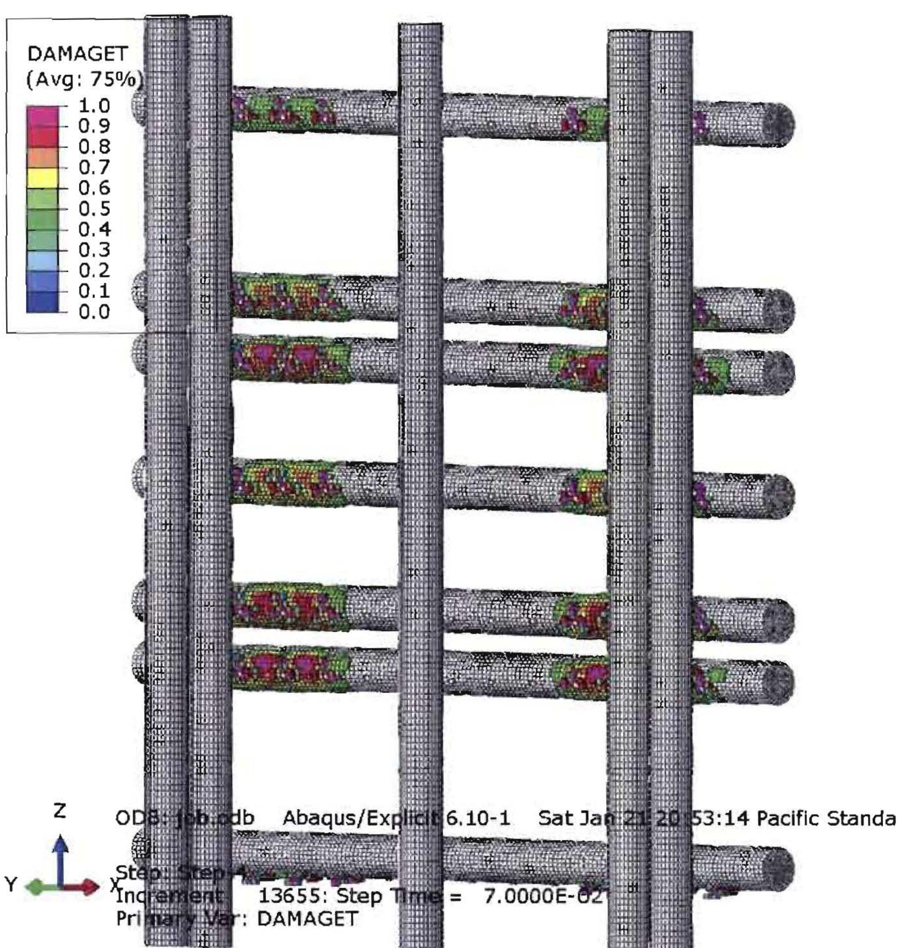
12" Verticals Only, 0.6% VF

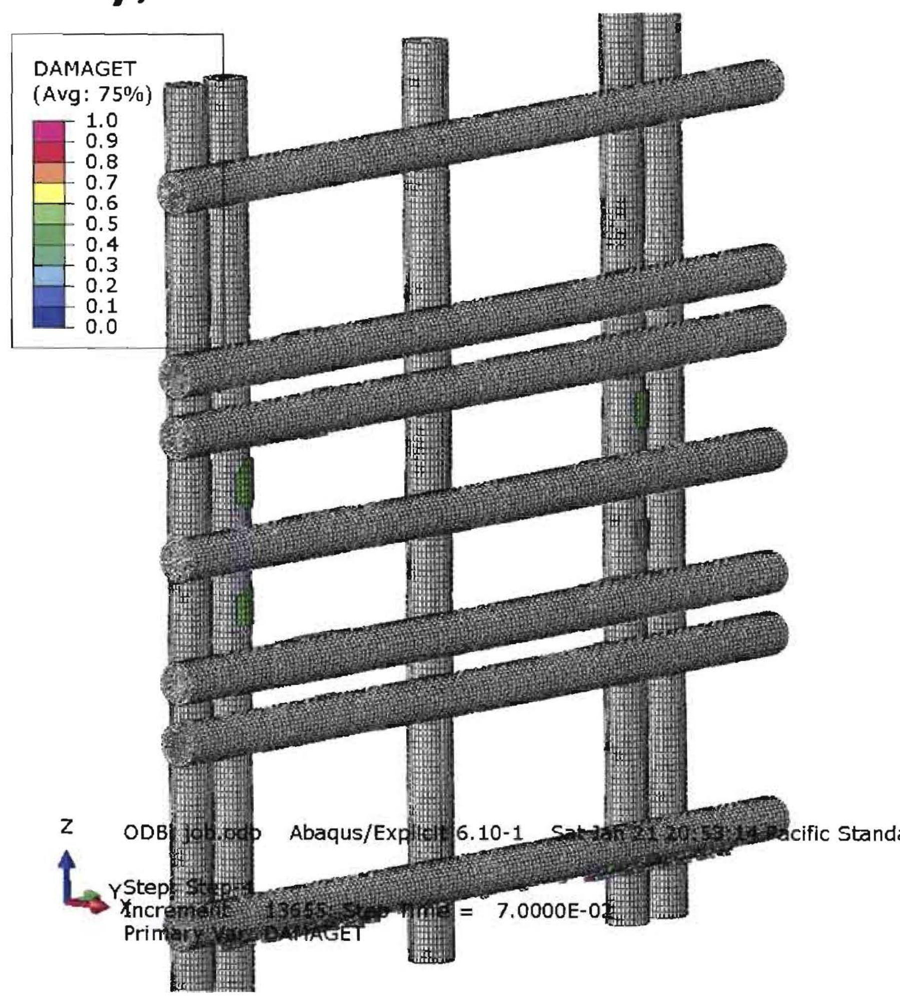




Outer 8” Frozen

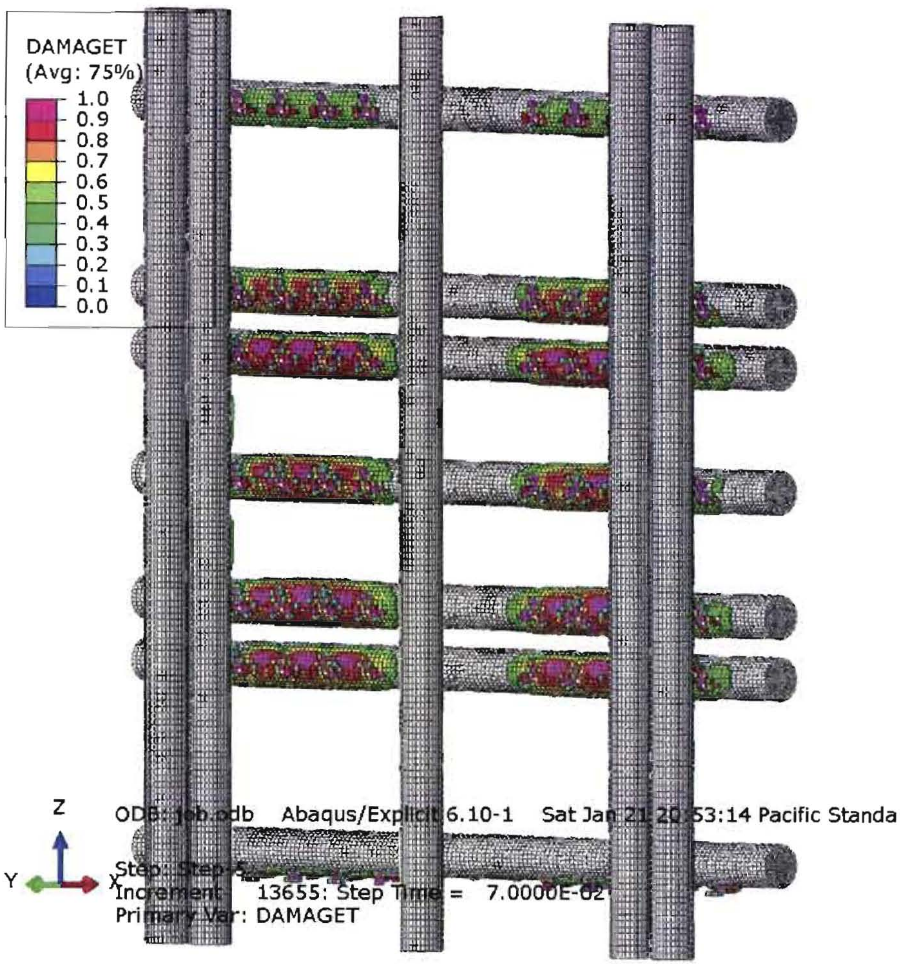
12” Verticals Only, 0.6% VF

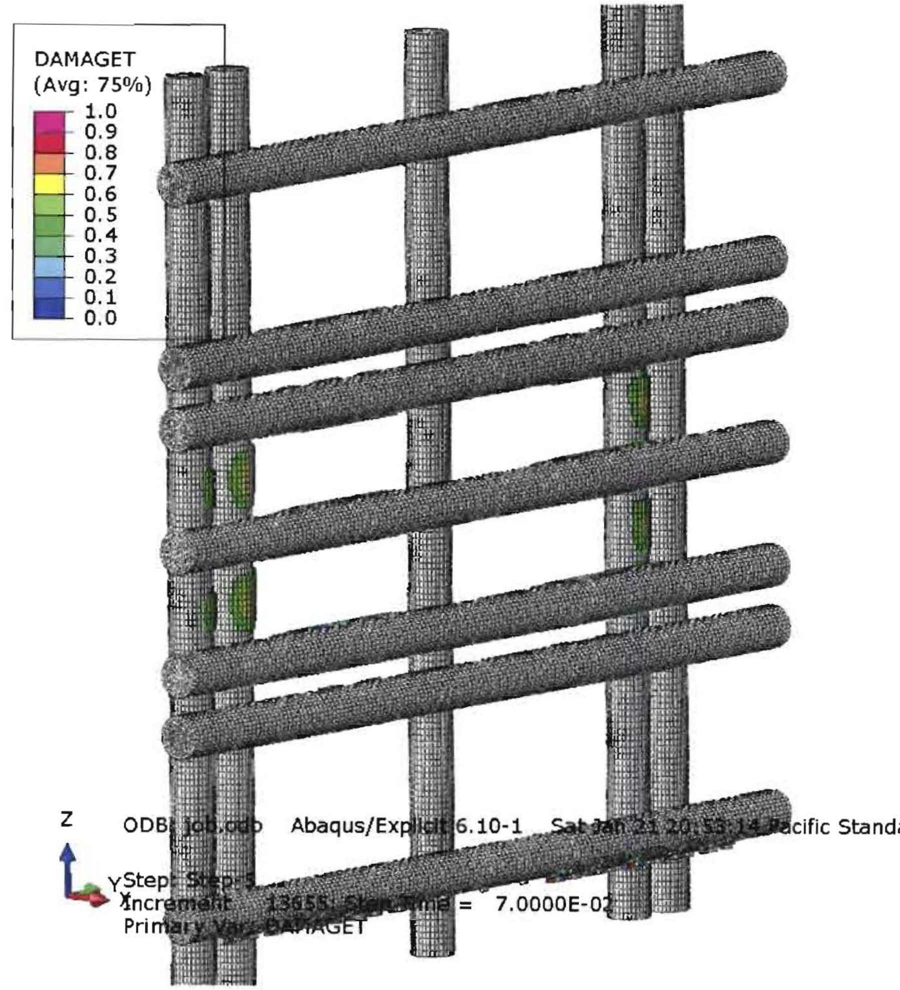




Outer 10" Frozen

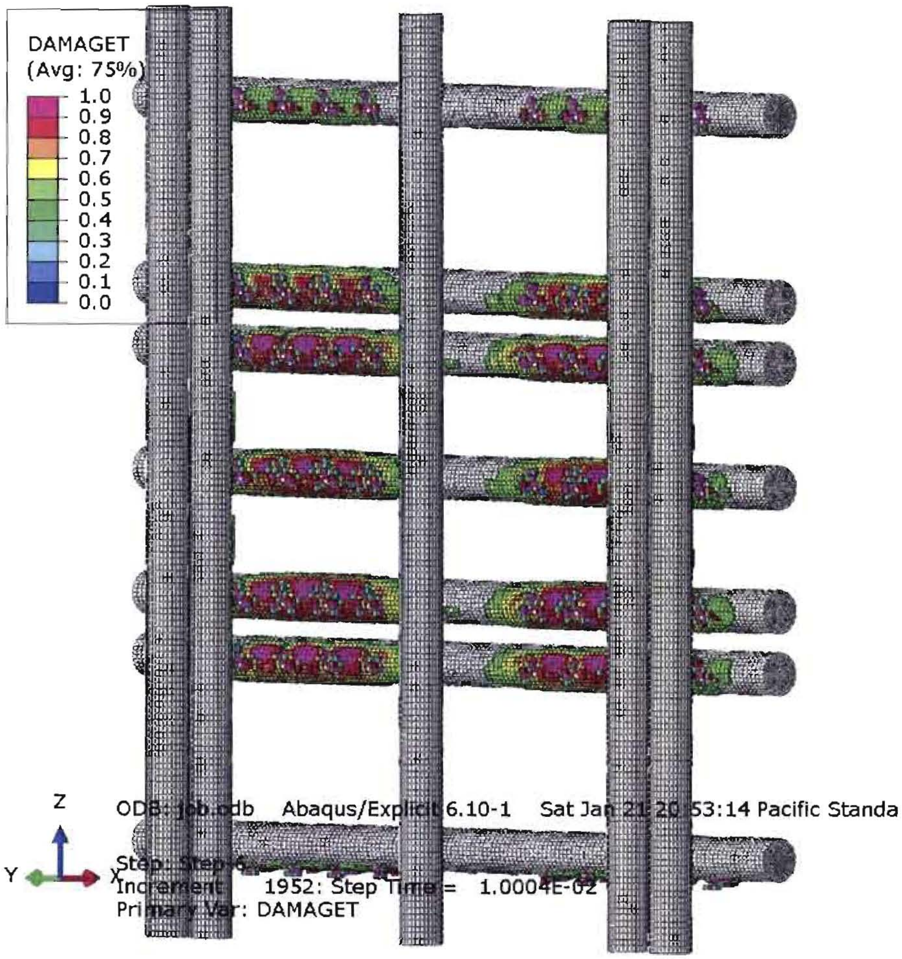
12" Verticals Only, 0.6% VF

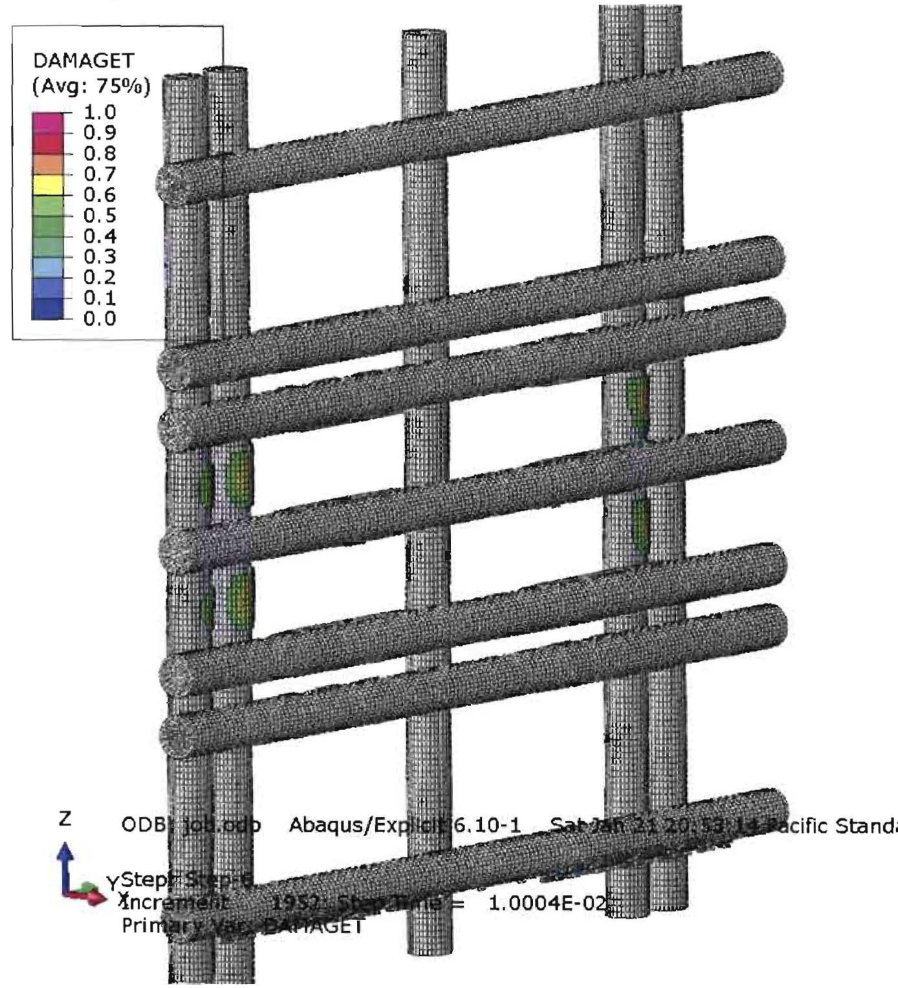




Inner 6" @ 1% Expansion

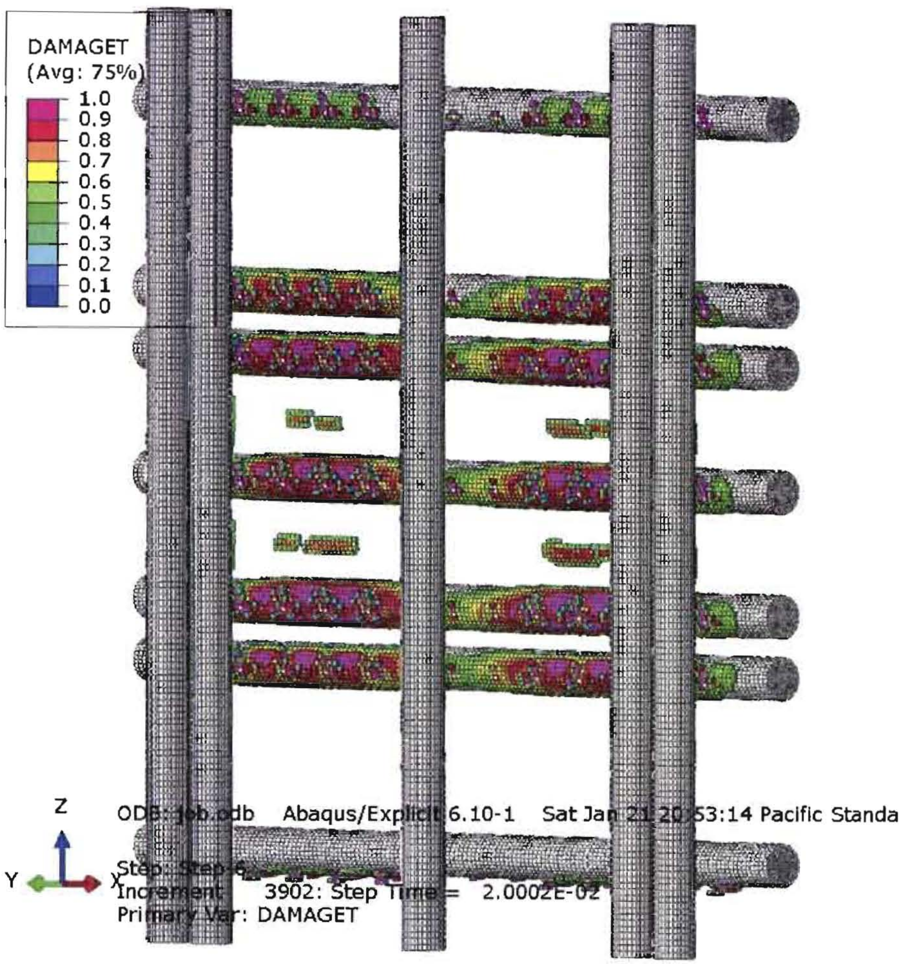
12" Verticals Only, 0.6% VF

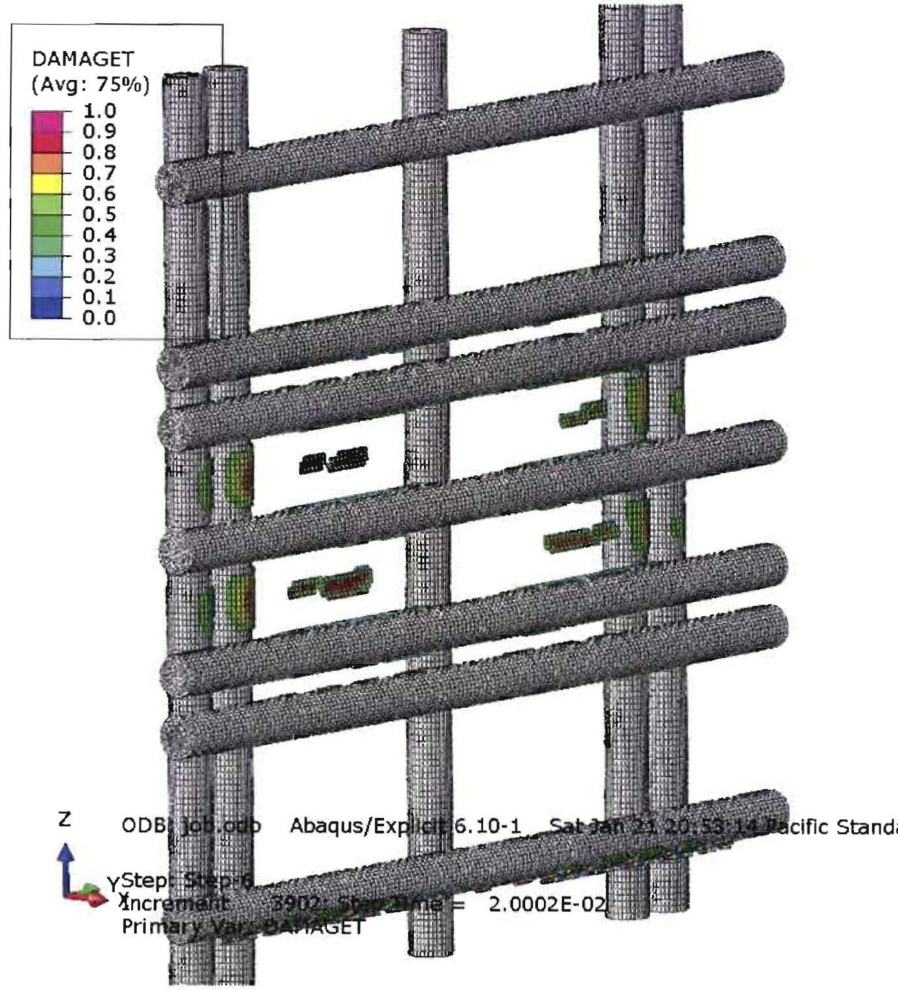




Inner 6" @ 2% Expansion

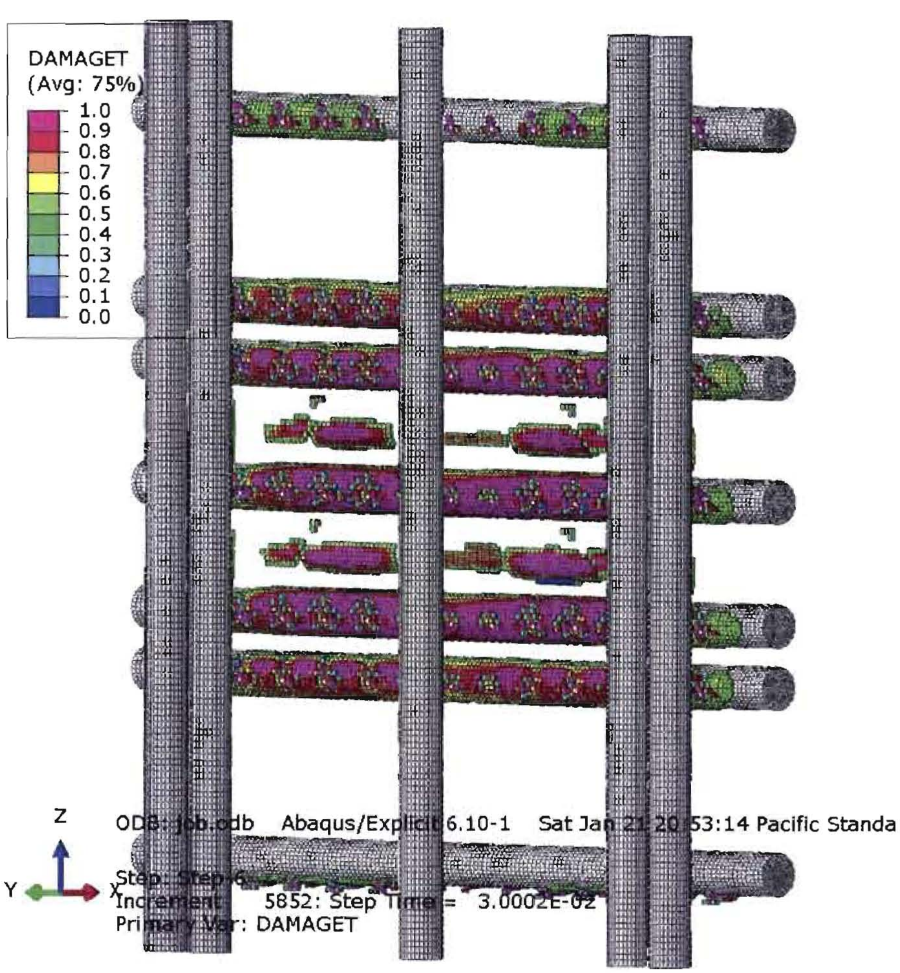
12" Verticals Only, 0.6% VF

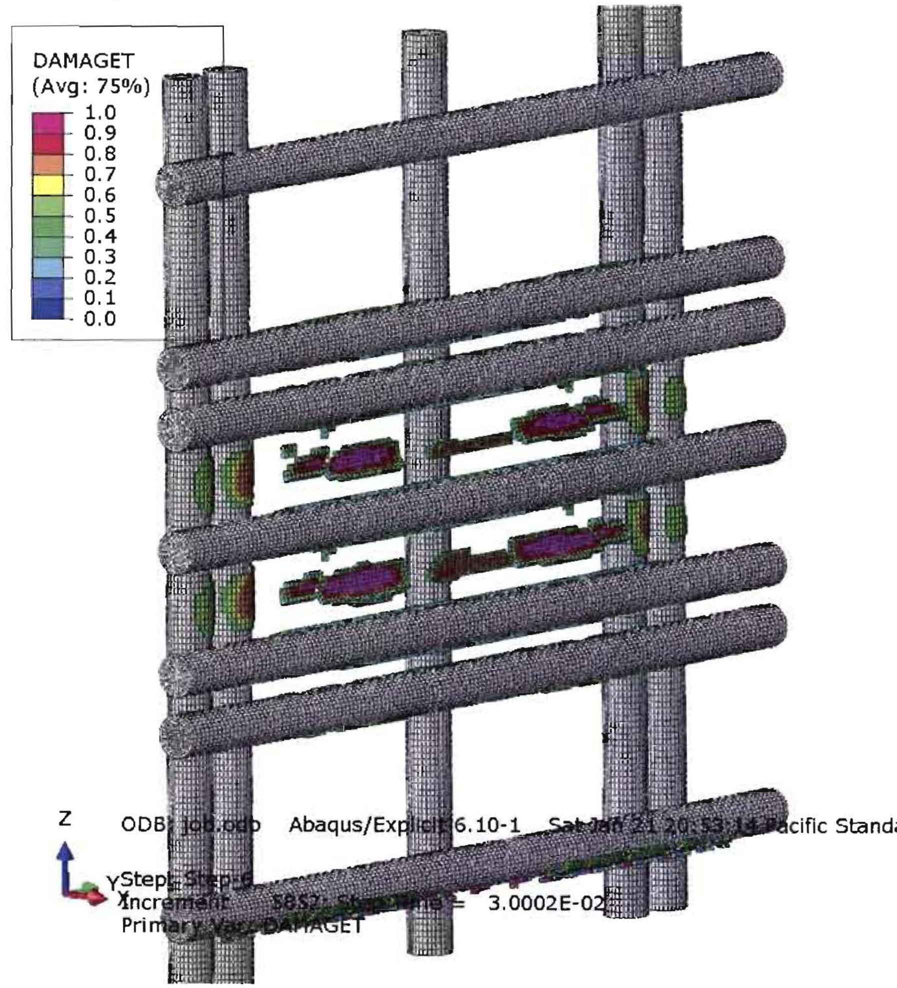




Inner 6" @ 3% Expansion

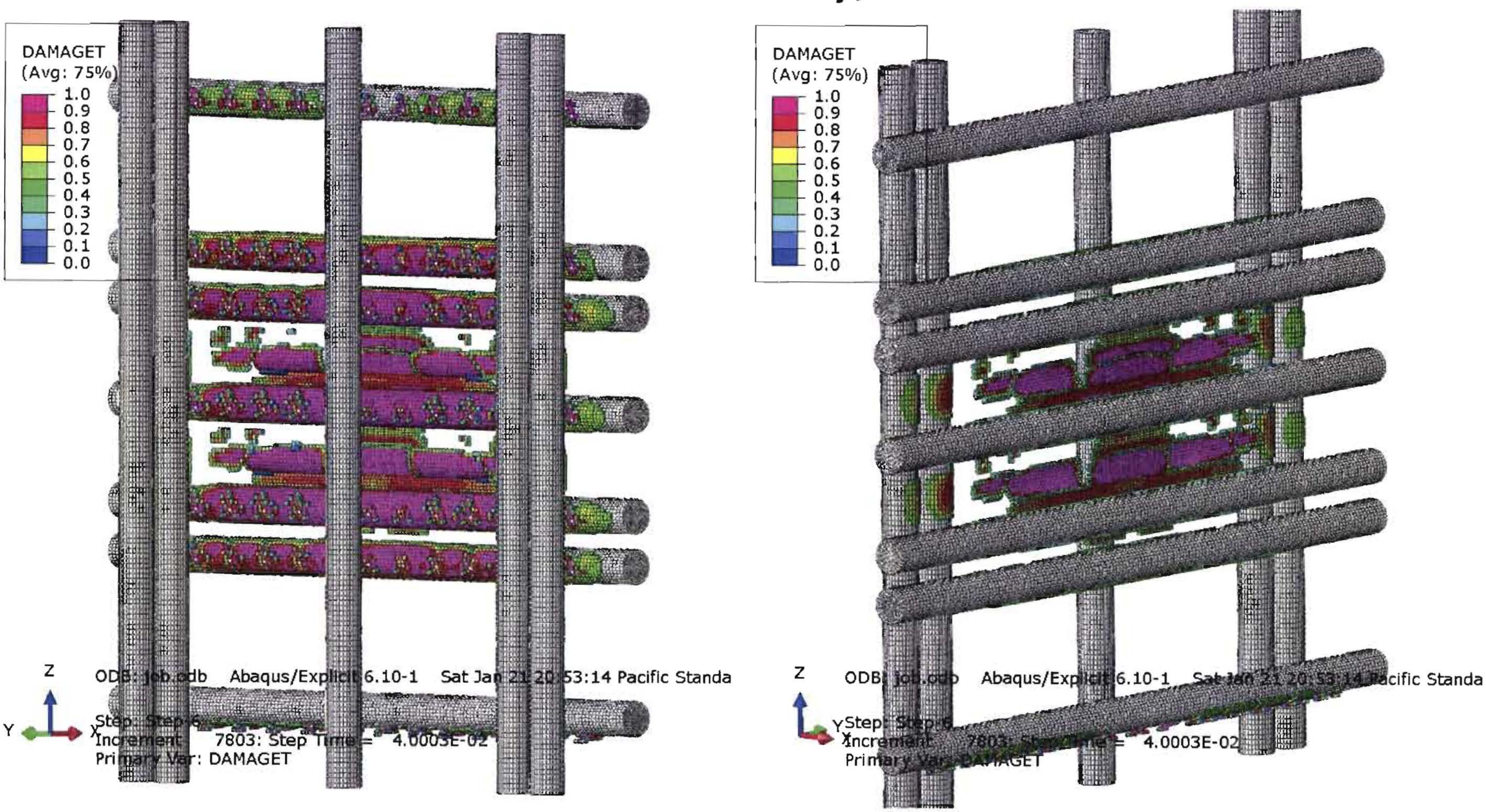
12" Verticals Only, 0.6% VF





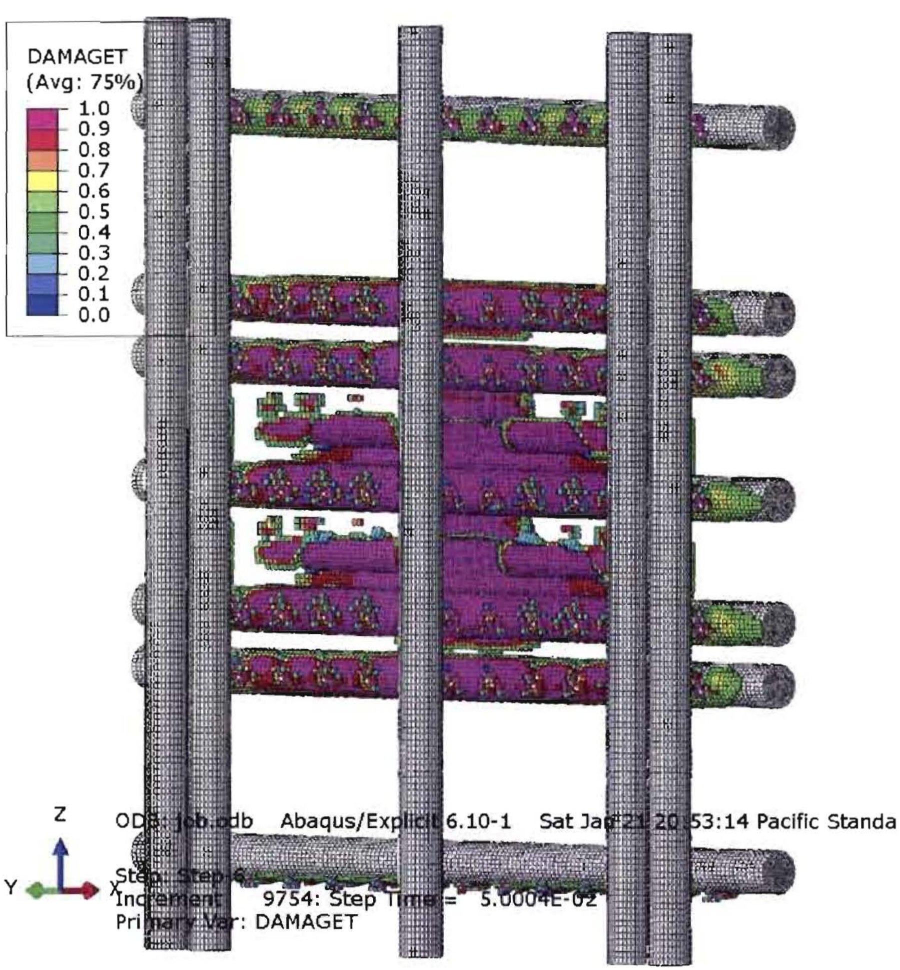
Inner 6" @ 4% Expansion

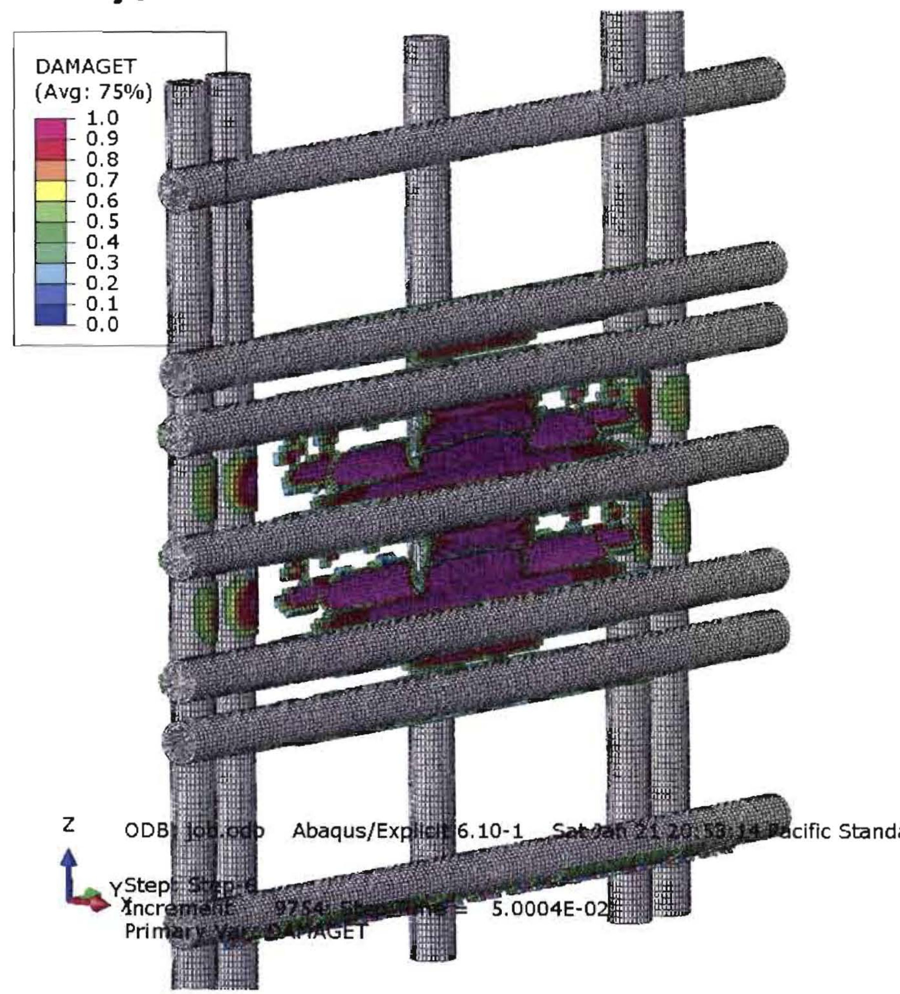
12" Verticals Only, 0.6% VF



Inner 6" @ 5% Expansion

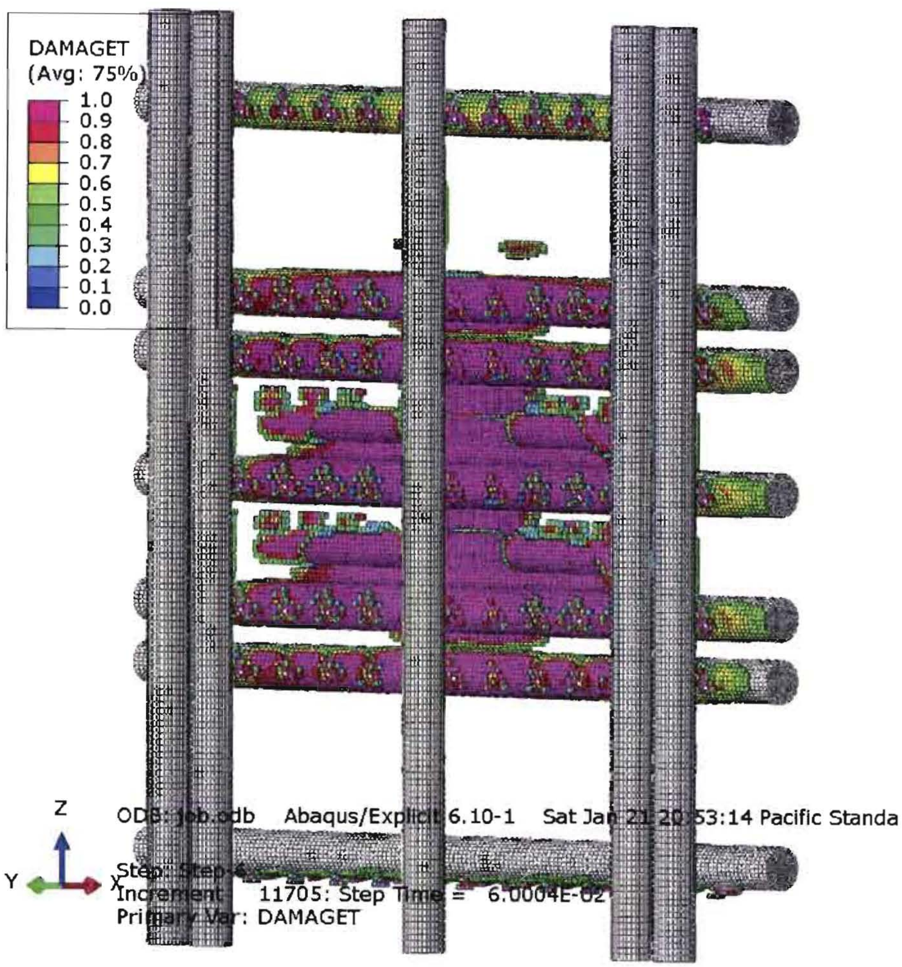
12" Verticals Only, 0.6% VF

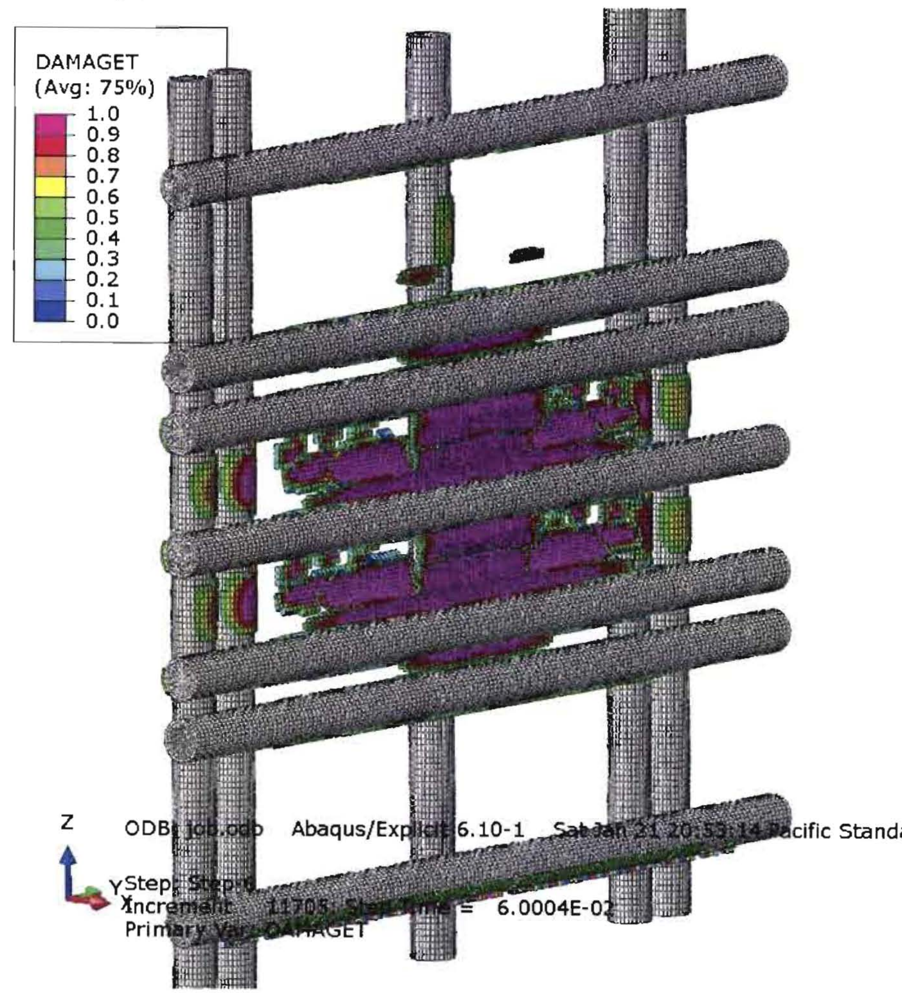




Inner 6” @ 6% Expansion

12” Verticals Only, 0.6% VF





All Frozen (7% Expansion)

12” Verticals Only, 0.6% VF