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MAY 13 2010

10 CFR 50
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10 CFR 54

LR-N10-0165

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555-0001

Salem Nuclear Generating Station, Unit No. 1 and Unit No. 2
Facility Operating License Nos. DPR-70 and DPR-75
NRC Docket Nos. 50-272 and 50-311

Subject: Response to NRC Request for Additional Information dated April 15, 2010, Related to Structures and Structures-Related Aging Management Programs for the Salem Nuclear Generating Station, Units 1 and 2 License Renewal Application

Reference: Letter from Mr. Donnie Ashley (USNRC) to Mr. Thomas Joyce (PSEG Nuclear, LLC) "REQUEST FOR ADDITIONAL INFORMATION REGARDING ASME SECTION XI, SUBSECTION IWE FOR THE SALEM NUCLEAR GENERATING STATION UNITS 1 AND 2 LICENSE RENEWAL APPLICATION (TAC ME1836 AND ME1834)", dated April 15, 2010

In the referenced letter, the NRC requested additional information related to certain Structures and Structures-Related Aging Management Programs associated with the Salem Nuclear Generating Station, Units 1 and 2 (Salem) License Renewal Application (LRA). Enclosure A contains the responses to the requests for additional information (RAI). Enclosure B contains updates to LRA Appendix A (UFSAR Supplement) and Appendix B Program descriptions that are affected by these RAI responses.

Enclosure C contains an update to the Salem License Renewal Commitment List, which details several changes being made to PSEG Nuclear's commitments associated with the Salem License Renewal Application.

If you have any questions regarding this submittal, please contact Mr. Ali Fakhar, PSEG Manager - License Renewal, at 856-339-1646.

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A047
NRR

MAY 13 2010

I declare under penalty of perjury that the foregoing is true and correct.

Executed on 5/13/10

Sincerely,

Paul J. Davison

Paul J. Davison
Vice President, Operations Support
PSEG Nuclear LLC

Enclosures:
A. Responses to Request for Additional Information
B. Updates to LRA Appendix A and Appendix B
C. Updates to License Renewal Commitment List
D. MPR Associates Report MPR-2613, Revision 3

cc:
S. Collins, Regional Administrator – USNRC Region I
B. Brady, Project Manager, License Renewal – USNRC
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Enclosure A

Responses to Request for Additional Information related to Structures and Structures-related Aging Management Programs for the Salem Nuclear Generating Station, Units 1 and 2 License Renewal Application (LRA)

RAI B.2.1.28-1

Background

GALL Report (NUREG-1801), AMP XI.S1, "ASME Section XI, Subsection IWE," Program Element 10 states that implementation of ASME Section XI, Subsection IWE, in accordance with 10 CFR 50.55a, is a necessary element of aging management for steel components of steel and concrete containments through the period of extended operation.

Issue

Program Element 10 for the Salem ASME Section XI, Subsection IWE aging management program discusses operating experience related to containment steel liner plate corrosion as described in NRC Information Notices IN 97-10 and IN 2004-09. However, Program Element 10 for the Salem ASME Section XI, Subsection IWE aging management program does not discuss operating experience related to liner plate corrosion recently reported at Beaver Valley. In addition, a review of the operating experience of the Salem Unit 1 (PIRS # 950706252-78) in 1995, (Notification # 20344017) in 2007, and Unit 2 (Notification #20235636) in 2005 indicate that borated water was running down the containment liner plate behind the insulation which resulted in indications of corrosion of the containment liner plate and seepage of water into moisture barrier. According to Notification # 20344017, borated water has been leaking in one area of containment for last 30 years.

Request

1. Provide details of borated water leakage, if any, observed inside Units 1 and 2 containments during the most recent refueling outages.

2. Explain why augmented inspection of the Unit 2 liner plate and moisture barrier was not performed in successive inspection intervals as required by IWE-1242 since 1995. According to IWE-1242, augmented inspection is required of areas exposed to standing water, repeated wetting and drying, and persistent leakage.

3. Provide a summary of the liner plate degradation, including loss of liner plate thickness due to corrosion, integrity of leak chase channels and condition of moisture barrier, as observed during the most recent inspections of Unit 1 and 2 containments.

4. Provide detailed future plans for determining corrective actions, including commitments and completion schedules, for addressing steel liner plate corrosion and moisture barrier deterioration in Unit 1 and 2 containments.

The staff needs the above information to confirm that the effects of aging of the containment pressure boundary metal will be adequately managed so that it's intended function will be maintained consistent with the current licensing basis for the period of extended operation as required by 10 CFR 54.21(a)(3).

PSEG Response:

1. During the most recent Salem Unit 1 outage, in the spring of 2010, no active leakage from the reactor cavity and fuel transfer canal telltales was observed.

   During the most recent Salem Unit 2 outage, in the fall of 2009, a 60 drip per minute leak of borated water was observed at the fuel transfer canal telltale, above the door to the Letdown Heat Exchanger Room. Borated water was observed on the containment liner plate moisture barrier under the fuel transfer canal. These leaks were attributed to reactor cavity leakage. The containment liner plate and moisture barrier were examined and found to meet the IWE acceptance criteria.

   There were other leaks identified during walkdowns of the Salem Unit 1 and Unit 2 Containments as part of the Boric Acid Corrosion program, but these other leaks were small and localized, such as at a pipe cap, and were generally inactive, so that there was no impact on the Containment.

2. Salem began implementation of containment inservice inspection (CISI) in accordance with ASME Section XI, Subsection IWE, as mandated by 10 CFR Part 50.55a in April 2000. Since that time, 100% of accessible surface areas of the Salem Unit 2 containment liner plate was examined each Inspection Period of the 1st CISI Interval in accordance with IWE-3500. The IWE program and examinations identified no surface areas of the containment liner plate that require augmented examinations as specified in IWE-1242. The moisture barrier was considered inaccessible and was not inspected prior to 2009. In 2009, Salem implemented a change to make the moisture barrier accessible, to the extent possible, and performed visual inspections of the moisture barrier in accordance with IWE. The 2009 containment liner plate examinations identified areas that require augmented examination. These augmented examination areas have been identified for inclusion in the Salem plan for the 2nd CISI Interval, which started in April of 2010.

   Prior to April 2000, inspection of the containment was performed under the Structures Monitoring Program in accordance with 10 CFR 50.65 and 10 CFR Part 50, Appendix J. Augmented Examination requirements of IWE-1242 did not apply.

3. A summary of the containment liner plate degradation, including loss of containment liner plate thickness due to corrosion, integrity of containment liner leak chase channels and condition of the moisture barrier, as observed during the most recent inspections of the Unit 1 and Unit 2 Containments, is summarized below. This degradation was found as a result of Enhancement #2 described in the LRA, Appendix B.2.1.28 (ASME Section XI, Subsection IWE), as shown on page B-132.

   This enhancement, which involved trimming the stainless steel lagging at the bottom of the containment liner insulation panels, has been implemented at both Unit 1 and Unit 2. As a result, areas that were previously inaccessible for inspection have been made accessible, examinations have been performed, and evaluations have verified the adequacy of existing conditions. Additionally, corrective actions to address degraded conditions found during the most recent inspections have been developed.

Unit 1:
The following examination results were obtained during the refueling outage in the spring of 2010.

Containment Component	Location	Examination Results
Liner plate - 3/4" knuckle plate	The knuckle plate connects the 1/4" liner plate below the floor to the 1/2" wall liner plate. The plate extends from 2 feet below the concrete floor to approximately 6" above the floor. See UFSAR Fig. 3.8-7	Some local corrosion was observed in the area above the floor. As a result, UT readings were taken at 1 foot intervals around the containment, except where interferences near the containment liner plate impeded UT testing, for a total of 349 readings. Minimum thickness measured after cleaning was 0.721". The thickness utilized in the containment analysis is 1/2". All readings met acceptance criteria for loss of material less than 10% of the thickness in the analysis.
Liner plate - 1/2" plate	The bottom starts at the top of the knuckle plate and extends to approximately 22 feet above the floor. See UFSAR Fig. 3.8-1 and 3.8-7	Four containment liner plate insulation panels were removed where corrosion was observed in the adjacent accessible areas. Visual examination of the containment liner plate covered by the insulation panels identified local surface corrosion at each of the four panels and local paint blistering at one of the panels. As a result, 102 UT thickness readings were taken at the four panels in the area of the surface corrosion. An additional 18 UT thickness readings were taken in the area of the blistered paint. The minimum thickness from UT measurement results was 0.452". All readings met acceptance criteria for loss of material less than 10% of the nominal thickness.
Vertical leak chase channels	Extends down from the lowest horizontal leak chase channel into and under the containment floor.	All of the accessible vertical leak chase channels were examined. One channel had corrosion that extended through the channel wall (hole). The leak chase channel with the hole was cleaned out and the channel and containment liner plate were visually examined with a boroscope beneath the containment floor. Only surface scale was observed and no evidence of significant moisture or loss of material was noted. The channel with the hole was cut at the floor and capped to prevent moisture intrusion.

Containment Component	Location	Examination Results
Moisture Barrier	At the containment liner plate to concrete floor interface at elevation 78'.	100% of the moisture barrier area was inspected and repaired or replaced where it did not meet the IWE acceptance criteria.

Unit 2:
The following examination results were obtained during the refueling outage in the fall of 2009.

Containment Component	Location	Examination Results
Liner plate - 3/4" knuckle plate	The knuckle plate connects the 1/4" liner plate below the floor to the 1/2" wall liner plate. The plate extends from 2 feet below the concrete floor to approximately 6" above the floor. See UFSAR Fig.3.8-7	Some local corrosion was observed in the area above the floor. As a result, UT readings were taken at 1 foot intervals around the Containment, except where interferences near the containment liner plate impeded UT testing, for a total of 368 readings. Minimum thickness measured before cleaning was 0.677". The thickness utilized in the containment analysis is 1/2". All readings met acceptance criteria for loss of material less than 10% of the thickness in the analysis.
Liner plate - 1/2" plate	The bottom starts at the top of the knuckle plate and extends to about 22 feet above the floor. See UFSAR Fig. 3.8-1 and 3.8-7	Seven UT measurements were taken on the 1/2" containment liner plate, just above the 3/4" knuckle plate due to an interference at the knuckle plate. The measured thickness was greater than the nominal thickness of 1/2". The minimum thickness measured was 0.509". In addition, containment liner insulation was removed at four panel locations in suspect areas as described in the response to RAI B.2.1.28-2, item #1. Some local surface corrosion was observed at all four of the panels. Some local paint blistering was observed at one of the panels. A total of 4 UT readings were taken at the one insulation panel with the paint blisters. After cleaning of indications, minimum thickness from UT results was 0.518". All readings met acceptance criteria for loss of material less than 10% of the nominal thickness.

Containment Component	Location	Examination Results
Vertical leak chase channels	Extends down from the lowest horizontal leak chase channel into and under the containment floor.	All of the accessible vertical leak chase channels were examined. Six channels had corrosion that extended through the channel wall (hole). The leak chase channels with the holes were cleaned out to the extent possible. The channel and containment liner plate were visually examined with a boroscope beneath the containment floor. The six channels with the holes were cut at the floor and capped to prevent moisture intrusion.
Moisture Barrier	At the containment liner plate to concrete floor interface at elevation 78'.	100% of the accessible moisture barrier was inspected and found to have performed its intended function but degradation was noted. A short segment of the moisture barrier was removed in an area with significant corrosion of the 3/4" knuckle plate above the moisture barrier, where the corrosion was suspected to occur below the moisture barrier. The moisture barrier was removed to a depth of approximately 1". Some corrosion of the 3/4" knuckle plate was noted below the surface of the moisture barrier at the floor level but the corrosion of the 3/4" knuckle plate did not extend below the portion of the moisture barrier that was removed. The 3/4" knuckle plate met the IWE acceptance criteria.

4. Degradation was found as a result of implementation of Enhancement #2 to the IWE program as described in the LRA, Appendix B.2.1.28 (ASME Section XI, Subsection IWE), as shown on page B-132. As a result, areas that were previously inaccessible for inspection have been made accessible, examinations have been performed, and evaluations have verified the adequacy of existing conditions as described in item #3 of this RAI. Some corrective actions have been completed and additional corrective actions have been specified, as described below.

Unit 1 - corrective actions completed during the refueling outage in the spring of 2010:

- Examination of 100% of the accessible 1/2" containment liner plate and moisture barrier.
- UT measurements of the 3/4" containment liner (knuckle plate) around the perimeter of the Containment.

- UT measurements of the 1/2" containment liner plate where insulation panels were removed and loss of material was observed.
- Coating repairs of the 3/4" containment liner (knuckle plate).
- The one vertical leak chase channel with a hole was capped.
- Coating repairs at areas where containment liner insulation panels were removed to allow for containment liner plate inspection and corrosion was observed.
- The moisture barrier was repaired or replaced.
- Evaluation to confirm the identified loss of material is acceptable.

Unit 1 - additional corrective actions to be completed prior to the period of extended operation:

- Perform augmented examinations of the 3/4" containment liner (knuckle plate) at 78' elevation in accordance with IWE-2420.
- Perform augmented examinations of the 1/2" containment liner plate behind insulation panels, where loss of material was previously identified, in accordance with IWE-2420.
- Remove 1/2" containment liner insulation panels, adjacent to accessible areas where there are indications of corrosion, to determine the extent of condition of the existing corroded areas of the containment liner plate.

Unit 2 - corrective actions completed during the refueling outage in the fall of 2009:

- Examination of 100% of the accessible 1/2" containment liner plate and moisture barrier.
- UT measurements of the 3/4" containment liner (knuckle plate) around the perimeter of the Containment.
- UT measurements of the 1/2" containment liner plate where insulation panels were removed and loss of material was observed.
- The six vertical leak chase channels with a hole were capped.
- Evaluation to confirm the identified loss of material is acceptable.

Unit 2 - additional corrective actions to be completed prior to the period of extended operation:

- Examine the accessible 3/4" containment liner (knuckle plate). If corrosion is observed to extend below the surface of the moisture barrier, excavate the moisture barrier to sound metal below the floor level and perform examinations as required by IWE.
- Perform remote visual inspections, of the six capped vertical leak chase channels, below the containment floor to determine extent of condition.
- Remove the concrete floor and expose the 1/4" containment liner plate (floor) for a minimum of two of the vertical leak chase channels with holes. Perform examinations of exposed 1/4" containment liner plate (floor) as required by IWE. Additional excavations will be performed, if necessary, depending upon conditions found at the first two channels.

- Remove 1/2" containment liner insulation panels, adjacent to accessible areas where there are indications of corrosion, to determine the extent of condition of the existing corroded areas of the containment liner plate.
- Perform augmented examinations of the 1/2" containment liner plate behind insulation panels, where loss of material was previously identified, in accordance with IWE-2420.
- Examine 100% of the moisture barrier in accordance with IWE-2310 and replace or repair the moisture barrier to meet the acceptance standard in IWE-3510.

Examinations and inspections will be performed in accordance with IWE-2000 and the acceptance standards will be in accordance with IWE-3500.

Updates to formalize these Salem commitments, to complete the future corrective actions described above, are made to LRA Appendix A, Section A.5, the License Renewal Commitment List, under line number 28, commitment #2, which can be found in Enclosure C.

RAI B.2.1.28-2

Background:

GALL Report (NUREG-1801), AMP XI.S1, "ASME Section XI, Subsection IWE," Program Element 1, requires inspection of steel containment components including liners, liner anchors, and integral attachments for loss of material due to general, pitting, and crevice corrosion. Inservice inspection (ISI) requirements of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code, Section XI, Subsection IWE for steel containments (Class MC) and steel liners for concrete containments (Class CC) are imposed by 10 CFR 50.55a.

Issue:

Program Element 10 for the Salem ASME Section XI, Subsection IWE aging management program discusses sampling inspections of normally inaccessible areas of steel liner plate located behind the insulation panels around the lower 30 feet of the Unit 2 containment completed in 2009. Similar inspections are scheduled for Unit 1. However, details of the sampling methodology used for the inspection is not described in the LRA and program basis document.

Request:

1. Describe the sampling methodology used in 2009 inspection to select the locations for inspecting containment liner plate and moisture barrier behind the insulating panels.

2. The sampling methodology planned for future inspections. Would the sampling methodology provide a statistical confidence level of at least 95% that the results of inspections will meet the acceptance criteria of IWE 3500.

The staff needs the above information to confirm that the effects of aging of the containment pressure boundary metal will be adequately managed so that it's intended function will be maintained consistent with the current licensing basis for the period of extended operation as required by 10 CFR 54.21(a)(3).

PSEG Response:

1. Random sampling was not used in 2009 to select the locations for inspecting the containment liner plate and the moisture barrier behind the containment liner insulation lagging. During the Salem Unit 2 outage in the fall of 2009, the bottom edge of the containment liner insulation lagging was trimmed to allow for visual inspection of 100 % of the liner at the juncture of the containment concrete floor and the moisture barrier; except for areas where access is restricted by permanent plant structures, equipment, or components. Salem recognized it is prudent to make this area accessible for visual examination prior to and during the period of extended operation to resolve concerns involving corrosion in this area.

   In addition to inspecting 100% of accessible containment liner plate and 100% of the moisture barrier, one containment liner insulation panel was removed in each quadrant, for a total of four containment liner insulation panels, to evaluate the acceptability of inaccessible areas covered by containment liner insulation. Selection of the containment liner insulation panels was based on the presence of and the extent of corrosion in accessible areas below the containment liner insulation panels, staining on the containment liner insulation panels, or staining at the floor below containment liner insulation panels. Visual inspection of exposed, inaccessible containment liner plate, during the 2009 Salem Unit 2 examinations, indicated only minor surface corrosion. Ultrasonic testing (UT) thickness measurements conducted at the insulation panel with the most loss of material showed that the measured containment liner plate thickness was greater than the nominal thickness of 0.5 inches.

2. In LRA Appendix A, Section A.2.1.28 (ASME Section XI, Subsection IWE), Enhancement #1, Salem committed to enhance the ASME Section XI, Subsection IWE, aging management program to require inspections of a sample of the inaccessible containment liner covered by containment liner insulation and lagging prior to the period of extended operation and every 10 years thereafter. The commitment is clarified as outlined below.

   Prior to the period of extended operation (PEO)

   - A sampling plan will be developed based upon guidance in EPRI TR-107514, “Age Related Degradation Inspection Method and Demonstration: in Behalf of Calvert Cliffs Nuclear Power Plant License Renewal Application”.
   - The population size of containment liner insulation panels in each unit is approximately 264 panels. A sample size of 57 will meet the statistical requirements of a 95% confidence level that 95% of the containment liner plate behind the containment liner insulation meets the acceptance criteria of IWE-3500.
   - The samples will be randomly selected.
   - The examination will be performed by either removing the containment liner insulation panels and performing a visual inspection, or by using a pulsed eddy current (PEC) remote inspection, with the containment liner insulation left in place, to detect evidence of loss of material. If evidence of loss of material is detected using PEC, the containment liner insulation panel will be subsequently removed to allow for visual and UT examinations.

- If acceptance criteria defined in IWE-3500 is not satisfied, the sampling plan will be modified as recommended in EPRI TR-107514.

During the period of extended operation

During the PEO, a reduced sample size will be randomly selected and examined each Containment Inservice Inspection Period contingent upon satisfactory results of the sample examined prior to the PEO.

- One containment liner insulation panel will be selected, at random, for removal from each quadrant, during each of the three Periods in an Inspection Interval. Therefore, a total of 12 containment liner insulation panels will be selected, in each unit, during each ten year Inspection Interval, to allow for examination of the containment liner behind the containment liner insulation.
- The randomly selected containment liner insulation panels in each quadrant will not include containment liner insulation panels previously selected.

The sampling plan during the PEO is considered to provide reasonable assurance that the containment liner plate, in the inaccessible areas behind the Containment liner insulation, will meet the acceptance criteria in IWE-3500 and the containment liner plate will perform its intended function during the PEO. This is based upon the evidence to date from inspections performed behind the containment liner plate, where all examinations met the acceptance criteria, and the enhanced inspections prior to the PEO described above.

Updates to LRA Appendix A and Appendix B as a result of the clarification of the enhancement can be found in Enclosure B. Updates to LRA Appendix A, Section A.5, the License Renewal Commitment List, under line number 28, commitment #1, can be found in Enclosure C.

RAI B.2.1.29-1

Background:

GALL Report, Section XI.S2, Element 6 states that ASME Section XI, Subsection IWL, Article IWL-3000 provides acceptance criteria for concrete containments. The GALL Report further states that quantitative acceptance criteria based on the "Evaluation Criteria" provided in Chapter 5 of ACI 349.3R may also be used to augment the qualitative assessment of the responsible engineer. Salem Generating Station Units 1 and 2, document SA-PBD- AMP-XI.S2, Rev. 2, Section 3.6 also states that quantitative acceptance criteria, developed based on Chapter 5 of ACI 349.3R, are included in the program implementing documents to augment the qualitative assessment by the responsible engineer.

Issue:

A review of the Salem Units 1 and 2 records indicate that IWL inspections performed in 2005, 2007, and 2009 indicate that Section 5.4 of S-C-CAN-SEE-1353, Rev. 0, "Acceptance Criteria for Containment Concrete Defects", has been used by the applicant for inspection of Salem Units 1 and 2 containment concrete surface examinations. According to this document, the acceptance criteria for concrete surfaces is significantly different and less stringent from the acceptance criteria specified in Section 5.1 of ACI 349.3R.

In addition, Notification 000020234570 describes the actual condition of the concrete on the north side of the Unit 2 containment involving surface spalling ranging up to 6 ft long and 16 inch wide, and spalling at joints that is up to 3 ft long and 4 in. wide. Notification 000020234570 also describes a condition on the north side of the containment between the equipment hatch and the fuel handling penetration area involving the protrusion of a pipe from the penetration wall. The applicant did not describe the purpose for the pipe, but the applicant reported that the pipe is broken at the flange. The notification also describes a piece of wood (1 in. by 8 in. by 4 in.) protruding from the penetration wall in the main steam area.

Request:

The applicant is requested to provide the following information:

1. The basis for the acceptance criteria in Section 5.4 of S-C-CAN-SEE-1353, Rev. 0, including the reasons for it being significantly less stringent than the ACI 349.3R requirements.

2. Provide information about broken pipe and flange protruding from the containment surface, and its impact on the containment leak tightness.

3. Confirm that the piece of wood (1 in. by 8 in. by 4 in.) is not embedded in the concrete containment wall.

4. Details of corrective actions that the applicant plans to implement for using the acceptance criteria described in Section 5.4 of S-C-CAN-SEE-1353, Rev. 0 which does not conform with the current industry practice and ACI 349.3R.

The staff needs the above information to confirm that the effects of aging of the concrete containment will be adequately managed so that it's intended function will be maintained consistent with the current licensing basis for the period of extended operation, as required by 10 CFR 54.21(a)(3).

PSEG Response:

1. S-C-CAN-SEE-1353 is no longer an active document in the Salem document control system. The acceptance criteria in Section 5.4 of S-C-CAN-SEE-1353 were never used since the containment concrete conditions never warranted using the acceptance criteria in Section 5.4 of S-C-CAN-SEE-1353. The Salem ASME Section XI, Subsection IWL program, examination procedures now use guidance provided in ACI 349.3R.

2. Notification 000020234570 describes a condition on the north side of the Salem Unit 2 Containment between the equipment hatch and the fuel handling penetration area involving the protrusion of a pipe from a wall. This pipe actually protrudes from a wall extending outward from the Fuel Handling Building. The wall extending outward from the Fuel Handling Building encloses the space between the Fuel Handling Building and the Containment. The pipe does not protrude from the Containment wall. Therefore, there is no impact on Containment leak tightness.

3. Notification 000020234570 describes a piece of wood (1 in. by 8 in. by 4 in.) that is not embedded in any concrete and is not touching the Containment. The piece of wood is wedged between miscellaneous steel and the mechanical penetration area wall of the Auxiliary Building, near the Containment wall. This piece of wood has no impact on containment integrity.

4. Corrective actions were initiated as a result of differences between the acceptance criteria provided in Section 5.4 of S-C-CAN-SEE-1353, Rev. 0, which does not conform with the current industry practice described in ACI 349.3R.

   S-C-CAN-SEE-1353 is no longer an active document in the Salem document control system and the acceptance criteria described in Section 5.4 of S-C-CAN-SEE-1353 is no longer being used. The ASME Section XI, Subsection IWL, aging management program has been enhanced to reflect the guidance provided in ACI 349.3R, which is now being used for the examination of the Salem Unit 1 and Unit 2 containments. This is a new enhancement to the ASME Section XI, Subsection IWL, aging management program which improves characterization of Containment degradation and revises the Containment inspection acceptance criteria. This will allow for more effective trending of degradation for future inspections.

   Site procedures have been revised to remove references to S-C-CAN-SEE-1353 and revise the acceptance criteria to reflect the ACI 349.3R acceptance criteria, as well as to add references to ACI 349.3R acceptance criteria, and consideration of long term aging management.

Concrete inspections for both Salem Unit 1 and Unit 2 Containments have recently been completed in April of 2010 using the ACI 349.3R tiered acceptance criteria. Examination results have been reviewed by the site Responsible Professional Engineer and satisfactorily met all ACI 349.3R acceptance criteria.

Updates to LRA Appendix A and Appendix B as a result of the enhancement can be found in Enclosure B. Updates to LRA Appendix A, Section A.5, the License Renewal Commitment List, under line number 29, can be found in Enclosure C.

RAI B.2.1.29-2

Background:

GALL Report (NUREG-1801), AMP XI.S2, "ASME Section XI, Subsection IWL," Program Element 10 states that implementation of ASME Section XI, Subsection IWL, in accordance with 10 CFR 50.55a, is a necessary element of aging management for concrete containments through the period of extended operation.

Issue:

Program Element 10 for the Salem ASME Section XI, Subsection IWL aging management program describes results of concrete inspections conducted in April 2001 and October 2005 for Unit 1, and November 2000, May 2005, and August 2009 for Unit 2. In addition to isolated areas of physical damage to concrete surfaces on both units, normal shrinkage cracking was also observed. Salem Units 1 and 2 containments are constructed from reinforced (non-prestressed) concrete; therefore, cracking of the concrete in some areas is likely and is expected over the 60-year operating life. In Notification 000020234570, the applicant reported cracks in the concrete coating over the entire outside of the Unit 2 containment. Long-term exposure of concrete cracks to salt spray originating from the Delaware Bay could result in corrosion of the embedded steel reinforcing bars located nearest to the outer surface of the containment concrete during the extended period of operation.

Request:

The applicant is requested to provide the following:

1. The extent and maximum width of the cracks observed in Salem Unit 1 and 2 containments.

2. Actions that are planned to mitigate the consequences of chloride ion penetration to the level of the embedded steel reinforcing bars over the period of extended operation. This may be necessary since the Salem Units 1 and 2 concrete containment surface inspection reports documented scaling and spalling of up to 3 inches.

3. If no actions are anticipated to mitigate the consequences of chloride ion penetration to the level of the embedded steel reinforcing bars, the applicant is requested to provide an assessment of this time-dependent phenomenon and the basis for this decision.

The staff needs the above information to confirm that the effects of aging of the concrete containment will be adequately managed so that it's intended function will be maintained consistent with the current licensing basis for the period of extended operation, as required by 10 CFR 54.21(a)(3).

PSE&G Response:

1. Concrete inspections for both Salem Unit 1 and Unit 2 Containment structures were completed in April 2010 using the ACI 349.3R tiered acceptance criteria. Examination results have been reviewed by the site Responsible Professional Engineer, and found acceptable, meeting ACI 349.3R acceptance criteria. These results, including extent and maximum width of cracks, are summarized below:

   General:
   The overall crack patterns are very similar for both Unit 1 and Unit 2 containments. The concrete surfaces exhibit general pattern cracking over the entire surface as well as cracking at random areas of mortar patches. The mortar patches were originally applied at the construction joints. Minor degradation of the mortar patches was noted.

   Cylindrical walls:
   Pattern cracking on about a 15" by 15" grid is evident over most of the cylindrical walls, with crack widths of about 0.015".

   Dome:
   There is similar pattern cracking on the tops of the Unit 1 and Unit 2 domes. The crack widths across most of the domes are about 0.015" wide with some areas at the top having cracks up to 0.040" wide.

   Maximum width and extent of cracking:
   - At the Unit 2 Containment, El. 130' airlock, some of the cracks were 0.0625" wide at the surface.
   - At the Unit 1 Containment, inside the Penetration area, above the floor at elevation 78', a circumferential crack 0.032" wide was noted.

   Comparison to original structural integrity tests:
   The above conditions were compared to those found during the original start-up structural integrity tests. The cracks are characterized as passive and inactive. The extent of the cracking and maximum crack widths is expected and consistent with the crack patterns exhibited following the original start-up structural integrity tests. Widening of cracks at the surface was identified and evaluated as part of the original structural integrity tests and accepted as a shallow, surface condition that was acceptable.

2. Salem will continue to monitor the condition of Unit 1 and Unit 2 containment concrete surfaces for spalling, scaling, cracking, and rust stains which are indicative of reinforcing bars corrosion. The monitoring activities will be in accordance with the applicable edition and addenda of ASME Section XI, Subsection IWL, as approved in 10 CFR 50.55a and recommended in the GALL Report, Rev. 1. Inspection and acceptance criteria will be in accordance with ACI 349.3R as described in LRA Section B.2.1.29. If acceptance criteria specified in ACI 349.3R for spalling, scaling, and cracking cannot be met, corrective actions will be implemented as required by the corrective action process to address corrosion of reinforcing bars. These

actions may include mitigative measures, such as repairs to scaled and spalled areas of concrete, and sealing of cracks to minimize penetration of chloride ions. If corrosion staining of reinforcing steel is observed on containment concrete surfaces, an engineering evaluation will be conducted to assess the condition of reinforcing bars and the impact of rebar corrosion on containment structural integrity. As described in the response to item 1 above, the Unit 1 and Unit 2 concrete containment surfaces were not spalled up to 3 inches, but rather had minor scaling and spalling. Therefore, there is currently no need for specific mitigative actions to prevent the potential of chloride ion penetration to the level of embedded reinforcing bars.

3. Chloride ions are common in nature and small amounts can be unintentionally contained in the concrete mixture ingredients. Potential external sources of chlorides include exposure to seawater or spray, deicing salts, or those from accelerating admixtures. The penetration and diffusion of chloride ions in concrete and their impact on reinforced concrete has been a subject of tests and studies as documented in the American Concrete Institute (ACI) ACI 222R, *"Protection of Metals in Concrete Against Corrosion"*, ACI 349.3R, *"Evaluation of Existing Nuclear Safety-Related Concrete Structures"*, ACI 365.1R, *"Service Life Prediction---State-of-the-Art Report"*, NUREG/CR-6927, *"Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures – A Review of Pertinent Factors"*, and other literature. NUREG/CR-6927 also includes the results of concrete condition surveys in ten nuclear power plants. The review of ACI 222R, ACI 349.3R, ACI 365.1R, and NUREG/CR-6927 indicates that chloride ion penetration and diffusion in concrete depends on concrete durability and serviceability which have been incorporated into codes such as ACI 318, *"Building Code Requirements for Structural Concrete"*. Durability of concrete has been included in ACI 318 through specification of maximum water-cement ratios, cement content, type of cement, entrained air, minimum cover over the reinforcing bars, and control of cracks. The Salem Containments are constructed of concrete that conforms to the applicable ACI 318 requirements. The minimum concrete clear cover over the reinforcing bars shown on the design drawings is $3^3/_8$ inches nominal which is greater than the 2 inch cover required by ACI 318 for concrete exposed to weather. Recent examinations of Unit 1 and Unit 2 Containment concrete surfaces using procedures that are based on ACI 349.3R inspection and acceptance criteria identified only minor spalling and scaling, but none that reduce the concrete cover over the reinforcing bars below the 2 inches required by ACI 318. Cracking is minor as described in the response to the item 1 of this RAI. In addition, the Containment concrete is observed to be free of large penetrating cracks that could permit significant chloride ion penetration to reach the level of reinforcing bars.

The primary concern associated with penetration and concentration of chloride ions over time is that it can lead to corrosion of the reinforcing bars. Reinforcing bars with adequate concrete cover should not be susceptible to corrosion because the highly alkaline conditions present within concrete cause a passive iron-oxide film to form on the surface of the reinforcing bars. Chloride ions, however, can destroy this passive film and initiate corrosion. Corrosion of reinforcing bars (i.e., the transformation of metallic iron to ferric oxide, or rust) is accompanied by an increase in volume of the metallic iron. The volume increase can cause cracking, spalling, and delamination of the concrete that can be visible in the form of loss of concrete material, rust spots and stains, and cracks in the concrete cover along the reinforcing bars. Visual inspection required to be conducted in accordance with ASME Section XI, Subsection IWL, would detect such conditions before the loss of containment intended function. To date, corrosion of containment concrete reinforcing bars has not been identified as a concern by Salem or industry operating experience.

In summary, chloride ion penetration and diffusion in concrete has been a subject of extensive research and studies. The extent of the penetration and diffusion depend on concrete durability, permeability, and cracking, which have been considered in concrete design codes and standards. Salem conforms to the applicable concrete codes and standards. In the event this time dependent phenomenon penetrates to the level of reinforcing bars and initiates corrosion, the increase in volume of the steel due to the creation of rust will result in spalling, cracking, delamination of concrete, and staining of concrete surfaces. Implementation of ASME Section XI, Subsection IWL, aging management program described in LRA B.2.1.29 is considered to provide reasonable assurance that these aging effects will be detected and corrective actions will be taken prior to the loss of the Containment intended function.

RAI B.2.1.33-1

Background:

NRC Information Notice 2004-05, "Spent Fuel Pool Leakage to Onsite Groundwater," notes that leakage of the spent fuel pools has occurred at Salem Unit 1.

Issue:

The licensee at Salem NGS in 2002 identified evidence of radioactive water leakage through the interior wall of the Unit 1 auxiliary building mechanical penetration room. In the years since initial startup, materials such as boric acid and minerals have accumulated in the leak collection and detection system that restricted normal drainage of fluid. Modification of the tell-tale drains that are used to detect, monitor, and quantify potential leakage from the spent fuel pool liner resulted in inadvertent further restriction of free drainage of leakage from the liner that resulted in accumulation of borated water between the liner and concrete and migration to other locations through penetrations, construction joints, and cracks. The seismic gap was confirmed to contain water with radionuclides characteristic of the Unit 1 spent fuel pool water and leakage into the seismic gap has continued. Leakage into the tell-tale drains is occurring at a rate of about 100 gallons per day.

Request:

a. Provide historical data on the leakage occurrence and volume, and available information from chemical analysis performed on the leakage.
b. Provide a summary of the root cause analysis that was used to identify the source of leakage through the liner that has resulted in accumulation of borated water between the liner and concrete, including information on the path of the leakage and structures that could potentially be affected by the presence of the borated water.
c. Discuss plans for remedial actions or repairs to address leakage through the spent fuel pool liner. In the absence of a commitment to fix the leakage prior to the period of extended operation, explain how the structures monitoring program, or other plant-specific program, will address the leakage to ensure that aging effects, especially in inaccessible areas, will be effectively managed during the period of extended operation.
d. Provide background information and data to demonstrate that the concrete and embedded steel reinforcement have not been degraded by exposure to the borated water and that the liner will not be impacted. If experimental results will be used as part of the assessment, provide evidence that the test program is representative of the materials and conditions that exist in the region between the spent fuel pool liner and concrete. This information should also include the MPR Associates report that documents the details of the tests performed and evaluation of SNP spent fuel pool concrete and rebar.
e. If a concrete sampling program (e.g., obtaining concrete cores in region affected) cannot be implemented, please explain why this is not feasible.

The staff needs the information to confirm that the potential effect of aging of the spent fuel pool reinforced concrete, liner, and steel reinforcement due to presence of borated

The staff needs the information to confirm that the potential effect of aging of the spent fuel pool reinforced concrete, liner, and steel reinforcement due to presence of borated water will be adequately managed so that the intended function of impacted structural members will be maintained consistent with the current licensing basis for the period of extended operation as required by 10 CFR 54.21(a)3.

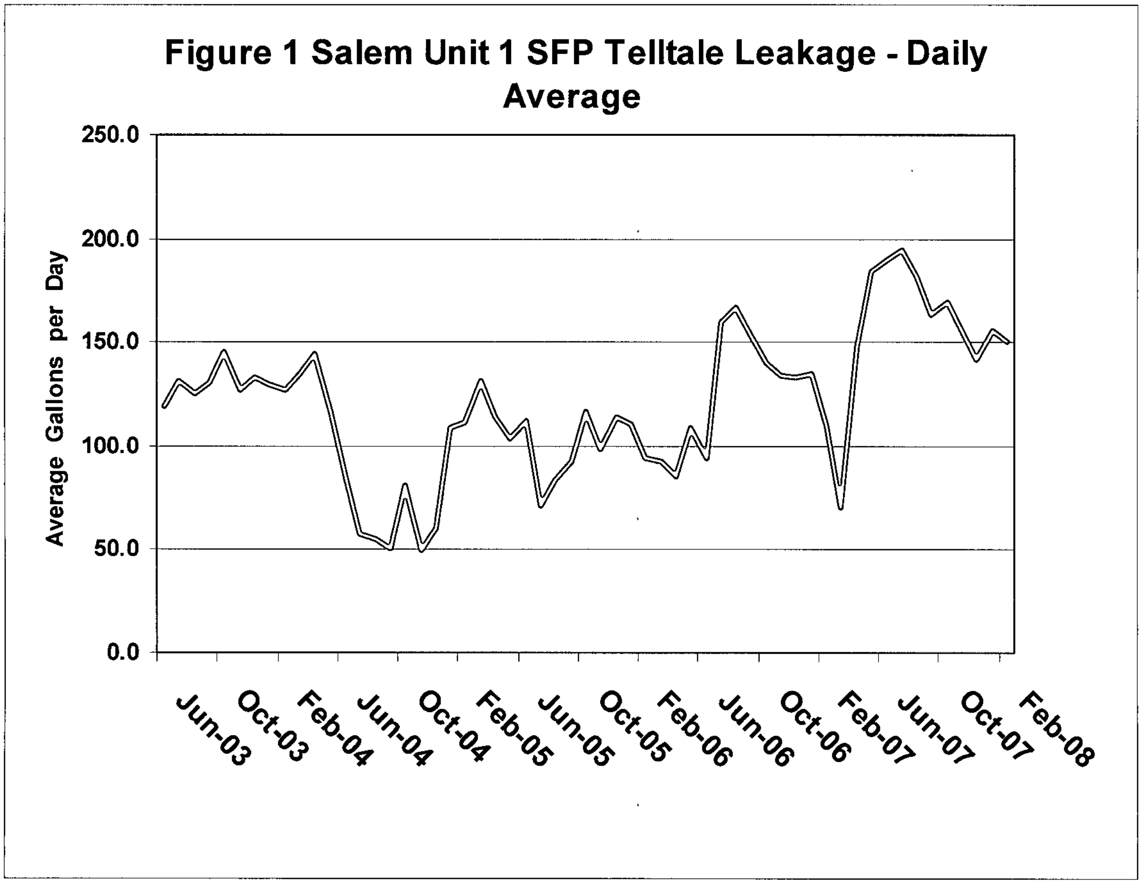
PSEG Response:

a. **Unit 1 History:**
The Unit 1 Spent Fuel Pool (SFP) leakage consists of leakage through the leak chase system drains (telltales) and leakage through concrete cracks and construction joints in the Fuel Handling Building (FHB) walls.

In 1980 a small leak was discovered in the Spent Fuel Pool (SFP) telltale drains. Underwater inspections determined that the cause was due to leaking seam welds. The identified leaking seam welds were repaired. After the repairs were implemented, the observed leakage was reduced to less than 0.2 gallons per day (gpd).

In 2002, Salem identified evidence of active water leakage through an exterior wall of the Unit 1 Auxiliary Building (AB) mechanical penetration room. Chemical analysis of the water verified the water was consistent with borated SFP water. Further investigations revealed that the SFP telltales were blocked. The blockage resulted in water accumulation behind the SFP liner and ultimately to migration of leakage through construction joints and small cracks in the Fuel Handling Building walls. The escaped water accumulated in the seismic gap between the FHB and AB and reached groundwater within the plant controlled area. Corrective actions were initiated to remove blockage in the telltales and drainage system. Analysis of the removed material showed that the deposits were largely quartz (SiO_2) and calcite ($CaCO_3$) with minor amounts of gismondine ($CaAl_2Si_2O_8.4H_2O$). These materials are consistent with FHB concrete. In addition the pH of water collected from the telltales after the removal of the blockage was 7.1 which is consistent with expected values for water in contact with concrete. The expected pH of SFP water not in contact with concrete should be equivalent to SFP chemistry which is 4.6.

In 2003, Salem began monitoring and trending leakage through the telltales. The volume of leakage varies as indicated in Figure 1 below but on an average is approximately 100 gallons per day. This figure shows average daily leakage up to February, 2008. Based on sump pump run times, daily telltale leakage rates have remained approximately the same for 2008, 2009 and early 2010. The trends of leakage through the telltales are used as an indicator that corrective actions are required to maintain proper drainage through the leak chase system, thus, preventing buildup of borated water between concrete and the liner.



Unit 2 History:
During an inspection of the Salem Unit 2 SFP in April of 2010, evidence of active leakage was identified from the telltale lines. Based on the inspection results, it was determined that for Unit 2 while there is leakage, the actual volume is on the order of a gallon per day and as such the use of measuring the sump pump run time is too small to provide meaningful quantification. In most instances the Unit 2 leakage actually evaporates before causing the sump pump to run.

The evidence obtained from the leakage identified on Unit 2 is consistent with the historical information derived from Unit 1 with respect to chemistry, radionuclide content, and pH information. These data suggest that the leakage from Unit 2 has been identified at an early stage of discovery and exhibits similar conditions that historically occurred at Salem Unit 1. Boroscopic inspection of the telltales in April 2010 showed that the telltales were open, however, based on the evidence that Unit 2 does have a relatively small but active leak, monitoring and trending will continue. In addition, telltales cleaning will be performed as necessary to ensure that the telltales remain open.

b. The evaluation of the leakage source for Unit 1 is documented in a focused self-assessment performed by Salem in 2003. Salem has since monitored and trended the leakage. The following is a summary of the self-assessment and the leakage monitoring.

Leakage Source:
Salem has concluded that leakage through the SFP liner is occurring in many small cracks in seam welds or plug welds located throughout the SFP liner. These cracks are too small to be readily identified and located. This leakage enters channels behind the SFP liner either directly from cracks in seam welds or indirectly by migrating over concrete from cracks in plug welds. The leakage drains to the channels then to 1 inch telltales and eventually to a sump.

Below are key points that led to the above conclusion.

- Chemical and isotopic analyses of the telltale leakage indicate the presence of boron and gamma activity at concentrations consistent with the SFP water.

- Isotopic analysis of the water discharged from the telltales after they were initially cleaned in 2003 was consistent with the SFP except it had aged several years suggesting that the leakage had been trapped between the SFP liner and concrete for some time.

- The leakage rate from the telltales varies, but is on the order of 100 gallons per day. Salem Engineering estimates this leakage rate could be a result of a single or multiple cracks 0.001 inch wide and 6 inches long.

- The leakage is likely from small cracks in seam welds (2,100 linear feet) of adjoining liners plates or the plug welds (1,400 total) that connect the liner plates to the steel embedded in the surrounding concrete. This conclusion is supported by the observation of leakage in all of the telltale drains.

- The conclusion that some of the leakage is through plug welds is based on chemical analysis of the deposits that were blocking the telltales. Analyses showed that the blockage debris was formed by materials consistent with FHB concrete, which can only occur if plug welds are leaking.

- The backing bar for the seam welds is tied to the embedded leakage channels and the plug welds are tied to the embedded steel. Therefore, the cause of the cracks is postulated to be due to differential thermal expansion between the liner and the concrete structure.

Leakage Path:
The leak path in the Unit 1 SFP begins at liner seam welds or liner plug welds. Leakage through seam welds is immediately directed into leak chase system collection channels and does not contact concrete or any structural elements. Leakage through plug welds may accumulate between the concrete and the SFP liner and migrates to leakage collection channels. Leakage proceeds down the leakage collection channel to a telltale and drains into the sump collection system.

Some time between 1995 and 1998 the telltales became blocked and the leakage accumulated in the space between the SFP liner and the concrete wall. The hydrostatic head associated with this accumulation drove migration of the leakage through the SFP structure along cracks and construction joints. The leakage reached the seismic gap between the Fuel Handling Building and the Auxiliary Building and entered the Auxiliary Building through a wall sleeve. In 2003 corrective actions were initiated to remove blockage in the telltales and drainage system. In addition, Salem has installed a drain in the seismic gap which drains water to the Auxiliary Building, thus, minimizing leakage to the groundwater.

Potential Impact to Salem Structures:
The accumulation of leakage in the seismic gap can impact the Fuel Handling Building and the Auxiliary Building. A description of the impact on the FHB is provided in the response to item "d" below. An assessment of the impact of borated water accumulation in the seismic gap on the Auxiliary Building was performed in 2009. The assessment used the concrete degradation curve developed from testing in support of the FHB evaluation with corrections to account for the difference in boron concentration and lower temperatures. The assessment concluded that potential degradation of the Auxiliary Building from exposure to the borated water in the seismic gap is minimal.

c. There are currently no plans to perform repairs to the SFP liner seam welds or plug welds. Repairs were considered previously but determined to be impractical because the pool is nearing capacity of stored spent fuel making access challenging. Also locating the cracks using the existing underwater NDE technology has not been successfully demonstrated. In 1995 Salem inspected 95% of all seam welds and found no through wall cracks. This suggests the leakage is occurring below the sensitivity of the test.

   Salem continues to participate in and monitor industry activities to develop crack detection methodologies and repair methods. This includes EPRI, the Material Aging Institute, and Électricité de France (EdF) activities. In addition, Salem will continue to evaluate repair methodologies as they become available for potential implementation.

   Salem has conducted extensive laboratory testing to characterize the extent of concrete and rebar degradation as a result of exposure to borated water through the period of extended operation (PEO). The results of the tests were used to evaluate the impact of the leakage on the FHB structural integrity. The evaluation concluded the impact on structural integrity is not significant and that the FHB will continue to perform its intended function through the PEO.

As indicated in the Salem License Renewal Commitment List in LRA Appendix A, Section A.5, page A-68, line item number 33, Enhancement 5 a, b, and c, the long-term aging management strategy for addressing borated water leakage from the SFP includes the following actions related to the spent fuel pool liner:

5. Require the following actions related to the spent fuel pool liner:

   a. Perform periodic structural examination of the Fuel Handling Building per ACI 349.3R to ensure structural condition is in agreement with the analysis.

   b. Monitor telltale leakage and inspect the leak chase system to ensure no blockage.

   c. Test water drained from the seismic gap for boron concentration.

Note – see below response to part B.2.1.33-1e for updates to Enhancement 5.

d. Salem has completed an assessment of the long-term structural adequacy of the Unit 1 FHB reinforced concrete structure under potential prolonged exposure of the concrete and reinforcing steel to borated water which has leaked from the pool. The results of the assessment are documented in MPR Report 2613, "Salem Generating Station Fuel Handling Building-Evaluation of Degraded Condition," Revision 3. This report is provided in enclosure "D". Below are summaries of the assessment elements and conclusions.

FHB Concrete Degradation Testing:
The most severe degradation is most likely occurring behind the SFP liner at the bottom and sides of the pool. It is neither practical nor safe to obtain samples of the degraded concrete. Instead Salem has performed concrete testing to quantify the potential degradation of the FHB structure.

Beginning in 2003 a series of borated water degradation tests were conducted on core samples taken from the Salem Auxiliary Building and new concrete specimens that were made to the original Salem specifications using aggregate from the original sources. Concrete test specimens were submerged in a borated water bath for varying periods up to 39 months. The borated water bath testing parameters (borated water concentration and temperature) were maintained to be representative of the SFP water. The tests were conducted in a manner which was conservative with respect to the actual conditions expected in the FHB. Test specimens were removed from the bath at periodic intervals and degradation of the exposed concrete surface was measured. Results show that degradation is diffusion-controlled, which is consistent with published studies of weak acid attack of concrete. A curve fit of degradation versus time was generated using the measured degradation from test specimens. The following figure 2 shows the curve fit.

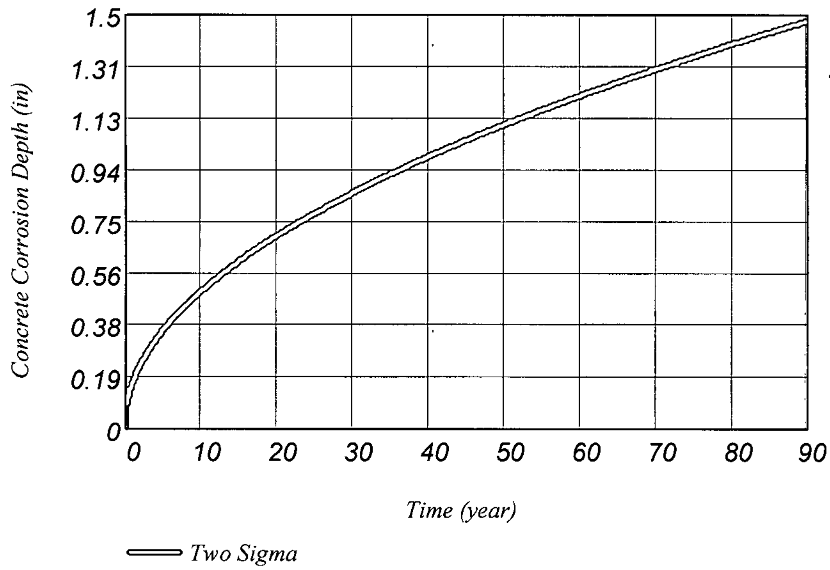


Figure 2 - Extrapolation of Degradation through End of Plant Life

Test Result Conclusions:

- The Borated water attacks the calcium hydroxide component of the cement paste causing loss of bonding of the coarse and fine aggregate.
- The predicted depth of penetration of concrete after 70 years exposure to spent fuel pool water at 100°F is 1.30 inches. This includes a two-sigma statistical uncertainty of the test data and an adjustment for temperature. Since the concrete clear cover for the walls and slab is greater than this projected degradation depth, the borated water will not reach rebar during the 70 year period.
- The wicking effect at the rebar/concrete interface was observed to be minor. That is, the degradation rate of the concrete at the rebar/concrete interface is similar to the general rate of attack of concrete without rebar. Therefore, degradation of rebar at the construction joints or cracks will not spread rapidly along the bond with the rebar. Rebar functionality is not compromised from a general loss of bond with the concrete.

Assessment Corroboration from the Connecticut Yankee SFP Cores:
In the fall of 2007 Salem learned that EPRI had possession of several samples taken from the Connecticut Yankee (CY) SFP during decommissioning. Salem and EPRI collaborated on evaluation of these concrete cores. The objective of the evaluation was to use actual CY plant observations to corroborate the assessment of the Salem FHB.

Overall, the CY cores corroborate the MPR results of testing and projections for Salem. The concrete used at CY is similar to that used at Salem in that the aggregates (coarse and fine) were inert with respect to borated water. The degraded concrete in the CY cores varied from less than 0.05 inch to 0.91 inch which demonstrates that the degradation depth can vary. Over the 37 year life of the CY Spent Fuel Pool, the concrete degradation reached a maximum depth of 0.91 inch. These results are consistent with the above degradation curve developed from the Salem concrete testing.

Although the surface closest to the SFP liner of the CY cores was degraded from borated water attack, the embedded reinforcing steel exhibited no corrosion or loss of bond with the cement. The presence of secondary deposits, including secondary deposits in cracks found in the CY cores, provide evidence that water migration occurred. These results confirm that cracks in concrete and other concrete defects do not promote reinforcing steel degradation.

Published Studies:
Published studies demonstrate that attack of Portland cement concrete by weak acids, such as borated water, usually results in low depths of penetration, and are diffusion-controlled. Curve fits of the test data show that the depth of degradation versus time follows a Fick's Law of Diffusion formulation where depth increases with the square root of time.

A published study has also investigated the effects of reinforcing steel corrosion due to borated water entering reinforced concrete through cracks. The tests showed that corrosion increases as crack width increases and pH decreases. In particular, the tests showed negligible reinforcing steel attack even when specimens were subjected to the most corrosive test environment (pH of 5.2) with the largest crack width (0.4 mm) for a period of two years. Corrosion was limited to surface scarring in the area of the crack.

Existing FHB Structure Design Margin:
MPR performed an assessment of the FHB given the leakage paths and borated water attack described above. Below is a summary of the assessment results.

The concrete and embedded steel degradation rates developed from the core testing were used to calculate the reduction in available design margin. The analysis assumed that leakage continued through the end of plant life, which was defined as 70 years.

The slab underneath the SFP has been exposed to borated water leakage since early in plant life. The areas in the vicinity of leaking welds have likely experienced the most degradation, since some leakage at plug welds must migrate along the slab surface to the leakage collection channels, the projected depth of concrete degradation in these areas of the slab is 1.30 inches, assuming exposure to borated water for 70 years. This has no impact on the structural capacity of the slab since the reinforcing steel is protected by a 6 inch thick leveling layer of concrete which is not credited in the structural evaluations and an additional 2.46 inches of concrete clear cover.

The walls surrounding the SFP were exposed to borated water during the time period when the telltales were blocked and leakage accumulated in the gap between the SFP liner and the walls. Exposure of the walls to borated water started between 1995 and 1998 when the leakage collection channels and telltales became blocked and extended to early 2003 when drain flow was re-established. General degradation of the walls from exposure to borated water over this 8 year period had a minimal impact on the capacity of the FHB structure. Specifically, the depth of degradation over this period is calculated as 0.44 inches. This depth is less than the concrete clear cover of 4.4 inches on wall locations where there are no leakage collection channels, and 2.9 inches where leakage collection channels exist in the walls. Therefore, the reinforcing steel was protected from exposure to borated water. A 0.44 inch reduction in wall thickness reduces the structural margin at the most limiting wall by 0.4% (compared to the code allowable). Degradation of the concrete walls in the immediate vicinity of a leaking plug weld will continue even though the stored inventory has been drained. Such degradation will be highly localized and has no impact on the overall capacity of the concrete wall. Localized reinforcing steel degradation from borated water migration through cracks and construction joints had a negligible impact on the FHB walls during this period.

The assessment also concluded that reinforcing steel located in construction joints and cracks which are exposed to borated water will not be significantly degraded through the PEO. Based on the corrosion rate of carbon steel in de-aerated boric acid, this reinforcing steel may experience a radial reduction of 0.011 inches in 70 years which is insignificant.

Therefore, the structural assessment concluded that there are no structural challenges to the design basis of the FHB through the end of plant life including the period of extended operation.

Independent ACI Structural Assessment:
Salem had an experienced site concrete structural engineer perform an independent structural assessment of the FHB in 2006. The assessment included review of building drawings, a visual inspection of the accessible portions of the FHB exterior walls, and a visual inspection in the Sump Room. The checklist in ACI 201.1R-92, "Guide for Conducting a Visual Inspection of Concrete in Service" was used to guide the inspections. Observations were compared to limits in ACI 349.3R, *"Evaluation of Existing Nuclear Safety-Related Concrete Structures"*. Key conclusions from the independent assessment are excerpted below.

- "Overall the concrete appears to be in good structural condition."
- The "appearance of leaching or chemical attack and corrosion staining of undefined source on concrete surfaces do not indicate significant structural deterioration at this time."
- There were "no indications of concrete surface expansion due to reinforcing steel corrosion."

Assessment of SFP Liner:
Salem has also assessed the impact of potential degradation of the slab on the integrity of the SFP liner. The primary concern is that local degradation of the slab can create "voids" underneath the SFP liner. If the void corresponds to the location of a rack foot, the foot may no longer be supported on a firm surface. A scoping assessment included in MPR-2613 demonstrates that the liner is sufficiently ductile to accommodate the load from the fuel racks even if the foot of the rack is positioned over an area of local concrete degradation.

e. Salem has conducted extensive testing to understand and quantify concrete and reinforcing rebar degradations. The results of these tests were used to evaluate the FHB structural integrity through the PEO. The evaluation concluded the FHB will perform its intended function through the PEO. Thus obtaining concrete cores in the region most affected by borated water is not required. However, a shallow concrete core will be taken to assess potential degradation of the FHB from borated water. The shallow core sample will be taken in the SFP wall where previous inspections have shown evidence of borated water migration through the concrete. This action is being added as item d under Enhancement 5 of the Structures Monitoring Program.

   In preparing this response it was noted that Structures Monitoring Program Enhancement 5.c was not clear with respect to sampling the water taken from the seismic gap for ph, chlorides, and sulfates; although it is generally described in enhancement 11. Therefore enhancement 5.c is revised to clarify this enhancement.

   These updates to Enhancement 5 are shown below, and are captured in updates to the Appendix A and Appendix B Structures Monitoring Program description (See Enclosure B) and the License Renewal Commitment List (See Enclosure C). The revisions to item 5.c and the new action, item 5.d, are shown in bolded italic text:

5. Require the following actions related to the spent fuel pool liner:

   a. Perform periodic structural examination of the Fuel Handling Building per ACI 349.3R to ensure structural condition is in agreement with the analysis.

   b. Monitor telltale leakage and inspect the leak chase system to ensure no blockage.

   c. Test water drained from the seismic gap for boron***, chloride, and sulfate concentrations; and pH.***

   d. Perform a shallow core sample in the SFP wall where previous inspections have shown ingress of borated water through the concrete. The core sample will be examined for degradation from borated water.

RAI B.2.1.33-2

Background:

The LRA states that leakage of borated water has occurred in SNGS Units 1 and 2 reactor cavities during refueling outages, but the leaks have been contained within the Containment Building.

Issue:

In April 2006 visual structural examinations of the accessible portions of the containment reinforced concrete structures for SNGS Units 1 and 2 indicated that the concrete was apparently in good structural condition. It is unclear to the staff that leakage of the borated water has not resulted in degradation of either the concrete or embedded steel reinforcement that is inaccessible for inspection.

Request:

a. Provide historical data on the leakage occurrence and volume, and available information from chemical analysis performed on the leakage.

b. Provide the root cause analysis that was used to identify the source of leakage, including information on the path of the leakage and structures that could potentially be affected by the presence of the borated water.

c. Discuss plans for remedial actions or repairs to address leakage. In the absence of a commitment to fix the leakage prior to the period of extended operation, explain how the structures monitoring program, or other plant-specific program, will address the leakage to ensure that aging effects, especially in inaccessible areas, will be effectively managed during the period of extended operation.

d. Provide background information and data to demonstrate that concrete and embedded steel reinforcement potentially exposed to the borated water have not been degraded. If experimental results will be used as part of the assessment, provide evidence that the test program is representative of the materials and conditions that exist.

The staff needs the information to confirm that the potential effect of aging of the reinforced concrete due to presence of borated water will be adequately managed so that the intended function of impacted structural members will be maintained consistent with the current licensing basis for the period of extended operation as required by 10 CFR 54.21(a)3.

PSEG Response:

a. A summary of available Unit 1 and Unit 2 reactor cavity and fuel transfer canal leakage history is provided below:

Unit 1:

- During the Salem Unit 1 2005 refueling outage white deposits were observed in several locations of the Unit 1 containment. Chemical analyses indicated that the deposits originated from the reactor cavity or fuel transfer canal borated water leakage. In addition, the deposits contained constituents consistent with concrete. Follow-up walkdowns found evidence of a recent leak (water) in the "N16" Decay Tunnel. Analysis of the water indicated it was consistent with the reactor cavity water. The six fuel transfer canal telltale drain lines were found blocked. After cleaning, four of the telltales showed evidence of leakage ranging from approximately 1 to 60 drops per minute. The leakage rates steadily decreased to zero during the refueling outage.

- A review of the corrective action database indicates that no active leakage associated with the reactor cavity and fuel transfer canal liner was documented during the Unit 1 2007 nor during the 2008 refueling outages. However, during the 2008 refueling outage chemical analysis was performed on deposits collected from the "N16" tunnel. Analysis showed that the residue originated from either the reactor cavity or the fuel transfer canal.

- During the 2010 Unit 1 refueling outage no active leaks were observed.

Unit 2:

- In November 2003 liquid was observed running down the containment liner plate and lagging under the fuel transfer canal inside containment, and pooling on the concrete floor. No evidence of corrosion was observed at the containment liner to floor joint.

- In April 2005 during the Salem Unit 2 refueling outage, water was observed dripping down the wall on the containment liner. Also, white deposits were observed in several locations on the Letdown Heat Exchanger Room walls under 2 of the 6 telltales, which were dripping. Analysis showed the deposits and water were from the reactor cavity. The two dripping telltales were probed and no blockage was found.

- During the 2006 Unit 2 refueling outage a small amount of leakage was observed coming from a telltale in the Letdown Heat Exchanger Room. During the 2008 Unit 2 refueling outage no active leakage was documented. However, during the 2009 Unit 2 refueling outage a 60 dpm active leak was found from the telltale located above the door to the Letdown Heat Exchanger Room. A sample of the water was analyzed. Analysis of the water indicated it was consistent with reactor cavity water.

- Evidence of boric acid deposits on the Unit 2 containment liner under the fuel transfer canal have been observed during multiple outages since November 2000.

b. Chemical analysis of collected leakage during multiple refueling outages at both Units 1 and 2 shows that the source of the leakage is from the reactor cavity or the fuel transfer canal. Assessments for Unit 1 and 2 have concluded that the potential cause of leakage is very small cracks in the reactor cavity or fuel transfer canal liner. The majority of the leakage enters the leak collection chases and drains through the telltales. Some of the telltales in the Letdown Heat Exchanger Room associated with the fuel transfer canal liner have been observed with active leaks during refueling outages. A second leakage path occurs in the vicinity of where the fuel transfer canal exits containment. The leakage path is postulated to be through the reactor cavity and fuel transfer canal liner, then through concrete construction joints and cracks, and then down the sides of the containment liner behind the lagging inside containment.

The leakage only occurs when the reactor cavity and fuel transfer canal are flooded up for refueling. Active leaks have been observed sporadically only during refueling outages and measured leakage rates are less than 100 drops per minute.

This leakage has the potential to impact the reactor cavity and fuel transfer canal reinforced concrete structure. In addition, the leakage has the potential to impact the containment liner. The impact of leakage on the containment liner is documented in Salem's response to RAI B.2.1.28-1.

c. Salem has concluded that based on the short duration of the refueling activities and the very long exposures needed to degrade reinforced concrete, that remedial actions are not needed. Salem will continue Structural Monitoring (B.2.1.33) and ASME Section XI, Section IWE Program (B.2.1.28) Inservice Inspections to ensure the continued integrity of the in-scope structures.

Salem will enhance the Structure Monitoring Program to perform periodic inspection of telltales associated with the reactor cavity and fuel transfer canal liner to ensure the telltales are free of significant blockage and to periodically monitor for leakage when the cavity is flooded. Keeping the telltales free of blockage will ensure that water that has entered the inaccessible areas between the liner and concrete will only contact the concrete for short durations. Salem will also inspect concrete surfaces for degradation where leakage has been observed, in accordance with this Program. In addition, the Structural Monitoring Program will be enhanced to require pH testing of leakage from the telltales.

LRA Appendix A, Section A.2.1.33, and Appendix B, Section B.2.1.33, the Structures Monitoring Program descriptions, are being revised to reflect these enhancements. See Enclosure B to see these updates.

Also, refer to Enclosure C of this letter to see the corresponding change to the License Renewal Commitment List, LRA Appendix A, Section A.5.

d. Salem has performed an assessment of the long-term structural adequacy of the Salem Unit 1 Fuel Handling Building (FHB) reinforced concrete structure under potential prolonged exposure of the concrete and reinforcing steel to borated water. The results of the assessment are documented in MPR Report 2613, "Salem Generating Station Fuel Handling Building—Evaluation of Degraded Condition," Revision 3, which is attached in enclosure "D".

Conclusions of the assessment are summarized below.

- The predicted depth of concrete degradation after 70 years of continuous exposure to borated water is 1.3 inches.
- The degradation rate of the concrete at the rebar and concrete interface is similar to the general rate of attack of concrete without rebar. Therefore, degradation of rebar at the construction joints or cracks will not spread rapidly along the bond with the rebar. Rebar functionality is not compromised from a general loss of bond with the concrete.

Salem has concluded that the findings for the FHB are directly applicable to the Unit 1 and 2 the reactor cavity and fuel transfer canal reinforced concrete structure.

The reactor cavity and fuel transfer canal are only filled with borated water during refueling outages, which occurs at each unit approximately 1 month out of every 18 months (about 5% of the operating cycle) since the Salem units perform refueling outages every 18 months. By contrast the Unit 1 FHB assessment assumed continuous borated water exposure for 70 years with a resulting depth of degradation of 1.3 inches. Therefore, the exposure duration of borated water on the reactor cavity and fuel transfer canal concrete is approximately 5% of the 70 year duration used as input to the Unit 1 FHB assessment. Therefore, the expected depth of concrete degradation on the reactor cavity and fuel transfer canal concrete will be substantially less (0.29 inches) than the 1.3 inches predicted in the Unit 1 FHB assessment. These insignificant depths of degradation will not approach the reinforcing steel. Therefore, as demonstrated in the Unit 1 FHB analysis, there would be insignificant degradation on the reinforcing steel in the reactor cavity and fuel transfer canal reinforced concrete structure.

In summary, based on reinforced concrete testing, reactor cavity and fuel transfer canal concrete degradation due to the borated water leakage will be insignificant. Therefore, Salem has concluded that the leakage associated with the reactor cavity and fuel transfer canal liner has no impact on the intended function of these structures.

RAI B.2.1.33-3

Background:

The LRA states that groundwater intrusion has been observed through seismic expansion joints, concrete construction joints, and expansion and shrinkage cracks in the concrete. Underground reinforced concrete structures and structures in contact with raw water at SNGS are subject to an aggressive environment. Groundwater and raw water chemistry results in 2008 and 2009 indicate chloride levels up to 15,000 ppm that exceeds the GALL Report threshold limit for chlorides (< 500 ppm). The applicant stated that inspection of below-grade structures will be done when exposed during plant excavations done for construction or maintenance activities. The LRA states that the structures monitoring program has been enhanced to require periodic sampling, testing, and analysis of groundwater chemistry for pH, chlorides, and sulfates, and assessing its impact on buried structures. Also the LRA states that the service water intake structure will be monitored to provide a bounding condition and indicator of the likelihood of concrete degradation for inaccessible portions of concrete structures.

Issue:

As noted in the LRA, there are several subgrade exterior walls at SNGS that have evidence of past or present groundwater penetration. During the on-site audit, the applicant was asked if they had any plans for inspections of inaccessible reinforced concrete areas prior to the period of extended operation to confirm the absence of concrete degradation. The applicant responded that they did not and that operating experience indicates that there is no evidence of corrosion appearing on the interior surfaces of the concrete structures having inaccessible exterior surfaces. ACI 349.3R-96 recommends an inspection frequency of ten years for below-grade structures. It was noted that the thickness of some of these walls however may be on the order of four feet. Since the applicant does not have plans for inspections of inaccessible areas, the groundwater is aggressive, there have been several incidences of groundwater penetration into the structures, and the interior of the walls may not indicate the condition of the exterior walls, it is unclear to the staff that this is an adequate approach to managing aging of inaccessible concrete structures subjected to aggressive groundwater.

Request:

a. Provide locations where groundwater test samples were/are taken relative to safety related and important-to-safety embedded concrete walls and foundations and provide historical results (i.e., pH, chloride content, and sulfate content) including seasonal variation of results.
b. In locations adjacent to embedded reinforced concrete structures where chloride levels exceed limits in GALL Report, provide any plans for inspections, or if no inspections or coring of concrete is planned to evaluate condition of structures (e.g., presence of steel corrosion or determination of chloride profiles), provide a basis to demonstrate that the current level of chlorides in the groundwater is not causing structural degradation of embedded walls or foundations.

The staff needs the information to confirm that the potential effect of aging of the reinforced concrete due to presence of high chloride levels will be adequately managed so that the intended function of impacted structural members will be maintained consistent with the current licensing basis for the period of extended operation as required by 10 CFR 54.21(a)3.

PSEG Response:

a. The following table provides well locations and tabulates the groundwater test results.

Salem Groundwater Sampling

Location	Well No.	Sample Date	pH	Chlorides ppm	Sulfates ppm
Unit 1 - Containment Structure - south wall	AC	1/30/09	–	8.36	32.6
Unit 1 - Fuel Handling Building - south wall	AM	1/8/09	6.83	6.0	26.8
Unit 1 - Fuel Handling Building - west wall	M	1/17/08	6.70	43.3	23.7
	M	1/27/09	7.00	125	33
	M	11/19/09	–	83.2	21.6
Unit 1 - Fuel Handling Building - south wall	N	1/17/08	5.92	7.0	93.5
	N	1/8/09	5.94	4200	1048
	N	11/19/09	–	6.2	65
Salem - Outside the construction cofferdams	K	1/17/08	6.96	2634	483
	K	1/19/09	7.02	2100	239
	BA	4/08	6.97	1124	1.21
Unit 2 - Fuel Handling Building - south wall	Temporary Wells 3, 4 & 5	4/11/10	–	15 - 80	–
Unit 2 - Fuel Handling Building - west wall	Temporary Well 1	4/11/10	–	35	–
Auxiliary Building - north wall	Temporary Well 2	4/11/10	–	75	–
Aggressive Environment - limits per the GALL report			<5.5	>500	>1500

The location of the safety related structures at Unit 1 and 2 are surrounded by the underground cofferdams used during construction, as shown on Salem UFSAR Figure 3.8-56. Wells AC, AM, M, N and the Temporary Wells are located within the these cofferdams at a sampling depth of approximately 20 feet and these wells are within 30 feet of the associated building walls. Well K is north of Salem Unit 2 Containment Structure, west of the Fire Pump House, outside of the cofferdams at a sampling depth of approximately 80 feet and is monitoring the Vincentown formation. Well BA is south of Salem Unit 1 Containment Structure, outside of the cofferdams near the Main Fuel Oil Storage Tank and adjacent to the Shoreline Protection and Dike.

Salem power-block structures, except for the Turbine building, were constructed in an area excavated down to the Vincentown Formation, approximately 70-80 feet below grade level and encircled with a system of below grade cellular cofferdams, as discussed in Salem UFSAR Section 3.8.1.6.8.7 (Construction) and shown on Figure 3.8-56. The underground walls and foundations were encased in lean concrete up to the top of the cofferdams, approximately 20 feet below grade level. This precludes routine interaction with the normal groundwater which minimizes exposure to chlorides. The seasonal variation of using deicing salts at walkways and roadways, within the cofferdam area, has resulted in chlorides above 500 ppm on one occasion, as shown on the table above.

The flow of the Delaware River at the Salem site is dominated by the tidal flow from the Delaware Bay, with the tidal flow exceeding the river runoff flow resulting in brackish water. The chlorides in the river are tide dependent as high tide would have the higher chloride levels when the water from the bay flows farther up the river. The Salem NJPDES Permit Renewal Application, February 1, 2006 lists the river water sulfates as 387 ppm and the ph range is 6.7 - 8.3. The Salem UFSAR Section 2.4.12 (Environmental Acceptance of Effluents) discusses the river salinity to be variable and averages from 10 to 15 ppt which converts to chlorides of 5500 to 8300 ppm. The LRA section 3.5.2.2.2.4.2 (Aging Management of Inaccessible Areas for Group 6 Structures) on page 3.5-49, inadvertently listed the river water chlorides as variable and ranging from 10,000 to 15,000 ppm. This was the salinity or salt concentration range. The correct chloride concentration range is 5500 to 8300 ppm.

b. The enhanced Structures Monitoring Program is adequate to manage the potential effect of aging of embedded walls and foundations for the elevated levels of chlorides in the groundwater. The Structures Monitoring Program will inspect the Service Water Intake wall splash zones exposed to waves and river tide changes as a leading indicator for the condition of below-grade embedded walls and foundations. The use of Service Water Intake splash zones as a leading indicator to identify potential degradation of below-grade embedded walls and foundations provides reasonable assurance that the degradation of embedded walls and foundations will be detected before a loss of an intended function.

   The groundwater and river water samples as listed above show pH values are above 5.5 indicating a non-aggressive environment, and sulfates are lower than 1500 ppm and would indicate a non-aggressive environment; but chlorides exceed 500 ppm which indicates an aggressive environment. The river water has higher chloride levels than the groundwater, therefore, the Service Water Intake splash zones are exposed to higher chloride levels than embedded walls and foundations.

ACI 222R-01 “Protection of Metals in Concrete Against Corrosion” at the end of Section 2.2.5 states parts of a structure in the splash zone experiences particularly aggressive conditions. The corrosion of steel in concrete is an electrochemical process and the continuous wetting and drying at the splash zones promotes macro-cell formation in the concrete allowing corrosion to occur. Also, high salt levels arise by salt water being transported via capillary action upward through the concrete cover and being concentrated due to evaporation in the splash zone. Therefore, the intake structure splash zone is a leading indicator for the condition of all below-grade embedded walls and foundations.

The intake structure splash zones are presently inspected by the Structures Monitoring Program with an inspection frequency of not greater than 5 years per Maintenance Rule requirements. The corrosion of concrete reinforcing bars due to penetration and concentration of chloride ions is accompanied by an increase in volume of the steel as it is converted to rust products. The volume increase can cause cracking, spalling, and delamination of the concrete that can be visible in the form of loss of concrete material, rust spots and stains, and cracks in the concrete cover along the reinforcing bars. Visual inspection conducted in accordance with Structural Monitoring Program would detect such conditions before the loss of an intended function. In the event the inspection identifies significant concrete degradation at the Service Water Intake Structure, the Structural Monitoring Program and the Corrective Action Program require an evaluation of the condition to include applicability to inaccessible portions of the other structures and determine if excavation for inspection of concrete is warranted.

The embedded walls and foundations of the safety related structures surrounded by the underground cofferdams are encased in lean concrete as shown on USFAR Figure 3.8-56. This precludes routine interaction with the normally aggressive groundwater which minimizes exposure to chlorides. The seasonal variation of using deicing salts at walkways and roadways, within the cofferdam area, has resulted in chlorides above 500 ppm on one occasion, as shown on the table above.

Since 2000 there have been five inspection reports for Unit 1 and Unit 2 Service Water Intake Structures. These inspections did not identify signs of distress due to aggressive chemical attack or corrosion of embedded steel. Also, based on review of past excavations of below grade embedded walls, the concrete was found to be in good condition. The most recent excavations included the Unit 1 Service Water Intake Structure, Fuel Handling Building, and the Containment Structure during the Salem Unit 1 refueling outage in April 2010.

The Service Water Intake Structure concrete embedded walls and foundation are comparable to all other safety related embedded walls and foundations since the same concrete mix design was used during site construction. Also, the Service Water Intake Structure is designed with a 2 inch concrete cover which is equal to or less than the concrete cover at all other safety related structures. Therefore, the Service Water Intake Structure splash zone inspections are considered to be an acceptable leading indicator to identify potential degradation of safety related below-grade embedded walls and foundations before a loss of an intended function.

RAI B.2.1.33-4

Background:

IN GALL Report AMP XI.S6, program elements 3 and 4 state that for each structure/aging effect combination the specific parameters monitored or inspected are selected to ensure that the aging degradation leading to loss of intended function will be detected and quantified before there is a loss of intended function.

Issue:

As a result of the field walk-down with the applicant's technical staff on February 12, 2010, the staff noticed minor indications of degradation in several areas (e.g., cracking, efflorescence, leaching, and water). At Salem Unit 1 Auxiliary Building Elevation 64 (below ground water level) there was evidence of water in-leakage through the wall and the area was roped off as an exclusion zone. The applicant was asked about this and informed the staff that the source of the contamination was from in-leakage of groundwater and that the groundwater had picked up the contamination external to the wall.

Request:

Provide information on how the in-leakage of contaminated groundwater will be addressed under your corrective action program.

The staff needs the above information to confirm that the effects of aging such as noted above will be adequately managed so that the intended function of impacted structural members will be maintained consistent with the current licensing basis for the period of extended operation as required by 10 CFR 54.21(a)(3).

PSEG Response:

In-leakage of contaminated groundwater into the Auxiliary Building was documented and evaluated under the corrective action program. The groundwater in-leakage has been identified at shrinkage cracks in the below-grade embedded concrete wall. A corrective action report was generated to identify the water intrusion on 10/2009. As part of the corrective action process a qualified structural engineer conducted an initial inspection to evaluate and document the condition. The structural engineer concluded that the current condition does not adversely impact the structure's intended function, and the current in-leakage will not impact the structure's safety function. A past corrective action was to inject a sealant into the cracks in this concrete wall. The sealant has deteriorated and is now seeping out of the concrete cracks. Proper housekeeping was used to eliminate standing water to ensure the condition does not create an environment that will promote deterioration of structural members. This below-grade embedded concrete wall shows efflorescence or other mineral deposits. The crack area is presently in the corrective action program to be cleaned so a detailed engineering inspection can be performed to ensure long term aging issues are identified and any other required corrective actions can be performed.

The Structures Monitoring Program is used to monitor for the effects of groundwater intrusion to ensure the condition does not result in corrosion of rebar, embedded steel, floor mounted component support members, and anchors with the potential of adverse impact on their structural integrity. The Structures Monitoring Program includes an enhancement to perform a chemical analysis of ground or surface water in-leakage when there is significant in-leakage or there is reason to believe the in-leakage may be damaging concrete elements or reinforcing steel, as shown on LRA page A-27. In summary, implementation of the enhanced Structures Monitoring Program is considered to provide reasonable assurance the aging effect associated with groundwater intrusion will be adequately managed through the period of extended operation.

Enclosure B

Salem Generating Station Unit 1 and Unit 2 License Renewal Application (LRA) Appendix A and Appendix B Program Description Updates

Note: To facilitate understanding, portions of the original LRA Appendix A and Appendix B Program Descriptions have been repeated in this Enclosure, with revisions indicated. Existing LRA text is shown in normal font. Changes are highlighted with bolded italics for inserted text and strikethroughs for deleted text.

This Enclosure contains portions of LRA Appendix A and Appendix B program descriptions affected by the RAI responses in this package. Some pre-existing text is repeated here to provide context for the changes. The existing LRA text is formatted in normal font; new text is bold and italicized; deleted text is indicated with strikethroughs.

The Appendix A and B changes for a given program are presented together, followed by the next program, etc.

A.2.1.28 ASME Section XI, Subsection IWE

Starting with the third paragraph, Section A.2.1.28 of the LRA is modified as follows:

The ASME Section XI, Subsection IWE aging management program will be enhanced to include:

1. Inspection of a sample of the inaccessible liner covered by insulation and lagging prior to the period of extended operation and every 10 years thereafter. Should unacceptable degradation be found additional insulation will be removed as necessary to determine extent of condition in accordance with the corrective action process.

   Prior to the period of extended operation

   - ***The samples shall include 57 randomly selected containment liner insulation panels per unit.***
   - ***The examination will be performed by either removing the containment liner insulation panels and performing a visual inspection, or by using a pulsed eddy current (PEC) remote inspection, with the containment liner insulation left in place, to detect evidence of loss of material. If evidence of loss of material is detected using PEC, the containment liner insulation panel will be subsequently removed to allow for visual and UT examinations***

   During the period of extended operation

   - ***One containment liner insulation panel will be selected, at random, for removal from each quadrant, during each of the three Periods in an Inspection Interval. Therefore, a total of 12 containment liner insulation panels will be selected, in each unit, during each ten year Inspection Interval, to allow for examination of the containment liner behind the containment liner insulation.***

   - ***The randomly selected containment liner insulation panels in each quadrant will not include containment liner insulation panels previously selected.***

2. Visual inspection of 100 % of the moisture barrier, at the junction between the containment concrete floor and the containment liner, will be performed in accordance with ASME Section XI, Subsection IWE program

requirements, to the extent practical within the limitation of design, geometry, and materials of construction of the components. The bottom edge of the stainless steel insulation lagging will be trimmed, if necessary, to perform the moisture barrier inspections. This inspection will be performed prior to the period of extended operation, and on a frequency consistent with IWE inspection requirements thereafter. Should unacceptable degradation be found, corrective actions, including extent of condition, will be addressed in accordance with the corrective action process.

As a follow-up to inspections performed during the 2009 refueling outage, the following specific corrective actions will be performed on Unit 2 prior to entry into the period of extended operation:

- ***Examine the accessible 3/4" knuckle plate. If corrosion is observed to extend below the surface of the moisture barrier, excavate the moisture barrier to sound metal below the floor level and perform examinations as required by IWE.***

- ***Perform remote visual inspections, of the six capped vertical leak chase channels, below the containment floor to determine extent of condition.***

- ***Remove the concrete floor and expose the 1/4" containment liner plate (floor) for a minimum of two of the vertical leak chase channels with holes. Perform examination of exposed 1/4" containment liner plate (floor) as required by IWE. Additional excavations will be performed, if necessary, depending upon conditions found at the first two channels.***

- ***Remove 1/2" containment liner insulation panels, adjacent to accessible areas where there are indications of corrosion, to determine the extent of condition of the existing corroded areas of the containment liner plate.***

- ***Perform augmented examinations of the areas of the 1/2" containment liner plate behind insulation panels, where loss of material was previously identified, in accordance with IWE-2420.***

- ***Examine 100% of the moisture barrier in accordance with IWE-2310 and replace or repair the moisture barrier to meet the acceptance standard in IWE-3510.***

As a follow-up to inspections performed during the 2010 refueling outage, the following specific corrective actions will be performed on Unit 1 prior to entry into the period of extended operation:

- ***Perform augmented examinations of the 3/4" containment liner (knuckle plate) at 78' elevation in accordance with IWE-2420.***

- ***Perform augmented examinations of the areas of the 1/2" containment liner plate behind insulation panels, where loss of material was previously identified, in accordance with IWE-2420.***

- ***Remove 1/2" containment liner insulation panels, adjacent to accessible areas where there are indications of corrosion, to determine the extent of condition of the existing corroded areas of the containment liner plate.***

These enhancements will be implemented prior to the period of extended operation, with the inspections performed in accordance with the schedule described above.

B.2.1.28 ASME Section XI, Subsection IWE

Starting at the top of LRA Appendix B page B-131, the Program Description of Section B.2.1.28 is modified as follows:

The program will be enhanced to include inspection of a random sample of the containment liner behind the containment liner insulation prior to the period of extended operation. The sampling plan was developed based upon guidance in EPRI TR-107514, "Age Related Degradation Inspection Method and Demonstration: in Behalf of Calvert Cliffs Nuclear Power Plant License Renewal Application". The population size of containment liner insulation panels in each unit is about 264 panels. A sample size of 57 will meet the statistical requirements of a 95% confidence level that 95% of the containment liner plate behind the containment liner insulation meets the acceptance criteria of IWE-3500. The samples will be randomly selected. If acceptance criteria defined in IWE-3500 is not satisfied, the sampling plan will be modified as recommended in EPRI TR-107514. Also, based on the satisfactory completion of the above sample plan, a reduced sample size will be randomly selected and examined each Containment Inservice Inspection Period during the period of extended operation.

In addition, Enhancement #1 on LRA page B-132 is clarified as follows:

1. Inspection of a sample of the inaccessible liner covered by insulation and lagging prior to the period of extended operation and every 10 years thereafter. Should unacceptable degradation be found additional insulation will be removed as necessary to determine extent of condition in accordance with the corrective action process. **Program Elements Affected: Scope of Program (Element 1)**

   Prior to the period of extended operation (PEO)

   - ***The samples shall include 57 randomly selected containment liner insulation panels per unit.***
   - ***The examination will be performed by either removing the containment liner insulation panels and performing a visual inspection, or by using a pulsed eddy current (PEC) remote inspection, with the containment liner insulation left in place, to detect evidence of loss of material. If evidence of loss of material***

is detected using PEC, the containment liner insulation panel will be subsequently removed to allow for visual and UT examinations.

During the period of extended operation

- ***One containment liner insulation panel will be selected, at random, for removal from each quadrant, during each of the three Periods in an Inspection Interval. Therefore, a total of 12 containment liner insulation panels will be selected, in each unit, during each ten year Inspection Interval, to allow for examination of the containment liner behind the containment liner insulation.***
- ***The randomly selected containment liner insulation panels in each quadrant will not include containment liner insulation panels previously selected.***

A.2.1.29 ASME Section XI, Subsection IWL

LRA Appendix A is revised as follows:

- Section A.1.1 (NUREG-1801 Chapter XI Aging Management Programs), item # 29 (ASME Section XI, Subsection IWL (Section A.2.1.29), on page A-6, is revised as follows:

  29. ASME Section XI, Subsection IWL (Section A.2.1.29) [Existing – ***Requires Enhancement***]

- Section A.2.1.29 (ASME Section XI, Subsection IWL), on page A-24, the following is added:

  The ASME Section XI, Subsection IWL, aging management program will be enhanced to include:

  1. Examination and acceptance criteria in accordance with the guidance contained in ACI 349.3R.

B.2.1.29 ASME Section XI, Subsection IWL

LRA Appendix B, Section B.2.1.29 is revised as follows:

- Section B.2.1.29 (ASME Section XI, Subsection IWL), on page B-136, the following change is made to the section on Enhancements:

  Enhancements

  ~~None.~~ ***Prior to the period of extended operation, the following enhancement will be implemented in the program elements:***

  1. Examination and acceptance criteria in accordance with the guidance contained in ACI 349.3R. Program Elements Affected: Acceptance Criteria (Element 6)

- Section B.2.1.29 (ASME Section XI, Subsection IWL), on page B-138, the following change is made to the Conclusion:

  Conclusion

  The ~~existing~~ ***enhanced*** ASME Section XI, Subsection IWL, aging management program ***will*** provide~~s~~ reasonable assurance that the identified aging effects are adequately managed so that the intended functions of components within the scope of license renewal will be maintained consistent with the current licensing basis during the period of extended operation.

A.2.1.33 Structures Monitoring Program

The fifth Enhancement to the Structures Monitoring Program, as described in LRA Section A.2.1.33, on pages A-26 and A-27 is modified as follows:

5. Require the following actions related to the spent fuel pool liner:

   a. Perform periodic structural examination of the Fuel Handling Building per ACI 349.3R to ensure structural condition is in agreement with the analysis.

   b. Monitor telltale leakage and inspect the leak chase system to ensure no blockage.

   c. Test water drained from the seismic gap for boron***, chloride, and sulfate concentrations; and pH.***

   d. ***Perform a shallow core sample in the Spent Fuel Pool wall where previous inspections have shown ingress of borated water through the concrete. The core sample will be examined for degradation from borated water.***

In addition, a new Enhancement (#15) is added to page A-27 of LRA Appendix A, Section A.2.1.33, as follows:

15. ***When the reactor cavity is flooded up, Salem will periodically monitor the telltales associated with the reactor cavity and refueling canal for leakage. If telltale leakage is observed, then the pH of the leakage will be measured to ensure that concrete reinforcement steel is not experiencing a corrosive environment. In addition, Salem will periodically inspect the leak chase system associated with the reactor cavity and refueling canal to ensure the telltales are free of significant blockage. Salem will also inspect concrete surfaces for degradation where leakage has been observed, in accordance with this Program.***

B.2.1.33 Structures Monitoring Program

Consistent with the changes made to LRA Section A.2.1.33, the fifth Enhancement to the Structures Monitoring Program, as described in LRA Section B.2.1.33, on page B-153 is modified as follows:

5. Require the following actions related to the spent fuel pool liner:

    a. Perform periodic structural examination of the Fuel Handling Building per ACI 349.3R to ensure structural condition is in agreement with the analysis.

    b. Monitor telltale leakage and inspect the leak chase system to ensure no blockage.

    c. Test water drained from the seismic gap for boron***, chloride, and sulfate concentrations; and pH.***

    d. ***Perform a shallow core sample in the Spent Fuel Pool wall where previous inspections have shown ingress of borated water through the concrete. The core sample will be examined for degradation from borated water.***

Also consistent with the changes made to LRA Section A.2.1.33, a new Enhancement (#15) is added to page B-154 of LRA Appendix B, Section A.2.1.33, as follows:

15. When the reactor cavity is flooded up, Salem will periodically monitor the telltales associated with the reactor cavity and refueling canal for leakage. If telltale leakage is observed, then the pH of the leakage will be measured to ensure that concrete reinforcement steel is not experiencing a corrosive environment. In addition, Salem will periodically inspect the leak chase system associated with the reactor cavity and refueling canal to ensure the telltales are free of significant blockage. Salem will also inspect concrete surfaces for degradation where leakage has been observed, in accordance with this Program.

A.5 License Renewal Commitment List

The following table identifies modifications made to license renewal commitments 28, 29 and 33 as a result of the responses to the RAIs contained in this package. Pre-existing text is formatted in normal font; new text is bold and italicized; deleted text is indicated with strikethroughs. The specific RAIs that led to the commitment modifications are listed in the "SOURCE" column adjacent to beginning of the new text. Any other actions described in this submittal represent intended or planned actions. They are described for the NRC's information and are not regulatory commitments.

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
28	ASME Section XI, Subsection IWE	ASME Section XI, Subsection IWE is an existing program that will be enhanced to include: 1. Inspection of a sample of the inaccessible liner covered by insulation and lagging once prior to the period of extended operation and every 10 years thereafter. Should unacceptable degradation be found additional insulation will be removed as necessary to determine extent of condition in accordance with the corrective action process. ***Prior to the period of extended operation*** • ***The samples shall include 57 randomly selected containment liner insulation panels per unit.*** • ***The examination will be performed by either removing the containment liner insulation panels and performing a visual inspection, or by using a pulsed eddy current (PEC) remote inspection, with the containment liner insulation left in place, to detect evidence of loss of material. If evidence of loss of material is detected using PEC, the containment liner insulation panel will be***	A.2.1.28	Program to be enhanced prior to the period of extended operation. Inspection Schedule identified in Commitment	Section B.2.1.28 ***Salem letter LR-N10-0165 RAI B.2.1.28-2***

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
		subsequently removed to allow for visual and UT examinations ***During the period of extended operation*** • ***One containment liner insulation panel will be selected, at random, for removal from each quadrant, during each of the three Periods in an Inspection Interval. Therefore, a total of 12 containment liner insulation panels will be selected, in each unit, during each ten year Inspection Interval, to allow for examination of the containment liner behind the containment liner insulation.*** • ***The randomly selected containment liner insulation panels in each quadrant will not include containment liner insulation panels previously selected.*** 2. Visual inspection of 100 % of the moisture barrier, at the junction between the containment concrete floor and the containment liner, will be performed in accordance with ASME Section XI, Subsection IWE program requirements, to the extent practical within the limitation of design, geometry, and materials of construction of the components. The bottom edge of the stainless steel insulation lagging will be trimmed, if necessary, to perform the moisture barrier inspections. This inspection will be performed prior to the period of extended operation, and on a frequency consistent with IWE inspection requirements thereafter. Should unacceptable degradation be found, corrective actions, including extent of condition, will be addressed in accordance with the corrective action process.			

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
		As a follow-up to inspections performed during the 2009 refueling outage, the following specific corrective actions will be performed on Unit 2 prior to entry into the period of extended operation: • *Examine the accessible 3/4" knuckle plate. If corrosion is observed to extend below the surface of the moisture barrier, excavate the moisture barrier to sound metal below the floor level and perform examinations as required by IWE.* • *Perform remote visual inspections, of the six capped vertical leak chase channels, below the containment floor to determine extent of condition.* • *Remove the concrete floor and expose the 1/4" containment liner plate (floor) for a minimum of two of the vertical leak chase channels with holes. Perform examination of exposed 1/4" containment liner plate (floor) as required by IWE. Additional excavations will be performed, if necessary, depending upon conditions found at the first two channels.* • *Remove 1/2" containment liner insulation panels, adjacent to accessible areas where there are indications of corrosion, to determine the extent of condition of the existing corroded areas of the containment liner plate.* • *Perform augmented examinations of the areas of the 1/2" containment liner plate behind insulation panels, where loss of material was previously identified, in*			*Salem letter LR-N10-0165 RAI B.2.1.28-1*

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
		accordance with IWE-2420. • ***Examine 100% of the moisture barrier in accordance with IWE-2310 and replace or repair the moisture barrier to meet the acceptance standard in IWE-3510.*** ***As a follow-up to inspections performed during the 2010 refueling outage, the following specific corrective actions will be performed on Unit 1 prior to entry into the period of extended operation:*** • ***Perform augmented examinations of the 3/4" containment liner (knuckle plate) at 78' elevation in accordance with IWE-2420.*** • ***Perform augmented examinations of the areas of the 1/2" containment liner plate behind insulation panels, where loss of material was previously identified, in accordance with IWE-2420.*** • ***Remove 1/2" containment liner insulation panels, adjacent to accessible areas where there are indications of corrosion, to determine the extent of condition of the existing corroded areas of the containment liner plate.***			
29	ASME Section XI, Subsection IWL	~~Existing program is credited.~~ ***ASME Section XI, Subsection IWL, is an existing program that will be enhanced to include:*** ***1. Examination and acceptance criteria in accordance with the guidance contained in ACI 349.3R.***	A.2.1.29	~~Ongoing~~ ***Program to be enhanced prior to the period of extended operation.***	Section B.2.1.29 ***Salem letter LR-N10-0165 RAI B.2.1.29-1***

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
33	Structures Monitoring Program	Structures Monitoring is an existing program that will be enhanced to include: 1. Additional structures and components as described in A.2.1.33. 2. Concrete structures will be observed for a reduction in equipment anchor capacity due to local concrete degradation. This will be accomplished by visual inspection of concrete surfaces around anchors for cracking, and spalling. 3. Clarify that inspections are performed for loss of material due to corrosion and pitting of additional steel components, such as embedments, panels and enclosures, doors, siding, metal deck, and anchors. 4. Require inspection of penetration seals, structural seals, and elastomers, for degradations that will lead to a loss of sealing by visual inspection of the seal for hardening, shrinkage and loss of strength. 5. Require the following actions related to the spent fuel pool liner: a. Perform periodic structural examination of the Fuel Handling Building per ACI 349.3R to ensure structural condition is in agreement with the analysis. b. Monitor telltale leakage and inspect the leak chase system to ensure no blockage. c. Test water drained from the seismic gap for boron, ***chloride and sulfate concentrations; and pH.*** ***d. Perform a shallow core sample in the Spent Fuel Pool wall where previous inspections have shown ingress of borated water through the concrete. The core sample will be examined for degradation from borated water.***	A.2.1.33	Program to be enhanced prior to the period of extended operation.	Section B.2.1.33 ***Salem letter LR-N10-0165 RAI B.2.1.33-1***

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
		6. Require monitoring of vibration isolators, associated with component supports other than those covered by ASME XI, Subsection IWF. 7. Add an Examination Checklist for masonry wall inspection requirements. 8. Parameters monitored for wooden components will be enhanced to include: Change in Material Properties, Loss of Material due to Insect Damage and Moisture Damage. 9. Specify an inspection frequency of not greater than 5 years for structures including submerged portions of the service water intake structure. 10. Require individuals responsible for inspections and assessments for structures to have a B.S. Engineering degree and/or Professional Engineer license, and a minimum of four years experience working on building structures. 11. Perform periodic sampling, testing, and analysis of ground water chemistry for pH, chlorides, and sulfates on a frequency of 5 years. Groundwater samples in the areas adjacent to Unit 1 containment structure and Unit 1 auxiliary building will also be tested for boron concentration. 12. Require supplemental inspections of the affected in scope structures within 30 days following extreme environmental or natural phenomena (large floods, significant earthquakes, hurricanes, and tornadoes). 13. Perform a chemical analysis of ground or surface water in-leakage when there is significant in-leakage or there is reason to believe that the in-leakage may be damaging concrete elements or reinforcing steel. 14. Implementing procedures will be enhanced to include additional acceptance criteria details specified in ACI 349.3R-96.			

NO.	PROGRAM OR TOPIC	COMMITMENT	UFSAR SUPPLEMENT LOCATION (LRA APP. A)	ENHANCEMENT OR IMPLEMENTATION SCHEDULE	SOURCE
		15. *When the reactor cavity is flooded up, Salem will periodically monitor the telltales associated with the reactor cavity and refueling canal for leakage. If telltale leakage is observed, then the pH of the leakage will be measured to ensure that concrete reinforcement steel is not experiencing a corrosive environment. In addition, Salem will periodically inspect the leak chase system associated with the reactor cavity and refueling canal to ensure the telltales are free of significant blockage. Salem will also inspect concrete surfaces for degradation where leakage has been observed, in accordance with this Program.*			*Salem letter LR-N10-0165 RAI B.2.1.33-2*

Enclosure D

MPR Report MPR-2613, "Salem Generating Station Fuel Handling Building—Evaluation of Degraded Condition," Revision 3 (305 pages) [PSEG Nuclear LLC VTD Number 326367], associated with response to RAI B.2.1.33-1 for Salem Generating Station Unit 1 and Unit 2 License Renewal Application

FORM-1

VENDOR INFORMATION

PSEG NUCLEAR LLC VTD NUMBER: 326367

☒	ACTIVE	Approved Documentation
☐	APCP	Approved, Pending Change Package (May only be used for Rev. 1)
☐	CAN	Canceled, Not Required
☐	VOID	No longer applicable, superseded by : ____________

Discipline Selection

☐ Electrical ☐ I&C ☐ Mechanical ☒ Other Specify: CIVIL

Safety Related: ☒ Yes ☐ No

UNIT APPLICABILITY

☐ Salem 1	☒ Salem Common	☐ Hope Creek & Salem
☐ Salem 2	☐ Hope Creek	☐ PSEG Nuclear LLC
☐ Salem 3		

System / Title: FUEL HANDLING BUILDING SAP Sys Code: FHB

Vendor Name: MPR

Vendor Code: 139M Vendor No.: MPR-2613

Vendor Category: ____________ (Category Codes are listed in DCRMS.)

Purchase Order No. ____________

Material Master: ____________

Originator: Amitava Ghose AMITAVA GHOSE Dept: DESIGN ENG Group: CIVIL

Date: 6/22/04 Ext: 3869

If changes are made to this form, initial and date the change and document in the revision summary.

USER RESPONSIBLE FOR VERIFYING REVISION, STATUS AND CHANGES
VTD 326367 000 1 PRINTED 20060531

ORM-2
REVISION SUMMARY

PSEG NUCLEAR LLC VTD No. 326367

PS REV	DOC REV	DATE	EVALUATOR *	SUPERVISOR *	DISCIPLINE INTERFACE *	REVISION DESCRIPTION
1	1	6/22/04	AMITAVA GHOSE	Alan Johnson	N/A	ORIGINAL ISSUE
2	2	5/31/06	G. Luh	Alan Johnson	A.T. Roberts 5/31/06	See record of revision.
3	3	3/9/09	G. LUH	Alan Johnson	—	INCORPORATED MPR REPORT MPR-2613, REV. 3, FEB, 2009

* SIGN & PRINT NAME

MPR
ASSOCIATES INC.
ENGINEERS

MPR-2613
Revision 3
February 2009
(PSEG Nuclear VTD 326367)

Salem Generating Station Fuel Handling Building Evaluation of Degraded Condition

QUALITY ASSURANCE DOCUMENT

This document has been prepared, reviewed and approved in accordance with the Quality Assurance requirements of 10CFR50, Appendix B, as specified in the MPR Quality Assurance Manual.

Prepared for

PSEG Nuclear LLC
Salem Generating Station
P.O. Box 236
Hancocks Bridge, NJ 08038

MPR ASSOCIATES INC. ENGINEERS

Salem Generating Station Fuel Handling Building Evaluation of Degraded Condition

MPR-2613
Revision 3
(PSEG Nuclear VTD 326367)

February 2009

QUALITY ASSURANCE DOCUMENT
This document has been prepared, reviewed and approved in accordance with the Quality Assurance requirements of 10CFR50, Appendix B, as specified in the MPR Quality Assurance Manual.

Prepared by: Matthew C. Frey

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RECORD OF REVISIONS

Revision	Affected Pages	Description
0	All	Initial Issue
1	All	• Changed terminology from "Spent Fuel Pool Building" to "Fuel Handling Building" throughout report for consistency with plant convention. • Moved the details of concrete and reinforcing steel testing to a separate report (MPR-2634).
2	All	• Included results and conclusions from additional concrete testing (including long-duration - 9 months - testing) documented in Revision 1 of MPR-2634. • Removed discussion of potential margin recovery from the ultimate strength method, and the associated calculation in Appendix A. • Revised the discussion of degradation of embedded rebar based on additional information from the open literature and reports regarding SFP leakage at another PWR. • Updated the degradation model. • Added an evaluation of the liner above localized degradation of the slab. • Added a "road map" to previous evaluations that form the current design basis for the FHB. • Added discussion of independent structural evaluation by an experienced concrete structural engineer. • Included a description of the mechanism for formation of the channel/telltale obstructions. • Provided an expanded description of a construction joint. • Added an Executive Summary.
3	All	• Updated the discussion of testing conducted by MPR to reflect results from the 39-month specimens documented in Revision 2 of MPR-2634. Updated degradation projections accordingly. • Updated evaluation of the liner above localized degradation of the slab. • Updated the assessment of potential rebar corrosion. • Incorporated results of evaluations of concrete cores from the CY SFP as corroboration of degradation modes and projections.

Executive Summary

This report, together with a companion report (MPR-2634, *Boric Acid Attack of Concrete and Reinforcing Steel*, Revision 2), represent the culmination of a multi-year effort to assess the structural adequacy of the Salem Unit 1 Fuel Handling Building (FHB). The FHB, a reinforced concrete structure, has experienced and will continue to experience degradation from boric acid leakage from the Spent Fuel Pool (SFP).

This report demonstrates the adequacy of the Unit 1 FHB through the end of plant life (70 years total). This evaluation uses results from long-duration concrete testing as well as other efforts completed after the initial assessment in 2003. These efforts have furthered the understanding of the degradation processes, supported quantification of the long-term concrete degradation rate and provided a more accurate corrosion rate for embedded rebar. This evaluation includes results from concrete cores removed from the SFP of a decommissioned plant as corroboration of the laboratory testing and degradation postulations for the Salem FHB.

Concrete Degradation

Boric acid reacts with the alkaline constituents of concrete, causing cracking and loss of bonding with the aggregate. The reacted concrete is soft and porous and has no strength. There is no impact on concrete strength other than an effective reduction in thickness corresponding to the depth of the corrosion layer.

The rate of degradation is controlled by diffusion of boric acid into the concrete. Results from the long-duration testing show that degradation is diffusion-controlled. Projections of concrete degradation are made using a square root of time curve fit of the test data, including uncertainty.

- The slab underneath the SFP has been exposed to boric acid leakage since early in plant life. This degradation is localized to the vicinity of leaking plug welds. However, as telltales became obstructed over time, the area exposed to boric acid increased until the entire slab was exposed. Re-establishing flow in the telltales and draining the stored inventory between the liner and concrete does not fully stop this mode of degradation because some telltales remain blocked and the leakage must migrate from the plug welds to channels with open telltales. The projected depth of concrete degradation in the slab is 1.3 inches assuming exposure to boric acid for 70 years.

- The walls surrounding the SFP were exposed to boric acid during the time period when the telltales were fully plugged and SFP leakage accumulated in the gap between the SFP liner and the walls. Exposure of the walls to boric acid started between 1995 and 1998 when the leakage channels and telltales became blocked and extended to early 2003 when drain flow was re-established. The projected depth of concrete degradation in the walls is 0.44 inch.

Reinforcing Steel Corrosion

Embedded reinforcing steel can potentially corrode from boric acid that migrates through the concrete. Since the concrete cover for all walls and the slab is markedly greater than the projected depth of concrete degradation, boric acid penetration into the concrete will not reach the reinforcing steel. Therefore, the only mechanism for degradation of reinforcing steel is migration of boric acid through cracks or construction joints.

Migration of boric acid through construction joints or cracks started prior to 2002, possibly as early as the 1995 to 1998 timeframe. The leakage from the building stopped subsequent to cleaning the telltales in early 2003. This mode of degradation is not expected to recur as PSEG Nuclear has implemented multiple measures to ensure that the telltales do not become entirely blocked again.

All evidence indicates that any degradation of reinforcing steel, particularly the outer reinforcing steel (i.e., the reinforcing steel of concern from a structural standpoint), is negligible. This evidence includes the following.

- Laboratory studies available in the literature of corrosion of: embedded rebar from boric acid flow through cracks; and corrosion of mild steel in de-aerated boric acid solutions.
- Inspections of potential rebar degradation from flow of boric acid leakage through a concrete crack at another US PWR.
- No visual indications of rebar degradation in the FHB, based on an independent concrete condition assessment of the Salem Unit 1 FHB by a concrete structural engineer in accordance with ACI-201.

It is conservatively estimated that the rebar has experienced a reduction in radius of less than 1 mil (0.001 inch). This is extent of degradation is negligible. Further, the fact that the actual rebar strength is greater than the specified compensates for the predicted reduction in margin by more than a factor of 10.

FHB Structural Capacity

The foregoing discussion shows that projected degradation through the end of plant life is minor and would have a small impact on available structural margin. However, per the current design basis analysis of the Salem FHB, the available margin is as low as 2% depending on the load case and the location.
Projected degradation through the end of plant life reduces the available margin in the limiting section by less than half percentage point to 1.6%. Therefore, the design basis analysis of record remains valid even with the postulated degradation.

CORROBORATION BY CY SFP CORES

Overall, the CY cores corroborate the results of testing for Salem and the projections for Salem. The maximum depth of concrete degradation in the CY cores is within that predicted using the correlation developed from the Salem testing. The rebar in the CY cores exhibited no corrosion even though the upper surface of the concrete was degraded by boric acid, the concrete was cracked and, based on the presence of secondary deposits within the concrete, there was water migration in the concrete.

Contents

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Introduction

1.1 Purpose

This report assesses the structural adequacy of the Salem Fuel Handling Building (FHB) reinforced concrete structure after prolonged exposure of the concrete and reinforcing steel to boric acid, which has leaked from the Spent Fuel Pool (SFP).

1.2 Scope of Assessment

This assessment evaluates the potential degradation of the FHB and its impact on the structural capacity by examining:

- Conservatisms in the current Salem FHB design basis,
- Results of tests, analyses, assessments and research documented in open literature that have reported the effects of boric acid on concrete and reinforcing steel,
- Results of evaluations of the impact of SFP leakage on the surrounding reinforced concrete structure at another PWR,
- Results of testing designed to determine the effect of boric acid on concrete and reinforcing steel,
- Chemical analyses of the liquid draining from the telltales and the material that blocked the telltales, and
- History of SFP leakage at Salem Unit 1.

In addition, results from petrographic examinations of concrete cores from the Connecticut Yankee Atomic Power Plant (CY) SFP are reviewed to corroborate the degradation modes and projections developed herein.

1.3 Spent Fuel Pool Description

Salem Unit 1 and Salem Unit 2 have SFPs, which are similar in construction. Each SFP is located in its corresponding unit's FHB. The Unit 1 FHB construction details are shown in References 9.4.1 through 9.4.9. The FHBs are reinforced concrete structures located on the west side of the containment structures, and each contains a new fuel storage pit, a spent fuel storage pool, and a fuel transfer pool. The buildings consist of reinforced concrete walls and a reinforced concrete roof and foundation mat. The walls vary in thickness from between 2'-2" to 10'-0". An outline of the FHBs is shown in Figure 1-1.

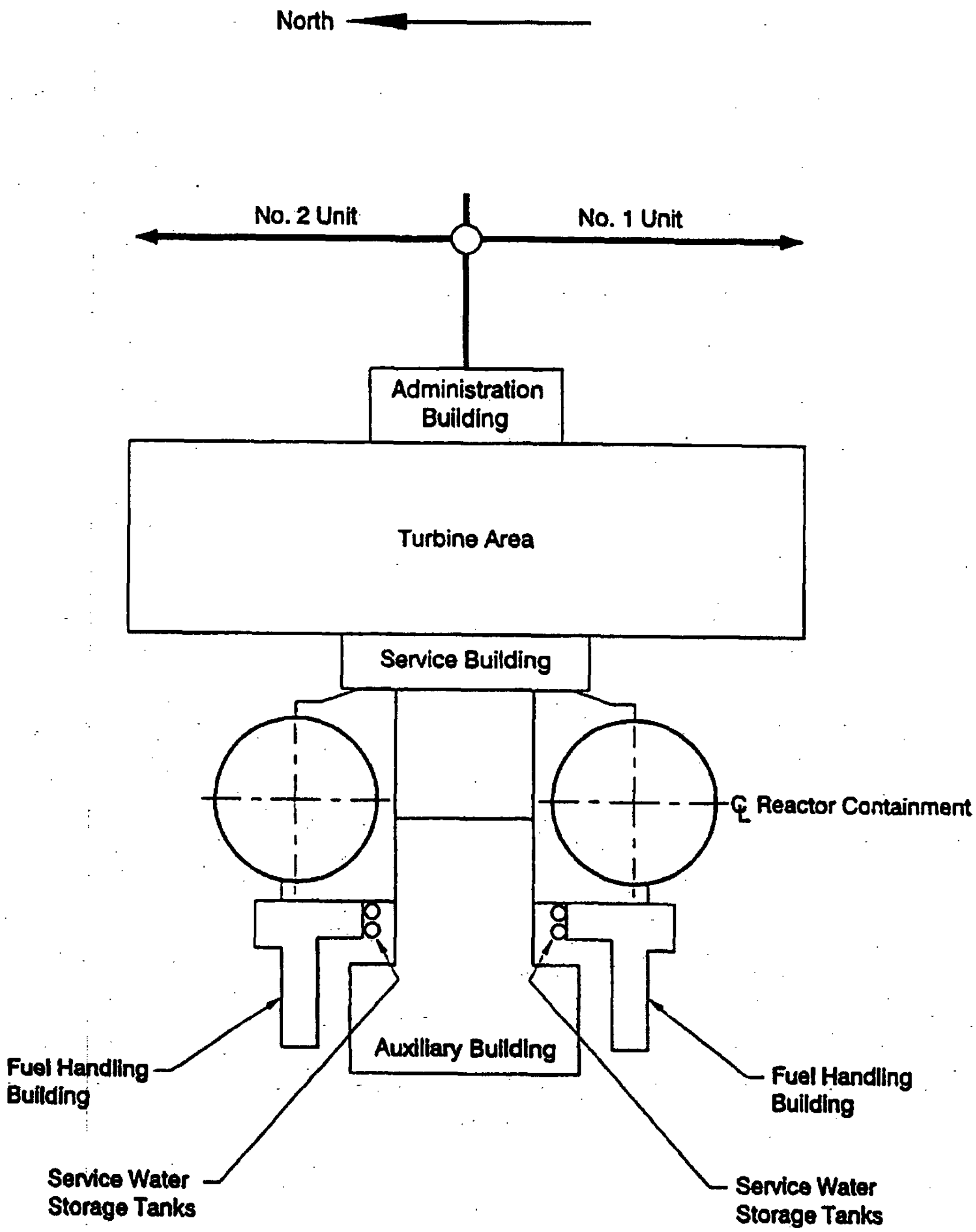

Figure 1-1. Location of Fuel Handling Buildings

At the intersection of the concrete walls and the floor slab is a construction joint. A construction joint is formed any time unhardened concrete is placed against concrete that has become sufficiently rigid that the new concrete cannot be incorporated into the old concrete by vibration.

Because of this, additional steps are taken to insure a bond between the two lifts. For example, the old concrete is usually sand blasted and a mortar or other material is used to ensure a bond. The bond is then considered to be of sufficient stiffness that the construction joint is accepted as monolithic for the purpose of determining the relative stiffness of the elements of the building.

Construction joints are prone to water migration because shrinkage and other minor cracks are likely to develop at some locations in the joint, and because the joint extends from the inside to the outside of the wall. This was evidenced by through-wall leakage at various points along the construction joints within the Salem FHB. The development of cracks sufficient to permit small amounts of water migration is not considered significant and does not affect the overall relative stiffness of the elements of the building.

The SFPs and transfer pools are lined with stainless steel plate. The liner seams are fully welded. Behind the seam welds, channels are embedded in the concrete walls and slab to collect and drain any leakage through the welds. Between the seams, the liner is attached to studs embedded in the concrete of the SFP and transfer pool structures.

Leakage from the channels is collected in a series of telltales (seventeen one-inch diameter drain pipes per unit). The channels have internal plates so that a given telltale corresponds to specific seam weld locations. The telltales are piped to a drainage canal in each unit's Sump Room.

The Sump Rooms run along the West walls of the SFPs. The telltales enter the Sump Rooms at about the floor elevation to discharge into the drainage canal. The drainage canal is separated from the remainder of the Sump Room floor by a barrier wall approximately one foot in height.

1.4 Background

On September 18, 2002, a technician working at the 78-foot elevation of the Salem Unit 1 Auxiliary Building contaminated his shoe. Investigation into the source of the contamination identified white deposits on the wall and active water inflow into the building. Further investigation determined that water from the SFP was leaking through the concrete wall into the Auxiliary Building and into the seismic gap between the buildings. Also, there was evidence of leakage into the Sump Room in the FHB via a construction joint at the base of the pool.

In early 2003 a videoscope of the telltales and leakage channels revealed that white deposits were present inside the telltale drains. Most of the telltale drains were completely blocked with the deposits. Apparently, the drains had become blocked causing leakage from the pool to accumulate in the gap between the liner and the pool. As the water level in the gap increased, hydrostatic pressure forced water into construction joints and cracks. Note that fluid may weep through the seams via minor gaps or discontinuities, however these seams should not be treated as through-wall cracks with significant amount of flow passing through them.

PSEG Nuclear snaked the telltales in early 2003 using a power auger to re-establish flow in the telltales and drain the stored inventory in the gap between the SFP liner and FHB structure. After snaking, the flow rate from the telltales increased to approximately 140 gallons/day (gpd) following the cleaning, and later decreased and held steady at 100 gpd.
The initial increase in the drainage rate following the cleaning indicates that leakage had collected in the blocked drains and channels and possibly between the liner plate and concrete.

Potential long-term exposure of the concrete structure to boric acid leakage from the SFP could degrade the structure as the acid reacts with the concrete and possibly corrodes the embedded reinforcing steel. This raises the issue of whether the potential degradation has challenged the structural adequacy of the FHB with respect to its design basis conditions.

PSEG Nuclear's strategy to address the boric acid leakage from the FHB is to:

- Demonstrate the structural adequacy of the FHB structure based on a combination of structural analyses and concrete testing; and

- Maintain flow in the leakage channels and telltales through periodic cleanings.

This report documents the assessment of the structural adequacy of the FHB.

2
Summary

2.1 Boric Acid Attack on Concrete

Formation of corrosion products is typical when concrete is exposed to acids. The corrosion layer is typically soft, cracked, and without bonding properties. In the drying process, the corrosion layer shrinks, cracks widen, and the layer crushes easily. When a corroded layer is formed, the mechanical properties of a specimen depend primarily on the quality of the 'non-corroded core' of the specimen.

The reaction between hardened cement paste or concrete and an acid solution is controlled by the diffusion of the acid into the concrete. The rate at which the concrete (or paste) degrades decreases over time as the distance acid must diffuse through degraded concrete to reach intact concrete increases. Degradation of concrete by acids follows Fick's Law of Diffusion formulation in which the depth of degradation varies with the square root of time. Hence, the rate of degradation decreases monotonically, approaching zero asymptotically. The degradation rate depends on the acid.

The acidic solution can also attack reinforcing steel embedded in the concrete. Studies of reinforcing steel corrosion due to boric acid entering reinforced concrete through a crack have shown ***negligible*** reinforcing steel attack. Tests documented in open literature demonstrate that, after an exposure time of two years, reinforcing steel corrosion was limited to scarring in the area of the crack. Studies by EPRI on corrosion of steel in de-aerated boric acid determined that the corrosion rate under de-aerated conditions (0.004 mm/year, Reference 9.5.5). Migration of boric acid through cracks and construction joints is expected to be de-aerated; however, corrosion of reinforcing steel will be even lower due to the elevated pH as the boric acid reacts with the alkaline constituents of the concrete.

Testing was performed as part of this overall assessment to determine how concrete and reinforcing steel are affected by exposure to boric acid in concentrations typical of SFP chemistry. More specifically, the testing was conducted to understand the mechanisms of concrete and reinforcing steel degradation and degradation rates. The testing used a combination of cores from the Auxiliary Building at Salem and specimens prepared using the same concrete mix and suppliers as for the concrete used at Salem. Several test series, covering exposure times up to 39 months, were conducted. The tests were conducted in a manner which is conservative with respect to the actual conditions expected in the FHB (e.g., boric acid bath was periodically refreshed to maintain acidic conditions). Key insights from the testing include the following.

- Boric acid reacts with the alkaline constituents of cement paste, causing cracking and loss of bonding with the aggregate. The reacted cement paste is soft and porous and has no strength. Fine aggregate particles are easily dislodged. This type of degradation is consistent with concrete degradation from attack by other acids.

- The wicking effect at the reinforcing steel/concrete interface is minor. That is, the degradation rate of the concrete at the reinforcing steel/concrete interface is similar to the general rate of attack of concrete away from the reinforcing steel. Hence, degradation of reinforcing steel at the construction joints or cracks with boric acid migration will not spread rapidly along the steel bar; i.e., rebar degradation is localized to the vicinity of the construction joint or crack. Functionality of the reinforcing steel is therefore maintained.
- The structural impact of boric acid attack on the concrete is reduction of the effective area carrying loads.
- The rate of concrete degradation follows a square root of time formulation, which is typical for diffusion-controlled processes. The degradation rate decreased substantially during the 39-month test series. Projected depth of affected paste through the end of plant life is 1.3 inches, including adjustment for uncertainty.

2.2 Degradation of Salem FHB

The Salem FHB has degraded from prolonged exposure to boric acid due to SFP liner leakage. The degradation has likely occurred in three different modes.

2.2.1 Local Degradation from Weld Leakage

Leakage through liner plug welds onto concrete or leakage from welds (seam or plug) overflowing blocked channels results in local degradation of the concrete structure, primarily the slab underneath the pool. The boric acid will attack the cement paste, weakening it and causing it to de-bond from the coarse and fine aggregate. As the degradation progresses, a rubble bed of coarse and fine aggregate may be formed after a significant time. In essence, local degradation will create a "pothole" with sand and coarse aggregate on top of the remaining concrete.

This mode of degradation most likely initiated prior to 1995 and is ongoing. Re-establishing flow in the telltales and draining the stored inventory between the liner and concrete did not stop this mode of degradation because the leakage must still migrate from the plug welds to channels with open telltales.

2.2.2 General Degradation from Water Trapped between the Liner and Concrete

As the leakage channels and telltales became plugged, leakage from the SFP accumulated in the gap between the liner and the concrete on the slab and the walls. The water level in the gap increased and may have equalized with the level in the pool, at which point leakage essentially stopped and conditions in the gap become stagnant. Degradation of the concrete is similar to that described above for weld leakage, except the degradation is widespread rather than localized. Virtually the entire structure surrounding the pool is potentially exposed to boric acid and subject to degradation.

The period of general degradation started between 1995 and 1998 when the leakage channels and telltales became blocked and extended to early 2003 when drain flow was re-established and degradation again became localized.

2.2.3 Rebar Degradation from Migration through Joints/Cracks

Once channels and telltales plugged and leakage accumulated in the gap between the liner and structure, the hydrostatic head forced the leakage into construction joints and cracks and ultimately into the sump room, the Auxiliary Building and the seismic gap (between the FHB and Auxiliary Building). Boric acid migration through the construction joints or cracks passed reinforcing steel, potentially initiating corrosion of the reinforcing steel. Boric acid migration through construction joints or cracks would react with the concrete prior to reaching the reinforcing steel. Hence, the pH of the leakage flow would likely be neutral or basic by the time it reaches the reinforcing steel. As noted above, studies of reinforcing steel corrosion from boric acid seepage through cracks showed negligible corrosion—only minor scarring—after two years of exposure (Reference 9.5.4). Further, measured corrosion rates of steel under static, de-aerated conditions with an acidic pH are low (< 4 microns per year) (Reference 9.5.5).

The combination of evidence - studies in the literature, inspections of the FHB, testing conducted for Salem and experience at another US PWR - suggests that corrosion of embedded reinforcing steel from boric acid migration through cracks and construction joints is negligible.

2.3 Structural Assessment

Assessment of the structural adequacy of the degraded Salem Unit 1 FHB is based on a combination of the following.

- Projections of degradation incurred to date:
  - Assessment of degradation drawing upon: evaluation of leakage from the SFP; chemical analyses of water draining through the telltales; and chemical analyses of the material blocking the telltales.
  - Degradation rates from the testing, with additional insights from other studies available in the open literature, and evaluations of similar issues at another PWR.
- Review of the existing design margin and quantification of the impact of the projected degradation on available margin.
- Evaluation of the margin that can potentially be recovered through reassessment of the actual strength of materials used (i.e., reinforcing steel and concrete).

Table 2-1 shows the available margin after each of the three degradation modes is taken into account. Positive margin is maintained in all sections.

Table 2-1. FHB Available Margin Based on Predicted Degradation Modes

FHB Wall	Percent Reduction in Capacity	Available Margin (WSD, No Degradation)	Location of Limiting Margin	Available Margin (WSD, Including Degradation)
North	0.4%	4%	Middle, Bottom	3.6%
South	0.7%	300%	West, Toward Bottom	299%
East	0.7%	5%	Middle, Toward Bottom	4.3%
West	0.4%	2%	Middle, Top	1.6%
Slab	0%	3%	Middle, Middle	3%

It is concluded that the Salem Unit 1 FHB is currently structurally adequate and can withstand the design basis load combinations for up to seventy years, total plant life. Hence, the design basis analysis of record is not invalidated by the postulated degradation.

2.4 Corroboration of Salem Assessment with Cores from CY's SFP

Overall, the CY cores corroborate the results of testing for Salem and the projections for Salem. The maximum depth of concrete degradation in the CY cores is within that predicted using the correlation developed from the Salem testing. The rebar in the CY cores exhibited no corrosion even though the upper surface of the concrete was degraded by boric acid, the concrete was cracked and, based on the presence of secondary deposits within the concrete, there was water migration in the concrete.

3 Review of Design Basis

The design basis for the FHB structures is provided in Report MPR-1863 (Reference 9.2.3), which documents the structural design analysis of the FHBs as modified to include two 10,000 gallon Service Water Storage Tanks in each unit. The most recent design analysis for the FHB before the addition of the Service Water Storage Tanks was performed during the FHB high density rack modification (Reference 9.3.1). In addition to the design analysis, the FHB was evaluated for beyond-design-basis thermal loading in EQE Calculation Number 200050-C-01 (Reference 9.3.2).

This section provides a summary of the analysis performed in Reference 9.2.3.

3.1 Analysis Method

Because the FHB and Service Water Storage Tanks are similar for Unit 1 and Unit 2, the design basis analysis in Reference 9.2.3 used bounding loads to perform a single analysis for both units. The analysis was performed by solving a three-dimensional finite element model, which included the SFP, the transfer pool, and the surrounding walls. The model divided the FHB structure into approximately seventy sections. Linearized shear and bending stresses were obtained for each section and were converted to equivalent shear loads and bending moments, respectively, for comparison with design allowables.

3.2 Design Conditions

This section defines the FHB design conditions used in the design basis analysis of Reference 9.2.3. The design conditions include the seismic category, material properties, design loads, load combinations, and design allowables.

3.2.1 Seismic Category

The FHB is a Seismic Category I Structure, per Section 3.1 of the Salem Structural Design Criteria (Reference 9.1.2).

3.2.2 Material Properties

The following material properties were defined in Reference 9.2.3 for the FHB.

- Concrete Compressive Strength: f'_c = 3,500 psi (Reference 9.4.1)
- Reinforcing Steel Yield Strength: f_y = 60,000 psi (Reference 9.4.1)

3.2.3 Load Combinations

The design load combinations for reinforced concrete structures at Salem are defined in Reference 9.1.2 and provided below. The acceptance criteria for the FHB structure, defined in Reference 9.1.2, Paragraph 7.2.1, are also listed below for each load combination.

Normal Operating Condition:
$S = D + L + I + H + B + C$

Where:

D = Dead Load
L = Live Load
I = Impact Load
H = Normal Operating Thermal Load
B = Buoyancy Load
C = Commodity Load

Stresses are limited to the working stress limits defined in Reference 9.1.3.

Operating Basis Earthquake (North-South):
$S = D + L + I + H + E_{NS} + E_V + B + C$

Where:

E_{NS} = Operating Basis Earthquake in North-South Direction
E_V = Operating Basis Earthquake in Vertical Direction

Stresses are limited to the 1-1/3 of the working stress limits defined in Reference 9.1.3.

Operating Basis Earthquake (East-West):
$S = D + L + I + H + E_{EW} + E_V + B + C$

Where:

E_{EW} = Operating Basis Earthquake in East-West Direction

Stresses are limited to the 1-1/3 of the working stress limits defined in Reference 9.1.3.

Design Basis Earthquake (North-South):
$S = D + L + I + H' + E'_{NS} + E'_V + B + P + J + C$

Where:

E'_{NS} = Design Basis Earthquake in North-South Direction
E'_V = Design Basis Earthquake in Vertical Direction
H′ = Maximum Thermal Load during Abnormal Conditions
P = Internal Pressure Load
J = Pipe Whip Load

Loads are based on Ultimate Strength Design, limiting the stresses in the reinforcing steel to 90% of the yield strength, per Reference 9.1.3.

Design Basis Earthquake (East-West):
$S = D + L + I + H' + E'_{EW} + E'_V + B + P + J + C$

Where:

E'_{EW} = Design Basis Earthquake in East-West Direction

Loads are based on Ultimate Strength Design, limiting the stresses in the reinforcing steel to 90% of the yield strength, per Reference 9.1.3.

Tornado Loading:
$S = D + L + I + H' + W_t + B + C$

Where:

W_t = Tornado Loading

Loads are based on Ultimate Strength Design, limiting the stresses in the reinforcing steel to 90% of the yield strength, per Reference 9.1.3.

3.2.4 Design Allowables

Working Stress Design Method
The method for determining the allowable loads based on working stress design are presented in Part IV-A of Reference 9.1.3. A summary of the analysis method is provided below.

Allowable Moment (M):
The assumptions for flexural design using the working stress design methods is provided in Reference 9.1.3, Paragraph 1101. These assumptions are:

- A plane section before bending remains plane after bending. Stresses vary linearly with the distance from the neutral axis.
- The stress-strain relationship for concrete is a straight line under service loads and within the allowable stresses.

- The steel takes all of the tensile stress due to flexure.
- The area of reinforcing steel is replaced by an equivalent area of concrete, scaled by the Modular Ratio (n).
- In the tensile stress zone the concrete is assumed cracked and is not supporting tensile stress. Therefore, the compressive stress above the neutral axis is all carried by the concrete, and the tensile stress below the neutral axis is carried by the equivalent area of the steel. The stress distribution is shown in Figure 3-1.

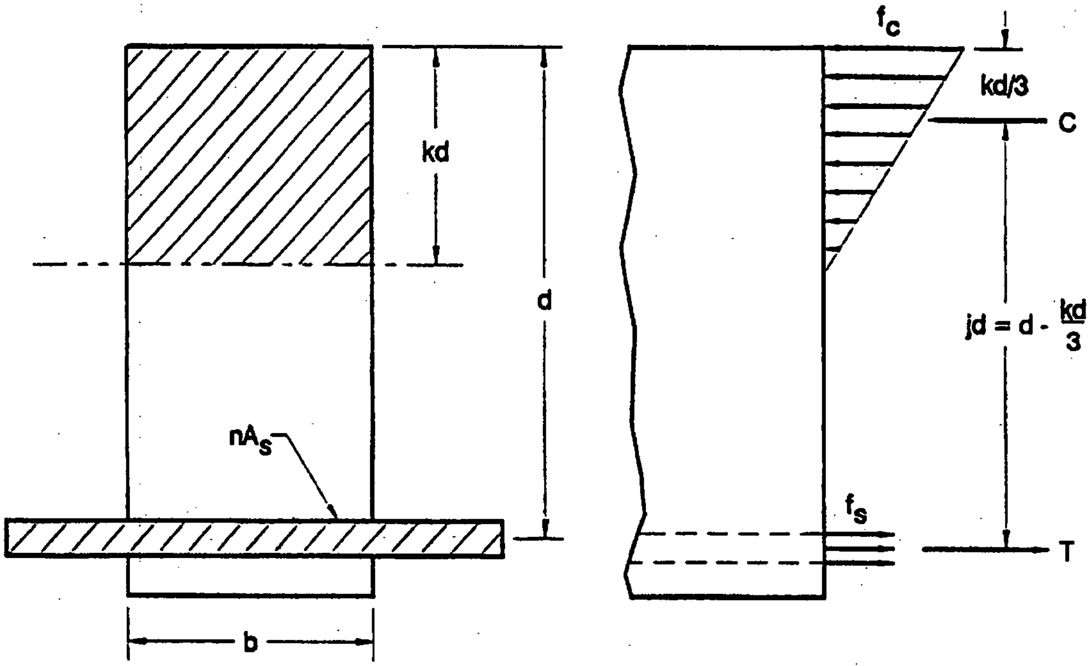

Figure 3-1. Stress Distribution for Working Stress Design

The allowable moment for working stress design, which is based on yielding of the steel limiting failure rather than the concrete limiting failure, is:

$M = A_s f_s jd$

Where:

A_s = equivalent area of steel reinforcement
f_s = allowable stress of steel for working stress design
= 24,000 psi (Reference 9.1.3, Paragraph 1003(a))
jd = distance between tension and compression forces
d = distance from extreme compression fiber to centroid of tension reinforcement

Allowable Shear Load (V):

The allowable average shear load across a section is:

$$V = v_c bd$$

where:

b = width of concrete section
v_c = allowable concrete shear stress
$= 1.1\sqrt{f_c'}$ for working stress design (Reference 9.1.3, Paragraph 1201(c))

Ultimate Strength Design

The method for determining the allowable loads based on ultimate strength design is presented in Part IV-B of Reference 9.1.3. A summary of the analysis method is provided below.

Ultimate Moment (M_u):

The assumptions for flexural design using the ultimate strength design method are provided in Reference 9.1.3, Paragraph 1503. These assumptions are:

- At ultimate strength the stress in the concrete is considered to be 0.85 f_c' distributed over a rectangular area bounded by the edges of the cross section and extending a distance (a) into the depth of the cross section.

- The stress in the steel is assumed to be at yield (note that this is modified by Reference 9.1.3 to a stress of 90% of yield).

- In the tensile stress zone the concrete is assumed cracked and is not supporting tensile stress. The assumed stress distribution is shown in Figure 3-2.

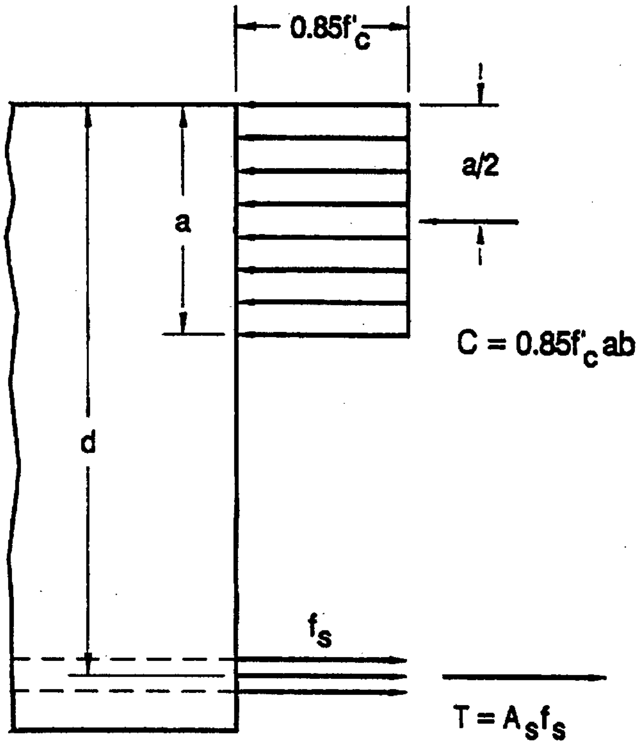

Figure 3-2. Stress Distribution for Ultimate Strength Design

The ultimate moment is given as:

$$M_u = \phi\left[A_s f_{s,u}\left(d - \frac{a}{2}\right)\right]$$ (Reference 9.1.3, Paragraph 1601(a))

where:

ϕ = 0.9 for flexure (Reference 9.1.2, Paragraph 7.2.1)
A_s = equivalent area of steel reinforcement
$f_{s,u}$ = 90% of yield strength of steel = $0.90f_y$ (Reference 9.1.2, Paragraph 7.2.1)
d = distance from extreme compression fiber to centroid of tension reinforcement
a = depth of compression zone (in.)

$$= \frac{A_s f_{s,u}}{0.85 f_c' b}$$ (Reference 9.1.3, Paragraph 1601(a))

f_c' = concrete compressive strength
b = width of compression face

Ultimate Shear Load (V_u):
The allowable average shear load across a section is:

$$V = v_c bd$$

where:

v_c = allowable shear stress for concrete
$= 2\phi\sqrt{f_c'}$ for ultimate strength design (Reference 9.1.3, Paragraph 1701(c))

3.3 CURRENT DESIGN MARGIN

Twenty-four design margins (equal to the applied load over the allowable load) were calculated in Reference 9.2.3 for each section of the FHB; one for each of the six load combinations under each of the following four load types.

- Horizontal Shear Load
- Vertical Shear Load
- Horizontal Bending Moment
- Vertical Bending Moment

Results are provided in Table 3-1 by load type. The most limiting design margin from among all FHB sections for each of the six load combinations is provided. Also, the applied and allowable loads are listed. Recall that the allowable loads for the normal operating condition and OBE are based on working stress design whereas the allowable loads for the other conditions are based on ultimate strength design. As shown, the normal operating condition and the OBE have the lowest margins for each load type. The low margins for the normal operating condition and OBE result from lower allowable loads associated with the working stress method.

Table 3-1. Loads and Margins for Each Load Combination

Load Combination	Limiting Location	Applied Load[1]	Allowable Load[1,3]	Margin[2]
Horizontal Shear Load Analysis				
Normal Operating	South Wall - East, Middle	-21.4	-42.4	1.99
East-West OBE	South Wall - East, Middle	-24.5	-56.6	2.31
North-South OBE	North Wall - West, Top	-31.0	-104.	3.36
East-West DBE	South Wall - East, Middle	-26.9	-65.5	2.44
North-South DBE	North Wall - West, Top	-40.0	-120.	3.00
Tornado	East Wall – North, Top	30.7	80.1	2.61
Vertical Shear Load Analysis				
Normal Operating	East Wall - Middle, Bottom	15.0	51.8	3.45
East-West OBE	West Wall - Middle, Bottom	-25.5	-114.	4.47
North-South OBE	East Wall - Middle, Bottom	15.2	69.1	4.55
East-West DBE	West Wall - Middle, Bottom	-28.6	-132.	4.62
North-South DBE	East Wall - Middle, Bottom	14.7	80.1	5.44
Tornado	East Wall - Middle, Bottom	19.9	80.1	4.02
Horizontal Bending Moment Analysis				
Normal Operating	Slab - Middle, Middle	-191.2	-197.	1.03
East-West OBE	West Wall - Middle, Top	-293.6	-299.	1.02
North-South OBE	Slab - Middle, Middle	-188.9	-263.	1.39
East-West DBE	West Wall - Middle, Top	-369.7	-469.	1.27
North-South DBE	Slab - Middle, Middle	-256.1	-409.	1.60
Tornado	East Wall - Middle, Top	-207.4	-285.	1.37
Vertical Bending Moment Analysis				
Normal Operating	North Wall - Middle, Bottom	-147.7	-154.	1.04
East-West OBE	North Wall - Middle, Bottom	-142.2	-206.	1.45
North-South OBE	East Wall - Middle, Toward the Bottom	-97.2	-137.	1.41
East-West DBE	North Wall - Middle, Bottom	-185.5	-321.	1.73
North-South DBE	West Wall - Middle, Toward the Bottom	-203.8	-353.	1.73
Tornado	West Wall - Middle, Toward the Bottom	-216.3	-353.	1.63

Notes:

1. Negative loads indicate compression on the pool side of the wall.
2. Margin is defined as Allowable Load / Applied Load.
3. Normal Operating and OBE allowables are based on working stress design methods, whereas the DBE and Tornado allowables are based on ultimate strength design methods.

Table 3-2 identifies all sections having design margins less than 10% from among all load combinations and load types. Also included in the table are the applied loads and allowable loads used to calculate the design margins, the locations of the limiting sections, and the load combinations and load types that produced each applied load and allowable load. All of the cases with less than 10% margin are for normal operation and OBE. Once again, the low margins result from low allowable loads associated with the working stress method.

Table 3-2. Limiting FHB Design Margins from the Design Basis Analysis

Load Combination	Load Type	Limiting Section Location	Applied Load[1] (kip-ft/ft)	Allowable Load[1] (kip-ft/ft)	Design Margin[2]
Normal Operation	Horizontal Moment	West Wall - Middle, Towards Bottom	-215	-225	1.04
		West Wall - Middle, Towards Top	-216	-225	1.04
		West Wall - Middle, Top	-208	-225	1.08
		East Wall - Middle, Towards Top	-125	-136	1.09
		Slab - Middle, West	-184	-197	1.07
		Slab - Middle, Middle	-191	-197	1.03
	Vertical Moment	North Wall - Middle, Bottom	-148	-154	1.04
		East Wall - Middle, Towards Bottom	-98	-103	1.05
East-West OBE	Horizontal Moment	West Wall - Middle, Towards Top	-279	-299	1.07
		West Wall - Middle, Top	-294	-299	1.02
		West Wall - South, Top	-274	-299	1.09

Notes:

1. Negative loads indicate compression on the pool side of the wall.
2. Margin is defined as Allowable Load / Applied Load.

Review of Tables 3-1 and 3-2 indicates that most of the limiting margin cases, including all of the cases with less than 10% margin, have negative loads which denotes compression on the pool side of the wall (or slab) and tension on the outside of the wall (or slab). Since reinforcing steel carries the tensile loads, the reinforcing steel of primary concern with regard to structural margin is the rebar near the outside of the wall - the side farthest from the pool and farthest from the spent fuel pool water which may reside in the gap between the liner and the wall. Because concrete carries compressive loads, the concrete of primary concern with respect to structural margin is that beside the liner gap.

4 Potential Margin Recovery

As discussed in Section 3, the current design basis analysis of the FHB shows that little design margin (less than 10%) exists in several areas of the FHB, allowing for very little degradation of the FHB concrete and reinforcing steel. This section evaluates the margin that can potentially be recovered through assessment of measured material properties.

4.1 Reinforcing Steel Capacity

Per Reference 9.4.1, the reinforcing steel in the FHB structure has a specified minimum yield strength of 60 ksi. However, the actual yield strength of reinforcing steel is typically higher than the specified minimum value. Using the actual yield strengths of the reinforcing steel is a potential method to recover margin in the FHB.

During construction of the Salem units, tensile testing of reinforcing steel was performed to verify that the yield and ultimate strengths met or exceeded the minimum specified value. MPR Calculation 108-275-02 (Reference 9.3.4, provided in Appendix B) documents a statistical analysis on a sample of this yield strength test data. The total sample population was comprised of sub-samples of each reinforcing steel size present in the FHB structure. The statistical analysis determined the mean yield strengths for each sub-sample and the total sample population. The analysis also characterized the distribution of yield strengths in terms of the percentage of each sub-sample and the total sample population greater than a given yield. Results from the statistical analysis are provided in Table 4-1. The yield strength distribution for the total sample population is shown graphically in Figure 4-1.

Table 4-1. FHB Reinforcing Steel Yield Strength Analysis Results

Rebar Type	Mean Yield Strength	Yield Strength Std. Dev	Sample Size	80% Lower Bound[1]	85% Lower Bound[1]	90% Lower Bound[1]	95% Lower Bound[1]
Total	69,840	6,370	394	64,100	63,300	62,200	61,300
No. 6	67,092	5,027	13	64,100	63,850	62,550	62,550
No. 8	69,410	7,376	123	63,500	63,000	61,500	60,750
No. 9	70,815	6,591	95	63,800	63,000	62,500	61,600
No. 10	70,934	5,024	47	66,150	65,400	62,200	62,100
No. 11	69,363	5,490	116	64,950	63,300	62,065	60,900

Notes:
1. The indicated percentages of the sample sizes have yield strengths greater than those shown.
2. All yield strengths are in psi.

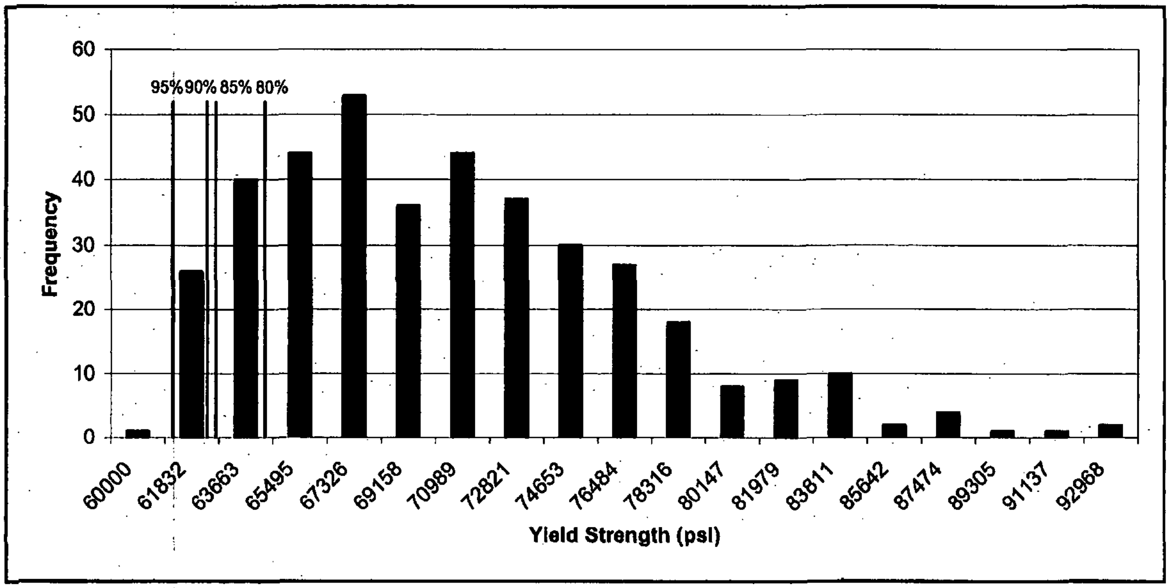

Figure 4-1. FHB Reinforcing Steel Yield Strength Distribution of Population Sampled

The above table and figure show that the actual yield strengths of the reinforcing steel in the Salem FHB are larger than the minimum specified 60 ksi. Based on the total population evaluated, the mean yield strength is almost 70 ksi and the 95% lower bound is 61.3 ksi (i.e., 95% of the data are greater than 61.3 ksi).

In the working stress design method, the allowable reinforcing steel stress is equal to 40% of the nominal yield strength of the material. For reinforcing steel with a yield strength of 60 ksi, the allowable stress is then 24 ksi. To assess the margin recovered using measured yield strength, the 95% lower bound value of 61.3 ksi is used to conservatively bound a significant portion of the reinforcing steel. Using 40% of 61.3 ksi (24.52 ksi) as the yield strength in the methodology described in Appendix D increases the design margin by about 2%.

4.2 CONCRETE CAPACITY

The testing documented in Reference 9.2.4 included compressive strength tests for concrete specimens prepared using the same mix design and same raw material suppliers as the concrete used in the FHB. The tests showed that the concrete mixture used at Salem has a compressive strength of about 6,000 psi, compared to a specified design value of 3,500 psi. The impact of concrete strength on the concrete capacity can be assessed by review of the calculation in Appendix D. The moment capacity of the concrete is not sensitive to the actual concrete strength. Specifically, the increase in concrete strength from 3,500 psi to 6,000 psi provides a very small (<< 1%) increase in the available margin. Accordingly, the potential for margin recovery from measured concrete strength is not considered further.

4.3 CONCLUSIONS

Design margins in the Salem FHB under normal operating conditions and OBE conditions may be slightly improved through the use of measured material properties as opposed to specified or nominal properties. In the working stress design method, the allowable reinforcing steel stress is equal to 40% of the nominal yield strength of the material. Using 40% of the actual yield strength of the reinforcing steel in the working stress design calculations in lieu of the specified normal allowable recovers 2% margin. Using the actual compressive strength of the concrete recovers a negligible amount of margin.

The limiting margin in the FHB can be increased from 1.02 to at least 1.04 by taking credit for the actual yield strength of the reinforcing steel. Subsequent sections will show that degradation expected from boric acid attack is less than that required to challenge the structural capacity of the FHB.

5 Boric Acid Attack of Concrete and Reinforcing Steel

Several activities were performed to assess the impact of boric acid on concrete and reinforcing steel. First, MPR performed a review of industry literature regarding the effects of boric acid and other acids on concrete and reinforcing steel. The results of the review were previously provided to PSEG Nuclear via Reference 9.6.3. Second, MPR conducted testing to determine how concrete and reinforcing steel are affected by exposure to boric acid. The testing was research-oriented in nature with the goals of understanding the mechanisms of concrete and reinforcing steel degradation and quantifying degradation rates. Details of the testing are documented in MPR-2634 (Reference 9.2.4). Third, MPR reviewed evaluations of concrete and embedded rebar degradation from SFP leakage at another US PWR. Insights identified from the literature review and testing are provided in the following sections.

5.1 Chemical Reactions between Acid and Concrete

References 9.5.2 and 9.5.3 provide excellent discussions of the mechanisms of concrete degradation from acid attack. Cement paste in concrete is easily attacked by acidic solutions due to its high alkalinity. As the acid attacks the concrete, the cement constituents are altered by decalcification, leading to degradation of the concrete properties. Portlandite ($Ca(OH)_2$) is the first cement constituent to react with the acid. Calcium silicate hydrate also reacts with the acid. In most cases of acidic attack, the chemical reactions result in the formation of calcium salts. The corrosive effect of an acid depends on the solubility of these salts; a higher solubility contributes to the progression of attack.

The reaction between hardened cement paste or concrete and an acid solution is controlled by diffusion of the acid into the concrete. The rate at which the concrete (or paste) degrades decreases over time as the distance acid must diffuse through degraded concrete to reach intact concrete increases. Degradation of concrete by acids follows a Fick's Law of Diffusion formulation in which the depth of degradation varies with the square root of time. Hence, the rate of degradation decreases monotonically, approaching zero asymptotically.
The degradation rate depends on the acid. The typical signs of acidic attack include a gradual loss of alkalinity, loss of mass, and loss of strength and rigidity.

Reference 9.5.3 provides additional insight regarding the degradation of concrete due to acid attack. Although concrete degradation is typically higher when soluble salts, as opposed to insoluble or nearly insoluble salts, are formed during the reaction process, formation of insoluble or nearly insoluble salts can create microcracks during crystallization, which can lead to spalling of the concrete.

If a cement matrix is continuously immersed in an acidic solution rather than exposed to alternate wetting and drying cycles, the expansion caused by salt crystallization is less, and may not occur at all. The paper also states that in the first few days or weeks of exposure to acid, cement based material can become denser with corresponding increases in weight and compressive strength. These phenomena, attributed to small amounts salt crystallization and deposition of corrosion products in the relatively open pore structure of the cement based material, are reported to be temporary until the salt crystallization is high enough to show deteriorating effects (spalling, cracking, etc.).

5.2 LITERATURE STUDIES ON BORIC ACID ATTACK

5.2.1 Degradation of Concrete and Cement Paste

Reference 9.5.2 investigated the effect of various acids (boric acid was not included) on concrete. Results of testing showed that formation and growth of a layer of reaction products is typical for concrete exposed to acids. The degraded layer is usually soft, cracked, and without bonding properties. In the drying process, the degraded layer shrinks, cracks widen, and the layer can be crushed easily. When a degraded layer is formed, the mechanical properties of a specimen depend primarily on the quality of the 'non-degraded core' of the cement paste.

The testing documented in Reference 9.5.2 also demonstrated that attack of Portland cement concrete by weak acids, such as boric acid, usually results in low depths of penetration, and is diffusion-controlled. Curve fits of the test data show that the depth of degradation versus time follows a Fick's Law of Diffusion formulation—depth increases with the square root of time. Further, this reference states that corrosion rates are dependent upon the pH value of the solution.

Reference 9.5.1 documents the results of testing on hardened Portland cement paste specimens that were cured in boric acid solutions, and on concrete exposed to boric acid in the field. The cement paste specimens were exposed to boric acid solutions for up to 127 days. The results of the testing showed that the weight, bulk density, and compressive strength of the specimens increased due to boric acid exposure, and the porosity of the specimens decreased. No specimen degradation was reported. The paper attributed these results to the formation of low-soluble hydrated calcium borates from the reaction between the boric acid solution and Portlandite, which filled up the pore system of the cement paste. The paper noted that the test results were different from typical acid attack, which usually results in a loss of weight, decrease in density and compressive strength, and increase in porosity.

Applying the discussion of Reference 9.5.3 to the results of the cement paste specimen testing documented in Reference 9.5.1, the cement paste specimens may have increased in density, weight, and compressive strength and showed a decrease in porosity because the reaction rate did not have the opportunity to increase to the point where salt crystallization could have deteriorating effects on the specimens. The slow increase in pH after the first week of testing shows that the reaction rate was low. As previously discussed, Reference 9.5.3 indicates that increases in density and compressive strength after exposure to acid is a temporary phenomena that can occur during the first days or weeks of exposure.

5.2.2 Compressive Strength of Concrete

Reference 9.5.1 documents the results of compressive strength tests performed on concrete exposed to boric acid in the field. The results showed that boric acid had no effect on the compressive strength of the concrete, or any other properties that were tested. No degradation of the concrete was reported. While the boric acid appeared to have no effect on the concrete, factors that affect attack, such as the pH of the solution, whether or not the solution was refreshed, and length of time the concrete was exposed to the solution, were not provided in the reference. Therefore a strong conclusion related to the effect of boric acid on concrete can not be made with respect to this test. The paper did report that the concrete aggregate was limestone. Because the limestone aggregate can react with the acid, the findings may not be applicable to concrete mixes using different aggregates.

5.2.3 Corrosion of Rebar

Reference 9.5.4 reports on testing performed to study the effects of reinforcing steel corrosion due to boric acid entering reinforced concrete through cracks. The tests showed that corrosion increases as crack width increases and pH decreases. In particular, the tests showed *negligible* reinforcing steel attack even when specimens were subjected to the most corrosive test environment (pH of 5.2) with the largest crack width (0.4 mm) for a period of two years. Corrosion was limited to scarring in the area of the crack.

5.3 MPR (CRT) Testing of Boric Acid Attack to Support Salem FHB Evaluation

Reference 9.2.4 documents testing conducted by MPR to support the FHB structural evaluation. Specifically, the testing determined how concrete and reinforcing steel are affected by exposure to boric acid. The testing used concrete cores from the Salem Auxiliary Building and additional specimens prepared using the same concrete mix and suppliers as for the Salem concrete. The specimens were soaked in a boric acid bath with a boron concentration consistent with the SFP. The bath was periodically refreshed to maintain acidic conditions. The testing was research-oriented in nature with the goals of understanding the mechanisms of concrete and reinforcing steel degradation and quantifying degradation rates, rather than to closely replicate the conditions behind the SFP liner. The testing was performed under MPR's 10CFR50 Appendix B Quality Assurance Program.

5.3.1 Boric Acid Attack on Concrete

The testing included exposure of concrete specimens to a boric acid solution for up to 9 months. The testing used a combination of cores from the Auxiliary Building at Salem and cylinder-rebar specimens prepared using the same concrete mix and suppliers as for the concrete used at Salem. Microscopic examinations and chemical analyses were performed on the specimens after exposure to the solution. The results showed that boric acid reacted with the alkaline constituents of the cement paste of concrete. This reaction caused cracking and a loss of bonding

from the fine and course aggregate, and left the reacted paste soft and porous with no strength; fine aggregate particles were easily dislodged.
The rate of degradation decreased with the square root of time, as is expected for diffusion-controlled processes. Based on Reference 9.5.2, the projected depth of degradation after 70 years[1] of exposure is 1.30 inches, including adjustments for temperature and uncertainty.

Compressive strength testing was performed to assess the impact of boric acid degradation on concrete strength after 56 days of exposure. The apparent compressive strength for specimens soaked in boric acid was lower than that for control specimens soaked in tap water. However, the difference in compressive strengths can be explained by accounting for the reduction in cross-sectional area from boric acid attack.

5.3.2 Boric Acid Attack on Reinforcing Steel

Boric acid attack of reinforcing steel was investigated by MPR (CRT) using concrete specimens with embedded rebar. The specimens were soaked in boric acid for up to 56 days. The cylinder-rebar specimens provided insights on rebar corrosion beneath the concrete surface and the wicking rate of boric acid along the rebar.

- Only one of the specimens exhibited any reinforcing steel corrosion below the concrete surface. This specimen showed very minor surface corrosion just beneath the concrete surface. This specimen had a surface discontinuity at the rebar-to-concrete interface, which allowed the boric acid solution to contact the rebar below the nominal concrete surface. Hence, the observed corrosion is not indicative of the corrosion of embedded rebar.

- The wicking rate along the concrete/rebar interface was minor. That is, the degradation of concrete at the concrete/rebar interface is similar to the general rate of attack of concrete without rebar. Therefore, any degradation of reinforcing steel will remain localized to the region where boric acid contacts the rebar.

5.4 EVALUATIONS FROM ANOTHER PWR

Other PWRs have also experienced SFP leakage and evaluated the impact of boric acid on the concrete structure surrounding the SFP. Reference 9.2.5 documents the evaluation at one of these plants. The plant in question experienced leakage from the SFP which migrated through a crack in the concrete to an adjacent space underneath the SFP. The leakage occurred over a period of several years. Concrete was chipped away to expose rebar in the vicinity of the crack. The crack ran parallel to the rebar, directly next to the rebar. Inspections of the exposed rebar revealed no discernable corrosion of the rebar. This observation is consistent with the negligible observed corrosion of rebar exposed to boric acid via concrete cracks determined in Section 5.2.3, above.

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[1] Degradation was projected over 70 years to provide a bounding projection that envelopes potential license renewal and storage of fuel in the SFP for 10 years after cessation of operations. Use of 70 years in the evaluation should not be interpreted as a commitment by PSEG Nuclear to pursue license renewal of Salem.

5.5 CONCLUSIONS

Key insights from the literature review, and the testing, as well as the experience at other PWRs are as follows.

- Formation and growth of a layer of reaction products is typical for concrete exposed to acids, including boric acid. The degraded layer is usually soft, cracked, and without bonding properties. In the drying process, the corrosion layer shrinks, cracks widen, and the layer can be crushed easily. When a degraded layer is formed, the mechanical properties of a specimen depend mainly on the quality of the 'non-degraded core' of the cement paste.

- The attack of Portland cement concrete by weak acids, such as boric acid, usually results in low depths of penetration, and is controlled by diffusion of the acid into the concrete. Curve fits of the test data show that the depth of degradation versus time follows a Fick's Law of Diffusion formulation—depth increases with the square root of time.

- The extent of reinforcing bar corrosion in reinforced concrete depends primarily on the crack width and the pH value of the solution. An increasing corrosion rate is observed for larger crack widths and lower pH values.

- Significant reinforcing steel corrosion is not expected when boric acid is introduced to the steel through a crack in the concrete because the rebar is protected by the alkaline concrete matrix. Laboratory studies and experience at another PWR show negligible corrosion of embedded rebar after 2 years of exposure to boric acid.

- The wicking effect at the interface of concrete and rebar is minor. Therefore, degradation of rebar at the construction joints or cracks with migration will not spread rapidly along the rebar; i.e., rebar degradation is localized to the vicinity of the construction joint or crack.

6
Assessment of Potential Damage to Structure

This section assesses the nature of potential degradation of the FHB structure from exposure to the boric acid solution leaking from the SFP. The assessment draws upon:

- Evaluation of leakage from the SFP,
- Chemical analyses of water draining through the telltales,
- Chemical analyses of the material blocking the telltales, and
- Independent structural assessment per ACI guidelines by an experienced concrete structural engineer.

Based on these evaluations, degradation of the structure is estimated for portions of the concrete that have potentially been exposed to boric acid from the SFP.

6.1 LEAKAGE EVALUATION

It is generally considered that the source of liner leakage is cracking of the liner seam welds and/or the plug welds attaching the liner to the studs embedded in the concrete. Since the backing bar for the seam welds is tied to the embedded leakage channels and the plug welds are tied to embedded studs, these welds can be highly stressed due to differential thermal expansion between the liner and the concrete structure. Given the large number of seam welds (about 2,100 feet) and plug welds (about 1,400), it is likely that there are multiple leaking cracks as opposed to a single large crack. The plug welds are considered more likely to crack and leak on the basis that the differential thermal expansion loads are more concentrated resulting in high stresses.

6.1.1 Crack Size/Leakage Rate

Scoping calculations were performed to estimate the crack size necessary to produce the nominal leakage rate of 100 gpd. The required crack length varies with the hydrostatic head across the crack (i.e., elevation of crack and water level in gap) and the crack width. If the crack is on the bottom of the pool and there is no water in the liner/wall gap, the crack length necessary to produce 100 gpd ranges from about 0.5 inch for a 0.003 inch wide crack to about 6 inches for a 0.001 inch wide crack.

The scoping calculations suggest that the crack or cracks causing the leakage are very small, particularly in comparison to the total length of seam welds and number of plug welds. Small, tight cracks are difficult to locate. It is unlikely that video inspections with underwater cameras or vacuum box testing would be able to successfully locate such cracks.[2]

6.1.2 Leakage from Seam Welds versus Leakage from Plug Welds

As discussed above, cracks could occur in the liner seam welds and/or the plug welds to embedded studs. The flow path for each leak location is described below.

- **Seam Weld Leakage.** Leakage through seam welds collects in the leakage channel embedded in the concrete and flows out the telltale to a trough in the Sump Room. Provided the leakage channels and telltales are not obstructed, the boric acid solution from the SFP does not contact the concrete of the FHB structure.

- **Plug Weld Leakage.** Leakage associated with a plug weld exposes concrete to the boric acid solution from the SFP. Leakage from a weld on the pool bottom drips onto the concrete slab, forming a puddle, which grows until it, overflows into a leakage channel and is routed to a telltale. For leakage through a plug weld in a wall, the boric acid solution runs down to the slab forming a puddle which grows until it overflows into a leakage channel. Exposure of the concrete to the boric acid solution is limited to the flow path from the leak to an open channel.

When the leakage channels and telltales are obstructed, leakage from the SFP accumulates in the gap between the FHB structure and the SFP liner, exposing much larger areas of the structure to the boric acid solution and potential degradation.

6.2 TELLTALE CHEMISTRY

PSEG Nuclear's Chemistry Department has analyzed samples of the liquid discharge from the telltales. The samples have been subjected to both chemical analysis and isotopic analysis. The analyses are documented in Reference 9.2.2; key results and insights are provided below.

- Isotopic analysis of liquid samples collected in December 2002 just prior to snaking of the telltales shows that the isotopic signature is consistent with the SFP chemistry with about five years of decay.

- The average pH of SFP telltale samples collected after cleaning the telltales was 7.1, compared to an expected pH of 4.6. The transfer pool telltales showed a similar trend: measured pH of 7.8 compared to an expected pH of 4.8. The high pH values indicate that the boric acid solution has reacted with alkaline constituents of the concrete.

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[2] In 1995, the Salem Unit 1 SFP was inspected for weld leaks using vacuum box testing. Almost 95% of the seam welds were inspected using vacuum box testing with no indications of a crack; the remainder of the welds could not be inspected due to limited access under the fuel racks (Reference 9.3.6).

The above results indicate that the liquid accumulated in the gap was about 5 years old and that the boric acid had reacted with the concrete structure. Since basic solutions are relatively benign to carbon steel, the high pH values suggest that active degradation of any carbon steel exposed to the boric acid solution had largely slowed over the time period the liquid was trapped behind the liner.

Analysis of telltale samples taken on 11/20/03 show pH values ranging from 6.1 to 7.2, with an average of 6.8 (Reference 9.6.4). As discussed above, pH values are higher than the typical SFP pH of 4.6 indicate that the boric acid is reacting with the concrete. Hence, even though the stored inventory of has been eliminated from behind the liner, boric acid is coming into contact with concrete. This indicates that concrete degradation is continuing.

6.3 Chemical Analysis of Material Blocking Telltales/Channels

PSEG Nuclear obtained samples of the solid material that was obstructing the channels/telltales and contracted with Framatome-ANP for analysis of the samples. The analyses indicate that the deposits are largely quartz (SiO_2) and calcite ($CaC0_3$) with minor amounts of gismondine ($CaAl_2Si_2O_8.4H_2O$) (Reference 9.6.5). In other words, the material obstructing the telltales and leakage channels derives from the concrete of the FHB.

The mechanism for formation of the blockages in the telltales/leakage channels is not well understood. The calcite likely precipitates out of solution as dissolved calcium compounds are carbonized by reaction with carbon dioxide from the air in the leakage channels and telltales. The dissolved calcium compounds derive from the concrete via one of the following processes.

- Attack of the concrete by boric acid leaking from the SFP plug welds results in the generation of dissolved calcium compounds such as calcium hydroxide and calcium borates.
- Groundwater in-leakage through cracks or constructions can leach calcium species from the concrete as it migrates through and over the concrete on its way to the leakage channels and telltales.

Precipitation of calcite and other species can also be impacted by evaporation of water and solubility changes as SFP leakage cools to the concrete temperature. In short, the process for forming the blockages is similar to the formation of stalactites and stalagmites in a cave.

The fact that the obstructions include materials derived from concrete indicates that concrete degradation was occurring prior to the drains becoming plugged. This is clear evidence that the liner plug welds are leaking (see Section 6.1.2), because seam weld leakage would be diverted immediately to the drain channels without contact with concrete.

6.4 Leakage Timeline

Drainage from the telltales has been noted since 1980. Key milestones regarding SFP leakage are identified below and presented as a timeline in Figure 6-1.

- **1981.** A modification to address seam weld leakage was implemented in 1981. This modification consisted of positioning seam encasements over leaking seam welds, and welding the encasements to the liner (Reference 9.6.1).

- **1995.** A project to install high-density fuel racks to increase SFP storage capacity was implemented in 1995. Significant leakage through the telltales was noted during the project (Reference 9.3.6), particularly during rack moves. This was likely the result of water being pushed out from between the liner and concrete slab during the changes in floor loading. Therefore, it appears that portions of the liner to concrete gap in the slab were flooded with boric acid from the pool.

- **1998.** Isotopic analysis of telltale samples from when the drains were cleaned in early 2003 suggest that the liquid accumulated in the gap between the liner and concrete was about 5 years old.

- **2002.** In the Fall of 2002, PSEG Nuclear identified leakage from the FHB into the Auxiliary Building and into the seismic gap. In addition, leakage into the sump room via a construction joint was noted. This indicates that the liner to concrete gap was flooded up to an elevation above the fuel pool slab.

- **2003.** In January 2003, PSEG Nuclear cleaned the drains to re-establish flow from the channels and telltales. The drain flow increased significantly following cleaning. Later in the year, PSEG Nuclear performed hydrolazing to remove more blockages.

It appears that leakage from the plug welds likely initiated sometime before the 1995 rerack project when water had already puddled or accumulated in the liner floor-to-slab gap. However, at that time at least some of the channels and telltales were not obstructed, so complete flooding of the gap had not occurred.

Blockage of the drains and accumulation of water in the gap between the pool and the walls occurred sometime between 1995 and 1998. In 1995 some of the channels and drains were open for flow. However, radioisotopic analysis indicates that by 1998 SFP leakage was accumulating in the gap rather than flowing out the telltales.

The 1995 re-rack project may have contributed to formation of telltale blockages. In general, the velocity of the leakage overflowing into the channels is low, too low to entrain sand and other particulate matter. However, leakage flow observed during the re-rack as water was forced into the channels likely was high enough to carry particulates into the channels.

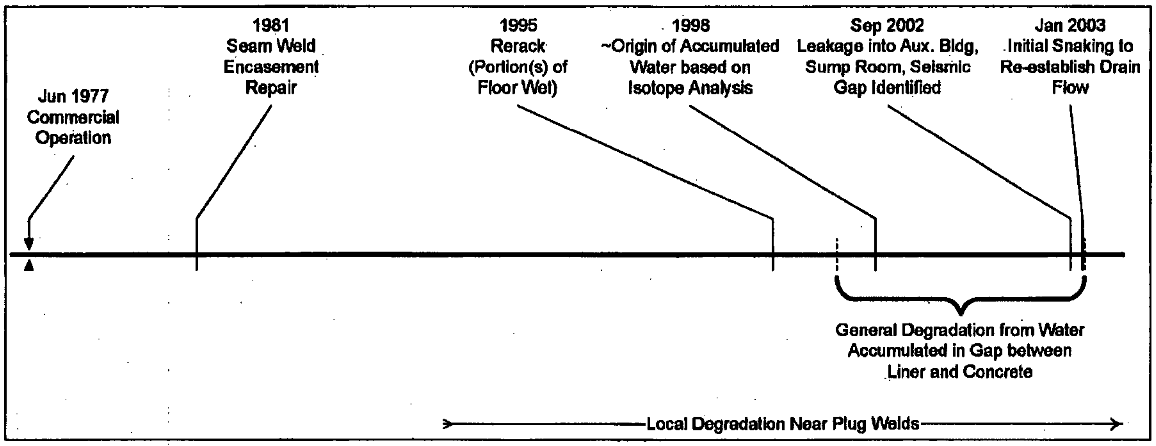


Figure 6-1. Leakage Timeline for Salem Unit 1

6.5 INDEPENDENT STRUCTURAL ASSESSMENT PER ACI GUIDELINES

PSEG Nuclear had an experienced concrete structural engineer perform an independent structural assessment of the FHB in 2006. The assessment included review of building drawings, a visual inspection of the accessible portions of the FHB exterior walls, and a visual inspection in the Sump Room. The checklist in ACI 201.1R-92 was used to guide the inspections. Observations were compared to limits in ACI 349.3R. The assessment is documented in Reference 9.2.6.

Key conclusions from the independent assessment are excerpted below.

- "Overall the concrete appears to be in good structural condition."
- The "appearance of leaching or chemical attack and corrosion staining of undefined source on concrete surfaces do not indicate significant structural deterioration at this time."
- There were "no indications of concrete surface expansion due to reinforcing steel corrosion."

It also recommends periodic inspections to trend the condition of the building.

6.6 DEGRADATION OF FHB STRUCTURE FROM BORIC ACID EXPOSURE

The above evaluations and the leakage timeline provide important insights into potential degradation of the FHB structure from exposure to the boric acid solution leaking from the SFP. As discussed above, seam weld leakage in and of itself is not a concern as the leakage is collected in the channels and discharged via the telltales. Plug weld leakage results in local degradation of the concrete structure. Once the channels and telltales become plugged, weld leakage (plug weld and seam weld leakage) accumulates in the gap between the liner and the structure and results in general wetting and degradation of the concrete structure.

Hydrostatic pressure of the water in the gap ultimately forced water through construction joints or cracks to the Auxiliary Building, the Sump Room and the seismic gap (between the FHB and the Auxiliary Building). Water migration through the construction joints or cracks flowed past reinforcing steel, potentially corroding the steel.

Once drain flow from the telltales was re-established, the accumulated leakage drained out and the hydrostatic pressure to force the boric acid solution into construction joints or cracks was eliminated. Consequently, general degradation from the stored inventory ceased and potential degradation of reinforcing steel from migration through joints or cracks ceased. However, active leakage from the pool continues to degrade concrete as it migrates to an open telltale. Since some telltales are partially or fully blocked, seam weld leakage could be contributing to concrete degradation.

Detailed discussions of the expected structural degradation are provided below. The discussion is divided by degradation mechanism: local degradation from weld leakage, general degradation from water accumulated in the gap, and reinforcing steel corrosion in construction joints or cracks.

6.6.1 Local Degradation from Weld Leakage

Local degradation of concrete from weld leakage occurs in two different ways.

- Boric acid leakage from a plug welds contacts the concrete in the vicinity of the leaking welds and degrades the concrete.
- If a telltale is blocked, weld leakage that normally collects in the channel and flows out the telltale, overflows the channel and migrates across concrete to a channel with an open telltale. The boric acid degrades the concrete along this migration path.

Each mechanism is described below and then the extent of degradation over remaining plant life is projected.

Plug Weld Leakage

Leakage associated with liner plug welds results in local degradation of the concrete structure, primarily the slab underneath the pool. Leakage from plug welds, whether on the wall or the bottom of the liner, puddles on the slab until it overflows into the channels. For a leaking plug weld in the bottom of the pool, the puddle will be located in the vicinity of the leaking weld. For a leaking plug weld in the pool wall, the puddle will be in the corners where the wall and slab intersect.

Concrete under the puddles will degrade from exposure to the boric acid solution. The boric acid will attack the cement paste, weakening it and causing it to de-bond from the coarse and fine aggregate. As the degradation progresses, a rubble bed of coarse and fine aggregate may be formed on top of the concrete as the cement in the top layer fully degrades. In essence, the local degradation will create a "pothole" with sand and coarse aggregate on top of the remaining concrete.

In the case of plug weld leaks in the FHB walls, there may be some local degradation of the wall as the leakage flows down the liner or the concrete wall. Such damage is expected to be limited to the immediate vicinity of the flow path of the leakage flow down the wall. Weakening of the cement paste and de-bonding of the aggregate results in debris falling down the wall to the slab. Falling debris increases the gap between the liner and the concrete in the upper portion of the wall reducing the potential for boric acid leakage to contact the concrete. Accumulation of debris in the gap in the lower portion of the wall increases the distance that boric acid must diffuse to come into contact with concrete, thereby reducing the degradation rate.

As previously discussed, leakage associated with plug welds started prior to the 1995 re-rack. However, pinpointing a date is very difficult. Degradation associated with this leakage will continue into the future even though drain flow has been re-established.

Migration of Leakage to Open Telltale

Once a given telltale is blocked, any weld leakage—seam weld leakage or plug weld leakage—that normally collects in the channel cannot flow out the telltale. This leakage overflows the channel and migrates across concrete to a channel with an open telltale. The concrete along the migration path to an open telltale is subject to degradation similar to that described above for plug weld leakage. This degradation mode primarily impacts the slab as leakage—from plug welds on the liner walls or the liner floor—collects on the slab and overflows into an open telltale.

Pinpointing the date that this mode of degradation started is difficult. However, it is clear that it started sometime prior to 1995 as some of the telltales were plugged at the time the re-racking was performed in 1995. This mode of degradation will continue into the future as not all telltales have been fully cleaned at this time and some may re-block between periodic telltale cleanings.

Projected Degradation

To assess the structural implications of this degradation mode, it is assumed that local areas of the concrete structure have been subjected to degradation for a period of 70 years. The 70-year time period is a conservative value that spans the entire plant life including: license renewal[3]; and maintaining fuel in the SFP for 10 years after cessation of operations. Per Reference 9.2.4, the projected depth of local concrete degradation after 70 years is 1.30 inches. Since the concrete cover for all walls and slab is greater than the projected depth of concrete degradation, no reinforcing steel degradation is expected for this mode of degradation.

The projected depth of local degradation is applied to the entire slab. While plug weld leakage results in degradation of only a local area, blockage of the telltales expands the area of the slab subjected to long term exposure to boric acid as the boric acid migrates to an open telltale. Applying the maximum depth of degradation to the entire slab is conservative.

[3] Consideration of license renewal in determining the plant operating life is not an indication that PSEG Nuclear has committed to pursuing license renewal. Instead, it is included to provide a bounding assessment.

6.6.2 General Degradation from Water behind Liner

As the leakage channels and telltales became plugged, leakage from the SFP accumulated in the gap between the liner and the concrete on the floor and the walls. The water level in the gap likely increased until it equalized with the level in the pool, at which point leakage essentially stopped and conditions in the gap became stagnant.

General degradation of the concrete is similar to that described above for plug weld leakage, except the degradation is widespread rather than localized. Virtually the entire structure surrounding the pool is exposed to boric acid and subject to degradation. The period of general degradation starts sometime between 1995 and 1998 when the leakage channels and telltales became blocked, and extends to early 2003 when drain flow was re-established. This mode of degradation is not expected to recur as PSEG Nuclear has implemented multiple measures to ensure that the telltales do not become entirely blocked (trending of telltale leakage rates, periodic videoprobe inspections and cleanings).

Reference 9.2.4 contains a calculation which projects degradation over a 70-year span. Using the projected degradation curve therein and the temperature adjustment, the projected general degradation over an 8-year interval is about 0.44 inch. Since the concrete cover for all walls and the slab is markedly greater than the projected depth of concrete degradation, no reinforcing steel degradation is expected for this mode of degradation.

The projected depth for general degradation is more appropriate to use in structural assessments of the walls than the local degradation projection. Although local areas of the walls near leaking plug welds could be degraded to deeper depths, there is no mechanism for expanding these local areas to a significant area. Further, structural margin is driven by the condition of the general area, not small localized areas.

6.6.3 Degradation from Migration through Construction Joints and Cracks

Once channels and telltales plugged and leakage accumulated in the gap between the liner and structure, the hydrostatic head forced the leakage into construction joints and cracks and ultimately into the Sump Room, the Auxiliary Building and the seismic gap. Migration through the construction joints or cracks passed reinforcing steel, potentially initiating corrosion of the reinforcing steel.

The combination of evidence—studies in the literature, inspections of the FHB, testing conducted for Salem and experience at another US PWR—indicates that reinforcing steel degradation in the FHB is minimal and structural capacity has not been impacted. The key points that support this case are as follows.

- The reinforcing steel of concern from a structural standpoint is the reinforcing steel near the outside of the wall (or slab)—the side farthest from the pool and farthest from the SFP water which may reside in the gap between the liner and the wall.

- Review of Tables 3-1 and 3-2 indicates that most of the limiting margin cases, including all of the cases with less than 10% margin, have negative loads which denotes compression on the pool side of the wall (or slab) and tension on the outside of the wall (or slab). Since reinforcing steel carries the tensile loads, the reinforcing steel of primary concern with regard to structural margin is the rebar near the outside of the wall. Hence, boric acid must migrate through the walls, which are several feet thick, to reach the reinforcing steel of concern.

- Boric acid migration through construction joints or cracks would react with the concrete prior to reaching the reinforcing steel. Hence, the pH of the boric acid would likely be neutral or basic by the time it reaches the reinforcing steel. Further, migration through the construction joint or crack would become de-aerated, which would markedly reduce the steel corrosion rate. Note that the reaction with concrete contributes to the observation of negligible corrosion of embedded of steel noted in References 9.5.4 and 9.2.5—see bullets below.

  – A study conducted by EPRI (Reference 9.5.5) concluded that the corrosion rate of steel in a de-aerated boric acid solution is 0.004 mm/year (0.157 mils/year). The study considered a range of temperatures and acid concentrations. The corrosion rate of 0.157 mils/year is for a 2400 ppm boron solution, which is consistent with SFP chemistry. This is conservative with regard to the situation in the FHB because the pH when the boric acid reaches the rebar will increase from reaction with the concrete.

  – Testing documented in Reference 9.5.4 showed ***negligible*** reinforcing steel attack from boric acid flow through a simulated crack after a period of two years; corrosion limited to scarring in the area of the crack. The tests covered a range of pressures, crack sizes and pH. The tests showed that corrosion increases as crack width increases and pH decreases. The observation of negligible corrosion was for the most aggressive conditions—widest crack (0.4 mm) and lowest pH (5.2). The lowest pH tested is similar to the pH of the SFP.

  – Experience at another US PWR showed ***no visible*** corrosion of embedded reinforcing steel from boric acid migration through a crack over several years (Reference 9.2.5). The source of the boric acid was SFP leakage and the concrete was six feet thick, which are similar to the situation at Salem.

- The Salem FHB does not show any signs of significant degradation of rebar from exposure to boric acid.

  – Rust staining on the walls in the sump room is very minor and the result of very small amounts of iron oxide.

  – An independent structural examination by an experienced concrete structural engineer concluded that the structure is sound and that there are "no indications of concrete surface expansion due to reinforcing steel corrosion was would be evidenced by a pattern of cracking, spalling or bulging of the concrete" (Reference 9.2.6).

Migration through construction joints or cracks is a relatively recent event at Salem that stopped in 2003. Migration through the construction joints or cracks started prior to 2002 (when leakage into the Auxiliary Building was noted), possibly as early as the 1995 to 1998 timeframe. Given the thickness of the walls, boric acid migrating through the walls would not have reached the outer reinforcing steel until well after the 1995 to 1998 timeframe. Reports of leakage into the Auxiliary Building and sump room stopped subsequent to cleaning the telltales in early 2003. Hence, the outer reinforcing steel was exposed to boric acid which migrated through construction joints or cracks for much less than 5 to 8 years. This mode of degradation is not expected to recur as PSEG Nuclear has implemented multiple measures to ensure that the telltales do not become entirely blocked (trending of telltale leakage rates, periodic videoprobe inspections and cleanings).

The preponderance of the evidence is that any degradation of reinforcing steel, particularly the outer reinforcing steel, is negligible. Based on Reference 9.5.5, the corrosion rate of the reinforcing steel in a de-aerated boric acid solution is 0.157 mils/year. For an exposure duration less than 7 years, the rebar has experienced a reduction in radius of less than 1 mil (0.001 inch). This is conservative estimate with regard to the situation in the FHB because the pH when the boric acid reaches the rebar will increase from reaction with the concrete.

It is important to note that any rebar degradation is limited to the immediate vicinity of the crack or construction joint. Testing documented in Reference 9.2.4 showed that wicking rate of boric acid along the reinforcing steel/concrete interface is about the same as the rate boric acid penetrates into the concrete. Any de-bonding of concrete from the reinforcing steel is localized. Accordingly, reinforcing steel functionality is maintained.

6.7 Conclusions

The condition of the FHB at the end of plant is projected using the foregoing discussions of the expected nature and timeline of building degradation. The projections consider degradation that has occurred to date and anticipated future degradation. The tables below summarize the projections; note that local degradation from weld leakage and general degradation from accumulation of boric acid behind the liner are combined into a single table. The depth of concrete degradation could be as high as 1.30 inches in the slab and 0.44 inch in the walls. It is estimated that the reinforcing steel has experienced a reduction in radius of less than 0.001 inch, which is negligible.

Table 6-1. Projected Degradation of Concrete Structure: Local Degradation from Weld Leakage and General Degradation from Water Trapped Behind Liner

Parameter	Value	Basis
Concrete Degradation		
Effective Loss of Concrete in Walls	0.44 inch (based on general degradation)	For the walls, loss of concrete is based on general degradation. Although local areas near leaking plug welds could be degraded to deeper depths, there is no mechanism for expanding these local areas to a significant area. Further, structural margin is driven by the condition of the general area, not small localized areas. The period of general degradation started sometime between 1995 and 1998 and extended to 2003. The maximum time period of 8 years is used. Using the projected degradation curve in Appendix D of Reference 9.2.4 and the temperature adjustment, the projected degradation over an 8-year interval is about 0.44 inch.
Effective Loss of Concrete in Slab	1.30 inches (based on local degradation)	For the slab, loss of concrete is based on local degradation. Plug weld leakage results in local degradation. However, blockage of the telltales expands the area of the slab subjected to long term exposure to boric acid as the boric acid migrates to an open telltale. Local exposure of the wall and slab to boric acid leakage started sometime before 1995. Local degradation from plug weld leakage and local degradation from leakage migration will continue into the future. For conservatism, this mode of degradation is assumed to occur over a period of 70 years. As the boric acid leakage may have puddled on the slab and created "potholes" of indeterminate size, the depth of local degradation is applied to the entire slab. Reference 9.2.4 projects a depth of degraded concrete of 1.30 inches after 70 years exposure to boric acid.
Reinforcing Steel Degradation		
Reinforcing Steel Corrosion	None	No degradation of the reinforcing steel is expected because the depth of concrete cover (3 inch minimum) exceeds the effective loss of concrete.

Table 6-2. Projected Degradation of Concrete Structure: Reinforcing Steel Degradation from Migration through Joints/Cracks

Parameter	Value	Basis
Reinforcing Steel Exposure Time	Possibly <7 years	Boric acid migration through the construction cracks started prior to 2002 (when leakage into the Auxiliary Building was noted), possibly as early as the 1995 to 1998 timeframe. However, given the thickness of the walls, the boric acid would not have reached the outer bar until well after the 1995 to 1998 timeframe. Reports of leakage into the Auxiliary Building and sump room stopped subsequent to cleaning the telltales in early 2003.
Reduction in Reinforcing Steel Radius	<1 mil (<0.001 inch)	Since the outer rebar was exposed to boric acid longer than two years, degradation may be greater than the "negligible" noted in the Reference 9.5.4 study and the experience at another US PWR. Based on the study documented in Reference 9.5.5, a reduction in radius of 0.001 inch (1 mil) is predicted. This is conservative with regard to the situation in the FHB because the pH when the boric acid reaches the rebar will increase from reaction with the concrete.
Length of Degradation at Concrete/ Reinforcing Steel Interface	Localized	The testing documented in Reference 9.2.4 showed that the wicking rate was low in acidic conditions. Therefore, the degradation would not spread considerably within the seven years the reinforcing steel is assumed to have been exposed to boric acid; i.e., degradation is localized to the immediate vicinity of the joint/crack.

7 Assessment of Structural Adequacy

The structural adequacy of the FHB can be evaluated using the estimated concrete and reinforcing steel degradation levels along with structural calculations for the FHB structure. Each of the degradation mechanisms discussed in Section 6.6 is addressed below to assess the current condition of the FHB.

7.1 Concrete Degradation from Boric Acid Exposure

The discussion in Section 6.6 concludes that the depth of concrete degradation may reach up to 0.44 inch on the walls and 1.30 inches on the slab. As shown in Reference 9.3.5 (provided in Appendix D) the impact of the degradation on the structure is contingent upon the section location within the SFP. The effects of degradation on the slab and walls are considered below.

7.1.1 Slab Degradation

As documented in Reference 9.2.3, the structural analysis of the slab does not credit the 6-inch layer of leveling concrete shown in Reference 9.4.6. Although no credit is taken for this concrete in any of the previously performed structural analyses, this layer of concrete is critical to understanding degradation depths.

The maximum estimated degradation depth of 1.30 inches would not penetrate this leveling layer and thus has no impact on the structural capacity of the slab. Accordingly, no structural concrete is lost and the margins documented in Appendix C of Reference 9.2.3 are unaffected.

7.1.2 Wall Degradation

For the FHB walls, the following equations were developed in Reference 9.3.5 to relate the percent reduction in allowable moment (y) to a concrete degradation level (x), in inches.

North Wall:	$y = 1.01x$
South Wall:	$y = 1.55x$
East Wall:	$y = 1.50x$
West Wall:	$y = 0.92x$

The pool-side of the FHB structure experienced concrete degradation to a depth of 0.44 inch during the time when boric acid leakage was trapped behind the liner. While local areas may experience more severe degradation from plug weld leakage, there is no mechanism for expanding these local areas to a significant area.

Further structural margin is driven by the condition of the general area, not small localized areas. Accordingly, general degradation of the wall is a more meaningful value to use for structural integrity calculations.

Using each of the above equations along with the predicted depth of general concrete degradation, percent reductions in allowable moments are obtained. Table 7-1 compares the percent reductions to the limiting available margins of each FHB wall to obtain the available margin in the FHB structure considering projected degradation from boric acid between the liner and concrete. Limiting available design margins, taken from Appendix C of Reference 9.2.3, are based on working stress design methods.

The limiting available margin is 1.6% in the middle section at the top of the West wall. At this region of the pool, concrete degradation is expected to be minimal due to the limited time the section should have been exposed to the boric acid. The concrete in this region would have only experienced sustained contact with boric acid during the time when the telltales were plugged and the pool had completely filled. As previously discussed, any leakage from plug welds in the FHB walls would be expected to only degrade concrete in the immediate vicinity of the flow path of the leakage down the wall. Weakening of the cement paste at these localized areas would not significantly impact the structural integrity of the wall.

Positive margin is maintained for all walls given the projected depth of degradation. Hence, the design basis analysis of record is not invalidated by the postulated degradation.

Table 7-1. FHB Available Margin Based on 0.44" General Concrete Degradation and Working Stress Design (WSD) Methods

FHB Wall	Percent Reduction in Capacity	Available Margin (WSD, No Degradation)	Location of Limiting Margin	Available Margin (WSD, Including Degradation)
North	0.4%	4%	Middle, Bottom	3.6%
South	0.7%	300%	West, Toward Bottom	299%
East	0.7%	5%	Middle, Toward Bottom	4.3%
West	0.4%	2%	Middle, Top	1.6%

7.2 Reinforcing Steel Degradation from Migration through Joints/Cracks

Reinforcing steel degradation from boric acid migration through cracks and construction joints has a negligible impact on the FHB structural capacity.

Key considerations are as follows.

- As noted in Table 6-2, migration of boric acid through construction joints and cracks has potentially degraded reinforcing steel by less than 0.001 inch (radial reduction). Using the equations provided in Appendix C, the maximum calculated reduction in margin is 0.2%, which is insignificant and well below the accuracy of the calculations.
- Fabrication tolerances for reinforcing steel are specified as 94% of total weight (Reference 9.1.5). For the reinforcing steel sizes used in the FHB, this is equivalent to a diametral variation of approximately 3% or about 30 times the estimated degradation. In light of this tolerance, it is apparent that any predicted reinforcing steel degradation is negligible relative to the imperfections that are inherent to the steel in its original form.
- As discussed in Section 4, the available margin for all sections under consideration may be increased by 2% if the actual yield strength of the FHB reinforcing steel is used in the working stress design calculation. The actual yield strength compensates for the predicted reduction in margin by more than a factor of 10.

Based on the above, there is no reduction in structural margin from potential reinforcing steel degradation from boric acid leakage through cracks and construction joints.

The conclusion on the adequacy of the reinforcing steel does not change even if reinforcing steel corrosion is assumed to occur over the entire 70-year period considered herein. Using the corrosion rate of carbon steel in de-aerated boric acid from Reference 9.5.5, the radial reduction is 0.011 inch after 70 years. Using the equations in Appendix C, the maximum calculated reduction in margin is about 2%, which is equal to the increase in margin that can be recovered by crediting the actual yield strength of the reinforcing steel in the working stress design calculation.

7.3 Voided Areas beneath the Liner

As discussed previously, boric acid will attack the cement paste, weakening it and causing it to de-bond from the coarse and fine aggregate. As the degradation progresses, a rubble bed of coarse and fine aggregate may be formed on top of the concrete as the cement in the top layer fully degrades. In essence, the local degradation will create a "pothole" with sand and coarse aggregate on top of the remaining concrete. This effect may produce a small voided depth below the ¼-inch stainless steel liner, but above the sand and rubble layer. With this void there is a concern that the load of the fuel racks may no longer be supported on a firm surface.
As stainless steel is a highly ductile material, it is expected to strain and deform to the voided depth without failure. Adequacy of the liner with the degraded under-layer was verified in a scoping assessment. The assessment considered both the water pressure load and fuel rack foot load. As discussed below, neither of these mechanisms are considered likely to cause liner failure.

Reference 9.2.4 calculated a degraded paste depth of 1.30 inches. This value considers the depth of cement that would be affected by the boric acid, but is not representative of the voided depth. The coarse and fine aggregate constitute approximately 71% of the volume of the concrete and 79% of the mass of the concrete. Although a small portion of the concrete constituents may have migrated to the telltales, the majority of the constituents (including almost all of the aggregate) are expected to remain in place. Assuming that 71% of the concrete constituents remain, the voided depth is expected to be no greater than 0.38 inch.

At a depth of 0.38 inch, the water pressure load (17.77 psi from Reference 9.3.1) and the single foot load (maximum of 62,600 lbs over a 12-inch by 12-inch pad, Reference 9.3.1) will likely plastically deform the liner to the rubble bed. The amount of strain experienced by the liner over this small depth is expected to be significantly less than the limiting strain of the material (<10%) and will not cause failure.

7.4 Conclusion

The FHB is structurally adequate through the end of plant life. As Table 7-1 shows, the structural capacity of the FHB is maintained for all degradation modes. The provided values are for the highest degradation conditions in the most limiting location in the pool; all other areas of the pool show higher available margin. Positive margin is maintained at all locations in the structure. Therefore, the design basis analysis of record is not invalidated by the postulated degradation.

A scoping assessment further demonstrates that the liner is sufficiently ductile to accommodate the load from the fuel racks even if the foot of the rack is positioned over an area of local concrete degradation.

8 Corroboration of Salem Assessment by Cores from the CY SFP

This section summarizes results from evaluation of cores removed from the floor of the Connecticut Yankee Atomic Power Plant (CY) SFP and demonstrates how the results corroborate the degradation modes and degradation projections in preceding sections.

In the fall of 2007 PSEG Nuclear learned that EPRI had possession of several samples taken from the CY SFP during decommissioning. The samples were taken in the form of cores that included the liner, leakage channel and concrete. B&W Technical Services Group, under contract from EPRI, had evaluated the liner welds to attempt to locate the leakage source and to support development of inspection and repair techniques. A thorough evaluation of the concrete had not been performed. PSEG Nuclear and EPRI agreed to collaborate on evaluation of the concrete. The objective of the evaluation was to use actual plant observations to corroborate the assessment of the Salem FHB.

8.1 Evaluation of CY Cores

The evaluation of the CY Cores was performed by Concrete Research & Testing (CRT), MPR's subcontractor for the testing described in Section 5.3, with assistance from B&W Technical Services Group. Reference 9.2.7 documents the examination of the CY cores.

8.1.1 Overview of CY SFP

The CY SFP was a reinforced concrete structure with a ¼-inch stainless steel liner. The pool had leakage collection channels located behind the liner seams. The channels are 3-inch wide by ½-inch thick stainless steel plates with a 1-inch wide by ¼-inch deep groove at the centerline. The liner plates were plug welded to the channel near the seam weld to align and support the plates for the closure weld. The channel is held in place by Nelson studs embedded into the concrete. It is likely that there were other embedded studs located in between the channels or alternate means for supporting the liner between seam welds, but the exact construction details are not known.

In leakage of water from behind the liner was noted during decommissioning. Specifically, after the pool was drained and dried, pools of water were noted in multiple locations on the floor. It was suspected that cracking in the liner allowed water from behind the liner to leak into the pool. The source of the water was either SFP leakage that had been trapped behind the liner or ground water.

Personnel who worked at CY during plant operation and decommissioning indicate that the SFP leakage was believed to have started early in plant life. CY began commercial operation in 1968 and was shutdown in 1996. Removal of fuel from the pool was completed in 2005. Therefore, the leakage occurred for approximately 37 years.

8.1.2 Description of Cores

Three cores were provided to EPRI and subsequently made available to PSEG Nuclear for examination. All three cores were from the floor of the SFP. The cores were taken from locations where water pooled after the fuel was removed and the pool was drained. The specimens are 6-inch diameter by 10-inch long cores. The cores included the liner, channel, concrete and embedded reinforcing steel. Two cores had been cut into disc specimens prior to shipment to EPRI. Note that the cores as examined by CRT did not include the liner sections as the liner had been removed as part of EPRI's evaluation of the liner welds.

8.1.3 Evaluation of Concrete

The cores were examined to characterize the concrete and to evaluate potential degradation of the concrete from boric acid attack. The examinations included petrographic examination of the cores by CRT personnel and chemical analyses by B&W Technical Services Group. The chemical analyses determined the presence of boron at varying depths from the top surface and to characterize secondary deposits observed in the concrete. The discussion below is based on Reference 9.2.7.

Petrographic Examination

Examination of the cores showed that both the coarse and fine aggregates are non-reactive[4] with respect to acid attack. Specifically, the coarse aggregate is diabase igneous rock and the fine aggregate is primarily quartz. CRT judged the cement paste to be fair quality with a water-cement ratio of 0.60.

The upper surface of the cores (i.e., the surface underneath the liner) showed evidence of boric acid attack. The concrete exhibited a light color and the paste was weak. In some cases, aggregate particles were exposed from loss of cement paste. Chemical analyses confirmed the presence of boron.

Table 8-1 lists the depth of degradation from each of the cores. As shown, the depth of degradation was typically minor (<0.3 inch). However, in some local areas the depth of degradation was markedly higher (up to 0.91 inch). The deepest areas were adjacent to the channel. Figure 8-1 shows Core 122 before it was sectioned. As shown, the deepest degradation is on one side of the channel where it appears that the channel had debonded from the concrete allowing the boric acid to access the concrete underneath below the channel. When the core was being sectioned for petrographic examination, it failed at a pre-existing crack. Figure 8-2 shows

[4] In the context of this report, "non-reactive aggregate" is used to denote either coarse or fine aggregates that do not react with acids. Non-reactive coarse aggregates include igneous rock and non-reactive fine aggregates include silica sand. Limestone and carbonate-based aggregates are considered reactive.

this fracture and its orientation is consistent with the deepest degradation. (Note that Figure 8-2 is from the opposite perspective as Figure 8-1.)

Table 8-1. Depth of Affected Paste in CY Cores

Core	Depth of Affected Paste (inch)		
	Away from Channel	Adjacent to Channel	Below Channel
122	0.06 – 0.12	0.28 – 0.91	0.20 – 0.31
123	0.05 – 0.12		<0.05
124	0.12 – 0.16	0.30 – 0.67	0.03 – 0.14

Figure 8-1. Core 122—As Received Core prior to Sectioning

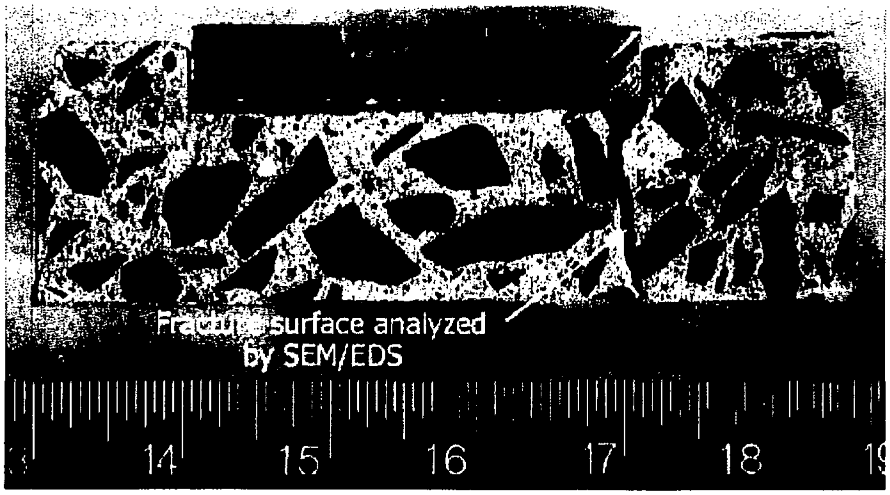


Figure 8-2. Core 122—Cross-Section View Showing Fracture along Crack

The three cores exhibited cracking as noted below.

- Core 122 had a vertical crack at the corner of the channel.
- Core 123 had a horizontal crack at the location of the top layer of reinforcing steel.
- Core 124 had three vertical cracks under the leakage channel.

Only the crack in Core 122 contributed to degradation of the concrete. This crack was the only crack that connected to the top surface of the concrete in an area wetted by boric acid. The horizontal crack in Core 123 did not connect to the surface. The vertical cracks in Core 124 connected to the surface of the concrete underneath the channel, but the lack of concrete degradation under the channel indicates that this area was not wetted by boric acid.

These results demonstrate that boric acid attack of concrete can be highly localized depending on where the boric acid pools. Cracks may provide a means to expand the degraded area, but only if they connect to the surface in an area where boric acid is present. Cracks did not lead to widespread degradation.

Chemical Analyses

Chemical analyses were performed on powder concrete samples drilled at various depths from the surface of the core. The analyses showed that boron was present and the boron concentration decreased with depth. These results confirm that the observed degradation is from boric acid attack.

The secondary deposits in the concrete were analyzed as well. The deposits were typically ettringite and calcite. The only location where the deposits contained boron was the vertical crack in Core 122. Recall that boric acid penetration into the crack led to an expanded area of degradation.

8.1.4 Evaluation of Reinforcing Steel

Cores 122, 123, and 124 all contained reinforcing steel. The cores were sectioned perpendicular to the reinforcing steel so reinforcing steel corrosion and the bond with the cement paste could be evaluated. Examination of the rebar is documented in Reference 9.2.7.

No corrosion was noted in any of the sections. However, the examinations showed areas where the concrete separated from the underside of the rebar. This is considered to be the result of settlement of the concrete prior to hardening and insufficient consolidation of the concrete around the rebar; it is not due to boric acid attack.

8.1.5 Evaluation of Liner Welds

EPRI performed non-destructive and destructive examinations of the liner welds in cores removed from the CY SFP. The scope of the examinations included liner seam welds and a plug weld to the channel. The cores did not include any plug welds to embedded studs that may have been located between channels. Reference 9.2.6 provides the details of some weld quality issues. Specifically, there was lack of fusion in the liner seam weld, an open root weld. Also, the plug welds were not completely filled. No through-wall defects were identified in the metallurgical evaluations.

8.2 Comparison of CY Cores to Salem SFP Assessment

8.2.1 Comparison of Concrete

The concrete used at CY and Salem can be compared as follows.

- The concrete at both CY and Salem has non-reactive aggregates.
- CY cores used fly ash while laboratory-prepared specimens used in the Salem long-term testing did not. Fly ash promotes hydration of the concrete and increases density. (Note that concrete used in structures at Salem contains fly ash.)
- The CY concrete had a higher water-cement ratio than Salem (0.6 versus 0.5). Permeability of concrete increases significantly for water-cement ratios above 0.5. Additionally, the strength of concrete is reduced as water ratio increases.

The net result of the differences identified in the second and third bullet likely increases the porosity of the CY specimens compared to the Salem test specimens.

8.2.2 Concrete Degradation

Degradation Modes
The degraded concrete in the CY cores varied from less than 0.05-inch to 0.91-inch demonstrating that the degradation can be highly localized. This is consistent with the postulated degradation of the Salem FHB as described in Section 6 of this report.

Depth of Degradation

Over the 37 year life of CY's SFP concrete degradation reached a maximum depth of 0.91 inch. The correlation developed in Reference 9.2.4 from the Salem testing predicts 0.94 inch of degradation at 37 years. Therefore, the CY degradation is within the expectation for the given exposure time. The increased porosity of the CY cores, as described in Section 8.2.1 should yield a result deeper than the model from Reference 9.2.4.

8.2.3 Rebar Corrosion

Although the upper surface of the CY cores was degraded from boric acid attack, the embedded reinforcing steel exhibited no corrosion or loss of bond with the cement from boric acid attack. However, it appears that the embedded rebar was not exposed to boric acid. The concrete degradation did not extend to the depth of the rebar, which would expose the rebar to boric acid. Further, the cracks present in the concrete CY cores did not connect from surface or the degraded concrete to the rebar.

It is important to note that the presence of secondary deposits, including secondary deposits in cracks found in the CY cores, provides evidence that water migration occurred. Yet the reinforcing steel exhibited no corrosion.

These results confirm that cracks in concrete and other concrete defects do not promote reinforcing steel degradation.

8.3 CONCLUSIONS

Overall, the CY cores corroborate the results of testing for Salem and the projections for Salem. The maximum depth of concrete degradation in the CY cores is within that predicted using the correlation developed from the Salem testing. The rebar in the CY cores exhibited no corrosion even though the upper surface of the concrete was degraded by boric acid, the concrete was cracked and, based on the presence of secondary deposits within the concrete, there was water migration in the concrete.

9
References

9.1 Specifications

9.1.1 Deleted.

9.1.2 PSEG Nuclear Technical Standard SC.DE-TS.ZZ-4201(Q), "Salem Structural Design Criteria," Revision 2.

9.1.3 ACI 318-63, "Building Code Requirements for Reinforced Concrete," American Concrete Institute, June 1963.

9.1.4 ACI 349-80, "Code Requirements for Nuclear Safety Related Concrete Structures," American Concrete Institute, April 1981.

9.1.5 ASTM A615, "Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement."

9.2 Reports

9.2.1 Deleted.

9.2.2 PSEG Nuclear Chemistry Technologies & Support Final Report, "Investigations of Salem Unit 1 Fuel Pool Leakage: Phase II Analyses," 2/21/2003.

9.2.3 MPR-1863, "Salem Generating Station Spent Fuel Pool Building Structural Design Analysis," Revision 0. (PSEG Nuclear VTD 326116)

9.2.4 MPR-2634, "Boric Acid Attack of Concrete and Reinforcing Steel," Revision 2. (PSEG Nuclear VTD 326561)

9.2.5 PSEG Nuclear Record Transmittal No. DES-060005; contains documents related to evaluation of concrete degradation from boric acid at another PWR.

9.2.6 PSEG Nuclear VTD 327194, "Salem Units 1 and 2 Structural Examination of Spent Fuel Pool Structures," Revision 1.

9.2.7 CRT Report No. R-140, "Petrographic Examination of Concrete Cores Removed from the Conn-Yankee Spent Fuel Pool," dated September 11, 2008. (included in Appendix A)

9.3 Calculations

9.3.1 PSEG Nuclear Calculation 6S0-1674, "Structural Analysis Report for the Salem Generating Station Spent Fuel Pool Storage," Revision 0, Holtec International.

9.3.2 EQE Calculation 200050-C-01, "Salem Spent Fuel Pool Evaluation for Beyond Design Basis Thermal Load," Revision 0, EQE Engineering.

9.3.3 Deleted.

9.3.4 MPR Calculation 108-275-02, "Statistical Analysis of Rebar Yield & Tensile Strengths for Salem Nuclear Generating Station," Revision 0 (included in Appendix B).

9.3.5 MPR Calculation 0108-0275-34, "Salem Spent Fuel Pool Structure Capacities Based on Degraded Concrete Conditions," Revision 0 (included in Appendix D).

9.3.6 PSEG Nuclear Calculation 6S1-1836, "Justification for Acceptability of Leakage from the Spent Fuel Pool - Salem Unit 1," Revision 0.

9.3.7 MPR Calculation 0108-0275-35, "Salem Spent Fuel Pool Reinforcing Steel Load Capacity at Degraded Conditions," Revision 0 (included in Appendix C).

9.4 Drawings

9.4.1 PSEG Nuclear Drawing No. 201075 A 8706-2, "No. 1 Unit – Fuel Handling Area, Plan at Elevation 78'-0", Revision 2.

9.4.2 PSEG Nuclear Drawing No. 201076 A 8706-4, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 84'-0"."

9.4.3 PSEG Nuclear Drawing No. 201077 A 8706-8, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 100'-0" and 116'-0"."

9.4.4 PSEG Nuclear Drawing No. 201078 A 8706-4, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 130'-0"."

9.4.5 PSEG Nuclear Drawing No. 201079 A 8706-3, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Roof Plan."

9.4.6 PSEG Nuclear Drawing No. 201080 A 8706-7, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections A-A & B-B."

9.4.7 PSEG Nuclear Drawing No. 201081 A 8706-6, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections C-C, D-D & E-E."

9.4.8 PSEG Nuclear Drawing No. 201082 A 8706-5, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections F-F & G-G."

9.4.9 PSEG Nuclear Drawing No. 201085 A 8706-5, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Elevation P-P & Str. Bar Schedule."

9.5 TECHNICAL PAPERS

9.5.1 A. Bajza, I. Rousekova, & M. Dubik, "Can Boric Acid Corrode Concrete?," International Symposium on the Non-Traditional Cement and Concrete, Brno, Czech Republic, June 11 - 13, 2002.

9.5.2 V. Pavlik, "Corrosion of Hardened Cement Paste by Acetic and Nitric Acids. Part I: Calculation of Corrosion Depth," Cement and Concrete Research, Vol. 24, No. 3, pp. 551-562, 1994.

9.5.3 A. Allahverdi and Frantisek Skvara, "Acidic Corrosion of Hydrated Cement Based Materials. Part 1 - Mechanism of the Phenomenon," Ceramics - Silikaty, Vol. 44, No.3, pp. 114- 120, 2000.

9.5.4 W. Ramm and M. Biscoping, "Autogenous Healing and Reinforcement Corrosion of Water-penetrated Separation Cracks in Reinforced Concrete," Nuclear Engineering and Design, No. 179, pp. 191 - 200, 1998.

9.5.5 R. E. Nickell, "Degradation and Failure of Bolting in Nuclear Power Plants," Volume 1, Electric Power Research Institute, NP-5769, April 1988.

9.5.6 G. Frederick, "Fuel Pool Inspection and Repair," RRAC Technical Program Meeting, December 2007.

9.6 OTHER DOCUMENTS

9.6.1 Deleted.

9.6.2 PSEG Nuclear DCR 1EC0995, "Provide Weld Seam Encasements Along the Seal Weld Joints in the FH#1 Building Spent Fuel Pool."

9.6.3 Letter from J. Simons (MPR Associates) to A. Roberts (PSEG Nuclear), "Results from Literature Search; Salem Nuclear Generating Station Spent Fuel Pool Building Assessment," 3/10/03.

9.6.4 Email from T. Taylor (PSEG Nuclear) to J. Simons (MPR), Subject: Spent Fuel Pool Telltales, dated 12/15/03.

9.6.5 Email from T. Taylor (PSEG Nuclear) to J. Simons (MPR), Subject: Telltale Pipe Deposit Status, dated 11/25/03.

A

Petrographic Examination of Concrete Cores Removed from the Conn-Yankee SFP

This appendix contains the following CRT report.

- CRT Report No. R-140, "Petrographic Examination of Concrete Cores Removed from the Conn-Yankee Spent Fuel Pool," dated September 11, 2008.

(This appendix originally contained a calculation that assessed the potential margin recovery if ultimate strength design was used for normal operation and OBE cases. The calculation was removed for Revision 2 of this report.)

REPORT NO. R-140

ON

PETROGRAPHIC EXAMINATION OF
CONCRETE CORES REMOVED FROM
THE CONN-YANKEE SPENT FUEL POOL

TO

MPR ASSOCIATES
ALEXANDRIA, VIRGINIA

SEPTEMBER 11, 2008

Concrete Research & Testing, LLC

REPORT NO. R-140

ON

PETROGRAPHIC EXAMINATION OF CONCRETE CORES REMOVED FROM THE CONN-YANKEE SPENT FUEL POOL

INTRODUCTION

Concrete cores were removed from the spent fuel pool of the decommissioned Connecticut Yankee nuclear power plant. Three of the cores were sent to B&W Technical Services Group in Lynchburg, Virginia for examination and testing of the stainless steel liner. All of the cores received at B&W were removed from the floor of the fuel pool.

Subsequently, it was decided to perform petrographic examinations on the cores to determine if and to what extent the concrete has been affected by exposure to boric acid from within the spent fuel pool. Results from the evaluation of the Conn-Yankee specimens will be used to augment MPR Associates previous assessment of the Salem Spent Fuel Pool leakage and Fuel Handling Building structure, including the long-term testing program.

Nick Scaglione of Concrete Research & Testing was contracted to examine the concrete cores at the B&W facility in Lynchburg, Virginia.

DESCRIPTIONS OF THE CORES

The concrete cores were labeled 122, 123 and 124. The cores have a diameter of 5 ¾ in. Core No. 122 is comprised of two sections each having a thickness of roughly 2 in. Section 122-1 represents the upper end of the core and contains the imbedded stainless steel channel. Section 122-2 represents a depth of 2 to 4 in. below the top surface of the core. This section contains a No. 7 steel rebar oriented parallel to the end surfaces. Photographs of the core are shown in Figure 1 of Appendix A.

Core 123 has a length of about 18 in. The core is separated into two pieces by a horizontal fracture located at a depth of 10 in. The top end of the core contains the embedded stainless steel channel. The core contains a No. 9 rebar at a depth of about 2 ¼ in. below the top surface. Photographs of this core are shown in Figure 2 of Appendix A.

Core No. 124 is comprised of two sections each having a thickness of roughly 2 in. Section 124-1 represents the upper end of the core and contains the imbedded stainless steel channel. Section 124-3 represents a depth of 4 to 6 in. below the top surface of the core. This section contains a No. 9 steel rebar oriented parallel to the end surfaces. Photographs of the core are shown in Figure 3 of Appendix A.

EXAMINATION AND TEST METHODS

Preliminary stereomicroscopic examinations were performed on the exterior surfaces of each core. Following this initial examination, the cores were saw-cut perpendicular to the end surfaces. For the Core 122 and Core 124 sections the saw cuts were made perpendicular to the orientation of the steel channel and perpendicular to the orientation of the rebar. Core 123 was initially saw cut parallel to the end surfaces at depths of 1 ½ in. and 4 ¼ in. below the top surface of the core. The upper section containing the steel channel will be referred to as Core 123, Section 1. The lower section contains the steel rebar and will be referred to as Core 123, Section 2. These two sections were then saw cut perpendicular to the end surfaces of the core. The saw cuts were made perpendicular to the orientation of the steel channel and perpendicular to the orientation of the rebar. The saw-cut sections of each core were prepared for microscopic examination by lapping with silicon carbide pads.

The lapped sections of the cores were examined under a stereomicroscope at magnifications of up to 50X. The examinations were performed following the guidelines outlined in ASTM C 856, "Standard Practice for Petrographic Examination of Hardened Concrete."

The water-cementitious ratios of the core specimens were estimated based on observations and qualitative assessments of cement paste hardness, textural features, color, relative amount of unhydrated cement particles and rate of water absorption. Features of the cement paste when probed (steel probe) were also used in this assessment.

The cores were tested to determine the depth of the boron penetration into the concrete. The testing for boron concentration was performed using Inductively Coupled Plasma / Mass Spectroscopy (ICP/MS). For this testing powdered concrete samples were obtained from each core by drilling with a ⅛ in. diameter carbide drill bit at various depths below the top surfaces of the cores. Photographs showing the drilling procedure are shown in Figure 4. The boron testing was performed by B&W.

Secondary deposits observed in the concrete specimens were analyzed using a Scanning Electron Microscope (SEM) with Energy Dispersive X-Ray Spectroscopy (EDS) capability. EDS analyses were performed to determine chemical compositions of the secondary deposits. The SEM/EDS analyses were performed by B&W.

EXAMINATION RESULTS

General Description of Concrete

The concrete represented by the cores can be described as a non air entrained concrete containing a 1 in. maximum size crushed coarse aggregate and a natural sand. The coarse aggregate particles are comprised predominantly of diabase igneous rock particles. The fine aggregate is comprised predominantly of quartz particles. The sand also includes smaller amounts of igneous rock lithics, feldspar particles, siltstone particles and hematite particles and minor amounts of mica, pyroxene and amphibole particles. The concrete contains both portland cement and fly ash as the cementitious constituents. The cement paste is judged to be of fair quality having an estimated water-cementitious ratio of 0.60. The concrete represented by the cores is well consolidated.

Core No. 122 - Section 1

Preliminary Exam

The top surface of the core exhibits a light color relative to the bulk of the core. This surface exhibits a light loss of cement paste, with fine aggregate particles and portions of a few coarse aggregate particles exposed. The cement paste comprising the top of the core is very weak and highly absorptive. The exterior surface of the core shows cement paste near the surface of the core which exhibits a distinctly lighter color than the cement paste lower in the core. Photographs showing this feature are presented in Figure 5 and Figure 6. These light colored areas represent cement paste that has been degraded by the boric acid exposure. As can be seen by the photographs, the depth of the degraded cement paste is significantly deeper adjacent to the steel channel compared to the areas away from the steel channel. The depths of affected cement paste are shown in Figure 5 and Figure 6.

The core fractured while being clamped into place for saw cutting. Secondary deposits observed on the fracture surface indicate that the fracture was due to a pre-existing crack present in the concrete. The fracture surface of the core showing the secondary deposits is presented in Figure 7.

Examination of Lapped Section

The lapped section of the core is shown in Figure 8. As can be seen in the photograph the stainless steel channel debonded from the concrete.

Depth of Degraded Cement Paste

The examination of this section shows degraded cement paste away from the channel to depths ranging from 1.5 mm to 3 mm below top surface of the core. Adjacent to the channel the cement paste is degraded to a depth of about 7 mm on one side and a depth of about 23 mm on the other side. The deeper depth of degradation correlates with the area of the pre-existing crack. Below the channel the cement paste is degraded to typical depths of 5 to 8 mm, although one area is degraded to a depth of less than 1 mm.

The cement paste affected by the penetration of boric acid is lighter in color and is significantly weaker and more absorptive than the cement paste in the bulk of the core.

Secondary Deposits

The concrete contains white colored secondary deposits within the air voids. These deposits typically have a fibrous habit. The secondary deposits are very common in air voids just below the degraded cement paste (see Figure 9). Occasional secondary deposits were observed throughout the full depth of the core section.

Examination of Fractured Surface

The fractured surface of the core was examined under the stereomicroscope. As previously mentioned, the fractured surface contains white secondary deposits. These deposits can be seen with the unaided eye and are present on only the lower portion of the section (see Figure 7). The concrete section has a thickness of about 1.8 in. The cement paste comprising the surface of the fracture plane has been degraded by boric acid penetration to a depth of about 0.9 in. The area of the affected cement paste does not contain the secondary deposits.

The crystal morphology of the secondary deposits could not be discerned under the stereomicroscope. SEM/EDS analyses were performed to identify the composition of the material. The SEM/EDS analyses identified boron as a major component of the secondary deposit material. The boron compound was not identified. An EDS spectrum showing the presence of boron is shown in Figure 10.

Examination of Bottom Lapped Surface

The bottom surface of Core 122, Section 1 was lapped to examine the concrete surrounding the steel stud that was used to anchor the stainless steel channel. The steel stud has a diameter of roughly ½ in. and is shown in Figure 11. The concrete is separated from the majority of the stud circumference (see Figure 11). The cement paste surrounding a portion of the stud appears slightly softer than typical. The thickness of this cement paste is only about 1 mm. It is unclear if this area has been affected by boric acid.

Two parallel cracks extend from the stud into the concrete.

Core No. 122 - Section 2

The lower section of Core 122 contains a No. 7 steel reinforcing bar (⅞ in. diameter). The core section was saw cut perpendicular to the rebar. The examination of the lapped section revealed that the rebar does not exhibit corrosion.

Void areas are present below the rebar and at the lower sides of the rebar. These voids appear to be related to both settlement of the concrete and to insufficient consolidation of the concrete around the rebar.

Cracks are present in the concrete that extend from the sides of the rebar up to the top of the core section. A vertically oriented crack was also observed within the concrete. This crack extends from the bottom of the rebar to the bottom of the core section.

Core No. 123

Preliminary Examination

The preliminary examination of the core revealed the presence of a horizontal crack located at a depth of 2 ¾ in. below the top surface of the core. The crack is present at the midpoint level of the No. 9 rebar. This crack is shown in Figure 12.

This examination also revealed the presence of a vertically oriented crack extending down from a corner of the stainless steel channel. The crack extended to a depth of 5 mm below the channel.

Core No. 123 - Section 1

Examination of Lapped Section

The lapped section of the core is shown in Figure 13. As can be seen in the photograph the stainless steel channel debonded from the concrete.

Depth of Degraded Cement Paste

The examination of the section shows degraded cement paste at the surface of the core to depths ranging from 1.3 mm to 3.0 mm below top surface of the core. Below the channel the cement paste is degraded to a maximum depth of 1.3 mm. Some areas of the cement paste below the channel show no degradation.

The cement paste affected by the penetration of boric acid is lighter in color and is significantly weaker and more absorptive than the cement paste in the bulk of the core.

Secondary Deposits

The concrete contains white colored secondary deposits within occasional air voids throughout the depth of the section. These deposits typically have a fibrous habit, although some having a blocky habit were also observed.

Cracking Distress

There was no cracking distress observed in the lapped section.

Core No. 123 - Section 2

The lower section of Core 123 contains a No. 9 steel reinforcing bar (1 ⅛ in. diameter). The core section was saw cut perpendicular to the rebar. The examination of the lapped section revealed that the rebar does not exhibit corrosion.

The concrete is separated from the underside of the rebar. The gap between the rebar and the concrete is about 0.8 mm. This feature is shown in Figure 14. The gap is judged to be due to settlement of the concrete below the rebar. A crack extends from one side of the rebar, upward at an angle of roughly 45°. This crack looks like a continuation of the settlement gap. The gap narrows into a tight crack (see Figure 14). The crack extends to the top of the section.

On the other side of the rebar a horizontal crack extends from the rebar (see Figure 14). This crack is relatively tight and passes through a coarse aggregate particle. This is the same crack shown in Figure 12.

Core No. 124 - Section 1

Examination of Lapped Section

The lapped section of the core is shown in Figure 15. In this core the stainless steel channel remained bonded to the concrete.

Depth of Degraded Cement Paste

In areas away from the channel, the cement paste is degraded to depths of 3 to 4 mm. The areas adjacent to the channel exhibit deeper levels of degradation. One side of the channel has degraded cement paste to a depth of 7.5 mm, while the other side of the channel shows degraded cement paste to depths of up to 17 mm below the core surface (see Figure 15). Below the channel, the depth of the degraded cement paste ranges from 0.8 mm to 3.5 mm. The areas of the degraded cement paste are shown in Figure 15.

The cement paste affected by the penetration of boric acid is lighter in color and is significantly weaker and more absorptive than the cement paste in the bulk of the core.

Secondary Deposits

The concrete contains white colored secondary deposits within occasional air voids throughout the depth of the section. These deposits typically have a fibrous habit, although some having a blocky habit were also observed. An example of fibrous secondary deposits taken under the stereomicroscope is shown in Figure 16.

SEM/EDS analyses were performed on some of these secondary deposits to identify their composition. The fibrous secondary deposits were identified as ettringite (see Figure 17). Analyses of the blocky secondary deposits were not performed, although the deposits are likely calcite.

Cracking Distress

The core section contains three vertical cracks below the channel. One of these cracks is very tight and does not extend through the full depth of the section. Two of the cracks are adjacent to each other and extend the full depth of the section (see Figure 15). These cracks have an actual width of about 0.06 mm. The width of the cracks at the lapped surface ranges from 0.3 to 0.5 mm. The larger width than actual is due to material lost within the cracks during the specimen preparation. The cement paste on either side of each crack does not appear to have been adversely affected by boric acid penetration. It is possible that the missing material within the cracks was affected by boric acid.

Other Features

Several large entrapped air voids are present below the channel. These voids are due to insufficient consolidation of the concrete against the steel channel. The voids can be seen in Figure 15.

Core No. 124 - Section 3

The lower section of Core 124 contains a No. 9 steel reinforcing bar (1 ⅛ in. diameter). The core section was saw cut perpendicular to the rebar. The examination of the lapped section revealed that the rebar does not exhibit corrosion.

Similar to Core 123, a settlement gap is present below the lower portion of the rebar. The gap has a width of 0.5 mm. Cracks extend from both sides of the rebar at 45° angles towards the top of the core section. One of the cracks extends to the top of the core section, while the other crack terminates about 13 mm from the rebar.

A very tight vertical crack is present below the rebar. The crack extends from the rebar to the bottom of the core section.

Examination of Channel Debris

Minor amounts of debris were observed within the stainless steel channel of each core. The debris was examined under the stereomicroscope. The material is comprised of a combination of cement paste particles, quartz particles and remnants of oxidized steel (rust). The rust particles are present throughout the debris and would indicate that this material was derived from the coring operation.

BORON CONCENTRATION

Boron concentration testing was performed on the concrete at various depths in the cores. This testing was performed by B&W Technical Services Group using ICP/MS. The results of this testing as reported by B&W are provided in Appendix B.

One measurement was performed on a sample taken from Core 123 at a depth of 14 in. below the top surface of the core. This sample was tested to determine the baseline level of the boron concentration for the concrete. The boron concentration of this sample was measured at 36 ppm.

For each core the boron concentration is highest near the top surface of the core. The core with the deepest depth of degraded cement paste (Core 122) exhibits the highest boron concentration (measured at 4370 ppm), while the core with the least amount of degraded cement paste (Core 123) shows the lowest boron concentration (measured at 806 ppm). These values are based on the measurements taken near the surface of the cores.

Although the boron concentrations were always lower at the deeper depths compared with the top measurement, the concentrations did not always decrease with depth. In Core 122, the measured boron concentration at the depth of 1 $^{9}/_{16}$ in. is significantly higher than the measured boron concentration at the depth of 1 $^{1}/_{16}$ in. This unusual result can possibly be explained by the fact that the sample taken at the 1 $^{1}/_{16}$ in. depth was from only one drilling location (see photo in Appendix B). Although the sample location was an area of cement paste at the drilled surface, it is possible that the majority of the sample was taken through an underlying coarse aggregate particle, yielding a lower than expected boron concentration.

The test results show that boron has penetrated into the concrete to significantly greater depths than shown by the degraded cement paste. The measurements of the samples taken from the lowest levels of the upper core sections (about 1 to 1 ½ in. depth) were significantly higher than the baseline level. Based on the examination, the boron penetration has not adversely affected the cement paste at the lower depths in the cores.

Measurement on a sample taken at the depth of the steel rebar in Core 123 (2 ½ in. below the core surface) showed a very low boron concentration of 64 ppm.

COMPARISON OF THE CONN-YANKEE AND SALEM CONCRETES

The concrete used for the Salem spent fuel pool was a non air entrained, portland cement concrete containing a diabase coarse aggregate and a natural siliceous sand. The concrete had a designed water-cementitious ratio of 0.49. The concrete used for the laboratory study at CRT used the same constituents and the same concrete mix design as the concrete of the Salem spent fuel pool.

The concrete represented by the Conn-Yankee cores contain fine and coarse aggregates that are similar to the aggregates of the Salem concrete. The major difference of the concrete represented by the Conn-Yankee cores with respect to boric acid penetration is the water-cementitious ratio. The water-cementitious ratio of the concrete represented by the Conn-Yankee cores was estimated at 0.60, which is significantly higher than the Salem concrete. The permeability of cement paste is directly related to the water-cementitious ratio. As the water-cementitious ratio increases, permeability increases.[1] A graph showing this relationship is shown in Figure 18. The Conn-Yankee concrete contains fly ash as a cementitious constituent, whereas laboratory prepared specimens for the Salem testing do not. The presence of fly ash in concrete decreases the permeability of the cement paste relative to a straight portland cement concrete having the same water-cementitious ratio.

SUMMARY & CONCLUSIONS

Three concrete cores removed from the floor slab of the spent fuel pool at the decommissioned Connecticut Yankee Nuclear Power Plant were examined petrographically. The examination was performed to determine the effect of the concrete's exposure to boric acid solution.

It is apparent from the examination of the Conn-Yankee cores that boric acid solution leaked from the fuel pool and came into contact with the underlying concrete. The solution apparently overflowed the stainless steel channels and flowed over the concrete surface. The depth of the affected cement paste is typically significantly deeper adjacent to the channel than away from the channel. This is due to the penetration of the boric acid solution along the channel/concrete interface. The penetration of the solution at this site was likely facilitated by poor bonding of the concrete to the steel channel. If the rate of boric acid solution flowing out of the channel was low, much of the solution would have penetrated along the channel/concrete interface as opposed to flowing over the concrete away from the channel. Vertical cracks located below the side of the channel in two of the cores (Core 122 and Core 123) would have contributed to deeper penetration of the solution in this area.

The boric acid solution has chemically attacked the near surface cement paste of the concrete. In general, acids attack portland cement paste by decalcification (calcium leaching) of the hydrated cement compounds, in particular Calcium Hydroxide and Calcium Silicate Hydrate. The chemical attack has not adversely affected either the diabase coarse aggregate or the natural sand. The cement paste affected by the chemical attack has been significantly weakened relative to the unaffected cement paste. The attacked cement paste is also highly absorptive. The cement paste comprising the immediate top surface of the cores is typically more severely deteriorated than the cement paste affected at lower depths.

The cement paste affected by the boric acid in the Conn-Yankee cores is not as severely deteriorated as the cement paste attacked by the boric acid in the laboratory studies previously performed by CRT (CRT Report No. R-125 dated 12-12-03 and CRT Report No. R-125-3 dated 2-24-06).

Based on the ICP/MS results (see Appendix B), boron has penetrated into the concrete well beyond the depth of the chemically attacked cement paste. This finding was also seen in the previous lab work performed at CRT (CRT Report No. R-125 dated 12-12-03). Following the reaction of the boric acid solution within the near surface concrete, the reacted solution further penetrates into the concrete carrying the boron. The penetration of the boron has not adversely affected the quality of the cement paste at the lower depths.

Another indication of solution moving through the concrete is the presence of secondary deposits (ettringite) observed in air voids at depths of up to 2 in. below the top surface of the cores. As water moves through concrete soluble salts are often dissolved out of the cement paste and re-deposited in air voids upon drying.

Through the SEM/EDS analyses boron was detected as secondary deposits within a vertical crack plane of Core 122. The crack extended from the top of the core to the bottom of the section (1.8 in. depth). Although the boric acid would have had direct access to this crack, the cement paste was adversely affected to a depth of only 0.9 in.

Nick Scaglione, President
Concrete Research & Testing, LLC.

References

1. Powers, T.C., Copeland, L.E., Hayes, J.C., Mann, H.M. "Permeability of Portland Cement Paste" Proceedings, American Concrete Institute, Volume 51, 1954, pp. 285-297.

APPENDIX A

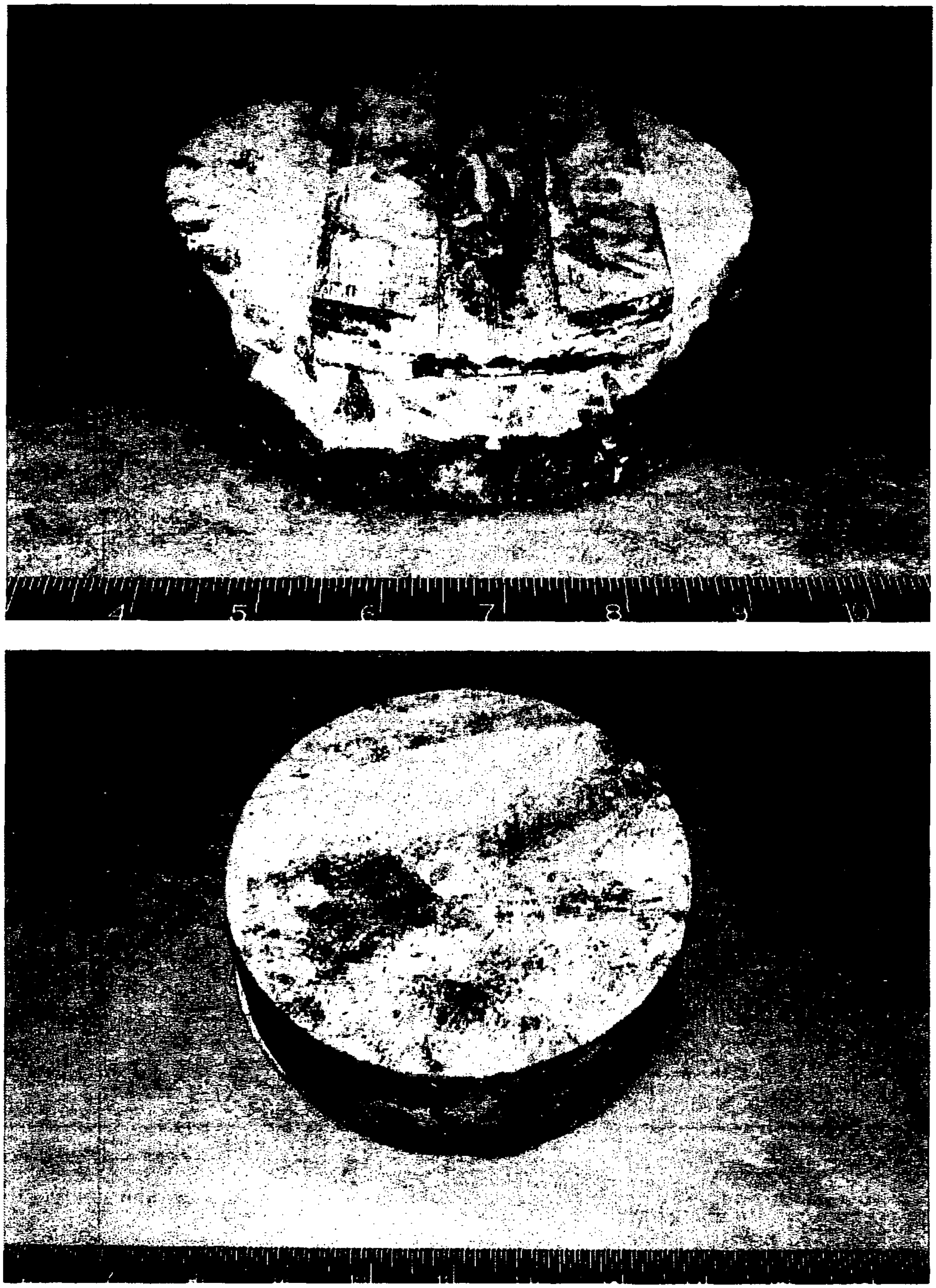


Figure 1. Photographs of Core 122. The upper photograph shows Section 1 (upper 2 in. of core). The stainless steel channel can be seen imbedded in the concrete. The lower photograph shows Section 2 (2 to 4 in. depth).

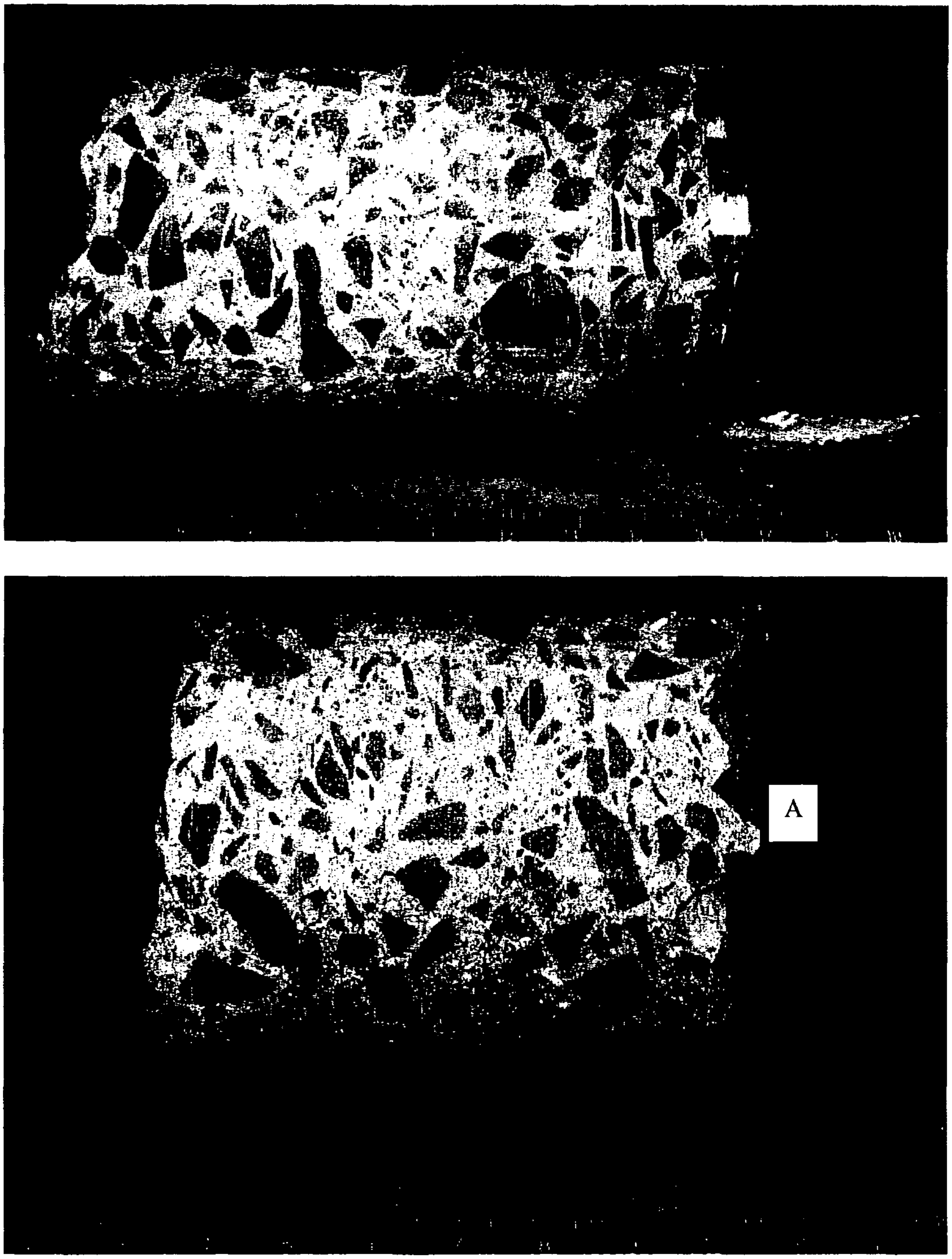
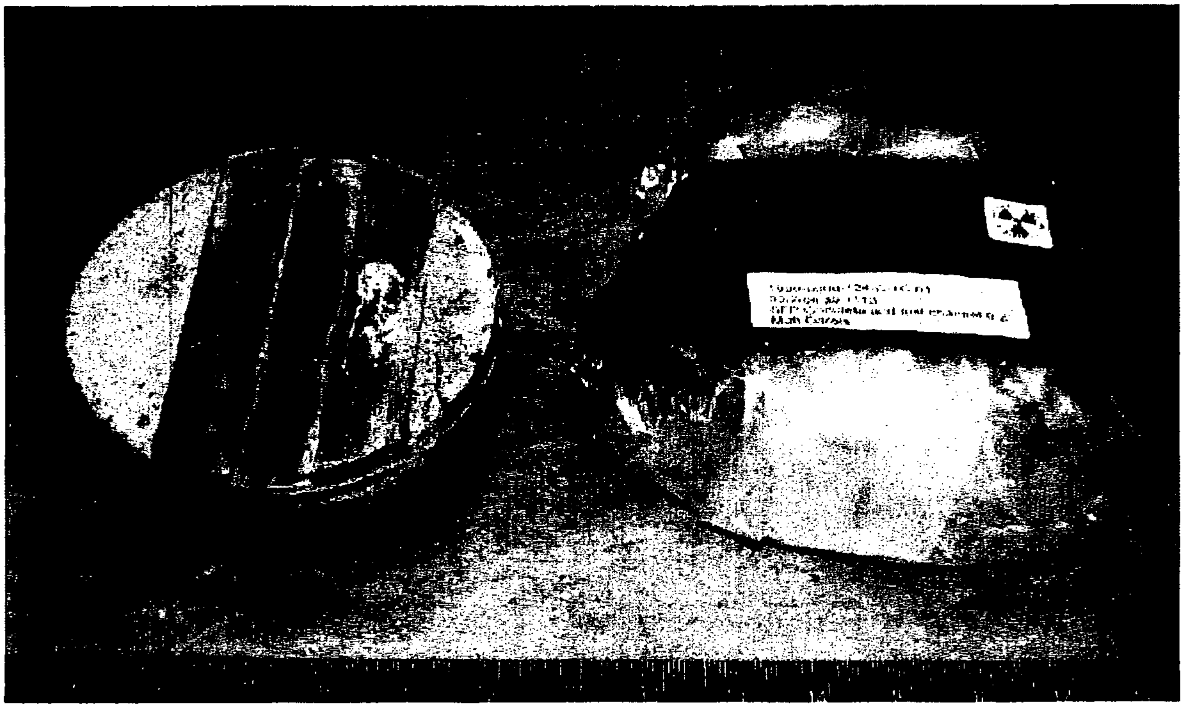


Figure 2. Photographs of Core 123. The upper photograph shows the top 10 in. of the core. The lower photograph shows the core from a depth of 10 to 18 in. These two pieces were originally connected at surface "A".



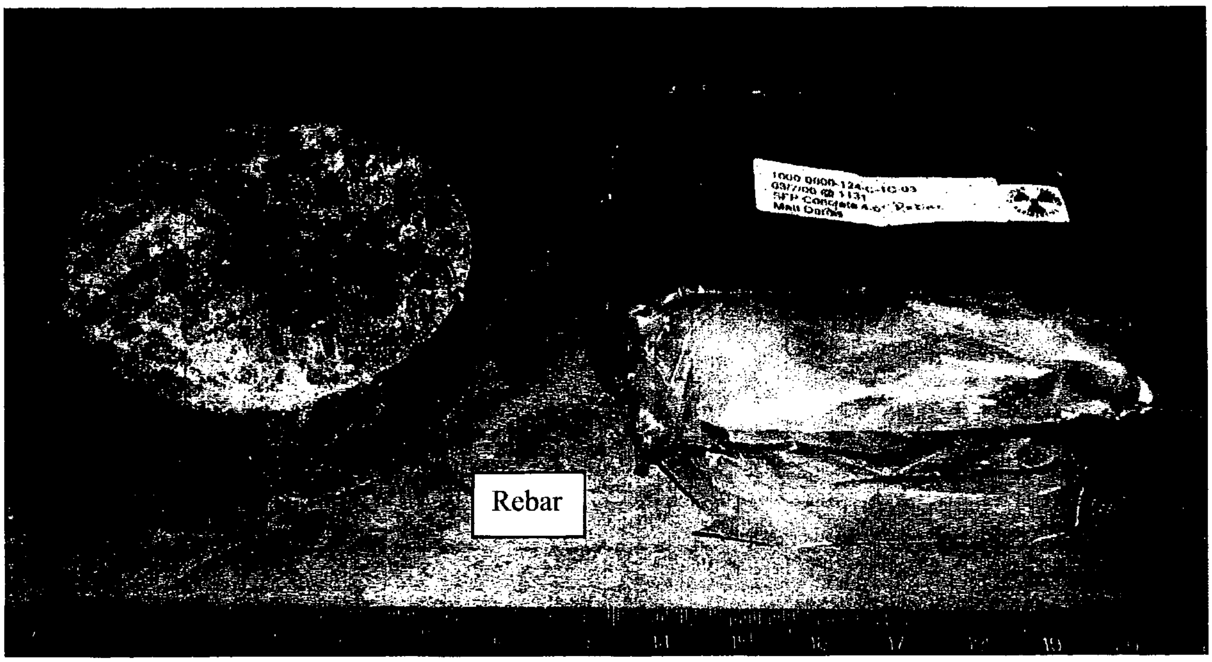


Figure 3. Photographs of Core 124. The upper photograph shows Section 1 (upper 2 in. of core). The lower photograph shows Section 3 (4 to 6 in. depth).

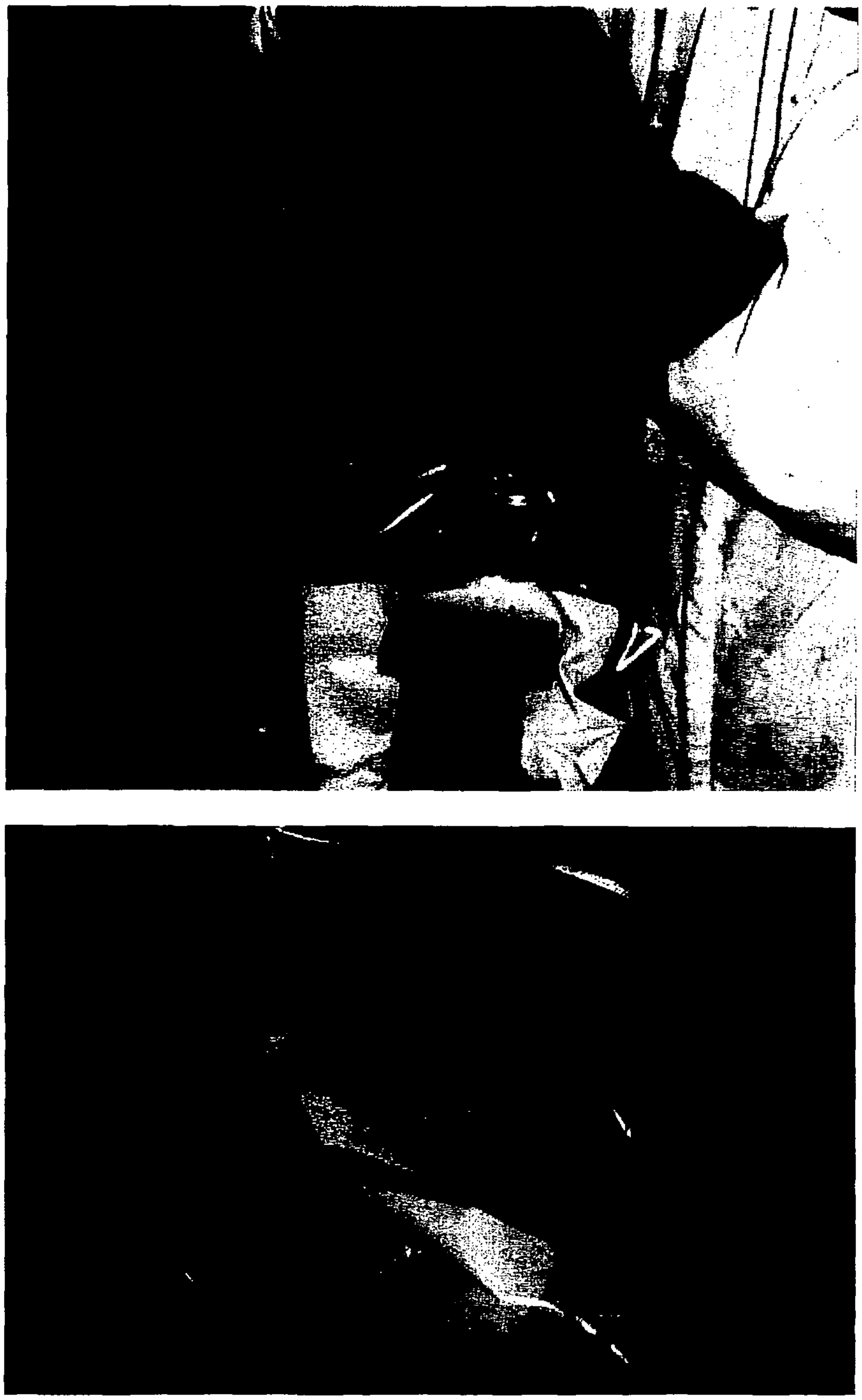
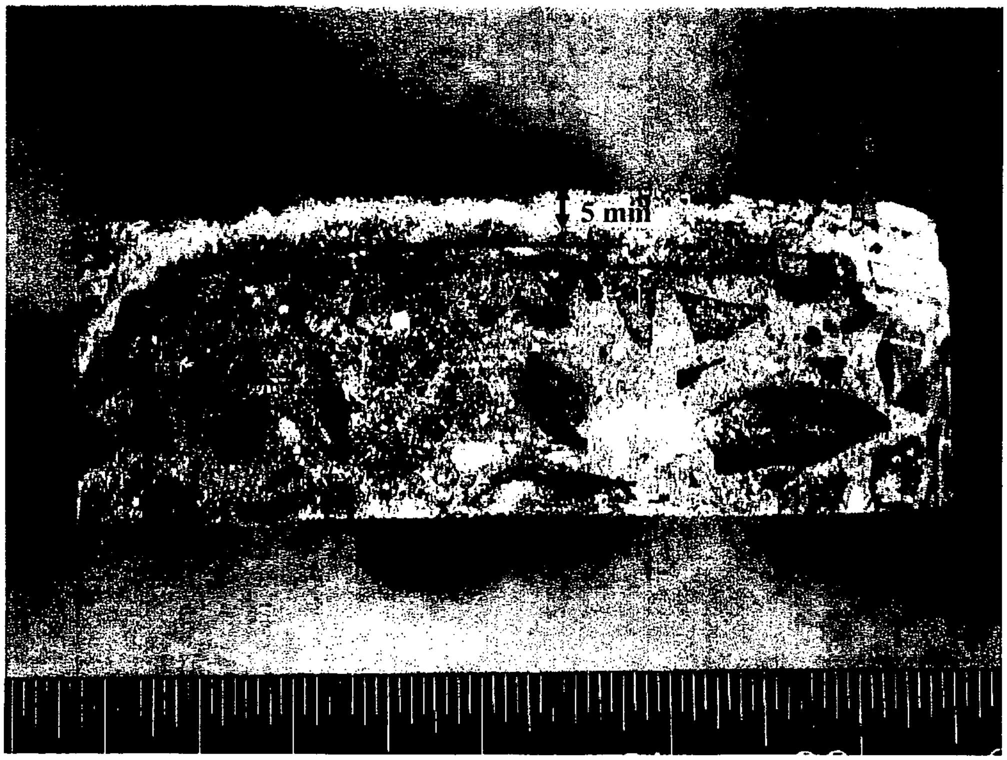


Figure 4. The photographs show the procedure used to remove the powdered concrete samples for the boron concentration testing.



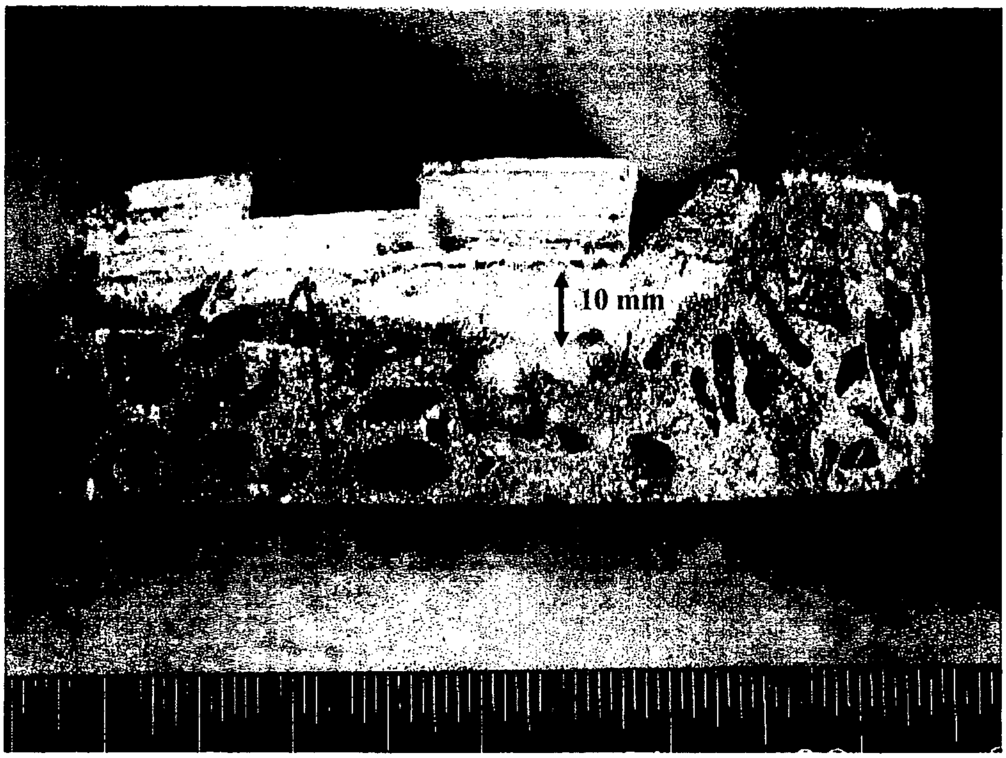


Figure 5. Exterior surfaces of Section 122-1. The light colored cement paste near the top surface of the core represents the degraded cement paste. It can be seen that the depth of the degraded cement paste in the vicinity of the channel is significantly greater than the depth of the degraded cement paste away from the channel.

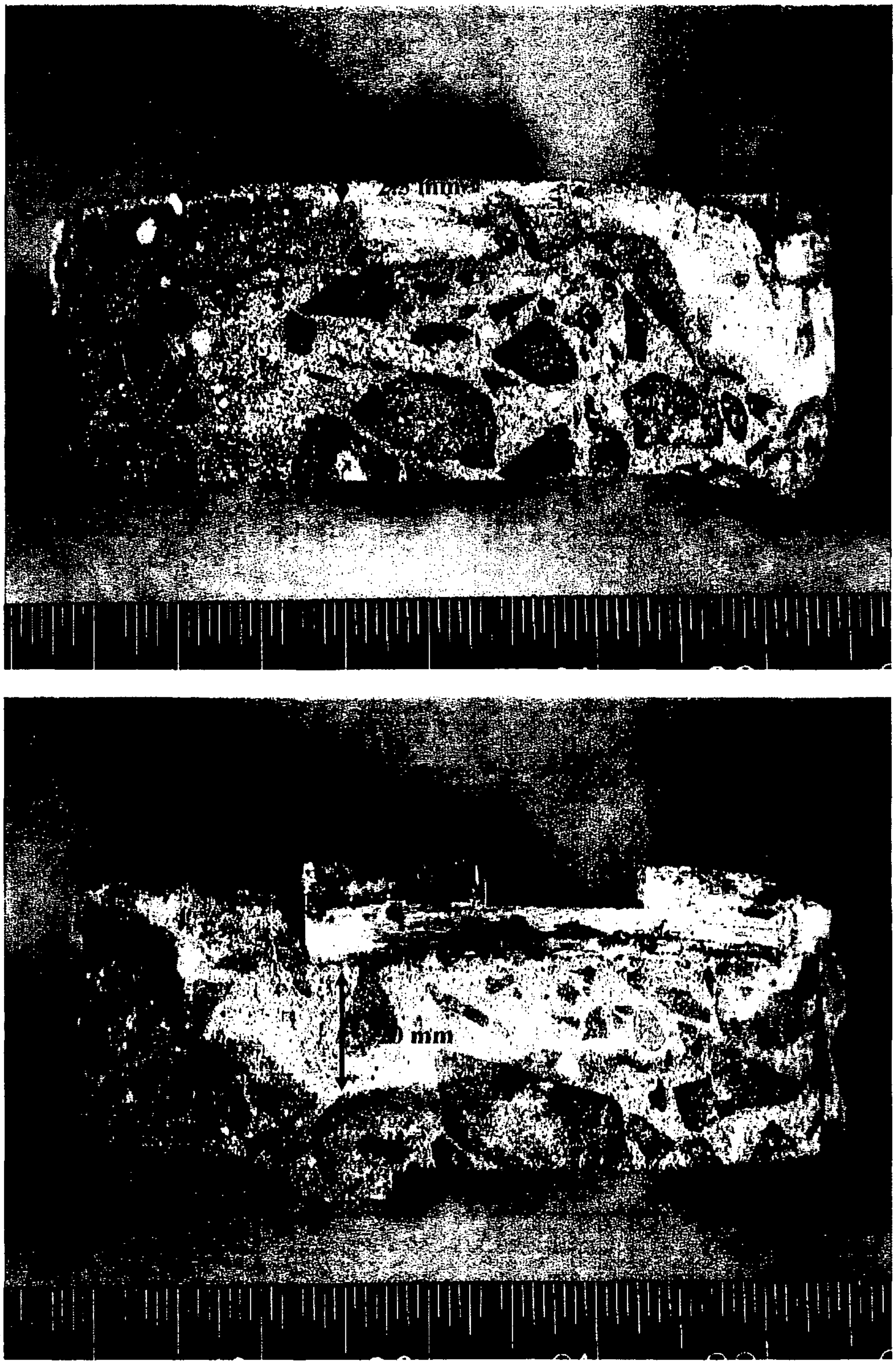


Figure 6. Exterior surfaces of Section 122-1. The light colored cement paste near the top surface of the core represents the degraded cement paste. It can be seen that the depth of the degraded cement paste in the vicinity of the channel is significantly greater than the depth of the degraded cement paste away from the channel.

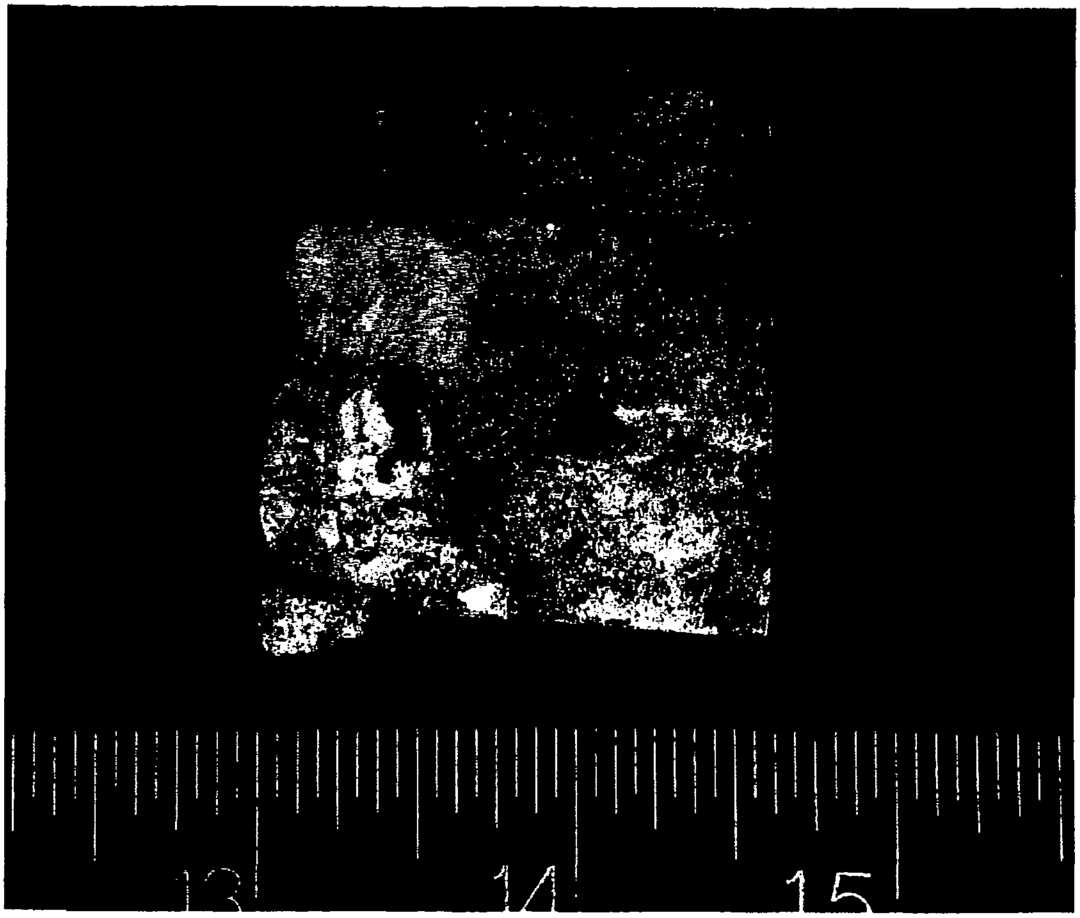


Figure 7. Fractured surface of Section 122-1. This portion of the core broke from the larger section during the saw cutting operation. White secondary deposits can be seen on only the lower portion of the sample where the concrete has not been affected by the boric acid penetration.

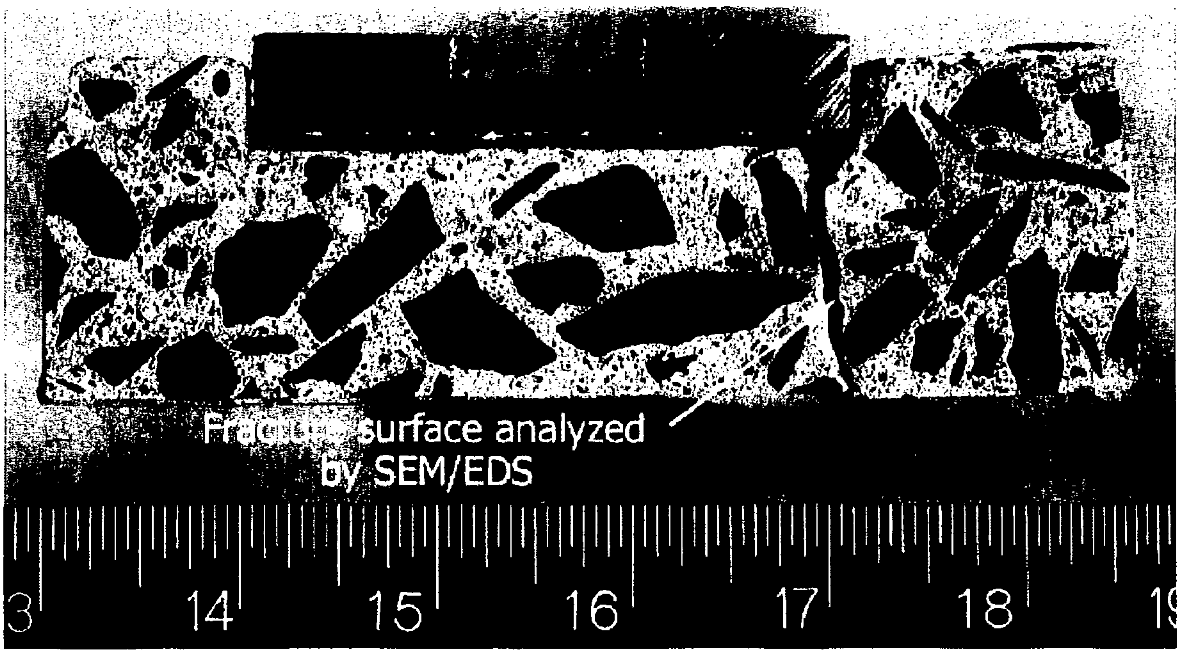


Figure 8. Cross-section view of Core 122, Section 1 (lapped surface).

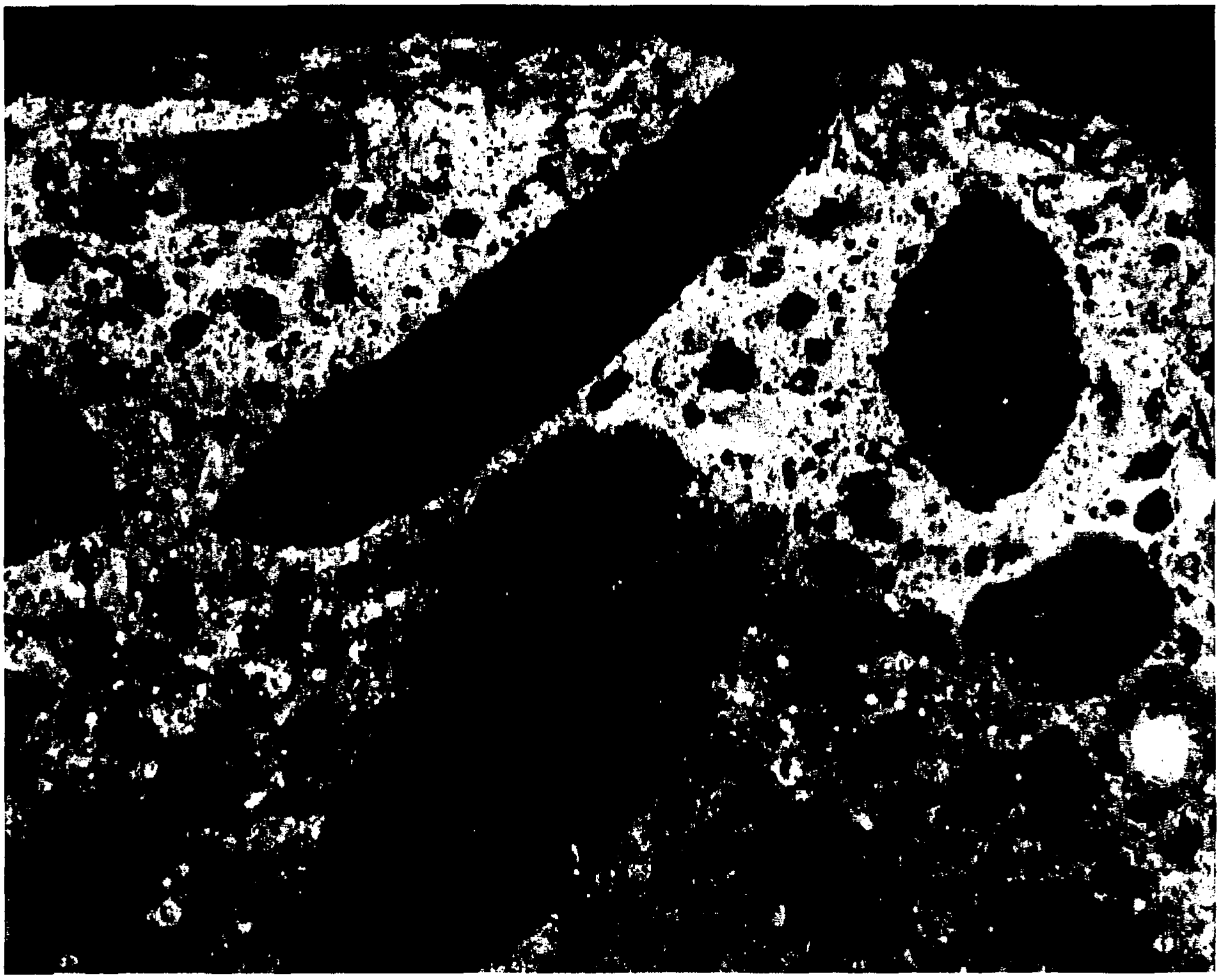
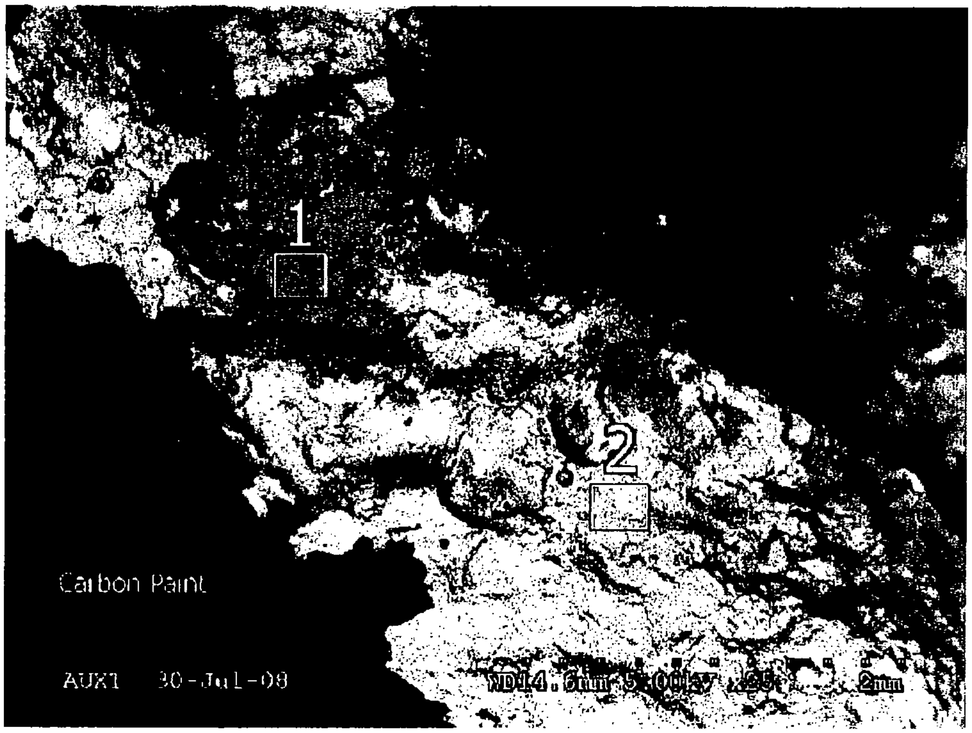


Figure 9. Cross-section view of Core 122, Section 1 taken under the microscope. The photograph shows small air voids filled with white secondary deposits. The air voids within the cement paste affected by the boric acid penetration does not contain the secondary deposits.



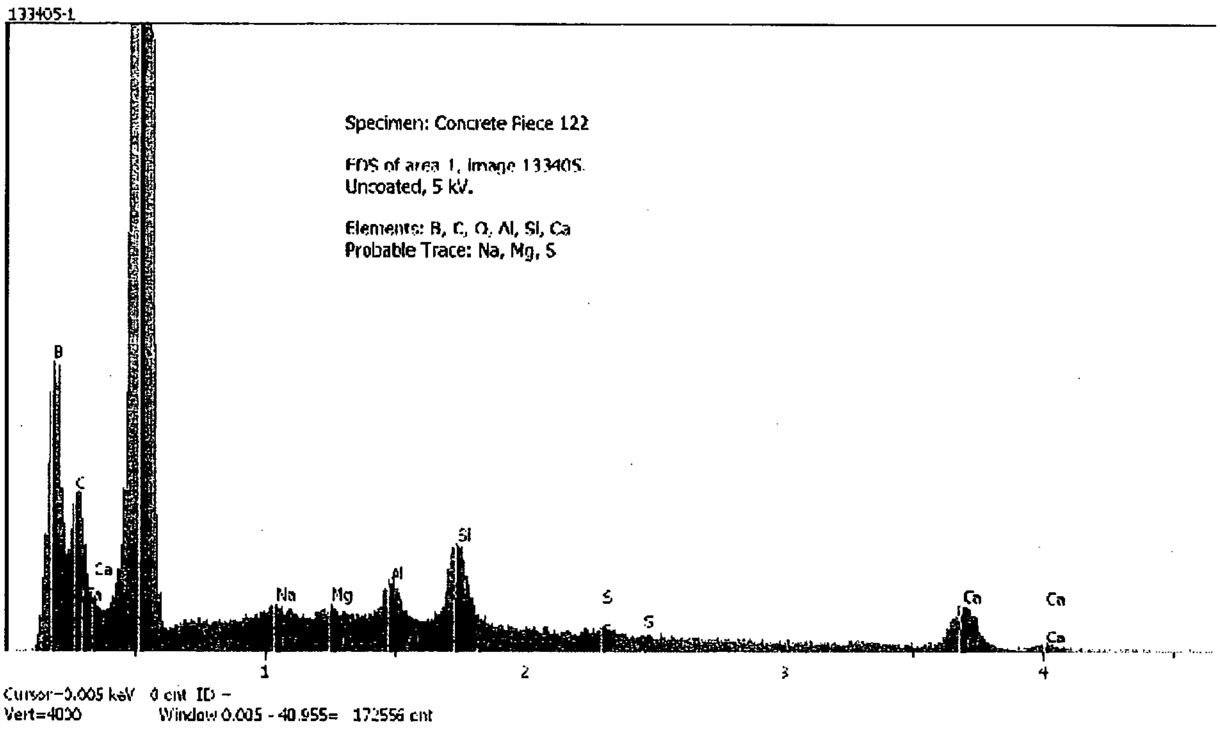


Figure 10. SEM Photograph and EDS analysis of secondary deposits on the fracture surface of Core 122 (see Figure 7). The EDS analyses of the secondary deposit material shows boron as a major element.

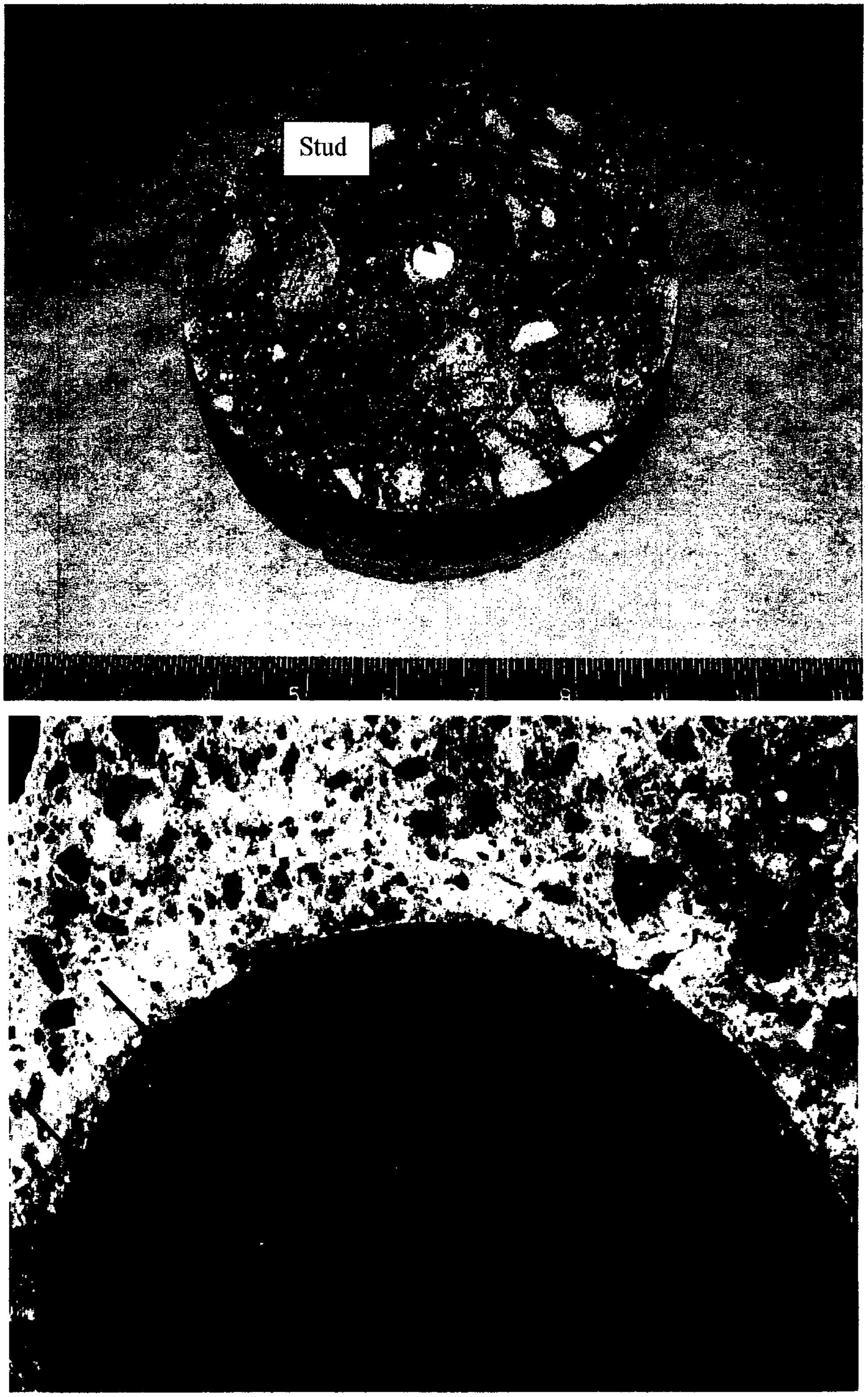


Figure 11. The upper photograph shows the bottom surface of Core 122, Section 1. The lower photograph was taken under the microscope and shows the stud/concrete interface. Note the separation between the concrete and the stud (black arrows) and the crack (blue arrows)

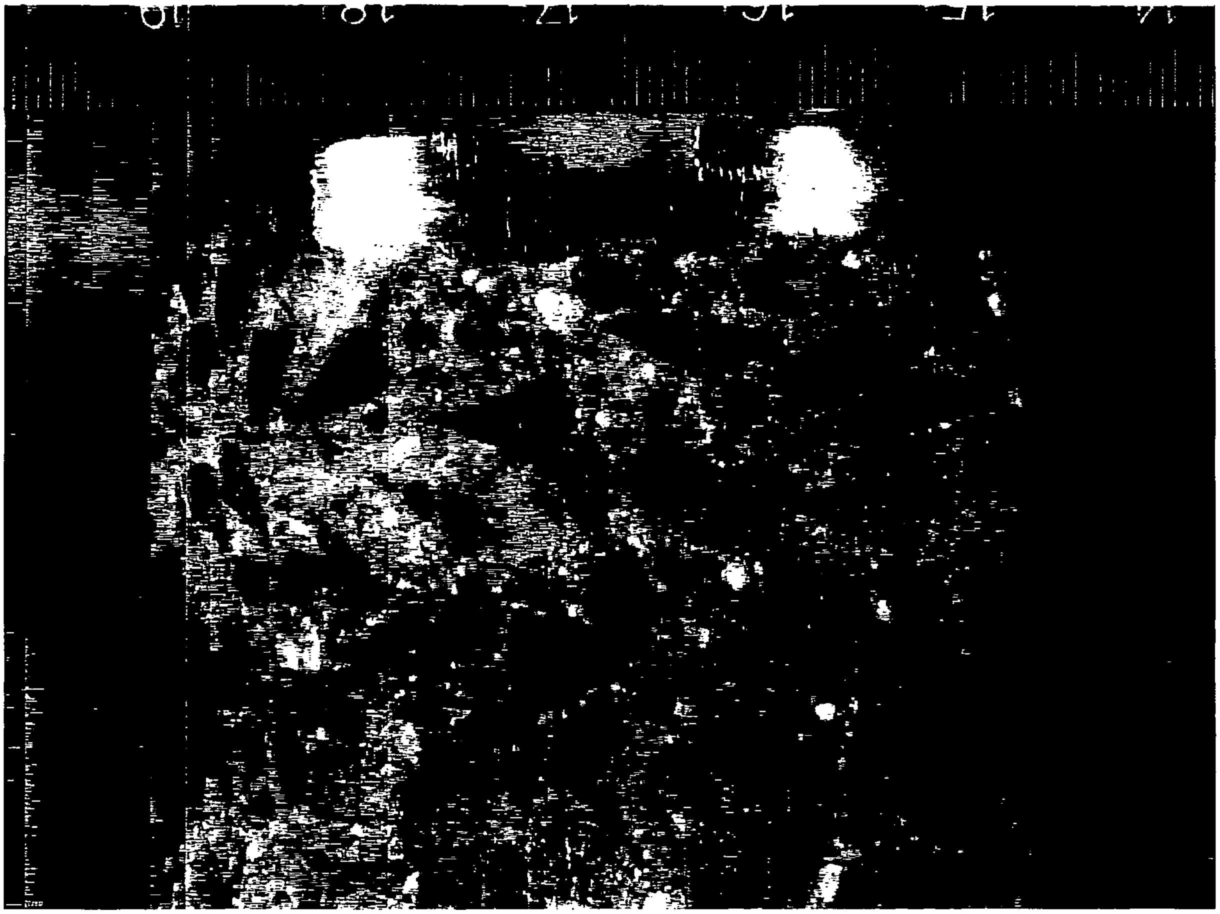


Figure 12. Photograph of Core 123 showing cracking distress at the level of the rebar. The crack was highlighted with black ink so it could be seen in the photograph.

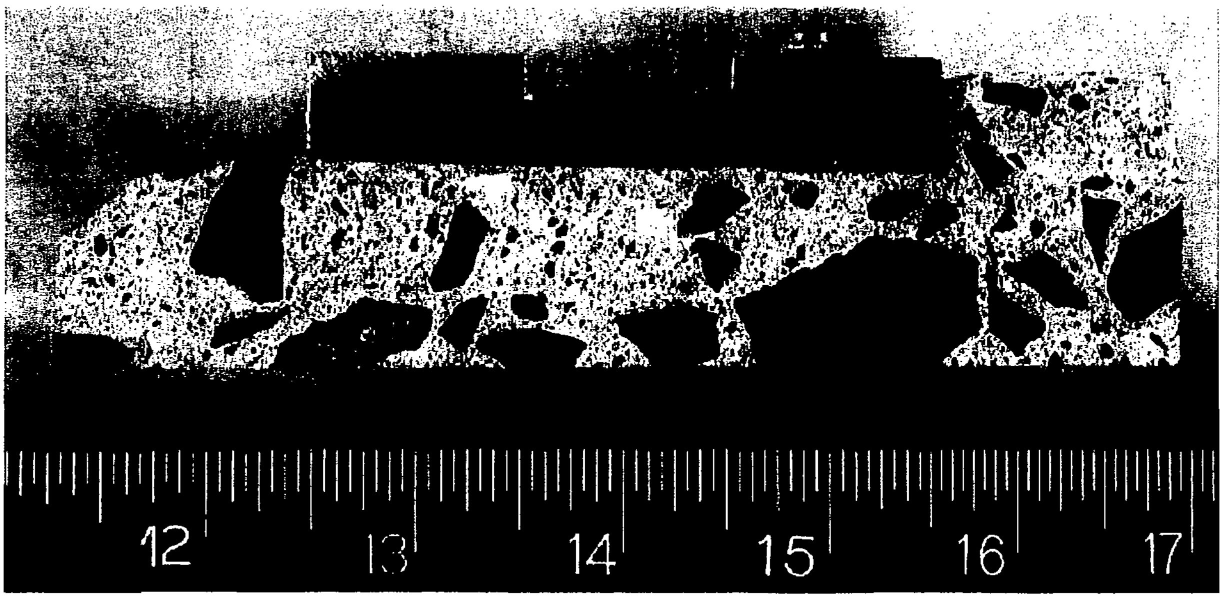


Figure 13. Cross-section view of Core 123, Section 1 (lapped surface). The upper left portion of the specimen was fractured during the removal of the stainless steel liner (see Figure 2).

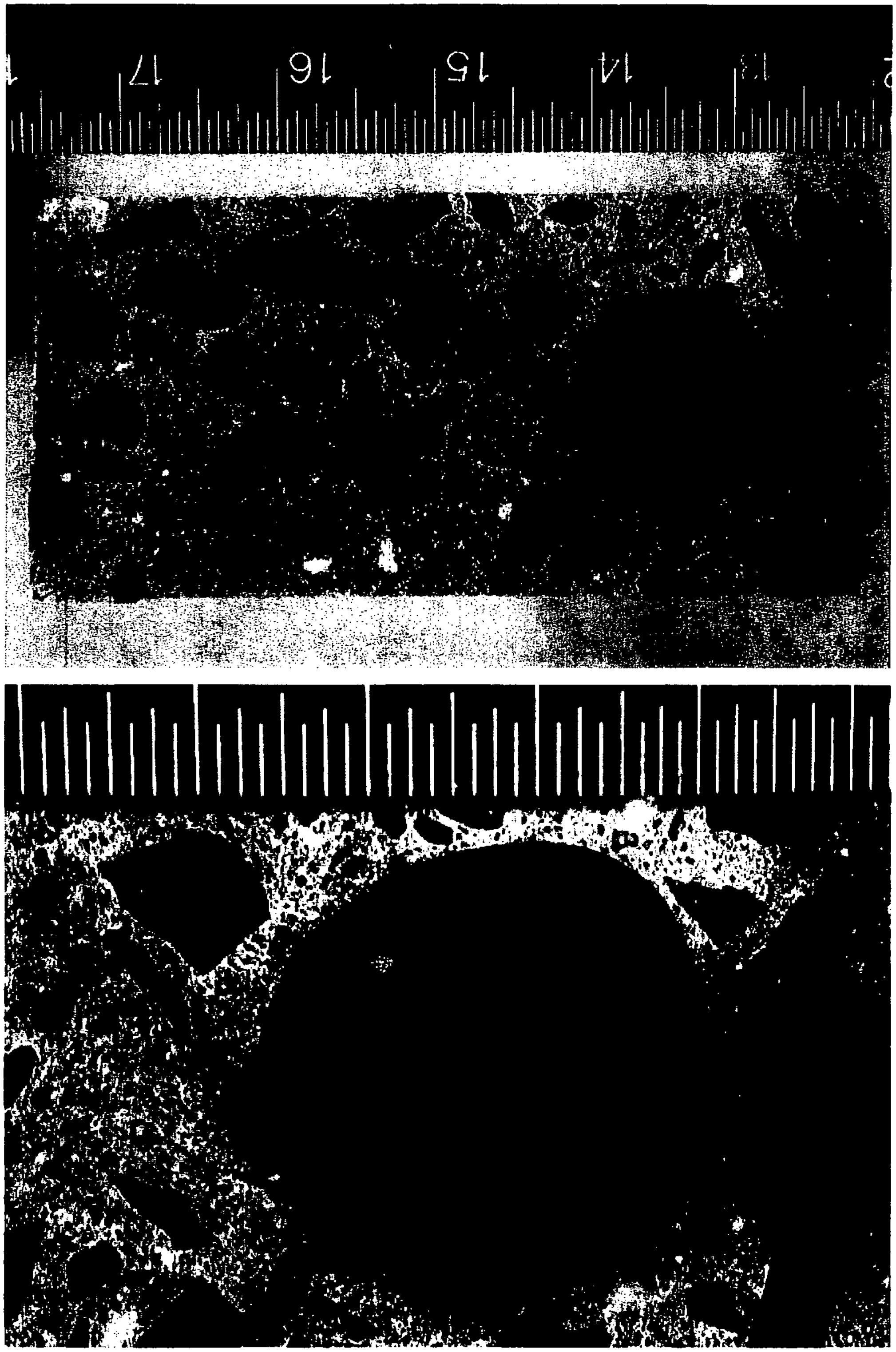


Figure 14. Cross-section views of Core 123, Section 2. The photographs show the cross-section of the No. 11 steel rebar. A gap can be seen at the lower rebar/concrete interface. This gap is due to settlement of the concrete prior to hardening. Cracking on either side of the rebar can also be seen.

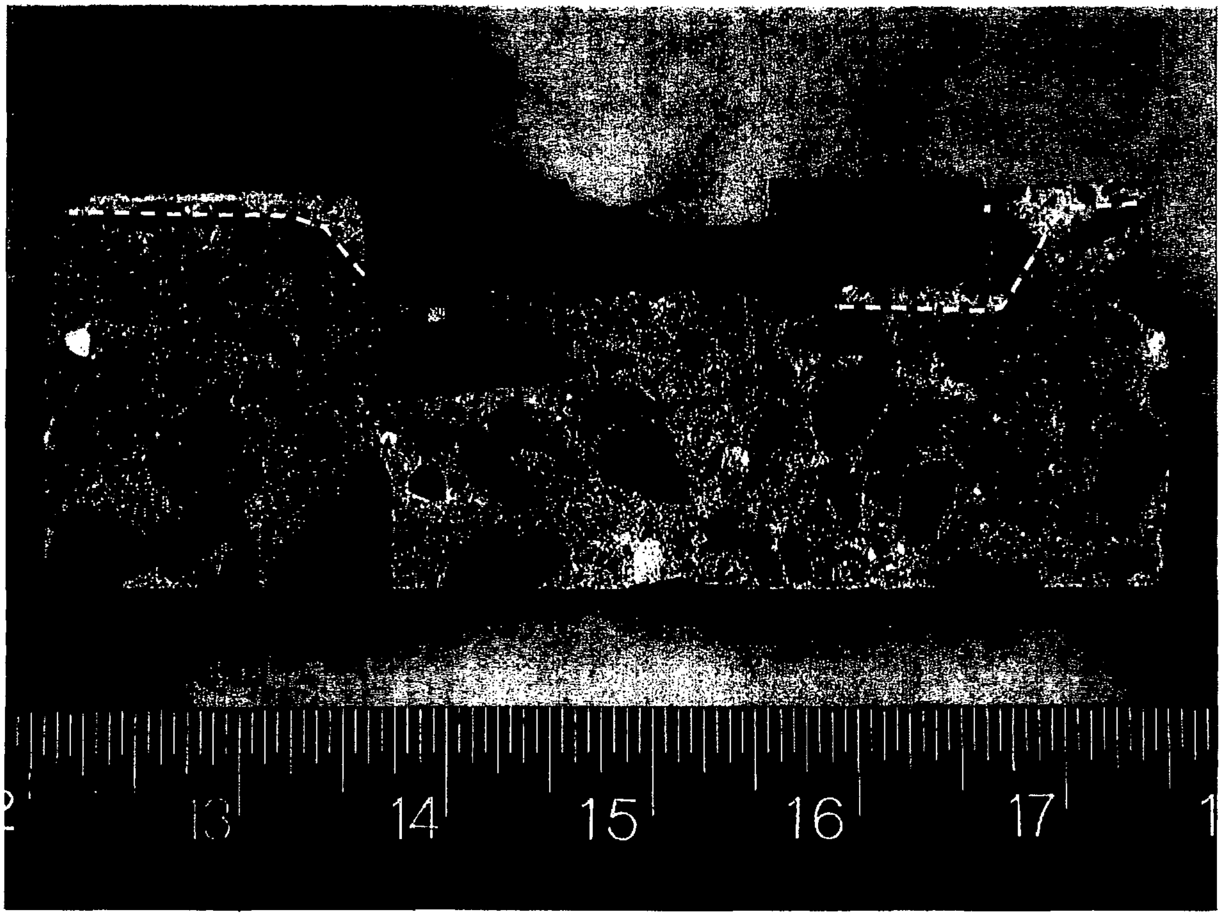


Figure 15. Cross-section view of Core 124, Section 1 (lapped surface). The dashed yellow line shows the area of the degraded cement paste. Two vertically oriented parallel cracks are shown by the arrows.

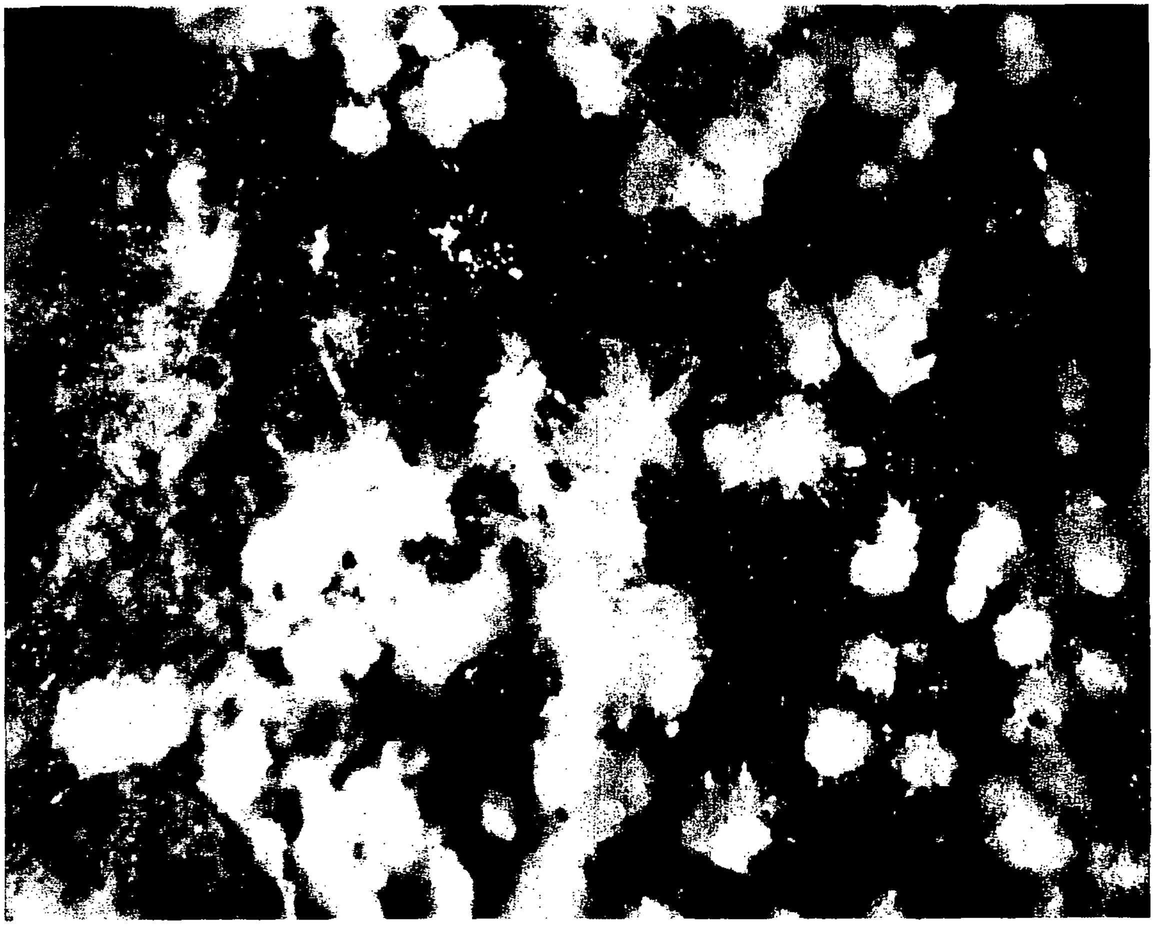


Figure 16. Photograph taken under the stereomicroscope showing the presence of fibrous, white secondary deposits present within an air void.

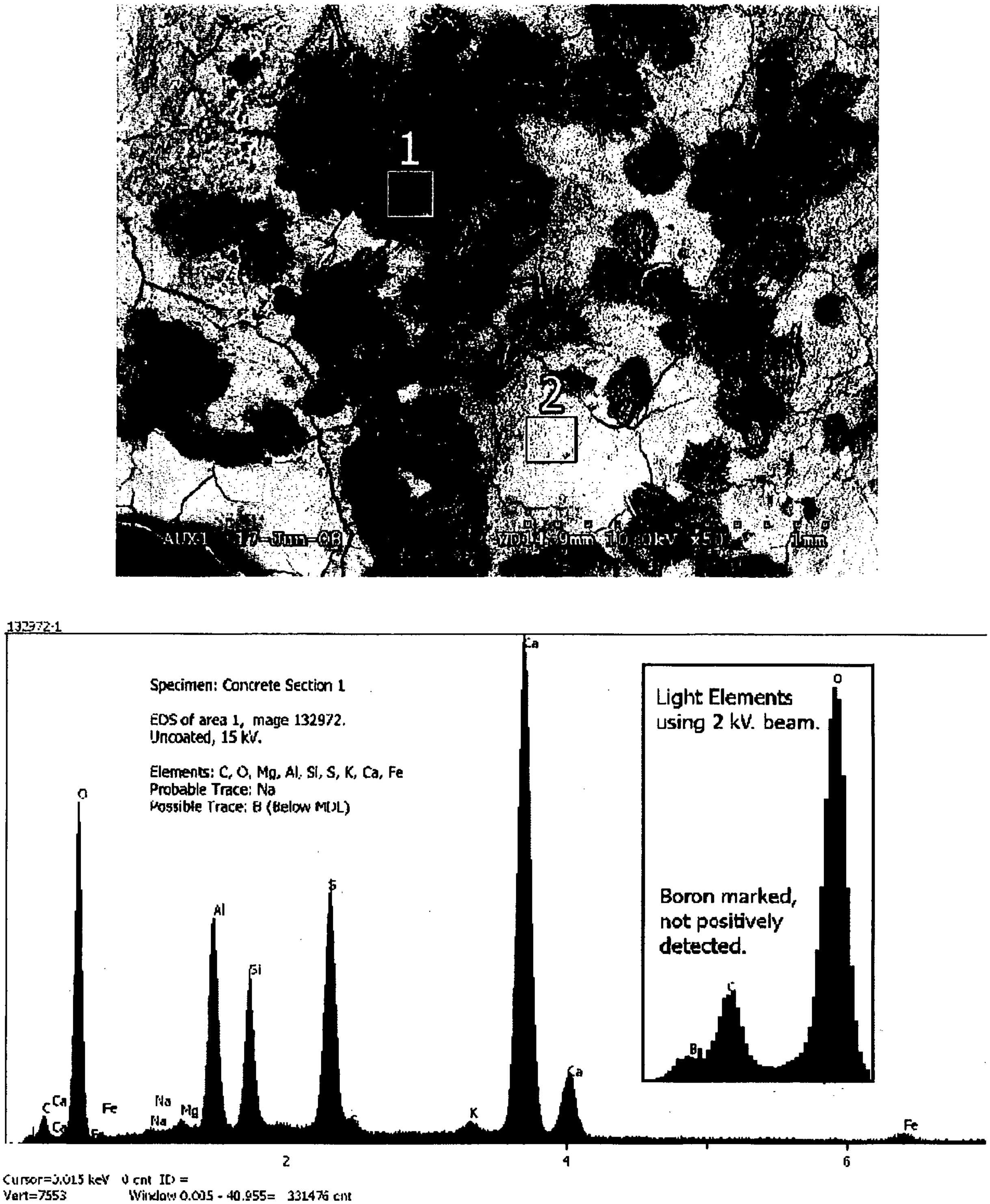


Figure 17. SEM Photograph and EDS analysis of fibrous secondary deposits present in an air void of Core 124. The photograph is the same area shown in Figure 16. The high sulfur and aluminum content indicates that the material is ettringite ($Ca_6Al_2(SO_4)_3(OH)_{12}\bullet26(H_2O)$).

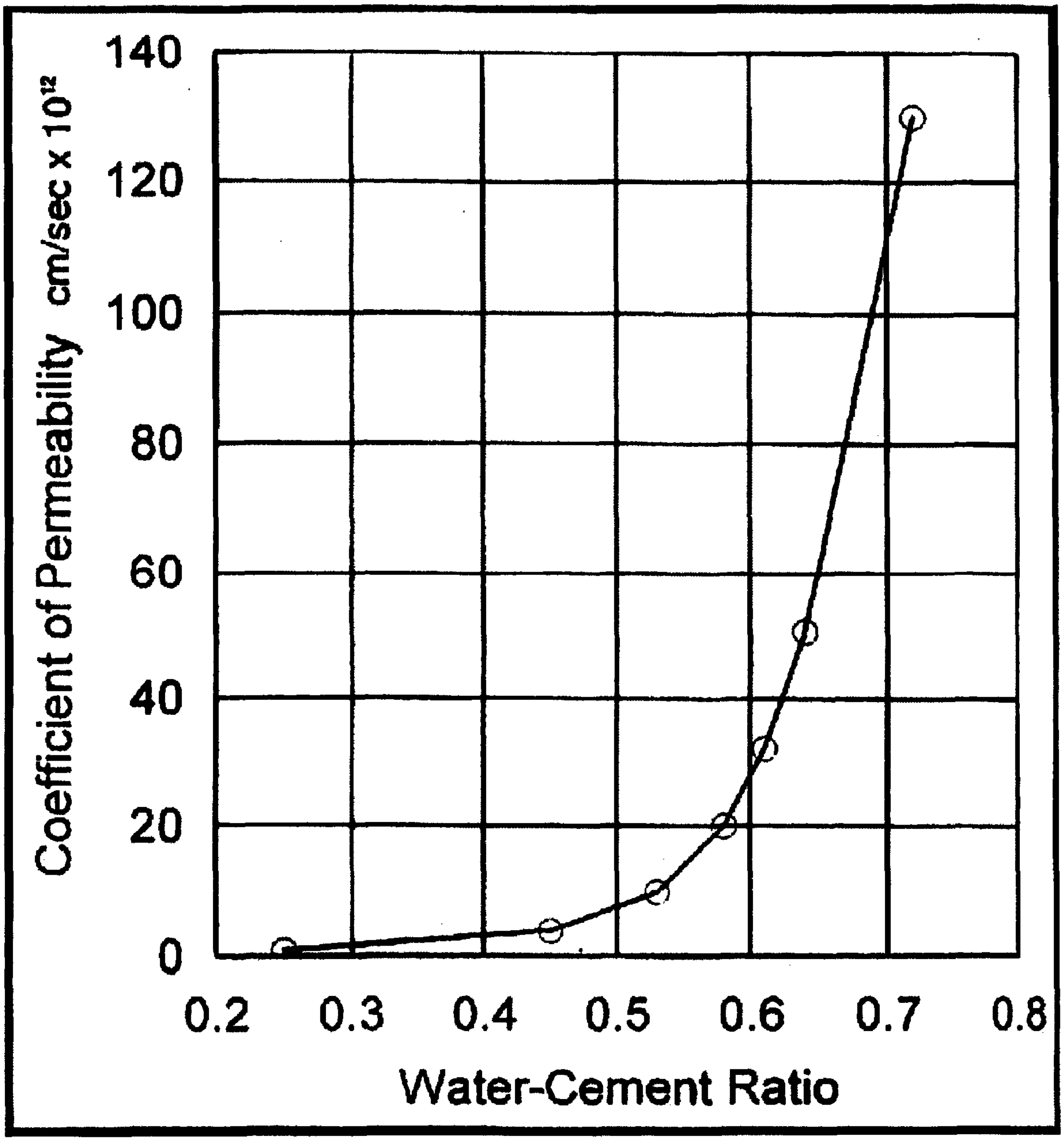
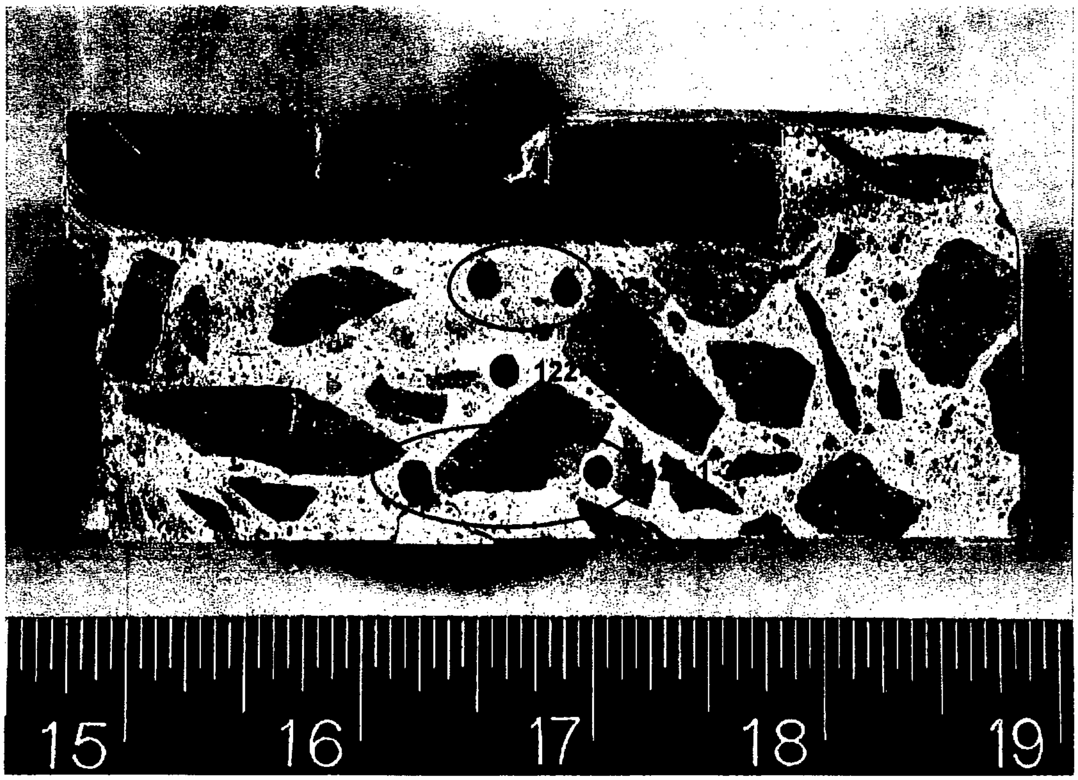


Figure 18. Relationship between permeability and water-cement ratio of mature portland cement paste (reference 1).

APPENDIX B

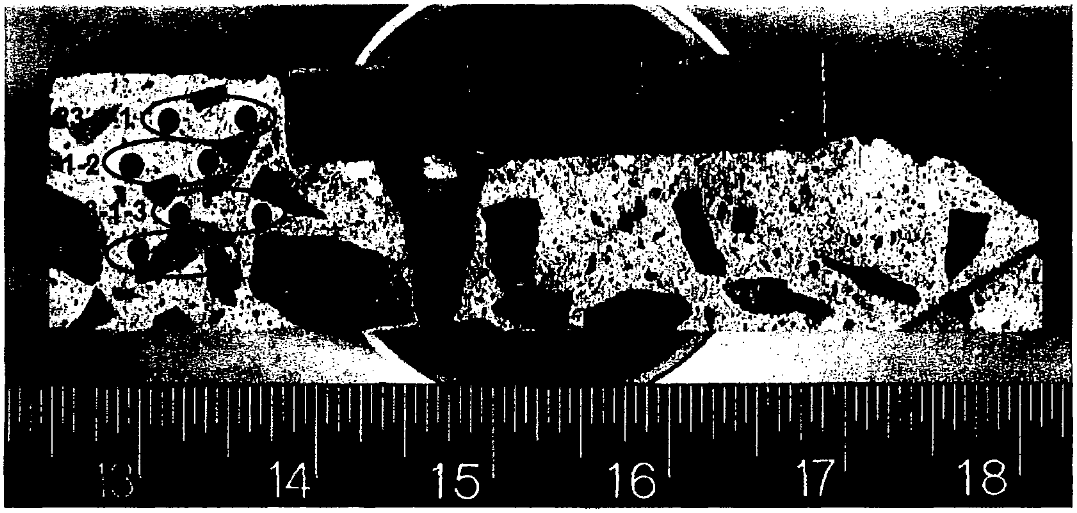
Summary of Boron Results



Core 122 - Boron Sample Locations

Sample ID	Distance below top surface of channel	Boron concentration, ppm
122-1-1	11/16"	4370
122-1-2	1-1/16"	593
122-1-3	1-9/16"	1410

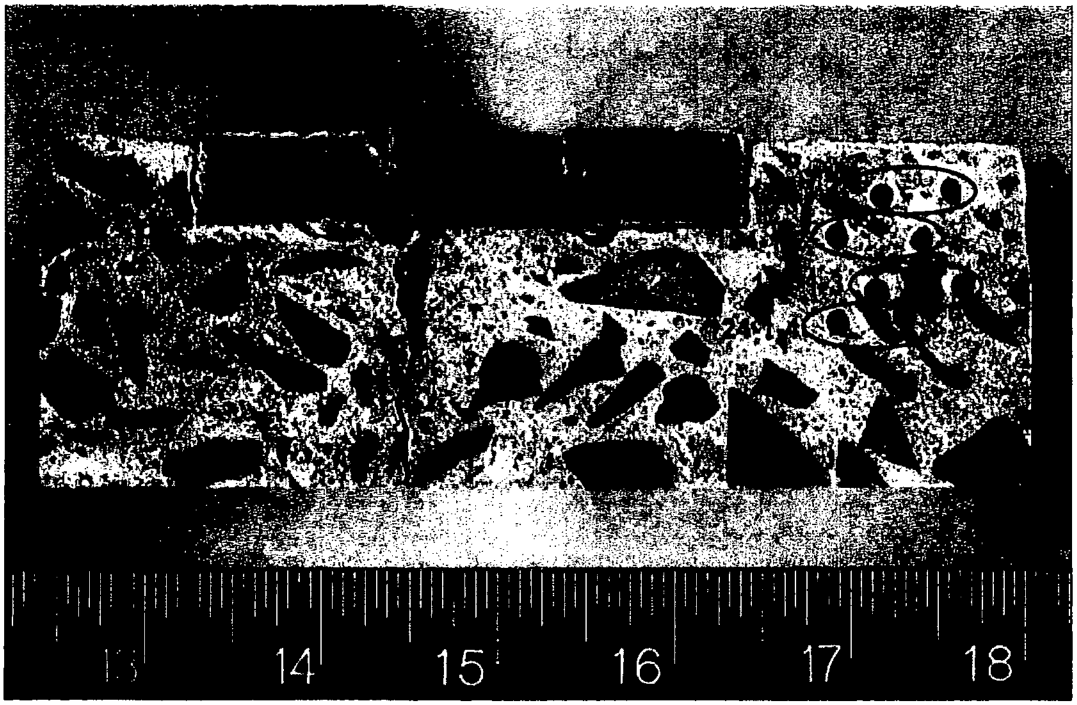
Summary of Boron Results



Core 123 - Boron Sample Locations

Sample ID	Distance below top surface of channel	Boron concentration, ppm
123-1-1	1/4"	806
123-1-2	1/2"	285
123-1-3	13/16"	387
123-1-4	1-1/16"	549
123-2-1	2-1/2"	64
123-baseline	~14"	36

Summary of Boron Results



Core 124 - Boron Sample Locations

Sample ID	Distance below top surface of channel	Boron concentration, ppm
124-1-1	1/4"	3670
124-1-2	1/2"	1000
124-1-3	11/16"	109
124-1-4	1"	136

B

Statistical Analysis of Rebar Yield and Tensile Strength Tests

This appendix contains the following MPR Calculation.

- MPR Calculation 108-275-02, "Statistical Analysis of Rebar Yield & Tensile Strengths for Salem Nuclear Generating Station," Revision 0.

MPR

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320 King Street
Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: PSEG Nuclear	Page 1 of 8 + Appendices
Project: Salem Spent Fuel Pool Leakage	Task No. 108-0303-275
Title: Statistical Analysis of Rebar Yield & Tensile Strengths for Salem Nuclear Generating Station	Calculation No. 108-275-02

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
Lisa Lichtenauer 7/29/03 Lisa Lichtenauer	Michelle Heinz 7/29/03 Michelle Heinz	R. Keating 7/29/03 Robert Keating	0

QUALITY ASSURANCE DOCUMENT
This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
108-275-02	Lisa Lichtenauer	M. [illegible]	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

The purpose of this calculation is to document the statistical analysis of the yield and tensile strengths of the rebar used in the construction of structures at Salem Nuclear Generating Station. The means, standard deviations, and percentages of rebar above certain yield and tensile strengths are calculated for the rebar used in the walls of the structures.

2.0 SUMMARY OF RESULTS

Table 2-1 summarizes the means, standard deviations, and sample sizes of the yield strength of the rebar in the structures at Salem Nuclear Generating Station. The percentages of rebar specimens that are stronger than a given yield strength are also summarized in the table below. For example, ninety percent (90%) of the rebar specimens have a yield strength of 62,200 psi or greater.

Table 2-1
Rebar Yield Strength Analysis

Rebar Type	Mean Yield Strength	Yield Strength Std. Dev	Sample Size	80% Lower Bound	85% Lower Bound	90% Lower Bound	95% Lower Bound
Total	69,840	6,370	394	64,100	63,300	62,200	61,300
No. 6	67,092	5,027	13	64,100	63,850	62,550	62,550
No. 8	69,410	7,376	123	63,500	63,000	61,500	60,750
No. 9	70,815	6,591	95	63,800	63,000	62,500	61,600
No. 10	70,934	5,024	47	66,150	65,400	62,200	62,100
No. 11	69,363	5,490	116	64,950	63,300	62,065	60,900

Table 2-2 summarizes the means, standard deviations, and sample sizes of the tensile strength of the rebar in the structures at Salem Nuclear Generating Station. The percentages of rebar specimens that are stronger than a given tensile strength are also summarized in the table below. For example, ninety percent (90%) of the rebar specimens have a tensile strength of 99,050 psi or greater.

Table 2-2
Rebar Tensile Strength Analysis

Rebar Type	Mean Tensile Strength	Tensile Strength Std. Dev	Sample Size	80% Lower Bound	85% Lower Bound	90% Lower Bound	95% Lower Bound
Total	105,850	5,116	394	101,600	100,300	99,050	97,400
No. 6	102,681	2,786	13	100,000	100,000	99,500	99,500
No. 8	105,751	5,164	123	101,300	100,000	99,700	98,000
No. 9	108,531	3,392	95	105,875	105,550	104,000	102,550
No. 10	107,336	4,773	47	104,350	103,600	102,400	98,800
No. 11	103,512	5,323	116	99,000	97,750	96,750	96,100

3.0 Calculation

The data used in this analysis, summarized in Appendix A, was obtained from PSEG Nuclear records (Reference 1) documenting chemical and physical tests of the reinforced bars at Salem. The tests were performed during the original plant construction. Copies of the original records are provided in Appendix B. The data provided by PSEG Nuclear for this analysis is a sample of test records for the sizes and grades of rebar used in the SFP Building (Grade 60 of Size Nos. 6, 8, 9, 10, and 11 – see MPR Calc in our previous report). The method of including the test data in the analysis is listed below.

1. In cases where the same test results were listed more than once within the records, only one entry is used in the statistical analysis of this calculation.
2. In some cases a tested sample resulted in a yield strength lower than the minimum required 60,000 psi.
   - In this case, if the retests of the heat resulted in values that satisfied the requirement, the retest data was included in the analysis, but not the failing original test value. The first failing test value was assumed to be invalid, but a passing original test was assumed acceptable.
   - If however, the retest yield strengths still did not satisfy the minimum requirement, the entire set of data for that rebar heat was not included in the analysis. Any rebar that did not have a yield strength of at least 60,000 psi was assumed to have not been used in the construction of the Salem Nuclear Generating Station, where Grade 60 was specified.
   - In the cases where there were multiple retests of a given sample, the values were averaged in such a way to maintain no more than three (3) sets of data per heat. This was done in an effort to prevent certain heats from being more heavily weighted in the averages solely because more tests were performed.

- The heats that were averaged, and their method of averaging, are noted in Appendix A with a footnote next to the heat number. For an even number of data values in a heat, the data was averaged in sets of two (2) or three (3) data values in order to obtain no more than 3 values per heat. If a heat included an odd number of data values, one value was maintained, while the others were averaged in groups of two (2).
- If a yield strength value of less than 60,000 psi was obtained for a heat, and no retests were performed, the entire heat was discarded from this analysis.

3. Note also that the tensile strength values were treated in the same manner as the yield strength data. That is, where the yield strength heats were averaged, the corresponding tensile strengths were averaged in the same way. If a set of data or an entire heat were discarded, the corresponding tensile strengths were also discarded.

The mean yield and tensile strengths were calculated for all samples, as well as for each different size rebar. The mean yield and tensile strengths were calculated using the following equation:

$$\overline{X} = \frac{\sum x}{n} \qquad (1)$$

Where: $\overline{X}$ = Mean
x = yield or tensile strength value
n = total number of values

The standard deviation of the samples was calculated using the equation below, where n and x are the same variables as mentioned above, and σ is the standard deviation.

$$\sigma = \sqrt{\frac{n\sum x^2 - (\sum x)^2}{n(n-1)}} \qquad (2)$$

Histograms of the yield and tensile strengths are presented in Figures 3-1 and 3-2 below in order to provide a visual representation of the analyzed data. Note that for the yield strength histogram, the distribution is a truncated distribution. This is due to the fact that, as stated above, rebar that had a yield strength less than 60,000 psi was assumed not included in the construction of the plant because it did not meet minimum specifications.

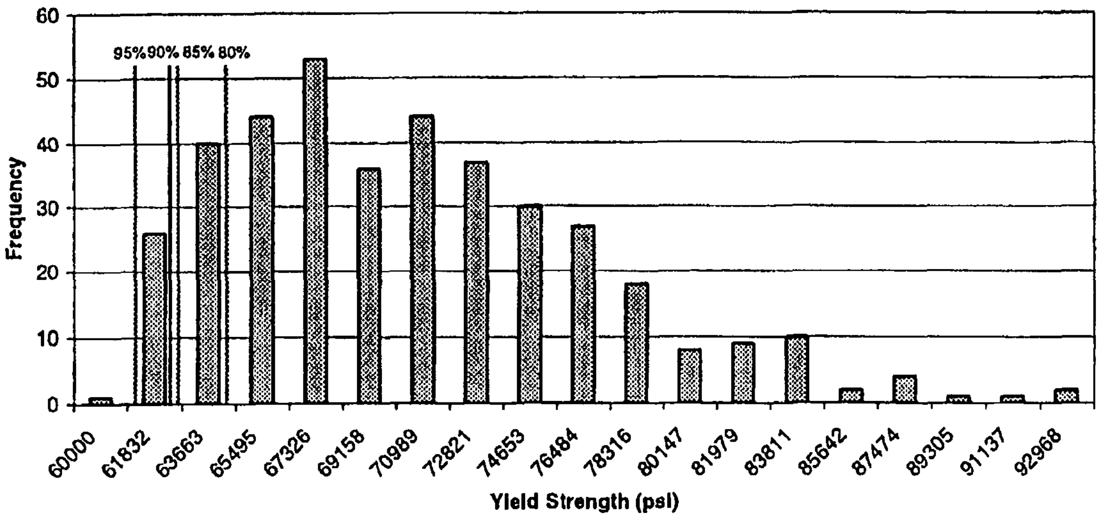

Figure 3-1. Yield Strength Distribution

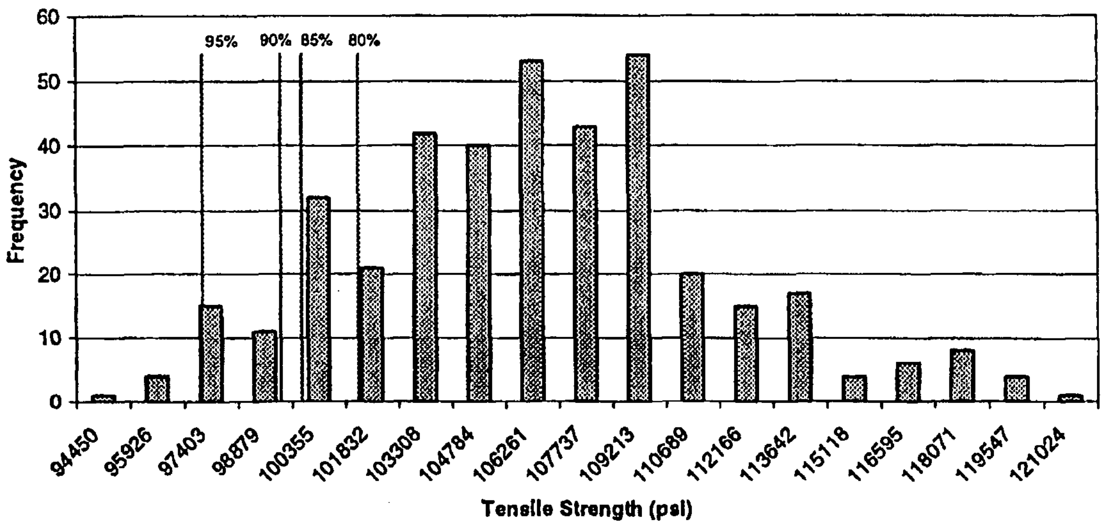

Figure 3-2. Tensile Strength Distribution

The four vertical lines on the two figures above demark the values for which a given percentage of the data has at least that yield or tensile strength. These values are given in Tables 2-1 and 2-2. Given the yield strength distribution, 85% of the rebar specimens had yield strengths equal to or greater than 63,300 psi, and 90% equal to or above 62,200 psi. Looking at Figure 3-2, 85% of the rebar specimens result in a tensile strength equal to or greater than 100,300 psi, and 90% equal to or above 99,050 psi.

4.0 REFERENCES

1. PSEG Nuclear records documenting reinforcing bar tests for Salem Nuclear Generating Station, performed during original plant construction (Provided in Appendix B). (Documents provided to MPR by express package from K. Fisher (PSEG Nuclear) to J. Simons (MPR) dated May 23, 2003).

Calculation No.	Prepared By	Checked By	Page: A-1
108-275-02	Lisa Lichtenauer	M. Ahuja	Revision: 0

A

Appendix A: Rebar Strength Data for Salem Nuclear Generating Station

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
PSEG	1825	64821[a]	8	66,850	108,750
PSEG	1825	64821[a]	8	65,850	108,100
Milton	24	134682	9	66,350	107,650
Milton	24	135386	9	69,400	108,650
PSEG	65	135386	9	65,800	107,400
Milton	24	135849	9	66,150	109,100
PSEG	64	135849	9	66,300	108,800
Milton	1678	135911	8	74,700	97,400
Milton	1678	135911	8	72,750	97,400
Milton	745	135984	11	64,750	107,050
Milton	746	135984	11	73,850	110,650
Milton	746	135984	11	75,500	110,650
PSEG	1089	136068	8	63,000	102,500
PSEG	1089	136068	8	60,800	102,300
PSEG	71	136069	9	63,000	111,800
PSEG	71	136069	9	80,600	112,000
PSEG	72	136099	9	62,000	104,000
PSEG	72	136099	9	61,600	104,600
PSEG	1088	136111	8	62,800	103,000
PSEG	1088	136111	8	61,500	103,000
PSEG	314	136155	11	78,900	117,000
PSEG	314	136155	11	82,000	114,100
Milton	24	136156	9	68,200	107,550
PSEG	60	136156	9	68,500	107,550
PSEG	60	136156	9	66,700	106,800
PSEG	303	136173	11	61,000	104,400
PSEG	303	136173	11	61,400	104,100
PSEG	302	136182	11	65,400	109,000
PSEG	302	136182	11	66,900	109,900
PSEG	70	136388	9	62,000	104,400
PSEG	70	136388	9	62,600	105,300
Milton	782	136663	11	61,688	101,948
Milton	781	136663	11	61,364	101,623
Milton	781	136663	11	67,834	109,873
Milton	513	136787	10	66,150	107,250

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
Milton	515	136787	10	73,000	105,650
Milton	515	136787	10	70,000	106,000
Milton	764	136907	11	72,900	115,150
Milton	780	136907	11	76,751	112,420
Milton	780	136907	11	78,662	112,738
Milton	514	136958	10	76,400	115,850
Milton	515	136958	10	82,250	116,150
Milton	515	136958	10	83,750	116,650
PSEG	296	137010	11	71,794	108,012
PSEG	296	137010	11	67,948	108,653
Milton	764	137010	11	65,050	109,950
Milton	1642	137023	10	62,200	104,900
Milton	515	137023	10	72,750	104,450
Milton	515	137023	10	72,750	104,050
Milton	20	137102	9	69,400	109,700
PSEG	46	137102	9	63,800	108,300
PSEG	46	137102	9	64,000	108,200
Milton	746	137248	11	71,400	99,350
Milton	745	137248	11	66,650	102,650
Milton	746	137248	11	66,100	99,650
Milton	10	137801	9	72,700	108,250
Milton	10	137801	9	74,000	107,150
Milton	10	137951	9	76,000	110,200
Milton	10	137951	9	77,800	108,600
Milton	13	137951	9	72,200	109,100
Milton	10	137958	9	80,600	113,250
Milton	10	137958	9	83,650	113,250
Milton	13	137958	9	74,500	113,250
Milton	10	138010	9	76,000	102,550
Milton	12	138010	9	65,150	102,550
PSEG	28	138010	9	87,250	102,550
Milton	10	138011	9	74,750	105,650
Milton	10	138011	9	72,700	106,200
PSEG	150	138333	11	65,800	100,300
PSEG	150	138333	11	68,750	100,300

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
PSEG	148	138344	11	82,250	111,500
PSEG	148	138344	11	82,050	111,100
PSEG	136	138434	11	65,050	94,450
PSEG	136	138434	11	63,600	95,700
Milton	6	138566	9	71,650	111,850
Milton	6	138566	9	70,100	111,850
Milton	1700	138624	8	63,000	101,300
Milton	1700	138624	8	66,250	101,950
Milton	1718	138958	8	74,050	107,800
Milton	1718	138958	8	75,950	108,450
PSEG	59	138989	11	74,500	104,900
PSEG	59	138989	11	76,300	103,900
PSEG	56	138990	11	69,200	99,050
PSEG	56	138990	11	70,450	99,050
PSEG	58	139000	11	75,150	102,300
PSEG	58	139000	11	76,450	102,950
PSEG	61	139001	11	82,800	102,250
PSEG	61	139001	11	82,450	103,250
PSEG	65	139039	11	71,350	97,750
PSEG	65	139039	11	73,250	98,400
PSEG	16	139639	11	67,950	100,300
PSEG	16	139639	11	68,900	100,950
Milton	1785	139736	9	71,700	109,100
Milton	1785	139736	9	70,700	109,100
Milton	1186	139900	8	63,300	100,650
Milton	1186	139900	8	60,750	100,650
Milton	1186	140843	8	70,150	104,550
Milton	1186	140843	8	70,150	104,550
Milton	1805	140909	8	72,400	106,400
Milton	1805	140909	8	71,400	107,800
Milton	1805	140913	8	68,600	104,500
Milton	1805	140913	8	68,600	104,500
Milton	1805	140935	8	71,500	106,300
Milton	1805	140935	8	71,200	107,700
Milton	1242	140935	8	63,000	103,250

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
Milton	1192	141392	6	64,900	100,250
Milton	1192	141392	6	62,550	100,000
Milton	1192	141470	8	72,750	106,500
Milton	1192	141470	8	76,600	106,500
Milton	1785	141473	9	73,200	105,650
Milton	1785	141473	9	73,200	106,200
Milton	1793	141515	10	71,750	105,250
Milton	1793	141515	10	73,150	106,100
Milton	1219	141842	11	67,500	97,400
Milton	1219	141842	11	65,300	97,400
Milton	1219	141843	11	66,000	96,800
Milton	1219	141843	11	65,100	96,500
Milton	1219	141865	6	64,100	99,500
Milton	1219	141865	6	64,300	100,000
Milton	1477	141898	10	72,000	108,000
Milton	1477	141898	10	70,400	108,000
Milton	1815	141901	9	70,700	105,550
Milton	1815	141901	9	70,900	106,100
Milton	1815	141902	9	71,950	108,150
Milton	1815	141902	9	70,100	108,750
Milton	1819	141953	8	77,250	107,150
Milton	1819	141953	8	71,450	107,150
Milton	1819	141954	8	66,250	101,950
Milton	1819	141954	8	64,950	101,300
PSEG	1875	142548	8	65,150	103,300
PSEG	1875	142548	8	66,450	103,300
PSEG	1877	143440	8	64,450	103,950
PSEG	1877	143440	8	63,650	103,250
Milton	1051	143668	6	73,000	106,050
Milton	1051	143668	6	80,450	106,050
Milton	1921	144032	9	73,750	115,650
Milton	1921	144041	9	79,000	116,850
Milton	1921	144042	9	67,350	107,150
Milton	1921	144043	9	65,300	107,650
Milton	1914	144076	9	69,700	106,050

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
Milton	1914	144597	6	64,650	103,250
Milton	1914	144597	6	64,400	102,800
PSEG	1915	144663	8	63,500	105,600
PSEG	1915	144663	8	64,000	106,600
Milton	1916	144663	8	70,100	109,300
Milton	1914	144691	8	64,450	103,300
Milton	1914	144691	8	65,800	103,950
Milton	1921	144691	8	63,150	103,950
PSEG	1920	144692	8	62,000	100,000
PSEG	1920	144692	8	61,300	99,500
Milton	1923	144692	8	63,150	104,600
Milton	1916	144708	8	66,250	105,850
Milton	1919	144724	8	66,250	105,200
Milton	1064	144854	8	67,100	107,250
Milton	1570	146213	11	71,850	103,300
Milton	1570	148742	11	63,450	102,550
Milton	1678	235532	8	71,450	107,150
Milton	1678	235532	8	75,350	107,800
Milton	24	235656	9	77,150	109,250
PSEG	61	235656	9	65,500	107,200
PSEG	1084, 1631	235722[a]	8	65,760	108,165
PSEG	1084, 1631	235722[a]	8	67,280	108,735
PSEG	311	235766[a,c]	11	61,470	103,415
PSEG	311	235766[a,c]	11	62,065	103,225
PSEG	311	235766[b,c]	11	60,880	104,150
PSEG	300	235783	11	63,200	105,000
PSEG	300	235783	11	60,500	104,900
PSEG	1021	235985[a]	10	62,100	104,850
PSEG	1021	235985[a]	10	62,100	103,350
PSEG	54	236029	9	77,000	115,200
PSEG	54	236029	9	74,400	104,400
Milton	24	236033	9	62,500	104,000
PSEG	63	236033	9	70,800	109,600
Milton	24	236137	9	62,500	106,500

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
PSEG	59	236137	9	66,000	108,000
Milton	24	236180	9	64,800	109,200
PSEG	62	236180	9	63,000	105,800
Milton	22	236429[a,c]	9	66,239	106,601
Milton	22	236429[a,c]	9	69,445	106,060
Milton	23	236429[b,c]	9	76,531	107,150
Milton	23	236431	9	69,697	105,555
Milton	21	236431	9	62,121	106,060
Milton	21	236436	9	63,265	106,133
Milton	23	236436	9	64,796	106,632
Milton	519	236458[a,c]	10	71,143	107,521
Milton	518, 519	236458[a,c]	10	70,326	107,317
Milton	519	236458[b,c]	10	70,732	107,724
Milton	514	236501	10	65,850	109,900
Milton	515	236501	10	67,850	108,350
Milton	515	236501	10	75,400	107,950
PSEG	43	236636[a,c]	9	61,450	103,950
PSEG	43	236636[a,c]	9	61,250	103,200
PSEG	20, 43	236636[a,c]	9	63,325	105,875
PSEG	249	236776	11	73,550	100,650
PSEG	249	236776	11	76,150	101,000
Milton	1363	236776	11	66,650	101,650
PSEG	247	236778	11	65,900	96,450
PSEG	247	236778	11	64,950	96,100
Milton	1363	236778	11	65,250	98,700
Milton	1672	236844	8	62,800	107,050
PSEG	1056	236884	8	73,700	108,350
PSEG	1056	236884	8	73,400	106,950
Milton	16	237124	9	65,300	109,700
PSEG	42	237124	9	80,800	109,100
PSEG	42	237124	9	71,950	109,050
Milton	10	237569	10	79,100	111,750
Milton	12	237569	9	76,000	113,250

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
Milton	10	237569	9	84,200	112,250
Milton	10, 12	237588[a]	9	75,000	108,950
PSEG	31	237588[a]	9	77,800	109,700
Milton	1682	237615	8	68,850	103,900
Milton	1682	237615	8	68,850	103,900
Milton	13	237640	9	72,950	112,250
Milton	10	237640	9	77,050	110,700
PSEG	32	237640	9	81,100	111,200
PSEG	147	237722	11	72,200	102,650
PSEG	147	237722	11	73,500	102,650
PSEG	132	237839	11	70,500	102,500
PSEG	132	237839	11	70,950	102,600
Milton	1718	238100	8	91,150	112,200
Milton	1718	238100	8	86,550	112,800
Milton	504	238190	10	73,400	112,100
Milton	504	238190	10	73,600	112,600
Milton	504	238202	10	77,400	118,550
Milton	504	238202	10	79,600	118,000
PSEG	64	238283	11	70,700	98,700
PSEG	64	238283	11	70,300	98,400
PSEG	63	238288	11	71,650	102,550
PSEG	63	238288	11	72,300	101,900
PSEG	57	238291	11	70,250	99,000
PSEG	57	238291	11	75,150	99,000
PSEG	60	238309	11	70,600	97,050
PSEG	60	238309	11	71,100	96,750
Milton	1785	238721	11	68,500	101,600
Milton	1785	238721	11	71,450	101,600
PSEG	18	238745	11	68,050	101,600
PSEG	18	238745	11	67,650	101,300
PSEG	19	238916	11	67,400	100,300
PSEG	19	238916	11	68,200	101,000
Milton	1186	240118	8	64,300	105,200
Milton	1186	240118	8	67,550	105,200
Milton	1793	240235	9	73,500	108,500

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
Milton	1793	240235	9	74,750	110,100
Milton	1785	240684	9	85,600	117,000
Milton	1785	240684	9	81,450	117,000
Milton	1785	240714	8	74,350	107,700
Milton	1785	240714	8	74,700	109,100
Milton	1192	240731	8	81,800	103,900
PSEG	1783	240731	8	86,200	105,250
Milton	1793	240734	9	78,800	114,650
Milton	1793	240734	9	83,350	115,150
Milton	1785	240736	8	86,200	105,250
Milton	1785	240760	10	70,750	105,300
Milton	1785	240760	10	73,600	105,300
Milton	862	240960	8	65,400	101,300
Milton	862	240960	8	64,100	101,900
Milton	1815	241147	8	76,650	110,400
Milton	1815	241147	8	77,900	110,400
Milton	1477	241204	10	71,100	113,800
Milton	1477	241204	10	76,400	113,800
Milton	1815	241213	9	68,900	106,650
Milton	1815	241213	9	67,850	106,100
Milton	1823	241236	8	68,400	95,400
Milton	1823	241236	8	65,100	95,400
Milton	1823	241238	8	75,300	112,300
Milton	1823	241238	8	77,300	110,400
PSEG	1820	241245	8	67,100	98,000
Milton	1823	241245	8	67,100	98,000
Milton	1819	241250	8	75,000	107,900
Milton	1819	241250	8	69,750	107,900
PSEG	1874	241968	8	76,300	111,850
PSEG	1874	241968	8	75,000	112,500
Milton	1911	243627	8	70,500	105,750
Milton	1911	243627	8	66,650	105,750
PSEG	1917	243908	8	61,000	99,700
PSEG	1917	243908	8	61,000	99,000
Milton	1919	243908	8	70,150	110,400

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
PSEG	1922	243912	8	60,500	97,700
PSEG	1922	243912	8	60,500	99,000
Milton	1923	243912	8	65,800	107,250
Milton	1923	243942	8	65,400	100,000
Milton	1064	244136	8	61,050	99,350
Milton	1570	247569	11	73,050	106,600
Milton	1570	247720	11	77,300	117,100
Milton	1570	247723	11	63,300	104,200
Milton	23	321698	9	67,857	107,653
Milton	21	321698	9	63,776	107,653
Milton	509	321737	10	62,000	103,600
Milton	509	321737	10	64,500	105,650
PSEG	312	321880	11	81,400	120,000
PSEG	312	321880	11	76,700	116,700
PSEG	301	321934	11	62,200	104,500
PSEG	301	321934	11	60,900	105,000
PSEG	305	322047	11	62,200	105,600
PSEG	305	322047	11	60,800	105,600
PSEG	1019	322064[a]	10	64,600	105,800
PSEG	1019	322064[a]	10	65,400	104,450
PSEG	55	322183	9	62,600	107,600
PSEG	55	322183	9	63,000	108,200
PSEG	307	322207	11	60,500	105,900
PSEG	307	322207	11	62,800	106,500
PSEG	56	322309	9	68,200	108,600
PSEG	56	322309	9	68,000	109,400
PSEG	50	322310	9	63,200	103,200
PSEG	50	322310	9	60,400	102,400
Milton	1678	322425	6	70,000	101,150
Milton	1678	322425	6	69,550	101,600
Milton	1634	322692	8	67,550	108,450
Milton	1637	322692	8	65,584	105,194
Milton	1637	322692	8	66,233	105,844
Milton	1637	322705	8	68,831	110,389
Milton	1637	322705	8	69,480	111,038

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
PSEG	291	322764	11	68,831	105,844
Milton	780	322764	11	69,300	100,980
Milton	764	322764	11	68,850	109,100
PSEG	299	322803	11	70,192	108,012
PSEG	299	322803	11	69,871	108,012
Milton	1642	322803	11	69,350	111,600
PSEG	250	322820[a]	11	70,266	107,235
PSEG	250	322820[a]	11	62,180	107,500
Milton	764	322820[b]	11	63,450	108,950
Milton	513	322836	10	72,200	104,350
Milton	515	322836	10	72,800	105,200
Milton	515	322836	10	73,200	105,200
Milton	513	322837	10	70,950	105,650
Milton	515	322837	10	70,800	104,400
Milton	515	322837	10	70,800	104,400
Milton	509	322844	10	67,050	102,850
Milton	509	322844	10	66,950	102,400
Milton	513	322845	10	66,550	99,200
Milton	515	322845	10	66,550	98,000
Milton	515	322845	10	69,350	98,800
PSEG	248	323068	11	79,600	103,600
PSEG	248	323068	11	75,650	103,600
Milton	1363	323068	11	68,550	103,300
PSEG	1057	323151	8	79,500	104,500
PSEG	1057	323151	8	73,400	103,800
Milton	1672	323151	8	63,450	102,550
Milton	242, 746	323632[a]	11	71,300	105,810
Milton	745, 746	323632[a]	11	72,275	107,100
PSEG	33	323827	9	74,000	110,700
PSEG	33	323827	9	82,150	111,750
PSEG	133	324227	11	73,700	105,900
PSEG	133	324227	11	74,350	106,250
PSEG	149	324235	11	68,500	96,100
PSEG	149	324235	11	66,150	96,100

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
PSEG	134	324320	11	67,000	99,650
PSEG	134	324320	11	67,000	99,350
PSEG	137	324324	11	64,700	95,750
PSEG	137	324324	11	65,150	96,700
PSEG	135	324325	11	65,450	96,400
PSEG	135	324325	11	65,900	97,350
PSEG	20	325500	11	67,400	99,350
PSEG	20	325500	11	67,300	99,050
PSEG	17	325501	11	67,650	98,400
PSEG	17	325501	11	67,750	99,050
Milton	1242	325780	6	66,350	104,200
Milton	1186	326413	6	64,100	101,600
Milton	1186	326413	6	63,850	108,400
Milton	1192	326580	8	77,250	109,100
Milton	1192	326580	8	76,600	109,100
Milton	1186	326742	8	63,900	101,900
Milton	1186	326742	8	64,750	103,850
Milton	1219	326810	10	72,000	109,200
Milton	1219	326810	10	71,200	109,200
Milton	1793	327609	8	88,450	122,450
Milton	1793	327609	8	92,300	122,450
Milton	1793	327610	8	90,250	118,200
Milton	1793	327610	8	94,800	118,200
Milton	981	327677	9	77,600	112,800
Milton	981	327677	9	75,500	112,800
Milton	1510	327885	8	65,600	107,150
Milton	1873	327885	8	65,600	107,150
PSEG	1876	329345	8	63,800	100,000
PSEG	1876	329345	8	66,450	101,300
PSEG	1918	330619	8	60,000	100,000
PSEG	1918	330619	8	60,100	100,300
Milton	1919	330619	8	64,100	100,000
Milton	1916	330646	8	61,550	100,000
Milton	1923	330660	8	63,800	103,300
Milton	1570	332081	11	75,000	109,200

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

Table A-1 Rebar Yield Strength Data

Company	Record	Heat	Bar Size	Yield Strength	Tensile Strength
Milton	1570	334582	11	71,300	105,150
Milton	755	336776	11	66,650	101,650
US Steel	1889	03M585	8	60,510	102,780
US Steel	1849	04M326[b]	8	69,620	107,850
PSEG	1848	04M326[d]	8	65,833	104,467
PSEG	1848	04M326[d]	8	65,533	104,733
US Steel	1826	06L821	8	80,510	118,730
PSEG	1850	209C680[a]	8	69,000	113,200
PSEG	1850	209C680[a]	8	69,500	113,300
Bethlehem	1852	209C680[b]	8	76,200	122,500
Bethlehem	1852	209C681[b]	8	66,800	107,800
PSEG	1851	209C681[a]	8	66,450	108,650
PSEG	1851	209C681[a]	8	66,400	108,800

a: average of 2 data from same heat
b: original data in a heat where other data was averaged
c: yield strength data <60ksi in this heat not included
d: average of 3 data from same heat

B

Appendix B: PSEG Nuclear Records of Reinforcing Steel Test Results

PSEG Nuclear records documenting reinforcing bar tests for Salem Nuclear Generating Station, performed during original plant construction (Provided in Appendix B). (Documents provided to MPR by express package from K. Fisher (PSEG Nuclear) to J. Simons (MPR) dated May 23, 2003).

AUG 7 1969

DATE 7-21-69

Taylor-Davis

THEIR ORDER NO.

OUR ORDER NO.

JOB

ROLLED TO MEET SPECIFICATIONS G60

TEST NO.	HEAT NO.	C.	MN.	P.	S.	SIZE	WEIGHT POUNDS PER FT.	NOM. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	BEND TEST
1	23553	.39	1.31	.018	.030	8	2.670	.79	2.00	5.24	2.620	.77	55,000	71,450	82,500	107,150	16		OK
2	23553	.39	1.31	.018	.030	8	2.670	.79	2.00	5.23	2.615	.77	58,000	75,350	83,000	107,800	16		
3	135911	.40	1.32	.025	.036	8	2.670	.79	2.00	5.24	2.625	.77	57,500	74,700	84,500	97,400	17		
4	135911	.40	1.32	.025	.036	8	2.670	.79	2.00	5.25	2.625	.77	56,000	72,750	84,500	97,400	17		OK
5																			
6	322425	.36	1.21	.022	.035	6	1.502	.44	2.00	3.01	1.505	.44	30,800	70,000	44,500	101,150	16		OK
7	322425	.36	1.21	.022	.035	6	1.502	.44	2.00	3.00	1.500	.44	30,600	69,550	44,700	101,600	18		OK
8		.27																	
9	3224[illegible]	.27	.036																
10																			
11	23553	.27	.395																
12	135911	[illegible]	.37																
13																			
14																			
15																			

SIGNED B. [illegible]

THEIR ORDER NO.

OUR ORDER NO.

JOB

SPECIFICATION [illegible] GRADE [illegible]

		C	MN	P	S		WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	ELONG.	BREAK	
1	23715	.38	1.25	.021	.030	#8	2.6[illegible]	.79	2.00	5.26	2.630	.77	53000	68.850	89000	103.900	17		ok
2	23715	.38	1.25	.021	.030	#8	2.6[illegible]	.79	2.01	5.25	2.625	.77	53000	68.850	80,000	103.900	17		ok
3																			
4																			
5																			
6																			
7																			

1636
1091

DATE 3-4-69

CUSTOMER TAYLOR-DAVIS INC

THEIR ORDER NO.

CONTRACTOR

OUR ORDER NO.

ROLLED TO MEET SPECIFICATION GRADE 60

JOB

No.	[illegible]	C	Mn	P	S	[illegible]	Weight Pounds per Ft.	Sec. Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds per Ft.	Sec. Area Sq. In.	Yield Point Machine Reading	Yield Point Lbs. per Sq. In.	Tensile Strength Machine Reading	Tensile Strength Lbs. per Sq. In.	% Elong.	[illegible]
1	322705	40	1.49	019	035	8	2.670	.79	200	5.22	2.610	.77	53,000	68,831	85,000	110,389	15	
2	322705	40	1.49	017	035	8	2.670	.79	200	5.26	2.630	.77	53,500	69,480	85,500	111,038	15	
3	322692	39	1.38	021	034	8	2.670	.79	2.01	5.27	2.622	.77	50,500	65,584	81,000	105,194	17	
4	322692	39	1.38	021	034	8	2.670	.79	2.06	5.26	2.630	.77	51,000	66,233	81,500	105,844	18	
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		

SIGNED [illegible]

87A-322705

[illegible]	[illegible]	161	021	034	67550	102450	[illegible]	[illegible]	[illegible]
[illegible]	35	51	019	042	47400	75750	19	9/20	011
[illegible]	36	58	022	036	42050	[illegible]	16	9/00	016
[illegible]	36	60	019	029	50050	84400	14	9/04	014
[illegible]	34	48	022	036	47100	80200	17	9/00	016
136930	34	60	021	036	49350	81900	19	9/00	014

			222803	2	39	145	0.20	028	69350	71600	9	9586	011
	24		157023	10	39	139	016	032	62200	104900	12	39796	01
	18	(40)	236586	a	352	57	018	038	51550	82450	18	8918	01
	18		236589	a	34	50	0.21	033	48450	77000	22	26251	01
	3-18		236582	a	34	51	0.21	040	50100	74600	24	18262	01
	3-10		[illegible]	a	33	53	0.22	039	69650	79100	20	27313	01

		236844		27	1.30	019	027	62808	102050	24	27393	02
	4-9	323151		38	1.30	021	039	63450	102550	26	27,323	
	(40)	236890		35	52	022	050	48600	76650	17	19118	
		236890		35	52	022	050	48600	76650	17	11 W8	
		137351		34	53	021	033	47850	75550	19	22 323	V
		236356		35	53	019	035	49607	79200	18	5/126	02

8-16A=137351
8-16B=236356
8-16C=236890
8-16D=236884
8-16E=323151

8-104

21

MILTON MANUFACTURING CO. Milton Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE 2-28-72

CUSTOMER TAYLOR - DAVIS - CORP.

THEIR ORDER NO. PS 545-A-B-C-D-581-581A-579

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC.

OUR ORDER NO. 7037F46 THRU 53

ROLLED TO MEET SPECIFICATIONS GRADE 60

JOB PUBLIC SERVICE ELECTRIC & GAS CO.

ITEM	HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH IN TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	BEND TEST
1	330393	.33	1.30	.014	.027	4	.668	.21	2.00	1.32	.660	.19	12.500	65,800	19.900	104,750	17		OK
2	330393	.33	1.30	.014	.027	4	.668	.20	1.99	1.32	.660	.19	12.500	65,800	19.900	104,750	18		
3	330331	.36	1.33	.012	.030	5	1.043	.31	1.99	2.03	1.020	.30	18,900	63,000	29,800	99,350	16		
4	330331	.36	1.33	.012	.030	5	1.043	.31	2.00	2.02	1.010	.30	19.000	63,350	29.900	99,650	15		
5	330450	.36	1.23	.015	.026	5	1.043	.31	1.99	2.04	1.025	.30	19,300	64,350	31,400	104,650	16		
6	330450	.36	1.23	.015	.026	5	1.043	.31	1.99	2.05	1.030	.30	19,900	66,350	31,400	104,650	15		
7	144597	.37	1.30	.013	.034	6	1.502	.44	2.00	2.93	1.465	.43	27,800	64,650	44,400	103,250	17		
8	144597	.37	1.30	.013	.034	6	1.502	.44	2.00	2.94	1.470	.43	27,700	64,400	44,200	102,800	17	8-8-58 B-[illegible]	8-85 A: 240960
9	243923	.37	1.23	.018	.029	7	2.044	.60	2.00	3.93	1.965	.58	40,000	68,950	60,200	103,910	17		
10	243923	.37	1.23	.018	.029	7	2.044	.60	2.00	3.93	1.965	.58	38,900	67,050	60,400	104,150	17		
11	240960	.39	1.40	.012	.025	8	2.670	.79	2.00	5.27	2.635	.78	51,000	65,400	79,000	101,300	15		
12	240960	.39	1.40	.012	.025	8	2.670	.79	2.00	5.27	2.635	.78	50,000	64,100	79,500	101,900	18		
13	144691	.37	1.30	.015	.025	8	2.670	.79	2.00	5.17	2.585	.76	49,000	64,450	78,500	103,300	17		
14	144691	.37	1.30	.015	.025	8	2.670	.79	1.99	5.14	2.583	.76	50,000	65,800	79,000	103,950	16	8-19	3
15	144076	.37	1.39	.012	.032	9	3.400	1.00	2.00	6.72	3.360	.99	69,000	69,700	105,000	106,050	17	8-19	4

[illegible]

8-96

RECEIVED

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST — 01918

IDENTIFICATION	MATERIAL	HEAT NO.	SIZE	DATE
330619 1	[illegible]	330619	8	[illegible]
330619 2	"	"	"	"

TEST RESULTS

[illegible]	IDENTIFICATION	YIELD (NOTE 1) LBS.	YIELD PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION %
[illegible]	330619 1	47,[illegible]00	60,000	79,000	100,[illegible]00	[illegible]
	330619 2	47,500	60,100	7[illegible],200	100,300	[illegible]

NOTE 1 — DIVIDER [illegible] OR DROP OF BEAM

[illegible] B. [illegible]

Remarks: Results meet [illegible] Specification Requirements. [illegible]

MATERIAL ENGINEER

7-44

MILTON MANUFACTURING [illegible]

[illegible]

DATE 8-26-71

CUSTOMER TAYLOR-DAVIS CORP. — CUSTOMER ORDER NO. PS-3621

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC. — OUR ORDER NO. 69307-1

ROLLED TO MEET SPECIFICATIONS GRADE 60 — JOB ARTIFICIAL ISLAND, SALEM COUNTY, N.J.

Item	Heat No.	C	Mn	P	S	Size	Weight Pounds per Ft.	Sec. Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds per Ft.	Sec. Area Sq. In.	Yield Point Machine Reading	Yield Point Lbs. per Sq. In.	Tensile Strength Machine Reading	Tensile Strength Lbs. per Sq. In.	% Elong.	Break	Bend Test
1	242525	39	1.20	.015	.036	7	2.044	.60	2.01	3.99	1.985	58	36,600	63,100	59,[illegible]	102,250	17		OK
2	242525	39	1.20	.015	.036	7	2.044	.60	2.00	3.95	1.975	58	36,800	63,450	59,000	101,700	14	[illegible]	—
3	241966	39	1.39	.016	.039	7	2.044	.60	2.01	3.98	1.980	59	37,400	64,500	60,400	104,150	15		OK
4	241966	39	1.39	.016	.039	7	2.044	.60	2.01	3.97	1.975	58	37,300	64,300	60,200	103,800	17		—
5	327885	38	1.35	.016	.037	8	2.670	.79	2.00	5.23	2.615	77	50,500	65,600	82,500	107,150	12	[illegible]	OK
6	327885	38	1.35	.016	.037	8	2.670	.79	2.00	5.23	2.615	77	50,500	65,600	82,500	107,150	15		—

7-28

Taylor Davis Corp — Mill Order No. PS 442 R
United Engineers & Constructors — Our Order No. 6150 F57
Grade [illegible] — Public Service Electric & Gas Co

Item	Heat No.	C	Mn	P	S	Size	Weight Pounds per Ft.	Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds per Ft.	Area Sq. In.	Yield Point Machine Reading	Yield Point Lbs. per Sq. In.	Tensile Strength Machine Reading	Tensile Strength Lbs. per Sq. In.	% Elong.	Remarks
1	24?,?51	33	1.32	015	039	4	.668	.20	2.00	1.35	.675	.20	13,900	69,500	20,000	[illegible]	17	
2	2?,?51	33	1.32	015	039	4	.668	.20	2.00	1.34	.675	.20	13,900	69,500	18,500	[illegible]	18	
3	32?,?23	38	1.30	017	033	5	1.043	.30	1.98	2.03	1.025	.30	21,100	70,300	32,600	108,700	14	
4	32?,?23	38	1.30	017	033	5	1.043	.30	2.00	2.06	1.030	.30	21,000	70,000	32,500	108,300	16	
5	14?,?07	39	1.37	014	040	7	2.044	.60	2.00	4.03	2.015	.59	41,600	70,500	62,800	106,400	15	
6	141,?07	39	1.37	014	040	7	2.044	.60	2.00	4.01	2.005	.59	41,200	69,800	62,700	106,200	15	
7	241,?04	41	1.44	014	039	10	4.303	1.27	2.0?	8.42	4.189	1.23	87,500	71,100	140,000	113,800	15	
8	241,?04	41	1.44	014	039	10	4.303	1.27	2.00	8.39	4.195	1.23	94,000	76,400	140,000	113,800	12	
9	141,878	37	1.43	015	036	10	4.303	1.27	1.99	8.43	4.236	1.25	90,000	72,000	135,000	108,000	11	outside
10	141,878	37	1.43	015	036	10	4.303	1.27	2.01	8.51	4.239	1.25	88,000	70,400	135,000	108,000	12	outside
11																		
12																		

Bend Test
OK

RECEIVED
FEB 2 1971
TAYLOR DAVIS CORP

7-28

858

0090

01631

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

PROJECT: SALEM NUCLEAR GEN. STA. [illegible] ORDER NO. [illegible]

IDENTIFICATION	MATERIAL	HEAT NO.	[illegible]	[illegible]
235772B-1	A432	235772	[illegible]	[illegible]
235772B-2	A432	235772	[illegible]	[illegible]

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE-1) LBS.	YIELD (NOTE-1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION [illegible]
[illegible]	235772B-1	50,700	64,180	83,600	105,8[illegible]	[illegible]
[illegible]	235772B-2	52,200	66,080	84,000	106,330	[illegible]

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM [illegible]

TESTED BY: Public Service Testing Laboratory [illegible]

Tensile Results Meet A432 Requirements [illegible]

[illegible]
LABORATORY ENGINEER

[illegible]

7-73

[illegible] REINFORCING BARS

[illegible] Cor. Number [illegible]

[illegible], New Jersey

Specification A.S.T.M. — A615-68 [illegible] 60

Project B/L 4085, 4086, 4076, 4073

[illegible]	[illegible]	[illegible]	[illegible]	Chemical analysis C	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		246040 X	11	39	1.38	013	032	63,150	102,550	13		OK
		247220	11	41	1.44	014	027	77,300	117,100	12		ok
		247569 X	11	34	1.14	013	028	73,050	166,600	15		ok
		247723 X	11	37	1.32	013	024	63,300	104,200	12		ok
		334582 X	11	31	1.16	014	030	71,300	105,150	12		OK
		246213 X	11	35	1.02	014	030	71,850	103,300	13		ok
		332061 X	11	32	.97	016	025	75,000	109,200	11		OK
	7-73	247890 X	7	40	1.42	013	026	68,100	108,300	12		OK
		X 332734 X	7	37	1.25	012	032	63,950	96,550	15		OK
		247569 X	7	37	1.31	013	020	63,600	100,850	18		ok
		247852 X	7	50	.83	014	031	65,950	108,300	12		ok
		247827 X	7	38	1.26	015	032	64,750	107,100	12		OK
		248820 X	7	48	.86	014	029	64,650	107,450	13		OK

Sworn and subscribed before me this 14th day of Sept 19 73

[illegible signature]

ROBERT W. BOWER, Notary Public
MILTON, NORTHUMBERLAND CO., PA.
My Commission Expires [illegible] 1975

C.S. Galbraith [illegible]

7-73

" 6-40, 6-41

				C	Mn	P	S	Yield point (lbs. per sq. in.)	Tensile strength (lbs. per sq. in.)			
PS 420	* 6150 F 35	138058	5	.49	.85	.019	.046	66,750	110,000	12	9111	ok
	*	325780	6	.36	1.34	.014	.044	66,350	104,200	15	2,081	
		141763	7	.37	1.32	.014	.037	62,950	100,700	14	1763	
		140935	8	.36	1.32	.015	.045	63,000	103,250	14	14,750	ok
	(40)											
PS 458	6150 F 36	327579	4	.32	.58	.015	.035	58,400	84,750	20	3751	ok
		240976	5	.30	.56	.015	.038	55,650	82,000	20	4143	
		241102	6	.33	.60	.014	.032	51,800	77,050	24	6542	
		137144	7	.23	.69	.018	.028	52,150	73,350	27	4094	
		327699	8	.34	.55	.017	.037	49,550	77,250	14	21,091	
		241605	9	.34	.55	.014	.055	50,500	77,250	21	772	
		141791	11	.36	.59	.015	.040	50,000	81,600	13	3H31	ok

NO USER Test

RECEIVED
JAN 21 1971
TAYLOR-DAVIS CORP.

6-42

Public Service Electric & Gas Co.

No.						Weight Pounds per Ft.		Length of Test Bar	Weight of Test Bar	Weight Pounds per Ft.	Sec. Area Sq. In.	Yield Point Machine Reading	Yield Point	Tensile Strength Machine Reading	Tensile Strength			
[illegible]	35	1.30	.015	.039	5	1.043	.31	2.00	2.04	1.020	.30	22,300	71,300	33,400	111,300	14		[illegible]
[illegible]	35	1.30	.015	.037	5	1.043	.31	2.01	2.05	1.020	.30	21,500	71,700	33,600	112,000	13		
241865	36	1.28	.012	.038	6	1.502	.44	2.00	2.97	1.485	.44	28,200	64,100	43,800	99,500	17		
241865	36	1.28	.012	.038	6	1.502	.44	2.00	2.97	1.485	.44	28,300	64,300	44,000	100,000	17		
241250	38	1.28	.014	.035	8	2.670	.77	2.01	5.25	2.612	.77	—	—	—	—	—		
241250	38	1.23	.014	.033	8	2.670	.77	2.03	5.31	2.616	.77	—	—	—	—	—		
327677	43	1.34	.014	.036	9	3.400	1.00	1.99	6.64	3.337	.98	76,000	77,600	110,500	112,800	13		9
327677	40	1.34	.014	.036	9	3.400	1.00	2.03	6.78	3.340	.98	74,000	75,500	110,500	112,800	14		
326910	37	1.33	.016	.032	10	4.303	1.27	2.02	8.61	4.262	1.25	90,000	72,0[illegible]	136,500	109,200	11	outside	
326910	37	1.33	.016	.032	10	4.303	1.27	2.00	8.50	4.250	1.25	89,000	71,200	136,500	109,200	12		
[illegible]	32	1.04	.014	.030	11	5.313	1.56	1.98	10.48	5.293	1.56	103,000	66,000	151,000	96,800	16		
[illegible]	32	1.04	.014	.030	11	5.313	1.56	1.97	10.34	5.296	1.56	101,500	65,100	150,500	[illegible]	16		
[illegible]	32	1.03	.012	.032	11	5.313	1.56	1.99	10.42	5.236	1.54	101,000	65,600	150,000	97,400	18		
[illegible]	[illegible]	[illegible]	.012	[illegible]	11	5.313	1.56	2.00	10.47	5.235	1.54	100,500	65,300	150,000	97,400	14		

[illegible]		28	1.25	.020	.025	57250	106750	1/2	1.25
250778		25	1.03	.027	.017	66250	114650	1/2	31.70
250780		24	1.05	.020	.035	65250	91702	3/4	[illegible]
252068		27.5	1.02	.021	.034	68500	103800	1/2	31.90

					Analysis								
			222803	21	39	1.16	0.20	0.28	69350	71600	19	9386	.011
	24		137023	11	39	1.38	0.16	0.32	62200	104900	12	39790	.02
	12	(40)	236596	8	35	.57	0.19	0.38	51550	82450	18	8918	.01
	12		236599	8	34	.50	0.21	0.33	48450	77000	22	26759	.01
	3-18		236582	8	34	.51	0.21	0.40	50100	78600	24	18262	.01
	3-17		[illegible]	8	35	.53	0.22	0.39	69650	79100	20	27315	.02

				ANALYSIS								
		236844		2.7	1.30	019	027	62800	107050	23	27,393	02
	4-9	323151		38	1.30	021	039	63450	102550	26	27,323	1
	(40)	236890		35	52	022	050	48600	76650	17	19,118	
		236890	H	35	52	022	050	48600	76650	17	11,658	
		137351		34	53	021	033	47850	75550	19	22,323	V
		236356		35	53	.019	035	49600	79200	18	5126	02

8-16A: 137351
8-16B: 236356
8-16C: 236890
8-16D: 336884
8-16E: 323151

[illegible] CO. [illegible]

AUG 7 1969   DATE 7-29-69

SOLD TO: TAYLR DAVIS   THEIR ORDER NO.

CONTRACT: OUR ORDER NO.

ROLLED TO MEET SPECIFICATION: C 60   JOB

No.	Heat No.	C	MN	P	S	SIZE	WEIGHT POUNDS PER FT.	AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	BEND TEST
1	235532	39	1.31	018	030	8	2.670	.79	2.00	5.24	2.620	.77	55,000	71,450	82,500	107,150	16		OK
2	235532	39	1.31	018	030	8	2.670	.79	2.00	5.23	2.615	.77	58,000	75,350	83,000	107,700	16		
3	135911	40	1.32	025	036	8	2.670	.79	2.00	5.24	2.62[illegible]	.77	57,500	74,700	84,500	97,400	17		
4	135911	40	1.32	025	036	8	2.670	.79	2.00	5.25	2.625	.77	56,000	72,750	84,500	97,400	17		OK
5																			
6	322425	36	1.21	022	035	6	1.502	.44	2.00	3.01	1.505	.44	30,800	70,000	44,500	101,150	16		OK
7	322425	36	1.21	022	035	6	1.502	.44	2.00	3.00	1.500	.44	30,600	69,550	44,700	101,600	18		OK
8		[illegible]																	
9	322425	.27	.036																
10																			
11	235532	.27	.393																
12	135911	[illegible]																	

[illegible]

[illegible] MANUFACTURING [illegible]

[illegible]

Taylor Bank [illegible]

THEIR ORDER NO. [illegible]

OUR ORDER NO. [illegible]

[illegible] SPECIFICATION: GRADE [illegible]

JOB [illegible]

	[illegible]	C	MN	[illegible]	[illegible]	[illegible]	WEIGHT POUNDS PER FT.	NET AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT: MACHINE READING	YIELD POINT: LBS. PER SQ. IN.	TENSILE STRENGTH: MACHINE READING	TENSILE STRENGTH: LBS. PER SQ. IN.	[illegible]	[illegible]	[illegible]
1	237615	.38	1.25	.121	.030	#8	2.67	.79	2.00	5.26	2.630	.77	53000	68,850	89000	103,900	17	[illegible]	ok
2	237615	.38	1.25	.021	.030	#8	2.6[illegible]	.79	2.00	5.25	2.625	.77	53000	68,850	80,100	103,900	17	[illegible]	ok
3																			
4																			
5																			
6																			
7																			

[illegible]

[illegible] Taylor-Davis [illegible] 3-4-69

[illegible] Grade 60

[illegible]	[illegible]	C	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Length of test bar ft.	Weight of test bar lbs.	Weight pounds per ft.	[illegible] area sq. in.	Yield point: machine reading	Yield point: lbs. per sq. in.	Tensile strength: machine reading	Tensile strength: lbs. per sq. in.	[illegible] elong.	[illegible]
1	322705	40	1.49	019	035	8	2.670	.79	2.00	5.22	2.610	.77	53,000	68,831	85,000	110,389	15	
2	322705	40	1.49	019	035	8	2.670	.79	2.00	5.26	2.630	.77	53,500	69,480	85,500	111,038	15	
3	322692	39	1.38	021	034	8	2.670	.79	2.01	5.27	2.622	.77	50,500	65,584	81,000	105,194	17	
4	322692	39	1.38	021	034	8	2.670	.79	2.00	5.26	2.630	.77	51,000	66,233	81,500	105,844	18	
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		

Signed [illegible]

87A-322705

[illegible]	[illegible]	3[illegible]	18	.021	.024	[illegible]	[illegible]	14	[illegible]	[illegible]
[illegible]	[illegible]	36	61	.019	.042	47400	79700	19	9/20	[illegible]
[illegible]	11	36	58	.022	.036	47050	[illegible]	16	9/00	.016
[illegible]	11	36	60	.019	.029	50600	84400	14	9/00	.014
[illegible]	11	34	48	.020	.036	47100	80200	17	9/00	.016
[illegible]	11	34	60	.021	.036	49350	81900	19	9/00	.016

... Test of Reinforcing Bars

TO [illegible] DAVIS | DATE 7-2[illegible]

THEIR ORDER NO.

CONTRACTOR

OUR ORDER NO.

ROLLED TO MEET SPECIFICATIONS G 60

JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	
1378[illegible]	.42	1.37	.019	.027	9	3.400	1.00	2.00	6.62	3.310	.97	70,500	72,700	105,000	107.[illegible]	13		
1378[illegible]	.44	1.37	.021	.021	9	3.400	1.00	2.00	6.66	3.333	[illegible]	72,500	74,700	105,000	107,150	13		
1378[illegible]	.40	1.42	.018	.033	9	3.400	1.00	2.00	6.6[illegible]	[illegible]	[illegible]	74,500	76,[illegible]	108,[illegible]	110.2[illegible]	13		
13775	.42	1.42	.015	[illegible]	9	[illegible]	1.00	2.00	[illegible]	3.33[illegible]	[illegible]	77,000	77,800	[illegible]	[illegible]	14		
1377[illegible]	.42	1.42	.017	[illegible]	9	3.400	1.00	[illegible]	6.7[illegible]	3.3[illegible]	[illegible]	79,000	80,[illegible]	111,000	113.[illegible]	[illegible]	[illegible]	
1373[illegible]	.42	1.47	.017	.036	9	3.400	1.00	[illegible]	6.60	[illegible]	[illegible]	82,000	83,650	111,[illegible]	113.[illegible]	13		
1377[illegible]	.42	1.37	.022	.037	9	3.400	1.00	2.00	6.6[illegible]	3.340	.98	74,500	76,000	10[illegible]	10[illegible].55	16		
1330[illegible]	.40	1.37	[illegible]	.037	9	3.400	1.00	2.00	6.67	3.335	.98	85,500	87,250	100,500	102.550	16		
1331[illegible]	.37	1.40	.016	.037	9	3.400	1.00	2.00	6.57	3.275	.97	72,500	74,750	103,500	105,650	13		
37[illegible]11	.37	1.40	.016	.037	9	3.400	1.00	2.00	6.61	3.305	.97	70,500	72,700	103,000	106.200	15		
3751[illegible]	.40	1.38	.018	.032	9	3.400	1.00	2.00	6.66	3.333	[illegible]	77,500	79,100	109,500	111,750	15		
37567	.40	1.38	.017	.032	9	3.400	1.00	2.00	6.66	3.333	[illegible]	82,500	84,200	110,500	112.25	11	J.G. Shear	
37522	.41	1.41	.019	.035	9	3.400	1.00	[illegible]	6.6[illegible]	3.34[illegible]	[illegible]	76,000	77,550	107,500	109.7[illegible]	12		
37522	.40	1.41	.019	.035	9	3.400	1.00	[illegible]	6.6[illegible]	3.345	[illegible]	76,500	78,050	107,500	109.700	12		
37640	.42	1.32	.019	.034	9	3.400	1.00	2.00	6.66	3.333	.98	75,500	77,050	108,500	110.7[illegible]	16		[illegible]

SIGNED [illegible] SIGNED B. E. Kurt

…t of Reinforcing Bars

DATE 11-19-69

…STOMER TAYLOR - DAVIS

THEIR ORDER NO.

…NTRACTOR

OUR ORDER NO.

…LLED TO MEET SPECIFICATIONS GRADE 60

JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR, LBS	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	BEND TEST
13856	40	1.34	.016	.043	9	3.400	1.00	1.99	6.57	3.302	.97	69,500	71,650	108,500	111,850	10		ok
13856	40	1.34	.016	.043	9	3.400	1.00	1.99	6.59	3.312	.97	68,000	70,100	108,500	111,850	10		ok

Witnessed By H. Obermann
Public Service Electric and Gas Co.

SIGNED B. E. Kett[illegible]

Report of Chemical and Physical Tests of REINFORCING BARS July 31, 1969

Customer: Taylor-Davis Corporation

Car Number

Shipped to: Wilmington, Delaware

Specification: GRADE 60

Project

Customer's Order No. and Date	Mill Order Number	Heat Number	Size Number	Analysis C	Analysis Mn	Analysis P	Analysis S	Yield Pt. Lbs. per Sq. In.	Ten. Strength Lbs. per Sq. In.	Elong. in 8" %	Wt.	Bend Test
		232569	9	40	1.38	018	032	76000	113250	12	[illegible]	OK
		138010	9	40	1.37	022	039	65150	102550	12	35[illegible]	OK
		237588	9	40	1.41	019	035	77950	110200	15	[illegible]	OK

We certify that the above figures are correct as contained in the company records. [signature]

This material made in USA by basic electric furnace process to meet specification ASTM A15-64 (or A432-64) and ASTM A305-64.

[illegible] CHEMICAL and PHYSICAL Tests of REINFORCING BARS July 30, 1969

Customer Taylor-Davis Corporation

Shipped to Wilmington, Delaware

Specification GRADE 60

Car Number

Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	[illegible]SED	BEND TEST
		137958	9	.40	1.42	.017	.036	74500	113750	13	[illegible]	OK
		237640	9	.40	1.32	.019	.034	72950	112250	12	14.745	OK
		137951	9	.40	1.42	.018	.033	72200	109100	13	15.745	OK

We certify that the above figures are correct as contained in the company records. [signature]

This material made in USA by basic electric furnace process to meet specification ASTM A15-64 (or A432-64) and ASTM A305-64.

8-52

Shipping Notice

Car Number

10723170 061823 10 09 2018 #11 Bar 80510 118730 12.0 11/1/62 Passed

RECEIVED
NOV 16 1970
DAVIS CORP

#8-52

8-45A

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
ELECTRIC DEPARTMENT
TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST 01848

PROJECT: SALEM NUCLEAR GEN. STA. | ORDER NO. | DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
04M326-1	A432	04M326	8	.0786 [illegible]
04M326-2	[illegible]	[illegible]	[illegible]	[illegible]
04M326-3	[illegible]	[illegible]	[illegible]	[illegible]
04M326-4	[illegible]	[illegible]	[illegible]	[illegible]
04M326-5	[illegible]	[illegible]	[illegible]	[illegible]
04M326-6	[illegible]	[illegible]	[illegible]	[illegible]

REPORT OF TEST RESULTS

DATE	IDENTIF. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) LBS. PER SQ. IN.	ULTIMATE LBS.	ULTIMATE LBS. PER SQ. IN.	ELONGATION IN. 2 IN.
8-1-71	04M326-1	54000	68,400	84,000	106,300	16.0
	04M326-2	[illegible]	65,300	80,200	105,500	[illegible]
	04M326-3	52,000	65,800	83,400	105,600	16.[illegible]
	04M326-4	49,400	62,500	80,000	101,300	[illegible]
	04M326-5	53,700	67,300	84,800	107,300	[illegible]
	04M326-6	52,800	66,800	83,400	105,600	[illegible]

NOTE 1: DIVIDER OVER 1 INCH OF DROP OF BEAM

TESTED BY R. [illegible]

Remarks: Results Meet A432 Specification Requirements

Testing Engineer

[illegible]

L-308 REV 6/60

527640	7	1.30	.817	.042	8	2.67	.79	2.00	5.22	2.60	.77	70.500	96.250	91.000	118.200	12	[illegible]
527642	7	1.35	.817	.075	8	2.67	.79	2.00	5.25	2.65	.77	73.000	94.800	91.000	118.000	12	
527649	7	1.25	.815	.037	8	2.670	.79	1.99	5.28	2.65	.78	69.000	88.450	95.510	122.450	13	
527649	7	1.25	.815	.037	7	2.670	.79	2.06	5.21	2.645	.78	72.000	92.300	95.500	122.450	12	
870285	5	1.45	.674	.047	9	3.400	1.00	1.97	6.71	3.39	1.00	73.500	73.500	108.500	108.50	14	
240235	5	1.45	.624	.047	9	3.400	1.00	1.00	6.74	3.390	.99	74.000	74.750	109.000	111.170	14	
241235	2	1.47	.617	.52	9	3.40	1.0	1.99	6.71	3.372	.99	78.000	78.800	115.500	114.650	14	
[illegible]	1	1.47	.617	.52	9	3.40	1.0	1.98	6.68	3.374	.99	82.500	83.350	114.100	118.150	13	
[illegible]		1.37	.734	.031	11	4.225	1.2	1.96	8.23	4.199	1.24	89.000	71.750	130.500	115.350	13	
[illegible]		1.37	.714	.31	12	4.225	1.2	2.00	8.39	4.195	1.25	90.000	73.450	120.500	[illegible]	14	[illegible]

RECEIVED

Customer: [illegible]

Specification: GRADE 60

THEIR ORDER NO.

OUR ORDER NO.

JOB

	Heat No.	C	Mn	[illegible]	[illegible]	[illegible]	Weight pounds per ft.	Sec. area sq. in.	Length of test bar ft.	Weight of test bar lbs.	Weight pounds per ft.	Sec. area sq. in.	Yield point: Machine reading	Yield point: Lbs. per sq. in.	Tensile strength: Machine reading	Tensile strength: Lbs. per sq. in.	% Elong.
1	140935	36	1.32	015	045	8	2.670	.79	2.00	5.34	2.670	.79	56,500	71,500	84,000	106,300	17
2	140935	36	1.32	015	045	8	2.670	.79	2.01	5.36	2.66	.78	55,500	71,200	84,000	107,700	17
3	140913	38	1.35	013	035	8	2.670	.79	2.00	5.28	2.640	.78	53500	68,600	81,500	104,500	18
4	140913	38	1.35	013	035	8	2.670	.79	2.00	5.30	2.650	.78	53,500	68,600	81,500	104,500	17
5	140909	37	1.39	014	033	8	2.670	.79	2.01	5.27	2.625	.78	56,500	72,400	83,000	106,400	14
6	140909	37	1.39	014	033	8	2.670	.79	1.99	5.23	2.628	.77	55,000	71,400	83,000	107,800	18
7																	
8																	
9																	
10																	
11																	
12																	

8-5A 140935
8-5B 140913
8-5C 140909

[illegible] LAYR-DAVIS CORP. DATE 1-28-71

THEIR ORDER NO. PS-402M-PS-422N

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC. OUR ORDER NO. 6150 FS4-55

ROLLED TO [illegible] SPECIFICATIONS GRADE 60 JOB PUBLIC SERVICE GAS & ELECTRIC CO.

HEAT NO.	C	Mn	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR, LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	[illegible]
141953	37	1.26	014	032	8	2.670	.79	2.00	5.26	2.630	.77	—	—	—	—	—	—	—
141953	37	1.26	014	032	8	2.670	.79	2.00	5.25	2.625	.77	—	—	—	—	—	—	—
141953	38	1.24	014	033	8	2.670	.79	2.00	5.23	2.615	.77	—	—	—	—	—	—	—
141953	38	1.24	014	033	8	2.670	.79	2.00	5.22	2.610	.77	—	—	—	—	—	—	—
241245	38	1.27	015	034	8	2.670	.79	2.00	5.28	2.600	.76	51,000	67,100	74,500	98,000	16		[illegible]
241245	38	1.27	015	034	8	2.670	.79	2.00	5.19	2.595	.76	51,000	67,100	74,500	98,000	15		[illegible]
241251	38	1.28	014	033	8	2.670	.79	2.01	5.25	2.612	.77	—	—	—	—	—	—	—
241251	38	1.28	014	033	8	2.670	.79	2.03	5.31	2.616	.77	—	—	—	—	—	—	—
241238	38	1.43	014	032	8	2.670	.79	2.00	5.22	2.610	.77	58,000	75,300	86,500	112,300	14	8-58A-241238	8-58A-241236
241238	38	1.43	014	032	8	2.670	.79	2.00	5.22	2.610	.77	59,500	77,300	85,000	110,400	13		
241236	39	1.27	015	027	8	2.670	.79	2.00	5.19	2.595	.76	52,000	68,400	72,500	95,400	17		
241236	39	1.27	015	027	8	2.670	.79	2.00	5.19	2.595	.76	49,500	65,100	72,500	95,400	15		

Rec'd for Cert [illegible]

2-17-71 [illegible] 8/48 A58.57

[illegible]

[illegible] 7-23-71

8591

01825

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

RE INFORCEMENT BAR TEST

[illegible] SALEM NUCLEAR GEN. STA. [illegible] ORDER NO. [illegible]

IDENTIFICATION NO.	MATERIAL	[illegible]	[illegible]	[illegible]
061821-1	A432	061821	6	[illegible]
061821-2	[illegible]	[illegible]	[illegible]	[illegible]
061821-3	[illegible]	[illegible]	[illegible]	[illegible]
061821-4	[illegible]	[illegible]	[illegible]	[illegible]

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES
11/12	061821-1	53,000	67,100	85,600	108,400	13
	061821-2	51,000	64,600	85,000	107,600	12
	061821-3	52,000	66,400	86,000	108,900	10
	061821-4	51,600	65,300	85,600	108,600	12

NOTE 1: [illegible] DIVIDE OVER 8 INCHES OR [illegible]

TESTED BY A. C. Lowe

REMARKS: MEETS A432 SPECIFICATION [illegible]

MATERIALS DIVISION [illegible]

[illegible] REV. [illegible]

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

01820

REINFORCEMENT BAR TEST

SALEM NUCLEAR GEN STA... ORDER NO

IDENTIFICATION NO.	MATERIAL	[illegible]	[illegible]	[illegible]
241243-1	A432	241243	18	[illegible]
241243-2	A432	241243	18	[illegible]

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) [illegible]	ULTIMATE LBS.	ULTIMATE [illegible]	ELONGATION %
11/23/71	241243-1	51,000	67,100	74,500	98,000	14
	241243-2	51,000	67,100	74,500	98,000	

NOTE 1 - DIVIDER OVER 8 INCHES OR DROP OF BEAM

[illegible] Milton Manufacturing Co. Witnessed By R.C. [illegible]

Remarks: Results Meet A432 Specification R[illegible]

[illegible signature]

[illegible]

OUR ORDER NO.

[illegible]	C	Mn	P	S	[illegible]	Weight Pounds per ft.	Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds per Ft.	Area Sq. In.	Yield Point Machine Reading	Yield Point [illegible]	Tensile Strength Machine Reading	Tensile Strength [illegible]	[illegible]	[illegible]
2401?51	.40	1.26	.015	.041	8	2.670	.79	2.00	5.20	2.600	.76	65,500	86,200	80,000	105,250	15	[illegible]
240714	.42	1.30	.016	.040	8	2.670	.79	2.00	5.28	2.640	.78	58,000	74,350	84,000	107,700	16	[illegible]
240714	.42	1.30	.016	.040	8	2.670	.79	1.99	5.24	2.633	.77	57,500	74,700	84,000	109,100	13	[illegible]
139736	.40	1.48	.016	.029	9	3.400	1.00	2.00	6.74	3.370	.99	71,000	71,700	108,000	109,100	13	[illegible]
139726	.40	1.48	.016	.029	9	3.400	1.00	2.00	6.74	3.370	.99	70,000	70,700	108,000	109,100	13	[illegible]
2406?4	.39	1.36	.014	.037	9	3.400	1.00	2.01	6.63	3.299	.97	83,000	85,600	113,500	117,000	12	[illegible]
2406??	.39	1.36	.014	.031	9	3.400	1.00	1.99	6.58	3.307	.97	79,000	81,450	113,500	117,000	11	[illegible]
141?53	.39	1.36	.014	.037	9	3.400	1.00	2.01	6.63	3.299	.97	71,000	73,200	102,500	105,650	16	[illegible]
141?73	.39	1.36	.014	.037	9	3.400	1.00	2.02	6.68	3.307	.97	71,000	73,200	103,000	106,200	17	[illegible]
[illegible]	.39	1.39	.014	.029	10	4.303	1.27	1.95	8.14	4.174	1.23	87,000	70,750	129,500	105,300	16	[illegible]
2407??	.39	1.39	.014	.029	10	4.303	1.27	1.98	8.27	4.177	1.23	90,500	73,600	129,500	105,300	17	[illegible]
2588??	.33	.99	.014	.028	11	5.313	1.56	2.00	10.50	5.250	1.54	105,500	68,500	157,500	102,600	12	[illegible]
[illegible]	.33	.99	.014	.028	11	5.313	1.56	1.99	10.48	5.31	1.54	110,000	71,450	157,500	102,600	12	[illegible]

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
[illegible]
TESTING LABORATORY REPORT

01783

REINFORCEMENT BAR TEST

[illegible]	[illegible] NUCLEAR GEN. ST[illegible]	ORDER NO. [illegible]	

IDENTIFICATION NO.	MATERIAL	HEAT NO.	[illegible]	[illegible]
[illegible]-1	A-432	[illegible]6731	[illegible]	[illegible]
[illegible]-2	[illegible]	[illegible]	[illegible]	[illegible]

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION [illegible]
10/15/[illegible]	[illegible]6731-1	63,000	81,800	80,[illegible]	[illegible]3,900	[illegible]
	[illegible]6731-2	65,[illegible]00	86,100	80,[illegible]	[illegible]	[illegible]

NOTE 1: [illegible] DIVIDER OVER 8 INCHES OR DROP OF BEAM

[illegible] MILTON MANUFACTURING CO. WITNESSED BY [illegible]

REMARKS: RESULTS MEET A432 SPECIFICATION [illegible]

[illegible]

[illegible]

L-106 REV [illegible]

MILTON MANUFACTURING CO. - Milton, Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE 11-19-69

CUSTOMER TAYLOR-DAVIS

THEIR ORDER NO.

CONTRACTOR

OUR ORDER NO.

ROLLED TO MEET SPECIFICATION GRADE 60

JOB

	HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	NOM. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	BEND TEST
1	238100	37	1.38	012	032	8	2.670	.79	1.76	4.65	2.642	.78	71,000	91,150	87,500	112,200	13		OK
2	238100	37	1.38	012	032	8	2.670	.79	1.75	4.64	2.651	.78	67,500	86,550	88,000	112,800	14		
3	138958	37	1.34	012	030	8	2.670	.79	1.76	4.59	2.608	.77	57,000	74,050	83,000	107,800	14		
4	138958	37	1.34	012	030	8	2.670	.79	1.76	4.62	2.625	.77	58,500	75,950	83,500	108,450	15		OK

OK

TAYLOR - DAVIS

DATE 10-1-62

THEIR ORDER NO.

OUR ORDER NO.

SPECIFICATION GRADE 60

		C	MN	P	S		NOMINAL WEIGHT POUNDS PER FT.	SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	
1	13862	.38	1.26	.018	.030	8	2.670	.79	2.00	5.26	2.630	.77	48,500	63,000	78,000	101,300	13		[illegible]
2	13826	.38	1.26	.018	.030	8	2.670	.79	2.00	5.23	2.615	.77	51,000	66,250	78,500	101,950	13		[illegible]
3																			
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PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

01875

[illegible] NUCLEAR GEN. STA. [illegible]		ORDER NO. [illegible]		DATE [illegible]
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	[illegible]
142548-1	A432	142548	8	0.76 [illegible]
142548-2	"	"	"	0.76 [illegible]

TENSION TEST RESULTS

DATE OF TEST	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
8/21/71	142548-1	49,500	65,150	78,500	103,300	16
	142548-2	50,500	66,450	78,500	103,300	16

NOTE 1 – DIVIDER [illegible] INCHES OR DROP OF BEAM

[illegible] Manufacturing Co. | Witnessed by: [illegible]

Remarks: Results meet A432 specification requirements.

[illegible]

L-388 REV. 6/60

8-03A

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

01874

REINFORCEMENT BAR TEST

[illegible] NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
241968-1	A 432	241968	8	0.76 sq. in.
241968-2	"	"	"	0.76 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION [illegible] INCHES
6/26/[illegible]	241968-1	58,000	76,300	85,000	111,850	14
	241968-2	57,000	75,00	85,500	112,500	13

NOTE 1. DIVIDERS OVER 8 INCHES OR DROP OF BEAM

[illegible] Milton Manufacturing Co. | Witnessed by R. [illegible]

Remarks: Results meet A-432 specification requirements

[illegible signature]

L-308 REV. [illegible]

8-47

DATE 1-26-71

CUSTOMER TAYLOR-DAVIS CORP. THEIR ORDER NO. BS-402K

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC. OUR ORDER NO. 6150F52

ROLLED TO MEET SPECIFICATIONS GRADE 60 JOB PUBLIC SERVICE ELECTRIC & GAS

	HEAT NO.	C	MN	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	BEND TEST
	141953	38	1.24	014	033	8	2.670	.79	2.00	5.24	2.620	.77	59,500	77,250	82,500	107,150	13		OK
	141953	38	1.24	014	033	8	2.670	.79	2.00	5.22	2.610	.77	55000	71,450	82,500	107,150	13		
	141955	37	1.26	014	032	8	2.670	.79	2.01	5.23	2.602	.77	51,000	66,250	78,500	101,950	13		
	141954	37	1.26	014	032	8	2.670	.79	1.99	5.20	2.613	.77	50,000	64,950	78,000	101,300	15		
	241251	38	1.28	014	033	8	2.670	.79	1.99	5.17	2.598	.76	57,000	75,000	82,000	107,900	15		
	241250	38	1.28	014	033	8	2.670	.79	1.96	5.09	2.597	.76	53,000	69,750	82,000	107,900	14		OK

Recorded for Cert.

1-28-71

9-57A 141959
8-57B 141954
8-57C 241250

Lot # 8-45 46 47

13 [illegible]

#8-44

[illegible]
Report of Chemical and Physical Test of Reinforcing Bars

DATE 1-18-71

CUSTOMER TAYLOR-DAVIS CORP. THEIR ORDER NO. PS 454-466

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC. OUR ORDER NO. 66415 11-12

ROLLED TO MILL SPECIFICATIONS G 60 JOB PUBLIC SERVICE ELECTRIC & GAS CO.

Heat No.	C	Mn	P	S	Size	Weight Pounds Per Ft.	Nom. Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds Per Ft.	Sec. Area Sq. In.	Yield Point Machine Reading	Yield Point Lbs. Per Sq. In.	Tensile Strength Machine Reading	Tensile Strength Lbs. Per Sq. In.	% Elong.	Break	Bend Test
241147	38	1.38	016	041	8	2.670	.79	1.99	5.22	2.623	.77	59000	76,650	85,000	110,400	17		OK
241147	38	1.38	016	041	8	2.670	.79	2.00	5.23	2.615	.77	60,000	77,900	85,000	110,400	12	outside	
141902	40	1.41	014	033	9	3.400	1.00	2.00	6.63	3.315	.98	70,500	71,950	106,000	108,150	12		
141902	40	1.41	014	033	9	3.400	1.00	1.99	6.58	3.307	.97	68,000	70,100	105,500	108,750	13		
141901	39	1.41	013	041	9	3.400	1.00	2.00	6.70	3.350	.99	70,000	70,700	104,500	105,550	12		
141901	39	1.41	013	041	9	3.400	1.00	2.00	6.69	3.345	.98	69,500	70,900	104,000	106,100	14		
241213	39	1.37	014	040	9	3.400	1.00	1.99	6.62	3.327	.98	67,500	68,900	104,500	106,650	12		
241213	39	1.37	014	040	9	3.400	1.00	2.00	6.64	3.320	.98	66,500	67,850	104,000	106,100	12		OK

Recorded for Cert
1-27-71
R. Baldwin

[illegible] #8-44

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST 01851

Identification No.	Material	[illegible]	[illegible]	[illegible]
2096661 1	A432	2096661	8	[illegible]
2096661 2	"	"	"	
2096661 3	"	"	"	
2096661 4	"	"	"	

TENSION TEST RESULTS

Identification No.	Yield (Note 1) Lbs.	Yield P.S.I.	Ultimate Lbs.	Ultimate P.S.I.	Elongation
2096661-1	54,200	68,600	88,400	112,200	15.2
2096661-2	50,800	64,300	83,000	105,100	11.1
2096661-3	54,000	[illegible]	[illegible]	[illegible]	[illegible]
2096661-4	50,900	64,400	83,300	105,400	11.1

Remarks: Results Meet A432 Specification Requirements

3-199

[illegible] COMPANY

CHEMICAL and PHYSICAL [illegible] March 21, 1972

Taylor-Davis Corporation

Wilmington, Delaware

[illegible] 60 Project S/L 2065

[illegible]	[illegible]	[illegible]	[illegible]	Analysis				[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
		243942	8	36	1.32	014	030	65400	100000	18	8010	OK
		243912	8	36	1.21	014	027	65800	107250	13	8010	
		330660	8	35	1.38	017	032	63800	103300	14	8010	
		144692	8	38	1.28	017	024	63150	104600	13	[illegible]	OK

RECEIVED

TAYLOR-DAVIS CORP.

[illegible] Corporation

Wilmington, Delaware

[illegible] 60

Period [illegible] 1972 1973

				ANALYSIS								
		144691	8	37	130	015	025	63500	105750	14	24030	012
		144692	8	37	138	017	024	63150	104600	13	8010	
		144724	8	37	140	015	030	62250	105200	14	8010	
		144042	9	40	136	014	032	67350	107150	12	10200	
	H 144032	144032	9	41	145	014	036	73750	115650	14	10200	
		144041	9	41	135	015	048	79000	116850	13	10200	
		144043	9	40	138	015	036	65300	107650	13	10200	012

RECEIVED

S CORP.

[illegible] 1

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

[illegible]

TESTING LABORATORY REPORT

01922

REINFORCEMENT BAR TEST

[illegible]	SALEM NUCLEAR GEN. STA.	[illegible]	[illegible]

IDENTIFICATION NO.	MATERIAL	HEAT NO.	[illegible]	[illegible]
443912-1	A-432	443912	B	[illegible]
443912-2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) P.S.I.	ULTIMATE LBS.	ULTIMATE P.S.I.	ELONGATION
[illegible]	443912-1	47,800	60,500	77,200	97,700	[illegible]
	443912-2	47,800	60,500	78,400	99,000	[illegible]

NOTE 1 - YIELD [illegible] INCHES OR DROP OF BEAM

[illegible] A. C. Lawe

REMARKS: RESULTS MEET A-432 SPECIFICATION REQUIREMENTS

[illegible]

[illegible]

[illegible]

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

01876

[illegible] IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
329345-1	A432	329345	8	0.762 in
329345-2	"	"	8	0.762 in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
6/26/71	329345-1	48500	63,800	76,000	100,000	17
	329345-2	50,500	66,450	77,000	101,300	14

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

Milton Manufacturing Co. Witnessed by R. [illegible]

Remarks: Results meet A432 specification requirements.

L-388 REV. 6/55

8-73D

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

01822

PROJECT SALEM NUCLEAR GEN. STA. | ORDER NO. [illegible] | DATE [illegible]

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
143440-1	A432	143440	8	.0576 sq. in
143440-2	"	"	"	.0774 sq. in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
8/26/71	143440-1	49,000	64,450	79,000	103,950	13
	143440-2	49,000	63,650	79,500	103,250	14

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | Witnessed by R. [illegible]

Remarks: Results meet A432 Specification requirements.

[illegible]

MATERIALS ENGINEER

[illegible]

L-308 REV. 6/68

8-8"

8-73

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST 61850

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2096680-1	A432	2096680	6	[illegible]
2096680-2	"	"	"	[illegible]
2096680-3	"	"	"	[illegible]
2096680-4	"	"	"	[illegible]

TENSION [illegible]

[illegible]	[illegible]	YIELD (NOTE 1)		[illegible]	ULTIMATE [illegible]	ELONGATION
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	2096680-1	54200	68600	89100	115400	[illegible]
	2096680-2	54800	69400	89700	113900	[illegible]
	2096680-3	54800	69400	89300	113000	[illegible]
	2096680-4	55000	69600	89600	113700	[illegible]

R. [illegible]

Remarks: Results Meet A432 Specification Requirements

[illegible]

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

16

PROJECT SALEM NUCLEAR GEN. STA. ORDER NO. DATE 2-17-70

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
139639-1	A-432	139639	11	1.56 sq. in
139639-2				1.56 sq. in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
2-17-70	139639-1	107,500	68,900	157,500	100,950	14
"	139639-2	106,000	67,950	156,500	100,300	11

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY KILTON MANUFACTURING CO. WITNESSED BY A. C. Law

REMARKS: RESULTS MEET A-432 REQUIREMENTS

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

(REV. 6/61)

MILTON, P[illegible]ANIA, 1784[illegible]

Report of CHEMICAL and PHYSICAL Test of REINFORCING BARS

May 23, 1969

76

Charged to Taylor-Davis, Corporation

Car Number

Shipped to Wilmington, Delaware

Specification GRADE A-432

Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND T.
	52[illegible]	237124	9	.40	1.37	.019	.024	65300	109700	12	307[illegible]	O.K.

We certify that the above figures are correct as contained in the company records. [signature]

This material made in USA by basic electric furnace process to meet [illegible]ation ASTM A15-64 (or A432-61) and ASTM A305-64.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

17

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE 2-17-70

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
325501-1	A432	325501	11	1.56 sq in
325501-2	"	"	"	1.55 "

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
2-17-70	325501-1	105,500	67,650	153,500	98,400	16
	325501-2	105,000	67,750	153,500	99,050	17

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Hilton Manufacturing Co. Witnessed By A. C. Law

REMARKS: RESULTS MEET A-432 REQUIREMENTS

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

[illegible] REV. [illegible]

MILTON MANUFACTURING COMPANY

MILTON, PENNSYLVANIA, 17847

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Report of CHEMICAL and PHYSICAL Tests of REINFORCING BARS — March 31, 1969

Charged to Taylor-Davis, Corporation — Car Number RDG. 31180

Shipped to Wilmington, Delaware

Specification GRADE 60 — Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	ANALYSIS Mn	ANALYSIS P	ANALYSIS S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND TEST
		137101	5	39	1.25	023	039	68000	101300	15	9074	OK
		137155	5	40	1.30	019	030	69000	110300	15	9074	OK
		236637	5	38	1.26	023	026	63050	109300	13	9074	OK
		236634	5	41	1.25	021	036	70630	111300	13	9074	OK
		322908	5	37	1.23	021	031	72000	112300	13	9074	OK
	(40) [illegible]	137060	8	31	[illegible]	023	030	48150	75250	25	27383	OK
		236593	8	34	51	021	040	50700	77600	24	9131	OK
		135919	9	34	53	020	030	47410	79300	15	27,540	OK
		236535	11	36	55	023	033	43350	79450	16	18,490	OK
		322702	11	34	50	020	036	47050	79400	16	9245	OK
		316022	11	33	50	018	029	44710	77800	16	9245	OK

We certify that the above figures are correct as contained in the company records. [illegible signature]

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	DATE 2-17-70
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238745-1	A 432	238745	#11	1.55 sq. in
238745-2	"	"	"	1.56 "

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
2-17-70	238745-1	105,500	68,050	157,500	101,600	14
"	238745-2	105,500	67,650	158,000	101,300	15

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By A. C. Lavoi

REMARKS: RESULTS MEET A 432 REQUIREMENTS

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA. | ORDER NO. | DATE 2-17-70

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238916-1	A-432	238916	11	1.55 sq. in
238916-2	"	"	"	1.54 "

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
2-17-70	238916-1	104,500	67,400	155,500	100,300	15
"	238916-2	105,000	68,200	155,500	101,000	15

NOTE 1 - DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By: A. C. Laws

Remarks: Results Meet A-432 Requirement

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

MILTON MANUFACTURING COMPANY
MILTON, PENNSYLVANIA 17847

Report of CHEMICAL and PHYSICAL Tests of REINFORCING BARS — Date: March 11, 196[illegible]

Sold to: Taylor-Davis Corporation — Car Number:

Shipped to: Wilmington, Delaware

Specification: GRADE [illegible]0 — Project:

Customer's Order No. and Date	Mill Order Number	Heat Number	Size Number	Analysis C	Mn	P	S	Yield Pt. Lbs. per Sq. In.	Ten. Strength Lbs. per Sq. In.	Elong. in 8"	Wt.	Remarks
		137102	9	40	1.45	.019	.032	69400	109700	11	[illegible]	[illegible]
		236636	9	42	1.36	.021	.039	64450	101750	12	[illegible]	[illegible]

We certify that the above figures are correct as contained in the company records. [signature]

This material made in USA by basic electric furnace process to meet specification ASTM A15-64 (or) A432-64) and ASTM A305-64.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

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REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE 2-17-70
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
325500-1	A-432	325500	11	1.55 sq. in.
325500-2	"	"	"	1.56 "

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
2-17-70	325500-1	104,500	67,400	154,000	99,350	14
"	325500-2	105,000	67,300	154,500	99,050	13

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY HILTON MANUFACTURING CO. WITNESSED BY A. G. Laws

REMARKS: RESULTS MEET A-432 REQUIREMENTS GRADE 60

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

MILTON MANUFACTUR[illegible] Milton Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

22

DATE

[illegible]STOMER

THEIR ORDER NO.

[illegible]NTRACTOR

OUR ORDER NO.

[illegible]LLED TO MEET SPECIFICATIONS

JOB

HEAT NO.	C	MN	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR. LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK
236458	.39	1.40	.018	.031	#10	4.303	1.27	2.00	8.34	4.170	1.23	80,000	65,050	132,500	107,724	10	
236458	.39	1.40	.018	.031	#10	4.303	1.27	2.00	8.34	4.170	1.23	95,000	77,236	132,000	107,317	12	
236458	.39	1.40	.018	.031	#10	4.303	1.27	2.00	8.36	4.180	1.23	83,000	67,480	132,000	107,317	13	
236458	.39	1.40	.018	.031	#10	4.303	1.27	1.99	8.34	4.191	1.23	87,000	70,732	132,500	107,724	16	
236429	.38	1.38	.016	.029	#9	3.400	1.00	2.00	6.71	3.355	.99	67,000	67,677	105,000	106,060	16	
236429	.38	1.38	.016	.029	#9	3.400	1.00	2.00	6.71	3.355	.99	70,500	71,212	105,000	106,060	16	
236429	.38	1.38	.016	.029	#9	3.400	1.00	2.00	6.72	3.360	.99	66,500	67,172	105,000	106,060	11	
236429	.38	1.38	.016	.029	#9	3.400	1.00	2.00	6.69	3.345	.98	64,000	65,306	105,000	107,142	11	

SIGNED [illegible]

[illegible]GNED

MILTON MANUFACTU[illegible] CO.

Report of Chemical and [illegible] Test of Reinforcing Bars

DATE 2-13-6[illegible]

CUSTOMER

THEIR ORDER NO.

CONTRACTOR

OUR ORDER NO.

ROLLED TO MEET SPECIFICATIONS GRADE 60

JOB

HEAT NO.	C	MN	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK	[illegible]
236431	38	1.30	019	036	#9	3.400	1.00	2.00	6.71	3.355	99	61,500	62.121	105,000	106,060	17		
236436	38	1.36	019	035	#9	3.400	1.00	2.00	6.67	3.335	98	62,000	63.265	104,500	106,632	14		
321698	38	1.33	025	050	#9	3.400	1.00	2.00	6.69	3.345	98	62,500	63.776	105,500	107,653	13		
236429	38	1.38	016	029	#9	3.400	1.00	2.00	6.75	3.375	.99	56,000	56,566	105,000	106,060	18		
~~236158~~	~~39~~	~~1.40~~	~~018~~	~~031~~	~~#10~~	~~4.303~~	~~1.27~~	~~2.00~~	~~8.40~~	~~4.200~~	~~1.24~~	~~61,500~~	~~49,597~~	~~131,500~~	~~106,078~~	~~15~~		
~~131663~~	~~.38~~	~~1.33~~	~~021~~	~~030~~	~~#11~~	~~5.313~~	~~1.56~~	~~2.00~~	~~10.44~~	~~5.220~~	~~1.54~~	~~95,000~~	~~61,688~~	~~157,000~~	~~101,948~~	~~16~~		
236431	.38	1.30	019	036	#9	3.400	1.00	2.00	6.72	3.360	.99	69,000	69.697	104,500	105,555	16		
236436	38	1.36	019	035	#9	3.400	1.00	2.00	6.66	3.330	.98	63,500	64.796	104,500	106,632	17		
321698	38	1.33	025	050	#9	3.400	1.00	2.00	6.69	3.345	98	66,500	67,857	105,500	107,653	18		
236429	3.8	1.38	016	029	#9	3.400	1.00	2.02	6.76	3347	98	75,000	76,531	105,000	107,150	17		
~~236158~~	~~39~~	~~1.40~~	~~018~~	~~031~~	~~#10~~	~~4.303~~	~~1.27~~	~~2.02~~	~~8.48~~	~~4.198~~	~~1.23~~	~~99,000~~	~~73,171~~	~~132,000~~	~~107,317~~	~~12~~		
~~136663~~	~~.38~~	~~1.33~~	~~021~~	~~031~~	~~#11~~	~~5.313~~	~~1.56~~	~~2.00~~	~~10.45~~	~~5.225~~	~~1.54~~	~~94,500~~	~~61,364~~	~~156,500~~	~~101,623~~	~~15~~		
~~131663~~	~~.38~~	~~1.33~~	~~021~~	~~.31~~	~~#11~~	~~5.313~~	~~1.56~~	~~2.00~~	~~10.65~~	~~5.325~~	~~1.57~~	~~106,500~~	~~67,834~~	~~172,500~~	~~109,873~~	~~10~~		
MAX	.45	1.30																

SIGNED [illegible] SIGNED

MILTON, PEN[illegible]ANIA 17847

Report of CHEMICAL and PHYSICAL Tests of REINFORCING BARS — February 11, 1969 — Z3

Customer: Taylor-Davis Corporation — Car Number PC 598289

Ship to: Wilmington, Delaware

Specification: Grade 60 — Project:

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8"	WT. USED	BEND
		236429	9	38	1.38	016	029	76531	107150	17	18360	[illegible]
		236431	9	39	1.30	019	036	69697	[illegible]	16	27540	[illegible]
		236436	9	38	1.36	014	035	64796	106632	17	36720	[illegible]
		321698	9	38	1.33	027	010	67857	107653	18	27540	[illegible]

INSPECTED BY PUBLIC SERVICE OF NEW JERSEY

We certify that the above figures are correct as contained in the company records. [signature]

This material made in USA by basic electric furnace process to specification ASTM A15-64 (or A432-64) and ASTM A305-64.

... Report of Chemical and Physical Te... of Reinforcing Bars

DATE 1-9-69

...STOMER T[illegible] DAVIS INC.

...TRACTOR WILMINGTON DEL.

THEIR ORDER NO.

OUR ORDER NO.

...LED TO MEET SPECIFICATIONS A432

JOB PUBLIC SERVICE GAS & ELEC. OF N.J.

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR. LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN. 60,000 Min	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ/IN. 90,000 Min.	IN 8" ELONG. 7%	BREAK	BEND TEST
236137	.38	1.35	021	026	9	3.400	1.00	2.01	6.80	3.383	1.00	62.500	62.500	106.500	106.500	19	ANGLE	OK
36156	.39	1.35	021	036	9	3.400	1.00	2.02	6.77	3.351	.99	67,500	68,200	106,500	107,550	13	ANGLE	
135656	.38	1.36	025	031	9	3.400	1.00	2.01	6.65	3.308	.97	69,000	71,150	106,000	109,250	12	SHEAR	
236180	.39	1.29	016	045	9	3.400	1.00	2.02	6.71	3.322	.98	63,500	64,800	107,000	109,200	15	ANGLE	
36033	.37	1.32	024	031	9	3.400	1.00	2.00	6.78	3.390	1.00	62,500	62,500	104,000	104,000	14	CUP	
35849	.40	1.34	020	040	9	3.400	1.00	2.02	6.83	3.381	.99	65,500	66,150	108,000	109,100	13	ANGLE	
35386	.38	1.40	022	037	9	3.400	1.00	2.00	6.63	3.315	.98	68,000	69,400	106,500	108,650	12	CUP	
34682	.38	1.25	024	044	9	3.400	1.00	2.00	6.68	3.340	.98	65,000	66,350	105,500	107,650	15	ANGLE	OK

...GNED

SIGNED

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE 7-29-69
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
138010-1	A-432	138010	9	0.98
138010-2	" "	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
7-29-69	138010-1	74,500	76,000	100,500	102,500	16
"	138010-2	85,500	87,250	100,500	102,500	16

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MFG. CO.

WITNESSED BY: A. C. [illegible]

REMARKS: RESULTS MEET A-432 REQUIREMENTS, GRADE 60.

[signature]
MATERIALS DIVISION CHIEF

[signature]
LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

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REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	DATE 7-29-64
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
237588-1	A-432	237588	9	0.98
237588-2	" "	"	"	"

TENSION TEST RESULTS

		YIELD (NOTE 1)		ULTIMATE		ELONGATION IN
DATE	IDENT. NO.	LBS.	PSI	LBS.	PSI	8 INCHES %
7-29-64	237588-1	76,000	77,550	107,500	109,700	12
"	237588-2	76,500	78,050	107,500	109,700	12

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Mfg. Co. | Witnessed By: A. C. Kunz

REMARKS: RESULTS MEET A-432 REQUIREMENTS, GRADE 60.

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
ELECTRIC DEPARTMENT
TESTING LABORATORY REPORT

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REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE 7-2-
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
237640-1	A-432	237640	9	0.98
237640-2	" "	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	PSI	ULTIMATE LBS.	PSI	ELONGATION 8 INCHES %
7-29-69	237640-1	75,500	77,050	[illegible]	[illegible]	16
"	237640-2	79,500	81,100	109,000	111,200	14

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton MFG. Co.

Witnessed By: A. C. [illegible]

REMARKS: RESULTS MEET A-432 REQUIREMENTS, GRADE 60.

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

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REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE 7-29-69
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
323827-1	A-432	323827	9	0.98
323827-2	" "	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
7-21-69	323827-1	72,500	74,000	108,500	110,700	14
"	323827-2	80,500	82,150	109,500	111,750	14

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Mfg. Co. Witnessed By W. C. [illegible]

Remarks: Results meet A-432 requirements, Grade 60.

[signature]
MATERIALS DIVISION CHIEF

[signature]
LABORATORY ENGINEER

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
237124-1	A432	237124	9	0.99
237124-2	A432	237124	9	0.98

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
5/22/67	237124-1	80,000	80,800	108,000	109,100	14
"	237124-2	70,500	71,950	107,500	109,650	14

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | Witnessed By: H. Ehrmann

REMARKS: MATERIAL MEETS A432 SPECIFICATION.

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

4/24/6

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0166

PROJECT SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236636-1	A 432	236636	9	1.00 sq. in
236636-2	A 432	236636	9	1.00 sq. in
236636-R1	"	"	"	"
236636-R2	"	"	"	"
236636-R3	"	"	"	"
236636-R4	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
4/4/69	236636-1	57,800	57,800	107,200	107,200	20.6
4/4/69	236636-2	62,400	62,400	105,500	105,500	11.9
4/19/69	236636-R1	60,500	60,500	102,400	102,400	11.6
"	236636-R2	61,700	61,700	102,700	102,700	11.9
"	236636-R3	60,800	60,800	103,700	103,700	12.5
"	236636-R4	62,200	62,200	104,000	104,000	10.0

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY H. Obermaier

Remarks: One Yield Strength Below A432 Specified 60,000 psi Mi
Four Retests Meet A432 Specified Minimum Requirement

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

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0151

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
137102-1	A432	137102	9	1.00 sq. in
137102-2	A432	137102	9	1.00 sq. in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
4/4/69	137102-1	63,800	63,800	108,300	108,300	12.5
4/4/69	137102-2	64,000	64,000	108,200	108,200	17.5

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY H. Obermaier

Remarks: Results Meet A432 Specification.

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0128

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322310-1	A 432	322310	9	1.00 sq. in.
322310-2	A 432	322310	9	1.00 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
3/20/69	322310-1	63,200	63,200	103,200	103,200	16.3
"	322310-2	60,400	60,400	102,400	102,400	16.3

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Public Service Testing Lab. H. Obermaier

Remarks: Results Meet A432 Specification:

H C [illegible]
LABORATORY ENGINEER

[illegible]
MATERIALS DIVISION CHIEF

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0015

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA.				ORDER NO.
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136394-15	A432	136394	9S	1.00 sq.in
136394-13	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
12/20/68	136394-15	57,000	57,000	95,100	95,100	13
12/18/68	136394-13	60,400	60,400	79,200	79,200	18.5

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY See footnote | Witnessed By: C. Cooper

Remarks: One Yield Strength Below A432 Specified 60,000 psi min

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

15 - Tested by Columbia University, Carleton Lab 12/20/68
13 - Tested by United States Testing Co., Inc 12/18/68

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

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0019

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236029-1	A 432	236029	9	1.00 sq. in.
236029-2	A 432	236029	9	1.00 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/8/69	236029-1	77,000	77,000	115,200	115,200	11.5
1/8/69	236029-2	74,400	74,400	104,400	104,400	15.5

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY U. S. Testing Co. Inc. | Witnessed By: Herbert J. [illegible]

Remarks: Results meet A432 Specification

[signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0020

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322183-1	A432	322183	9	1.00 sq. in.
322183-2	A432	322183	9	1.00 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/8/69	322183-1	62,600	62,600	107,600	107,600	17.5
1/8/69	322183-2	63,000	63,000	108,200	108,200	17.5

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY U. S Testing Co. Inc. Witnessed By: Herbert J. Olesmond

Remarks Results meet A432 Specification

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT	SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
138990-1	A 432	138990	11	1.59 sq. in.
138990-2	A 432	138990	11	1.59 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
11/26/69	138990-1	110,000	69,200	157,500	99,050	18
"	138990-2	112,000	70,450	157,500	99,050	20

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

W. H. [illegible]

MATERIALS DIVISION CHIEF

J. O. [illegible]

LABORATORY ENGINEER

[illegible] REV. (7/8)

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0021

PROJECT SALEM NUCLEAR GEN. STA.				ORDER NO.
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322309-1	A 432	322309	9	1.00 sq. in.
322309-2	A 432	322309	9	1.00 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN
		LBS.	PSI	LBS.	PSI	8 INCHES, %
1/8/69	322309-1	68,200	68,200	108,600	108,600	13.0
1/8/69	322309-2	68,000	68,000	109,400	109,400	14.5

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY U. S. Testing Co. Inc. | Witnessed By: Herbert J. Ackermann

Remarks: Results meet A432 Specification

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

57

PROJECT	SALEM NUCLEAR GEN. STA.	ORDER NO.		DATE	

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238291-1	A432	238291	11	1.53 sq. in.
238291-2	A432	238291	11	1.53 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
11/26/69	238291-1	107,500	70,250	151,500	99,000	14
"	238291-2	115,000	75,150	151,500	99,000	16

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | Witnessed By: H. Obermaier

Remarks: Results meet A432 specification requirements.

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE
IDENTIFICATION NO.	**MATERIAL**	**HEAT NO.**	**SIZE**	**AREA**
139000-1	A432	139000	11	1.53 sq. ins.
139000-2	A432	139000	11	1.53 sq. ins.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
11/26/69	139000-1	115,000	75,150	156,500	102,300	15
"	139000-2	117,000	76,450	157,500	102,950	12

NOTE 1, DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING CO. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

58 0014

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136-371-1	A432	136-371	9 S	1.015 sq in
-2	"	"	9 S	0.876 sq in

T E N S I O N   T E S T   R E S U L T S

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/10/69	136-371-1	58,000	57,120	97,800	97,700	14.0
1/10/69	136-371-2	54,600	62,343	98,200	102,700	11.0

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY United States Testing Co., Inc.

Witnessed By: [illegible]

Remarks: [illegible] Yield Strength Below A432 Specified [illegible] psi

[illegible signature]
LABORATORY ENGINEER

[illegible signature]
MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
138989-1	A432	138989	11	1.53 sq. in.
138989-2	A432	138989	11	1.54 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
11/26/69	138989-1	114,000	74,500	160,500	104,900	14
"	138989-2	117,500	76,300	160,000	103,900	15

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | Witnessed By: H. Obermaier

Remarks: Results meet A432 Specification Requirements

W. G. [illegible]

MATERIALS DIVISION CHIEF

J. O'Brady HHC

LABORATORY ENGINEER

L-388 REV 6/69

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0043

PROJECT SALEM NUCLEAR GEN. STA.
ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236137-1	A432	236137	9S	1.00
236137-2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/9/69	236137-1	62,500	62,500	106,500	106,500	19.0
2/7/69	236131-2	66,000	66,000	108,000	108,000	18.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY See footnote.
Witnessed By W. Kap[illegible]

Remarks: Results Meet A432 Requirements

[signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

1 - Tested by Milton Manufacturing Co.

2 - Tested by United States Testing Co. Inc.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238309-1	A 432	238309	11	1.53 sq. in.
238309-2	A 432	238309	11	1.54 sq. in.

T E N S I O N   T E S T   R E S U L T S

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
11/26/69	238309-1	108,000	70,600	148,500	97,050	12
"	238309-2	109,500	71,100	149,000	96,750	12

NOTE 1, DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING Co. | WITNESSED BY: H. Obermann

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

[illegible]
MATERIALS DIVISION CHIEF

[illegible]
LABORATORY ENGINEER

L-388 REV 8/[illegible]

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0040

PROJECT SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136156-1	A 432	136156	9S	1.00
136156-2	"	"	"	1.00

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/9/69	136156-1	68,500	68,500	107,550	107,550	13.0
2/7/69	136156-2	66,700	66,700	106,800	106,800	14.8

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY See footnote. WITNESSED BY W Kaptan

REMARKS: RESULTS MEET A432 REQUIREMENTS

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

1- Tested by Milton Manufacturing Co.
2- Tested by United States Testing Co. Inc.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
ELECTRIC DEPARTMENT
TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
139001-1	A432	139001	11	1.54 sq. in.
139001-2	A432	139001	11	1.54 sq. ins.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN
		LBS.	PSI	LBS.	PSI	8 INCHES %
11/26/69	139001-1	127,500	82,800	157,500	102,250	10
"	139001-2	127,000	82,450	159,000	103,250	10

NOTE 1, DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By: H. Oberman

Remarks: Results Meet A432 Specification Requirements.

W. A. [illegible]
MATERIALS DIVISION CHIEF

J. O'Grady [illegible]
LABORATORY ENGINEER

L-388 REV. 6/68

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

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0041

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
235656-1	A432	235656	#5	1.00
235656-2	"	"	"	1.00

T E N S I O N   T E S T   R E S U L T S

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/9/69	235656-1	71,550	71,550	109,250	109,250	12.0
1/7/69	235656-1	65,500	65,500	107,200	107,200	13.2

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY See footnote. WITNESSED BY W Kapetan

Remarks Results Meet A432 Requirements

H C [signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

1- Tested by Milton Manufacturing Co.
2- Tested by United States Testing Co. Inc.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238288-1	A432	238288	11	1.57 sq. in.
238288-2	A432	238288	11	1.57 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	PSI	ULTIMATE LBS.	PSI	ELONGATION IN 8 INCHES %
11/26/69	238288-1	112,500	71,650	161,000	102,550	12
"	238288-2	113,500	72,300	160,000	101,900	12

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING CO. WITNESSED BY: H. Ebermann

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

W. H. Pelman Jr.

MATERIALS DIVISION CHIEF

J. Prodzynski

LABORATORY ENGINEER

C-380 REV 6/69

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

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0044

PROJECT SALEM NUCLEAR GEN. STA.				ORDER NO.
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
Y36180-1	A432	Y36180	95	1.00
Y36180-2	"	"	"	1.00

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/9/69	Y36180-1	64,800	64,800	109,200	109,200	15.0
2/7/69	Y36180-2	63,000	63,000	105,800	105,800	16.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY See footnote — WITNESSED BY W. Kaplan

REMARKS: RESULTS MEET A432 REQUIREMENTS

H.C. Miller Jr.
LABORATORY ENGINEER

J. J. Palmer
MATERIALS DIVISION CHIEF

1 - Tested by Milton Manufacturing Co.
2. Tested by United States Testing Co. Inc.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238288-1	A432	238288	11	1.57 sq. in.
238288-2	A432	238288	11	1.57 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	PSI	ULTIMATE LBS.	PSI	ELONGATION IN 8 INCHES %
11/26/69	238288-1	112,500	71,650	161,000	102,550	12
"	238288-2	113,500	72,300	160,000	101,900	12

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By: H. Obermann

Remarks: Results Meet A432 Specification Requirements.

W. H. [signature]

MATERIALS DIVISION CHIEF

[signature]

LABORATORY ENGINEER

C-328 REV 8/69

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0044

PROJECT SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236180-1	A 432	236180	9S	1.00
236180-2	"	"	"	1.00

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/9/69	236180-1	64,800	64,800	109,200	109,200	15.0
2/7/69	236180-2	63,000	63,000	105,800	105,800	16.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY Sar Fortnok — WITNESSED BY Eli Kaplan

REMARKS: RESULTS MEET A432 REQUIREMENTS

H C Miller Jr
LABORATORY ENGINEER

J. L. Palmer
MATERIALS DIVISION CHIEF

1- Tested by Milton Manufacturing Co.
2. Tested by United States Testing Co. Inc.

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

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0042

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
Y36033-1	A 432	Y36033	9S	1.00
Y36033-2	"	"	"	1.00

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/9/69	Y36033-1	62,500	62,500	106,000	106,000	14.0
1/7/69	Y36033-2	70,800	70,800	109,600	109,600	10.8

NOTE 1. DIVIDER OVER 8 INCHES ~~[illegible]~~

TESTED BY See footnote - | Witnessed By. W. Kapetan

Remarks: Results Meet A432 Requirements.

H. C. [illegible] — LABORATORY ENGINEER

[illegible] — MATERIALS DIVISION CHIEF

1- Tested by Milton Manufacturing Co.

2- Tested by United States Testing Co. Inc.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT	SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
238283-1	A432	238283	11	1.52 sq. in.
238283-2	A432	238283	11	1.55 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
11/26/69	238283-1	107,500	70,700	150,000	98,700	10
"	238283-2	109,000	70,300	152,500	98,400	11

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING Co. WITNESSED BY: H. Hermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

W. G. [illegible]

MATERIALS DIVISION CHIEF

J. C. Grady

LABORATORY ENGINEER

[illegible]

Rec 2/13/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
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0039

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
135849-1	A432	135849	9S	1.00
135849-2	"	"	"	1.00

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/9/69	135849-1	66,150	66,150	109,100	109,100	13.0
2/7/69	135849-2	66,300	66,300	108,800	108,800	16.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY See footnote — WITNESSED BY W. Kapstein

REMARKS: RESULTS MEET A432 REQUIREMENTS

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

1- Tested by Milton Manufacturing Co.
2- Tested by United States Testing Co. Inc.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT	SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
139039-1	A432	139039	11	1.57 sq. in.
139039-2	A432	139039	11	1.57 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN
		LBS.	PSI	LBS.	PSI	8 INCHES %
11/26/69	139039-1	112,000	71,350	153,500	97,750	10
"	139039-2	115,000	73,250	154,500	98,400	10

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

W. H. [illegible]

MATERIALS DIVISION CHIEF

J. O. [illegible]

LABORATORY ENGINEER

L-388 (REV. 8/60)

Rec 2/13/69

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0038

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
135386-1	A 432	135386	9S	1.00
135386-2	"	"	"	1.00

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/9/69	135386-1	69,400	69,400	108,650	108,650	12.0
2/7/69	135386-2	65,800	65,800	107,400	107,400	18.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY See footnote - | Witnessed By W. Kapton

Remarks: Results Meet A 432 Requirements

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

1- Tested by Milton Manufacturing Co.

2- Tested by United States Testing Co. Inc.

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0013

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
Testing Laboratory
REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA.				ORDER NO.
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136388-1	A432	136388	9	1.00 [illegible]
136388-2	A432	136388	9	1.00 [illegible]

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
1/2/69	136388-1	62,000	62,000	104,400	104,400	13.0
1/2/69	136388-2	62,600	62,600	105,200	105,200	18.5

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY United States Testing Co. Inc. | Witnessed By: Herbert J. Obermaier

Remarks: Results meet A432 Specification

[signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
ELECTRIC DEPARTMENT
TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

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PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
324227-1	A432	324227	11	1.52 sq. in.
324227-2	A432	324227	11	1.52 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
9/11/69	324227-1	112,000	73,700	161,000	105,900	9
"	324227-2	113,000	74,350	161,500	106,250	9

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUREMENTS.

[signature]
MATERIALS DIVISION CHIEF

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LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC DEPARTMENT

TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

132

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
237839-1	A432	237839	11	1.56 sq. in.
237839-2	A432	237839	11	1.55 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
9/11/69	237839-1	110,000	70,500	159,500	102,500	11
"	237839-2	110,000	70,950	159,000	102,600	12

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING Co. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

MATERIALS DIVISION CHIEF

LABORATORY ENGINEER

L-388 REV. 6/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
ELECTRIC DEPARTMENT
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REINFORCEMENT BAR TEST

135

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
324325-1	A 432	324325	11	1.52 sq. in.
324325-2	A 432	324325	11	1.51 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	PSI	ULTIMATE LBS.	PSI	ELONGATION IN 8 INCHES %
9/11/69	324325-1	99,500	65,450	146,500	96,400	13
"	324325-2	99,500	65,900	147,000	97,350	13

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING CO. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY

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REINFORCEMENT BAR TEST

134

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	DATE
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
324320-1	A 432	324320	11	1.53 sq. in.
324320-2	A 432	324320	11	1.53 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	PSI	ULTIMATE LBS.	PSI	ELONGATION IN 8 INCHES %
9/11/69	324320-1	102,500	67,000	152,500	99,650	10
"	324320-2	102,500	67,000	152,000	99,350	11

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING Co. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATION REQUIREMENTS.

J. Palmer
MATERIALS DIVISION CHIEF

H C [illegible]
LABORATORY ENGINEER

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ELECTRIC DEPARTMENT
TESTING LABORATORY REPORT

REINFORCEMENT BAR TEST

137

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
324324-1	A 432	324324	11	1.53 sq. in.
324324-2	A 432	324324	11	1.52 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	PSI	ULTIMATE LBS.	PSI	ELONGATION IN 8 INCHES %
9/4/69	324324-1	99,000	64,700	146,500	95,750	14
"	324324-2	99,000	65,150	147,000	96,700	13

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING CO. WITNESSED BY: H. Oberman

REMARKS: RESULTS MEET A 432 SPECIFICATION REQUIREMENTS.

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L-388 REV 6/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
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REINFORCEMENT BAR TEST

136

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
138434-1	A 432	138434	11	1.53 sq. in.
138434-2	A 432	138434	11	1.51 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
9/11/69	138434-1	99,500	65,050	144,500	94,450	14
"	138434-2	96,000	63,600	144,500	95,700	13

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By: H. Obermaier

REMARKS: Results meet A 432 specification requirements.

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L-388 REV. 6/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
ELECTRIC DEPARTMENT
TESTING L[illegible]RATORY REPORT

REINFORCEMENT BAR TEST

148

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.		DATE
IDENTIFICATION NO.	**MATERIAL**	**HEAT NO.**	**SIZE**	**AREA**	
138344-1	A-432	138344	11	1.52 sq.in.	
138344-2	"	"	"	1.53 sq.in.	

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
8-27-69	138344-1	125,000	82,250	169,500	111,500	11
"	138344-2	125,500	82,050	170,000	111,100	12

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | Witnessed By: A. C. Laws

REMARKS: RESULTS MEET A-432 SPECIFICATION REQUIREMENTS FOR GRADE 60.

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LABORATORY ENGINEER

L-308 REV 6/69

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REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	DATE
IDENTIFICATION NO.	**MATERIAL**	**HEAT NO.**	**SIZE**	**AREA**
237722-1	A-432	237722	11	1.51 sq. in.
237722-2	"	"	"	1.51 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES
[illegible]	237722-1	109,000	72,200	155,000	102,650	12
"	237722-2	111,000	73,500	155,000	102,650	11

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING CO. | WITNESSED BY: [illegible]

REMARKS: RESULTS MEET A-432 SPECIFICATION REQUIREMENTS FOR [illegible]

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L-388 REV. 6/69

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
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REINFORCEMENT BAR TEST

150

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.	DATE

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
138333-1	A-432	138333	11	1.52 sq.in.
138333-2	"	"	"	1.52 sq.in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
8-27-69	138333-1	100,000	65,800	152,500	100,300	10
" "	138333-2	104,500	68,750	152,500	100,300	11

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MANUFACTURING CO. | WITNESSED BY: A. C. Laux

REMARKS: RESULTS MEET A-432 SPECIFICATION REQUIREMENTS FOR GRADE 60.

J. J. Palmer
MATERIALS DIVISION CHIEF

H. C. Anguilla
LABORATORY ENGINEER

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

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REINFORCEMENT BAR TEST

149

PROJECT SALEM NUCLEAR GEN. STA.			ORDER NO.	DATE
IDENTIFICATION NO.	**MATERIAL**	**HEAT NO.**	**SIZE**	**AREA**
324235-1	A-432	324235	11	1.54 sq. in.
324235-2	"	"	"	1.53 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
8-27-69	324235-1	105,500	68,500	148,000	96,160	15
"	324235-2	101,000	66,150	147,000	96,100	17

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By: A. C. Laws

REMARKS: Results meet A-432 specification requirements for Grade 60.

J. J. Palmer
MATERIALS DIVISION CHIEF

H. C. [signature]
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L-388 REV. 6/69

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REINFORCEMENT BAR TEST REPORT

248 058

PROJECT: SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
323068-1	A432	323068	11	1.52 sq
323068-2	A432	323068	11	1.52 sq

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS	YIELD (NOTE 1) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES %
4/9/69	323068-1	[illegible]	[illegible]	[illegible]	[illegible]	15
4/9/69	323068-2	115,000	75,650	157,500	103,600	15

NOTE: 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: MILTON MANUFACTURING CO. | WITNESSED BY: H. Chesman

REMARKS: RESULTS MEET A432 SPECIFICATION.

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REINFORCEMENT BAR TEST REPORT

247 0159

PROJECT: SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236778-1	A 432	236778	11	1.54 sq in
236778-2	A 432	236778	11	1.54 sq in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS	PSI	LBS	PSI	
4/9/69	236778-1	101,500	65,900	148,500	96,450	17
4/9/69	236778-2	100,000	64,950	148,000	96,100	19

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: MILTON MANUFACTURING CO. WITNESSED BY: H. Obermaier

REMARKS: RESULTS MEET A432 SPECIFICATIONS.

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REINFORCEMENT BAR TEST REPORT

250 0068

PROJECT: SALEM NUCLEAR GEN. STA.

ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322820-1	A432	322820	#11	1.56 sq in
322820-2	"	"	"	"
322820-3	"	"	"	"
322820-4	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES %
		LBS	PSI	LBS	PSI	
[illegible]	322820-1	[illegible]	[illegible]	[illegible]	107,051	10.0
"	322820-2	113,500	73,225	166,500	107,419	
4/4/69	322820-3	96,000	61,540	163,600	104,870	[illegible]
"	322820-4	98,000	62,820	171,800	110,130	[illegible]

NOTE (1): DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: SEE FOOT NOTE H. Abramovic

REMARKS: SAMPLES 1&2 TESTED BY MILTON MANUFACTURING CO., 3&4 TESTED BY PUBLIC SERVICE ELECTRIC & GAS CO. RESULTS MEET A432 SPECIFICATION.

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REINFORCEMENT BAR TEST REPORT

249 0157

PROJECT SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236776-1	A432	236776	11	1.53 sq. in
236776-2	A432	236776	11	1.53 sq. in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS	PSI	LBS	PSI	
4/9/69	236776-1	112,500	73,550	154,000	100,650	18
4/9/69	236776-2	116,500	76,150	154,500	101,000	17

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By: H. Obermaier

Remarks: Results Meet A432 Specification.

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REINFORCEMENT BAR TEST REPORT

276 0047

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
137010-1	A432	137010	11	1.56 sq.
137010-2	A432	137010	11	1.56 sq.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1.) LBS	YIELD (NOTE 1.) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES %
2/24/[illegible]	137010	[illegible]	[illegible]	[illegible]	[illegible]9,012	9
〃	137010	106,000	67,948	169,500	108,653	9

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. Witnessed By F. Oleaminin

Remarks: Results Meet A432 Specification Requirements.

[signature]
LABORATORY ENGINEER

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REINFORCEMENT BAR TEST REPORT

291 0066

PROJECT: SALEM NUCLEAR GEN. STA. | ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322764-1	A432	322764	11	1.53 sq. in.
322764-2	A432	322764	11	1.54 sq. in.

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) P.S.I.	ULTIMATE LBS.	ULTIMATE P.S.I.	ELONGATION, IN. 8 INCHES, %
2/27/69	322764-1	106,000	69,300	154,500	100,980	8
"	322764-2	106,000	68,837	163,000	105,844	8

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: Milton Manufacturing Co. | Witnessed By: [illegible]

Remarks: Results Meet A432 Specification Requirements

[illegible]
LABORATORY ENGINEER

[illegible]
MATERIALS DIVISION CHIEF

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REINFORCEMENT BAR TEST REPORT

299 0010

PROJECT - SALEM NUCLEAR GEN. STA. | ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322803-1	A 432	322803	11	1.56 sq in
322803-2	A 432	322803	11	1.56 sq in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES %
		LBS	PSI	LBS	PSI	
2/27/69	322803	107,000	69,477	168,500	108,012	9
"	322803	109,500	70,192	168,500	108,012	9

NOTE: 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | Witnessed By: H. [illegible]

Remarks: Results Meet A432 Specification Requirements

[illegible]
LABORATORY ENGINEER

[illegible]
MATERIALS DIVISION CHIEF

301 0057

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REINFORCEMENT BAR TEST REPORT

PROJECT: SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
321934-1	A432	321934	11S	1.56
321934-2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS	YIELD (NOTE 1) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES %
1/14/69	321934-1	97,500	[illegible]	163,000	104,500	12.5
1/14/69	321934-2	95,000	60,900	163,800	105,000	11.0

NOTE: 1. DIVIDER OVER 8 INCHES [illegible]

TESTED BY: United States Testing Co. Inc. | Witnessed By: [illegible]

Remarks: Results Meet A432 Specifications

[signature]
LABORATORY ENGINEER

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REINFORCEMENT BAR TEST REPORT

300
0058

PROJECT: SALEM NUCLEAR GEN. STA.			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	[illegible]
235783-1	A-432	235783	#15	1.56
235783-2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES %
		LBS	PSI	LBS	PSI	
1/19/69	235783-1	98,600	63,200	163,800	105,000	12.0
1/19/69	235783-2	94,400	60,500	163,700	104,900	15.5

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY United States Testing Co. Inc. Witnessed By O. C. Lowe

Remarks: Results Meet A432 Specifications

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LABORATORY ENGINEER

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REINFORCEMENT BAR TEST REPORT

303 0055

PROJECT: SALEM NUCLEAR GEN. STA. | ORDER NO.:

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136173-1	A432	136173	1.15	1.56
136173-2		"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS	YIELD (NOTE 1) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/19/69	136173-1	95,200	61,000	162,500	104,400	16.0
1/19/69	136173-2	95,800	61,400	162,400	104,100	18.5

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY United States Testing Co. Inc. | Witnessed By U. C. Lauz

Remarks: Results meet A432 specifications

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LABORATORY ENGINEER

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REINFORCEMENT BAR TEST REPORT

302 0056

PROJECT SALEM NUCLEAR GEN. STA. | ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136182-1	A432	136182	11S	1.56
136182-2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS	YIELD (NOTE 1) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
2/14/69	136182-1	102,000	65,400	170,000	109,000	11.0
2/14/69	136182-2	104,400	66,900	171,500	109,900	12.0

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY United States Testing Co. Inc. | Witnessed By A. C. Lawl

Remarks: Results meet A432 specifications

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REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322207-1	A432	322207	#11S	1.56
322207-2	"	"	"	1.56

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS	YIELD (NOTE 1) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
2/7/69	322207-1	94,400	60,500	165,700	105,900	12.5
2/7/69	322207-2	98,000	62,800	166,400	106,500	11.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY United States Testing Co. Inc. | Witnessed By [illegible]

Remarks: Results Meet A432 Requirements

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

305-0015

PROJECT - SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322047-1	A432	322047	11S	1.56
322047-2	A432	〃	11S	1.56

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES %
		LBS.	PSI	LBS	PSI	
2/7/69	322047-1	97,000	62,200	164,800	105,600	12.0
2/7/69	322047-2	94,800	60,800	164,800	105,600	14.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY United States Testing Co. Inc. | Witnessed By [illegible]

Remarks: Results Meet A432 Requirements

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LABORATORY ENGINEER

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY

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REINFORCEMENT BAR TEST REPORT

PROJECT: SALEM NUCLEAR GEN. STA. | ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
327880-17	A432	821880	#18	4.00 sq in
327880-16	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS	YIELD (NOTE 1) PSI	ULTIMATE LBS	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
12/20/68	327880-17	119,100	76,100	182,100	116,700	10.6
12/25/68	" " -16	127,000	81,400	187,000	120,000	8.5

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: [illegible] | Witnessed By W. [illegible]

Remarks: Results meet A432 requirements

[illegible signature]
LABORATORY ENGINEER

[illegible signature]
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#17 – Tested by [illegible] [illegible] 12/20/68
16 – Tested by [illegible] 12/25/68

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Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
235766-14	A432	235766	11	1.56 sq in
235766-15	A432	235766	"	"
235766-R1	"	"	"	1.534 sq in
235766-R2	"	"	"	1.550 sq in
235766-R3	"	"	"	1.550 sq in
235766-R4	"	"	"	1.544 sq in

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES %
		LBS	PSI	LBS	PSI	
12/20/68	235766-14	92,600	59,400	156,500	100,300	18.6
12/28/68	235766-15	94,200	60,380	160,600	103,000	18.0
1/13/69	235766-R1	96,400	62,640	159,800	103,830	15.0
1/13/69	235766-R2	96,600	62,320	160,200	103,350	14.0
1/13/69	235766-R3	95,800	61,810	159,800	103,100	17.0
1/13/69	235766-R4	94,000	60,880	160,800	104,150	11.0

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY See foot note | Witnessed By [illegible]

Remarks: One Yield Strength Below A432 Specified 60,000 psi Min. Four Retests Meet A432 Specified Min Requirements

[signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

14 - Tested by Columbia University, [illegible] Lab. 12/20/68

15 - Tested by United States Testing Co. Inc. 12/28/68

R1, R2, R3, R4 Tested by United States Testing Co. Inc. 1/13/69

MILTON MANUFACTURING CO., Milton Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE 11-19-69

TOMER TAYLOR-DAVIS — THEIR ORDER NO.

TRACTOR — OUR ORDER NO. 504

ED TO MEET SPECIFICATIONS GRADE 60 — JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR, LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG	BREAK	BEND TEST
38190	40	1.35	020	036	10	4.303	1.27	1.91	8.05	4.215	1.24	91,000	73,400	139,000	112,100	11		OK
38190	40	1.35	020	036	10	4.303	1.27	1.99	8.35	4.196	1.23	90,500	73,600	138,500	112,600	10		
38202	40	1.45	016	039	10	4.303	1.27	1.98	8.36	4.222	1.24	96,000	77,400	147,000	118,550	10		
38202	40	1.45	016	039	10	4.303	1.27	1.97	8.34	4.234	1.25	99,500	79,600	147,500	118,100	11		OK

IED Witnessed By: H. Obermann — SIGNED B. E. Hurst

314-0012

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT: SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136155-18	A 432	136155	#11	1.56 sq. in.
136155-17	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES, %
		LBS	PSI	LBS	PSI	
12/20/68	136155-18	123,000	78,900	182,400	117,000	10.9
12/20/68	-17	128,000	82,000	178,000	114,100	7.5

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: [illegible] | Witnessed By [illegible] Cooper

Remarks: Results meet A432 Requirements

[illegible]
LABORATORY ENGINEER

[illegible]
MATERIALS DIVISION CHIEF

-18 - Tested by [illegible] [illegible] 12/20/68
-17 - Tested by [illegible] [illegible] 12/20/68

MILTON MANUFA[illegible]RING COMPANY

MILTON, PE[illegible]ANIA, 17847

eport of CHEMICAL [illegible] PHYSICAL Test of REINFORCING BARS March 5, 1969

harged to Taylor-Davis, Corporation Car Number PRR 442438 513

hipped to Delaware, Wilmington

pecification GRADE 60 Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND TE[ST]
[illegible] 2-24	[illegible]	136787	10	39	1.36	014	036	66150	107250	12	27,108	[illegible]
[illegible]	[illegible]	136958	10	40	1.34	022	034	76400	115850	11	18,072	[illegible]
[illegible]	[illegible]	322836	10	36	1.06	021	030	72200	104350	13	27,108	[illegible]
[illegible]	[illegible]	322837	10	36	1.09	026	039	70950	105650	10	27,108	[illegible]
[illegible]	[illegible]	322845	10	34	.97	019	033	66550	99200	14	27,108	[illegible]

We certify that the above figures are correct as contained in the company records. Thomas a Fisher

MILTON MANUFACTURI[illegible] Milton Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE 8-18-69

TOMER TAYLOR - DAVIS | THEIR ORDER NO.

TRACTOR | OUR ORDER NO. 509

LED TO MEET SPECIFICATIONS GRADE 60 | JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR. LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG.	BREAK
22844	36	99	020	038	10	4.303	1.27	2.01	8.39	4.195	1.23	82,500	67,050	126,500	102,850	14	
22841	36	99	020	038	10	4.303	1.27	1.99	8.38	4.211	1.24	83,000	66,950	127,000	102,400	15	
21737	42	1.45	022	034	10	4.303	1.27	2.00	8.47	4.235	1.25	77,500	62,000	129,500	103,600	15	
21737	42	1.45	022	034	10	4.303	1.27	2.00	8.42	4.210	1.24	80,000	64,500	131,000	105,650	16	

ED [illegible]

MILTON MANUFACTURI CO., Milton Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE 3-4-69

TOMER TAYLOR-DAVIS CORP. THEIR ORDER NO.

TRACTOR OUR ORDER NO. 515

ED TO MEET SPECIFICATIONS G 60 JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG	BREAK
7023	39	1.38	016	132	10	4.303	1.27	2.01	839	4.174	1.23	89,500	72,750	128,500	104,450	17	
7023	39	1.38	016	032	10	4.303	1.27	2.01	8.40	4.179	1.23	89,500	72.750	128,000	104,050	15	
650	39	1.36	020	037	10	4.303	1.27	2.00	8.55	4.275	1.26	85,500	67,850	136,500	108,350	13	
3650	39	1.36	020	037	10	4.303	1.27	1.98	8.49	4.288	1.26	95,000	75,400	136,000	107,950	13	
6958	40	1.34	022	034	10	4.303	1.27	1.99	8.36	4.201	1.24	102,000	82.250	144,000	116,150	10	
6958	40	1.34	022	034	10	4.303	1.27	2.00	8.38	4.190	1.23	103,000	83,750	143,500	116,650	10	
6781	39	1.36	019	036	10	4.303	1.27	1.99	8.42	4,231	1.24	91,500	73,000	131,000	105,650	15	
6787	39	1.36	019	036	10	4.303	1.27	2.00	8.49	4.245	1.25	87,500	70,000	132,500	106,000	14	
2831	36	1.06	021	030	10	4.303	1.27	2.00	8.51	4.255	1.25	91,000	72.800	131,500	105,200	12	
2836	36	1.06	021	030	10	4.303	1.27	2.00	8.56	4.259	1.25	91,500	73,200	131,500	105,200	15	
2837	36	1.09	026	039	10	4.303	1.27	2.01	8.51	4,234	1.25	88,500	70,800	130,500	104,400	14	
2837	36	1.09	026	039	10	4.303	1.27	2.01	8.51	4.234	1.25	88,500	70,800	130,500	104,400	16	
2845	34	.97	019	033	10	4.303	1.27	2.00	8.40	4.200	1.24	82,500	66,550	121,500	98,000	15	
2845	34	.97	019	033	10	4.303	1.27	2.00	8.41	4.205	1.24	86,000	69,350	122,500	98,800	15	

MILTON MANUFACTURING COMPANY
MILTON, PENNSYLVANIA, 17847

Report of CHEMICAL and PHYSICAL Test of REINFORCING BARS — March 7, 1969

Charged to: Taylor-Davis, Corporation — Car Number PRR 441660

Shipped to: Wilmington, Delaware — 514

Specification: GRADE 60 — Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	ANALYSIS Mn	ANALYSIS P	ANALYSIS S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND TEST
	2-24	136958	10	40	1.34	.022	.034	76400	115850	11	19,873	ok
		235501	10	39	1.36	.020	.037	65850	109900	11	39,796	ok
		~~125907~~	~~11~~	~~39~~	~~1.44~~	~~.022~~	~~.036~~	~~75900~~	~~115150~~	~~10~~	~~10,736~~	~~ok~~
		~~137010~~	~~11~~	~~40~~	~~1.34~~	~~.022~~	~~.032~~	~~65050~~	~~109950~~	~~9~~	~~9,365~~	~~ok~~
	3-4	~~322764~~	~~11~~	~~39~~	~~1.41~~	~~.021~~	~~.030~~	~~68850~~	~~109100~~	~~9~~	~~18,731~~	~~ok~~
		~~333593~~	~~11~~	~~39~~	~~1.45~~	~~.020~~	~~.023~~	~~69350~~	~~111600~~	~~9~~	~~15,336~~	~~ok~~
		~~333530~~	~~11~~	~~40~~	~~1.40~~	~~.020~~	~~.026~~	~~65450~~	~~108950~~	~~10~~	~~9,[illegible]~~	~~ok~~

We certify that the above figures are correct as contained in the company records. Thomas O'Neil

MILTON MANUFA[illegible]RING COMPANY
MILTON, P[illegible]ANIA, 17847

745

[R]eport of CHEMICAL and PHYSICAL Test of REINFORCING BARS June 25, 1969

[C]harged to Taylor-Davis, Corporation Car Number

[S]hipped to Wilmington, Delaware

[S]pecification GRADE: 60 Project

CUSTOMERS ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS				YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND TE[ST]
				C	Mn	P	S					
	✓	137248	11	35	1.00	019	033	66650	102650	8	27735	OK
	✓	135984	11	40	1.32	022	030	64750	117050	10	9861	
	✓	323632	11	35	1.10	020	038	73250	105050	7	18,490	
	(40) 7-10	322453	11	32	55	021	034	45350	79950	17	18,490	OK

We certify that the above figures are correct as con[tain]ed in the company records

[signature]

Report of Chemical and Phy[illegible] Test of Reinforcing Bars

DATE

CUSTOMER THEIR ORDER NO.

CONTRACTOR OUR ORDER NO. 519

ROLLED TO MEET SPECIFICATIONS JOB

HEAT NO.	C	MN	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR. LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG	BREAK	[illegible]
236458	39	1.40	018	031	#10	4.303	1.27	2.00	8.34	4.170	1.23	80,000	65,050	132,500	107,724	10		
236458	39	1.40	018	031	#10	4.303	1.27	2.00	8.34	4.170	1.23	95,000	77,236	132,000	107,317	12		
236457	39	1.40	018	031	#10	4.303	1.27	2.00	836	4.180	1.23	83,000	67,480	132,000	107,317	13		
236458	39	1.40	018	031	#10	4.303	1.27	1.99	8.34	4.191	1.23	87,000	70,732	132,500	107,724	16		
236429	38	1.38	016	029	#9	3.400	1.00	2.00	6.71	3.355	.99	67,000	67,677	105,000	106,060	16		
236427	38	1.38	016	029	#9	3.400	1.00	2.00	6.71	3.355	.99	74,500	71,212	105,000	106,060	16		
236429	38	1.38	016	029	#9	3.400	1.00	2.00	6.72	3.360	99	66,500	67,172	105,000	106,060	11		
236429	38	1.38	016	029	#9	3.400	1.00	2.00	6.69	3.345	98	64,000	65,306	105,000	107,142	11		

SIGNED [illegible] SIGNED

MILTON, PE[illegible]A, 17847

755

April 16, 1969

eport of CHEMICAL and PHYSICAL Test of REINFORCING BARS

harge Taylor-Davis, Corporation

Car Number PC 598157

hipped to Wilmington, Delaware

pecification GRADE 60

Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND
	~~4-1~~	~~137219~~	~~7~~	~~38~~	~~1.25~~	~~020~~	~~025~~	~~65250~~	~~106450~~	~~13~~	~~9075~~	OK
	4-1	236776	11	35	1.03	021	027	66650	101650	11	36,980	OK
		236778	11	24	1.05	020	025	65250	98700	13	18,490	OK
		323038	11	33	1.08	021	034	68550	103300	12	36,980	OK

We certify that the above figures are correct as contained in the company records. Jack P. Edlund

This material made in USA has been [illegible]

MILTON MANUFACTUR[illegible] , Milton Pennsylvania

746

Report of Chemical and Physical Test of Reinforcing Bars

DATE 6-24-69

STOMER Public Service

THEIR ORDER NO.

NTRACTOR

OUR ORDER NO.

LED TO MEET SPECIFICATIONS G 60

JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR. LBS	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT		TENSILE STRENGTH		% ELONG	BREAK
												MACHINE READING	LBS. PER SQ. IN.	MACHINE READING	LBS. PER SQ. IN.		
135984	.40	1.32	.022	.030	11	5.313	1.56	2.00	10.53	5.265	1.55	117,000	75,500	171,500	110,650	12	
135984	.40	1.32	.022	.030	11	5.313	1.56	2.00	10.51	5.255	1.55	114,500	73,850	171,500	110,650	12	
323632	.35	1.10	.020	.038	11	5.313	1.56	1.98	10.45	5.278	1.55	110,500	71,300	164,500	106,150	17	
323632	.35	1.10	.020	.038	11	5.313	1.56	1.99	10.49	5.271	1.55	110,500	71,300	164,000	105,800	15	
37248	.35	1.00	.019	.033	11	5.313	1.56	2.00	10.34	5.170	1.52	108,500	71,400	151,000	99,350	15	
37248	.35	1.00	.019	.033	11	5.313	1.56	2.00	10.34	5.153	1.52	100,500	66,100	151,500	99,650	17	

NED [illegible]

MILTON [illegible] CO. Milton Pennsylvania

780

Report of Chemical and Physical Test of Reinforcing Bars

DATE 2-27-69

CUSTOMER Tyler-DAVIS INC.

THEIR ORDER NO.

CONTRACTOR

OUR ORDER NO.

ROLLED TO MEET SPECIFICATIONS GRADE 60"

JOB

	HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG
1	32274	[illegible]	1.41	.022	.030	11	5.313	1.56	2.01	10.47	5.209	1.53	106,000	69,300	154,500	100,980	8
2	[illegible]	[illegible]	1.41	.021	.030	11	5.313	1.56	2.01	10.52	5.234	1.54	106,000	68,831	163,000	105,844	8
3	[illegible]	[illegible]	1.39	.022	.022	11	5.313	1.56	2.01	10.65	5.299	1.56	112,000	71,794	168,500	108,012	9
4	[illegible]	[illegible]	1.39	.022	.022	11	5.313	1.56	2.01	10.65	5.299	1.56	106,000	67,948	169,500	108,653	9
5	[illegible]	[illegible]	1.40	.020	.026	11	5.313	1.56	2.01	10.64	5.294	1.56	105,000	67,307	167,000	107,051	10
6	[illegible]	[illegible]	1.40	.020	.026	11	5.313	1.56	2.00	10.55	5.275	1.55	113,500	73,225	166,500	107,419	10
7	136907	.37	1.44	.022	.036	11	5.313	1.56	1.99	10.61	5.332	1.57	120,500	76,751	176,500	112,420	8
8	136907	.37	1.44	.022	.036	11	5.313	1.56	2.00	10.68	5.340	1.57	123,500	78,662	177,000	112,738	8
9	3227-3	.37	1.45	.020	.028	11	5.313	1.56	2.01	10.67	5.308	1.56	108,000	69,871	168,500	108,012	9
10	3227-3	.39	1.45	.020	.028	11	5.313	1.56	2.00	10.60	5.300	1.56	109,500	70,192	168,500	108,012	9
11																	
12																	
13																	
14																	
15																	

12 8 71

SIGNED [illegible]

MILTON, PENNS[illegible]NIA, 17847

764

Report of CHEMICAL and PHYSICAL Test of REINFORCING BARS March 7, 1969

Charged to Taylor-Davis, Corporation Car Number PRR 441560

Shipped to Wilmington, Delaware

Specification GRADE: 60 Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS. PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND TEST
	~~2-24~~	~~136953~~	~~10~~	~~40~~	~~1134~~	~~022~~	~~034~~	~~76400~~	~~117830~~	~~11~~	~~17377~~	~~01~~
		~~233501~~	~~10~~	~~39~~	~~1136~~	~~020~~	~~037~~	~~65850~~	~~109900~~	~~11~~	~~31,196~~	~~01~~
		136907	11	39	1.44	022	036	72900	115150	10	18,736	01
		137010	11	40	1134	022	032	61050	109950	9	9368	01
	3-4	322764	11	39	1.41	021	030	68850	109100	7	18731	01
		322203	11	39	1145	020	028	69350	111600	9	18736	01
		322220	11	40	1140	020	026	65410	108910	10	9715	01

We certify that the above figures are correct as contained in the company records. [signature]

Report of Chemical and Physical Test of Reinforcing Bars

781

DATE 2-13-69

CUSTOMER

THEIR ORDER NO.

CONTRACTOR

OUR ORDER NO.

ROLLED TO MEET SPECIFICATIONS GRADE 60

JOB

HEAT NO.	C	MN.	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR. LBS	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	% ELONG	BREAK	BEND TEST
236431	38	1.30	019	036	#9	3.400	1.00	2.00	6.71	3.355	99	61,500	62,121	105,000	106,060	17		
236436	38	1.36	019	035	#9	3.400	1.00	2.00	6.67	3.335	98	62,000	63,265	104,500	106,633	14		
321695	38	1.33	025	050	#9	3.400	1.00	2.00	6.69	3.345	98	62,500	63,776	105,500	107,653	13		
236429	38	1.38	016	029	#9	3.400	1.00	2.00	6.75	3.375	99	56,000	56,566	105,000	106,060	18		
236458	39	1.40	018	031	#10	4.303	1.27	2.00	8.40	4.200	1.24	61,500	49,597	131,500	106,048	15		
136663	.38	1.33	021	030	#11	5.313	1.56	2.00	10.44	5.220	1.54	95,000	61,688	157,000	101,918	16	✓	
236431	.38	1.30	019	036	#9	3.400	1.00	2.00	6.72	3.360	.99	69,000	69,697	104,500	105,555	16		
236436	38	1.36	019	035	#9	3.400	1.00	2.00	6.66	3.330	.98	63,500	64,796	104,500	106,632	17		
321698	38	1.33	025	050	#9	3.400	1.00	2.00	6.69	3.345	98	66,500	67,857	105,500	107,653	18		
236429	38	1.38	016	029	#9	3.400	1.00	2.02	6.76	3.347	.98	75,000	76,531	105,000	107,150	17		
236458	39	1.40	018	031	#10	4.303	1.27	2.02	8.48	4.198	1.23	90,000	73,171	132,000	107,317	12		
36663	.38	1.33	021	031	#11	5.313	1.56	2.00	10.45	5.225	1.54	94,500	61,364	156,500	101,623	15		
136663	.38	1.33	021	.031	#11	5.313	1.56	2.00	10.65	5.325	1.57	106,500	67,834	172,500	109,873	10		
MAX.	.45	1.30																

SIGNED

SIGNED

5-17

DATE 1-29-71

TAYLOR [illegible] CORP.

THEIR ORDER NO. PS-3206

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC.

OUR ORDER NO. 6150 F49

ROLLED TO MEET SPECIFICATIONS GRADE 60

JOB PUBLIC SERVICE ELECTRIC & GAS CO.

HEAT NO.	C	MN	P	S	SIZE	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	LENGTH OF TEST BAR FT.	WEIGHT OF TEST BAR LBS.	WEIGHT POUNDS PER FT.	SEC. AREA SQ. IN.	YIELD POINT MACHINE READING	YIELD POINT LBS. PER SQ. IN.	TENSILE STRENGTH MACHINE READING	TENSILE STRENGTH LBS. PER SQ. IN.	[illegible]	BREAK
241190	35	1.30	015	039	5	1.043	.31	2.06	2.04	1.020	.30	22,200	71,300	33,400	111,300	14	
241190	35	1.30	015	[illegible]	5	1.043	.31	2.01	2.05	1.020	.30	21,500	71,700	33,600	[illegible]000	15	
141865	36	1.28	012	038	6	1.502	.44	2.00	2.97	1.485	.44	28,200	64,100	43,800	99,500	17	
141865	36	1.28	012	038	6	1.502	.44	2.00	2.97	1.485	.44	28,300	64,300	44,000	100,000	17	
241250	38	1.28	014	033	8	2.670	.79	2.01	5.25	2.612	77	—	—	—			
241250	38	1.28	014	033	8	2.670	.79	2.03	5.31	2.616	77	—	—	—			
327677	40	1.34	014	036	9	3.400	1.00	1.99	6.64	3.337	98	76,000	77,600	110,500	[illegible]	13	
327677	40	1.34	014	036	9	3.400	1.00	2.03	6.78	3.340	98	74,000	75,500	110,500	112,800	14	
326810	39	[illegible]	016	032	10	4.303	1.27	2.02	8.61	4.262	1.25	90,000	72,000	136,500	109,200	11	outside
326810	39	1.33	016	032	10	4.303	1.27	2.00	8.50	4.250	1.25	89,000	71,200	136,500	109,200	17	5-15A-241190
141843	32	1.04	014	030	11	5.313	1.56	1.98	10.43	5.293	1.56	103,000	66,000	151,000	96,800	16	
141843	32	1.04	014	030	11	5.313	1.56	1.99	10.54	5.296	1.56	101,500	65,100	150,500	[illegible]	19	
141842	32	1.03	012	032	11	5.313	1.56	1.99	10.42	5.236	1.54	104,000	67,600	150,000	97,400	18	
141842	32	1.03	012	032	11	5.313	1.56	2.00	10.47	5.235	1.54	100,500	65,300	150,000	97,400	14	

[illegible] SIGNED [illegible] 5-17

[illegible] 1/26/71

182

Report of CHEMICAL and PHYSICAL Test of REINFORCING BARS — February 10, 1969

Charged to Taylor-Davis, Corporation — Car Number

Shipped to Wilmington, Delaware

Specification Grade: 60 — Project

CUSTOMER'S ORDER NO. AND DATE	MILL ORDER NUMBER	HEAT NUMBER	SIZE NUMBER	ANALYSIS C	Mn	P	S	YIELD PT. LBS PER SQ. IN.	TEN. STRENGTH LBS. PER SQ. IN.	ELONG. IN 8" %	WT. USED	BEND TEST
		136663	11	38	1.33	021	030	61688	101948	16	9,245	ok
		~~236458~~	~~10~~	~~39~~	~~1.40~~	~~017~~	~~031~~	~~73171~~	~~107317~~	~~12~~	~~9,036~~	~~ok~~
		~~322580~~	~~5~~	~~43~~	~~1.54~~	~~021~~	~~034~~	~~78310~~	~~120610~~	~~10~~	~~9,074~~	~~ok~~
		~~322446~~	~~6~~	~~39~~	~~1.40~~	~~023~~	~~042~~	~~70410~~	~~110410~~	~~13~~	~~9,012~~	~~ok~~

TESTED AND APPROVED BY PUBLIC SERVICE OF NEW JERSEY

ONLY #10 AND #11 TESTED BY MR. OBERMAIER

We certify that the above figures are correct as contained in the company records. [signature]

This material made in USA by basic electric furnace process [illegible] ification ASTM A15 64 (or A432 64) and ASTM A305 64.

4-44

MILTON MANUFACTURING CO., Milton Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE 2-28-72

CUSTOMER TAYLOR-DAVIS CORP.

THEIR ORDER NO. PS 545-A-B-C-D-581-581A-579

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC

OUR ORDER NO. 7037F46 THRU 53

ROLLED TO MEET SPECIFICATIONS GRADE 60

JOB PUBLIC SERVICE ELECTRIC & GAS CO.

Item	Heat No.	C	MN	P	S	Size	Weight Pounds Per Ft.	Sec. Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds Per Ft.	Sec. Area Sq. In.	Yield Point Machine Reading	Yield Point Lbs. Per Sq. In.	Tensile Strength Machine Reading	Tensile Strength Lbs. Per Sq. In.	% Elong.	Break	Bend Test
1	330393	.33	1.30	014	027	4	.668	.20	2.00	1.32	.660	.19	12.500	65,800	19.900	104,750	17		OK
2	330393	.33	1.30	014	027	4	.668	.20	1.99	1.32	.660	.19	12.500	65,800	19.900	104,750	18		
3	330331	.36	1.33	012	030	5	1.043	.31	1.99	2.03	1.020	.30	18,900	63,000	29,800	99,350	16		
4	330331	.36	1.33	012	030	5	1.043	.31	2.00	2.02	1.010	.30	19,000	63,350	29,900	99,650	15		
5	330450	.36	1.23	015	026	5	1.043	.31	1.99	2.04	1.025	.30	19,300	64,350	31,400	104,650	16		
6	330450	.36	1.23	015	026	5	1.043	.31	1.99	2.05	1.030	.30	19,900	66,350	31,400	104,650	15		
1	144597	.37	1.30	013	034	6	1.502	.44	2.00	2.93	1.465	.43	27,800	64,650	44,400	103,250	17	4 330 393	
3	144597	.37	1.30	013	034	6	1.502	.44	2.00	2.94	1.470	.43	27,700	64,400	44,200	102,800	17		
2	243923	.37	1.23	018	029	7	2.044	.60	2.06	3.93	1.965	.58	40,000	68,950	60,200	103,800	17		
1	243923	.37	1.23	018	029	7	2.044	.60	2.00	3.93	1.965	.58	38,900	67,050	60,400	104,150	17		
2	240960	.39	1.40	012	025	8	2.670	.79	2.00	5.27	2.635	.78	51,000	65,400	79,000	101,300	15		
2	240960	.39	1.40	012	025	8	2.670	.79	2.00	5.27	2.635	.78	50,000	64,100	79,500	101,900	18		
1	144691	.37	1.30	015	025	8	2.670	.79	2.00	5.17	2.585	.76	49,000	64,450	78.500	103,300	17		
2	144691	.37	1.30	015	025	8	2.670	.79	1.99	5.14	2.583	.76	50,000	65,800	79,000	103,950	16		
3	144076	.39	1.39	019	030	9	3.400	1.00	2.00	6.72	3.360	.99	69,000	69,700	105,030	106,050	17	4-44	

Witnessed by R. Herren

SIGNED [illegible]

2/6/69

1019
0062

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT [illegible]	ORDER NO.			
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
322064A 1	A 432	322064A	10	1.27
322064A 2	"	"	"	"
322064B 1	"	322064B	"	"
322064B 2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
1/20/69	322064A-1	81,600	64,300	134,400	105,800	14.5
1/20/69	322064A 2	82,400	64,900	134,400	105,800	11.0
1/20/69	322064B-1	82,000	64,900	132,800	104,600	16.0
1/20/69	322064B 2	81,300	63,700	132,400	104,300	16.0

NOTE 1. DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY [illegible] Inc. | Witnessed By [illegible]

Remarks: [illegible] A432 [illegible]

[illegible signature]
LABORATORY ENGINEER

[illegible signature]
MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

1021
0061

PROJECT [illegible]			ORDER NO.	
IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
235985A-1	A432	235985A	10	1.27
235985A-2	"	"	"	"
235985B-1	"	235985B	"	"
235985B-2	"	"	"	"

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
[illegible]	235985A-1	78,000	61,400	133,200	104,900	13.0
[illegible]	235985A-2	79,800	[illegible] 800	133,000	104,800	17.0
[illegible]	235985B-1	79,400	62,500	131,200	103,300	14.0
[illegible]	235985B-2	78,400	61,700	131,300	103,400	15.5

NOTE 1. DIVIDER OVER 8 INCHES OR ~~DROP OF~~ BEAM

TESTED BY: [illegible]	[illegible]

Remarks: Results Meet A432 Specification

LABORATORY ENGINEER

MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

ELECTRIC [illegible] PARTMENT

TESTING LABORATORY REPORT

1051

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REINFORCEMENT BAR TEST

PROJECT SALEM NUCLEAR GEN. STA.		ORDER NO.		DATE 7-29-69
IDENTIFICATION NO.	MATERIAL	HEAT NO.	[illegible]	AREA
235J32-1	A 432	235J32	8	.77
235J32-2				

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES %
7-29-69	235J32-1	55,000	71,410	82,500	107,150	12
"	235J32-2	58,000	75,350	83,000	107,800	16

NOTE 1 - DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY MILTON MFG. CO. — WITNESSED BY: [illegible]

REMARKS: RESULTS MEET A432 REQUIREMENTS GRADE 60

[illegible]
MATERIALS DIVISION CHIEF

[illegible]
LABORATORY ENGINEER

[illegible] REV. 8/68

[illegible] Pennsylvania

Report of Chemical and Physical Test of Reinforcing Bars

DATE [illegible]

CUSTOMER TAYLOR - DAVIS CORP. THEIR ORDER NO. PS 586D

CONTRACTOR UNITED ENGINEERS & CONSTRUCTORS INC OUR ORDER NO. 7037F36

ROLLED TO MEET SPECIFICATIONS GRADE 60 JOB PUBLIC SERVICE ELECTRIC & GAS CO.

Item	Heat No.	C	Mn	P	S	Size	Weight Pounds per Ft.	Sec. Area Sq. In.	Length of Test Bar Ft.	Weight of Test Bar Lbs.	Weight Pounds per Ft.	Sec. Area Sq. In.	Yield Point Machine Reading	Yield Point Lbs. per Sq. In.	Tensile Strength Machine Reading	Tensile Strength Lbs. per Sq. In.	% Elong.	Break	Bend Test
1	329314	.38	1.30	.015	.023	5	1.043	.31	1.99	2.01	1.010	.30	20.200	67.350	31,100	103650	15		OK
2	329314	.38	1.30	.015	.023	5	1.043	.31	1.99	2.01	1.010	.30	20.200	67.350	31.200	104,000	15		
3	143668	.38	1.35	.016	.034	6	1.502	.44	1.99	2.89	1.452	.43	31.400	73.000	45.600	106.850	13		
4	143668	.38	1.35	.016	.034	6	1.502	.44	2.00	2.93	1.450	.43	34.600	80.450	45.600	106.050	14		
5	329756	.37	1.34	.017	.036	7	2.044	.60	1.99	4.01	2.015	.59	37.700	63.900	61.600	104.400	16		
6	329756	.37	1.34	.017	.036	7	2.044	.60	1.99	4.01	2.015	.59	37.800	64.050	61.600	104.400	17		
7	243627	.38	1.39	.016	.033	8	2.670	.79	1.99	5.29	2.658	.78	55.000	70.500	82.500	105.750	16		
8	243627	.38	1.39	.016	.033	8	2.670	.79	1.99	5.29	2.658	.78	52.000	66.650	82.500	105.750	16		
9																			
10																			
11																			
12																			
13																			
14																			
15																			

[illegible] SIGNED [illegible]

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0155

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT: SALEM NUCLEAR GEN. STA. ORDER NO.

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
236884-1	A-432	236884	8	0.787
236884-2	A-432	236884	8	0.774

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES
		LBS.	P.S.I.	LBS.	P.S.I.	
11/9/69	236884-1	57,500	73,700	84,500	108,950	14
11/9/69	236884-2	58,000	73,400	84,500	106,950	11

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY: Milton Manufacturing Co. | WITNESSED BY: H. Oberman

Remarks: Results Meet A-432 Specification

[signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

5-63

Taylor-Dr[illegible] Corporation

Wilmington, Delaware

Specification GRADE 60

March 2[illegible], 197[illegible]

Car Number

Project [illegible]S/L32131, 32137

Customer's Order No. and Date	Mill Order Number	Heat Number	Size Number	C	Mn	P	S	Yield Pt. Lbs. per Sq. In.	Tensile Strength Lbs. per Sq. In.	Elong.	Wt.	Bend Test
[illegible]	[illegible]	244136	8	37	[illegible]26	018	028	61050	99350	17	30277	OK
[illegible]	[illegible]	144854	8	37	1.31	016	044	67100	107250	15	10093	[illegible]
[illegible]	[illegible]	330780	5	37	1.34	017	031	65150	102400	15	10013	[illegible]
[illegible]	[illegible]	144738	5	39	1.28	014	050	65650	103650	14	10013	[illegible]
[illegible]	[illegible]	330778	5	37	1.35	015	047	64000	101650	[illegible]	10013	[illegible]
[illegible]	[illegible]	243920	5	37	1.11	018	037	66200	103100	16	10013	OK

5-50A-330780

5-50B-330778

Recvd. for cert.

5-62

5-6[illegible]

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104
0156

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR GEN. STA. | ORDER NO

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
323151-1	A432	323151	8	0.784
323151-2	A432	323151	8	0.7[illegible]

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES
		LBS	PSI	LBS	PSI	
4/9/69	323151-1	62,000	79,300	81,500	104,500	14
4/9/69	323151-2	58,000	73,400	82,000	103,800	17

NOTE 1. DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY Milton Manufacturing Co. | WITNESSED BY H. [illegible]

REMARKS - RESULTS MEET A432 SPECIFICATION

[signature]
LABORATORY ENGINEER

[signature]
MATERIALS DIVISION CHIEF

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

7084

0089

PROJECT SALEM NUCLEAR GEN STA.	ORDER NO.

IDENTIFICATION NO.	MATERIAL	[illegible]	[illegible]	[illegible]
235722 A-1	A-432	235722	8	[illegible]
235722 A-2	A-432	235722	8	[illegible]

TENSION TEST RESULTS

DATE	IDINT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION IN 8 INCHES
		LBS.	[illegible]	LBS.	[illegible]	
3/6/69	235722A-1	53,200	67,340	87,300	110,510	10.1
"	235722A-2	54,100	68,480	87,800	111,140	9.0

NOTE 1 — DIVIDER ... INCHES ... DROP ...

TESTED BY: Public Service Testing Laboratory

Remarks: Results Meet A-432 Requirements

LABORATORY ENGINEER

MATERIALS DIVISION

1088

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT SALEM NUCLEAR [illegible]	ORDER NO. [illegible]

IDENTIFICATION NO.	MATERIAL	HEAT [illegible]	SIZE	[illegible]
136111-1	A432	[illegible]	8	[illegible]
136111-2			8	

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1)		ULTIMATE		ELONGATION [illegible] 8 INCHES, %
		LBS.	PSI	LBS.	PSI	
7/10/69	136111-1	49,600	62,800	81,400	103,000	14.5
7/10/69	136111-2	48,600	61,500	81,400	103,000	16.0

NOTE 1: DIVIDER OVER 8 INCHES ~~OR DROP OF BEAM~~

TESTED BY United States Testing Co. Inc. | WITNESSED BY A. C. [illegible]

Remarks: Results meet A432 Specification

[illegible]
LABORATORY ENGINEER

[illegible]
MATERIALS DIVISION [illegible]

1089

0059

PUBLIC SERVICE ELECTRIC AND GAS COMPANY

Testing Laboratory

REINFORCEMENT BAR TEST REPORT

PROJECT: SALEM NUCLEAR [illegible]	ORDER NO. [illegible]

IDENTIFICATION NO.	MATERIAL	HEAT NO.	SIZE	AREA
136068	[illegible]	136068	[illegible]	[illegible]
136068-2				

TENSION TEST RESULTS

DATE	IDENT. NO.	YIELD (NOTE 1) LBS.	YIELD (NOTE 1) PSI	ULTIMATE LBS.	ULTIMATE PSI	ELONGATION IN 8 INCHES, %
2/10/69	136068-1	49,800	63,000	81,000	102,500	10.5
2/10/69	136068-2	48,000	60,800	80,800	102,300	15.0

NOTE 1: DIVIDER OVER 8 INCHES OR DROP OF BEAM

TESTED BY United States Testing Co. Inc. | Witnessed by: [illegible]

Remarks: [illegible]

HC Miller
LABORATORY ENGINEER

[illegible]
MATERIALS DIVISION CHIEF

C
Margin Reduction from Rebar Corrosion

This appendix contains the following MPR Calculation.

- MPR Calculation 0108-0275-35, "Salem Spent Fuel Pool Reinforcing Steel Load Capacity at Degraded Conditions," Revision 0.

MPR

MPR Associates, Inc.
320 King Street
Alexandria, VA 22314

CALCULATION TITLE PAGE

Client: PSEG Nuclear	
Project: Salem Spent Fuel Pool Leakage	Task No. 0108-0303-0275
Title: Salem Spent Fuel Pool Reinforcing Steel Load Capacities at Degraded Conditions	Calculation No. 0108-0275-35

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
Michelle Heinz 12-29-03 Michelle Heinz	Lisa Lichtenauer 12/29/03 Lisa Lichtenauer	12/29/03 Robert Keating	0

QUALITY ASSURANCE DOCUMENT
This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR	MPR Associates, Inc. 320 King Street Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0108-0275-35	M. Henig	L. Lichtenauer	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

The purpose of this calculation is to determine the percent reductions in the Salem spent fuel pool reinforcing steel load capacities due to various levels of reinforcing steel degradation.

2.0 RESULTS

Table 2-1 shows the percent reductions in the load capacities of the reinforcing steel sizes present in the spent fuel pool structure due to various levels of reinforcing steel degradation.

Table 2-1. Percent Reductions in Reinforcing Steel Load Capacity at Various Degradation Levels

	Size	#8	#9	#10	#11
	Diameter (In.)	1.000	1.128	1.270	1.410
Degradation (inches)	0.000	0.00	0.00	0.00	0.00
	0.010	1.99	1.77	1.57	1.41
	0.020	3.96	3.51	3.12	2.82
	0.030	5.91	5.25	4.67	4.21
	0.040	7.84	6.97	6.20	5.59
	0.050	9.75	8.67	7.72	6.97
	0.060	11.64	10.36	9.23	8.33
	0.070	13.51	12.03	10.72	9.68
	0.080	15.36	13.68	12.20	11.03

The expressions relating the percent reduction in reinforcing steel load capacity and the reinforcing steel degradation level are provided below. In each equation, 'x' represents the reinforcing steel degradation level, and 'y' represents the percent reduction in load capacity.

#8: $y = 192x + 0.0933$
#9: $y = 171.02x + 0.0734$
#10: $y = 152.52x + 0.0579$
#11: $y = 137.82x + 0.0469$

3.0 CALCULATION

The yield strength of reinforcing steel, f_y, is defined as the force that the steel can withstand at a certain cross-sectional area:

$$f_y = F/A_s \tag{1}$$

where:

$$A_s = \frac{\pi d^2}{4} \tag{2}$$

d = reinforcing steel diameter
F = reinforcing steel load capacity

Substituting Equation (2) into Equation (1) and solving for the load capacity yields:

$$F = \frac{f_y \pi d^2}{4} \tag{3}$$

The percent reduction in the original load capacity due to degradation of the reinforcing steel is calculated as:

$$\%\,\text{Reduction} = \left[1 - \frac{F_{\text{With-Degradation}}}{F_{\text{No-Degradation}}}\right] * 100 = \left[1 - \frac{(d - d_d)^2}{d^2}\right] * 100 \tag{4}$$

where:

d_d = reinforcing steel degradation level

Note that d_d represents the degradation off the reinforcing steel diameter, and not the radius.

The reinforcing steel sizes present in the Salem spent fuel pool structure are provided in References 1-9. This information is summarized in Table 3-1. Note that only the reinforcing steel around the wetted perimeter of the structure is included in the table (e.g., reinforcing steel in the portion of the North wall above the water is not included).

Table 3-1. Spent Fuel Pool Steel Reinforcement Sizes

Wall	Horiz, Tension Inside	Horiz, Tension Outside	Vert, Tension Inside	Vert, Tension Outside
North	#8	#8	#8	#8
West	#9	#8	#9	#8
South	#11	#11	#11	#11
East	#10	#8	#10	#8
	N-S, Tens. Top	N-S, Tens Bottom	E-W, Tens. Top	E-W, Tens. Bottom
Slab	#9	#8	#8	#8

Notes: 1. 'Inside' refers to the water side of the wall. 'Outside' refers to the side of the wall remote from the water.

The diameters of the reinforcing steel under consideration in this calculation are provided in Table 3-2. This information is taken from Reference 10.

Table 3-2. Reinforcing Steel Diameters

Size	Diameter
#8	1.000
#9	1.128
#10	1.270
#11	1.410

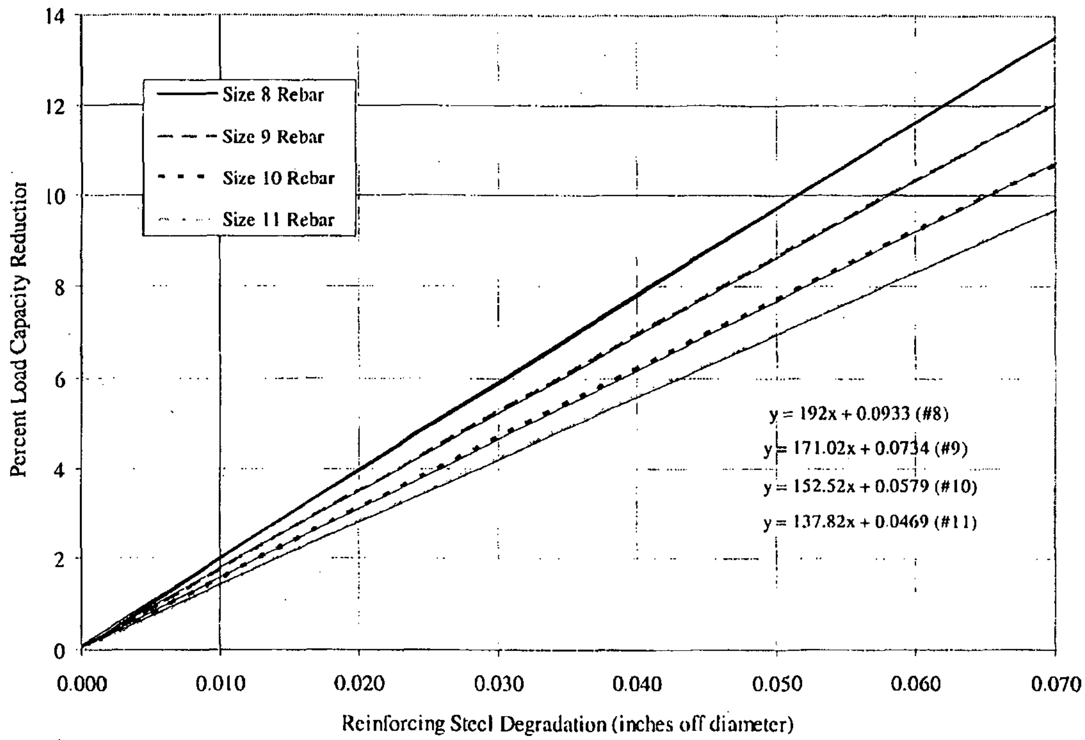
Degradation, d_d in Equation (4), is applied in increments of 0.01 inches to determine the effect on the capacity of each size of reinforcing steel. Results are presented in Table 3-3.

Table 3-3. Percent Reductions in Reinforcing Steel Load Capacity at Various Degradation Levels

	Size	#8	#9	#10	#11
	Diameter (in.)	1.000	1.128	1.270	1.410
Degradation (inches)	0.000	0.00	0.00	0.00	0.00
	0.010	1.99	1.77	1.57	1.41
	0.020	3.96	3.51	3.12	2.82
	0.030	5.91	5.25	4.67	4.21
	0.040	7.84	6.97	6.20	5.59
	0.050	9.75	8.67	7.72	6.97
	0.060	11.64	10.36	9.23	8.33
	0.070	13.51	12.03	10.72	9.68
	0.080	15.36	13.68	12.20	11.03

Figure 3-1 shows a plot of the percent reduction in reinforcing steel load capacities versus degradation for the various reinforcing steel sizes present in the spent fuel pool. Included on the plot are equations relating the percent load capacity reduction of each reinforcing steel size to the degradation level.

Figure 3-1. Reductions in Reinforcing Steel Load Capacities at Various Diameter Degradation Levels



4.0 References

1. PSEG Nuclear Drawing No. 201075 A 8706-2, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 78'-0"."

2. PSEG Nuclear Drawing No. 201076 A 8706-4, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 84'-0"."

3. PSEG Nuclear Drawing No. 201077 A 8706-8, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 100'-0" and 116'-0"."

4. PSEG Nuclear Drawing No. 201078 A 8706-4, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 130'-0"."

5. PSEG Nuclear Drawing No. 201079 A 8706-3, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Roof Plan."

6. PSEG Nuclear Drawing No. 201080 A 8706-7, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections A-A & B-B."

7. PSEG Nuclear Drawing No. 201081 A 8706-6, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections C-C, D-D & E-E."

8. PSEG Nuclear Drawing No. 201082 A 8706-5, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections F-F & G-G."

9. PSEG Nuclear Drawing No. 201085 A 8706-5, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Elevation P-P & Str. Bar Schedule."

10. Spiegel, Leonard, and George F. Limbrunner, "Reinforced Concrete Design," Prentice-Hall, Inc.: Englewood Cliffs, NJ, 1980.

D

Margin Reduction from Concrete Degradation

This appendix contains the following MPR Calculation.

- MPR Calculation 0108-0275-34, "Salem Spent Fuel Pool Structure Capacities Based on Degraded Concrete Conditions," Revision 0.

MPR

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CALCULATION TITLE PAGE

Client: PSEG Nuclear	Page 1 of 15 + Appendices
Project: Salem Spent Fuel Pool Leakage	Task No. 0108-0303-0275
Title: Salem Spent Fuel Pool Structure Capacities Based on Degraded Concrete Conditions	Calculation No. 0108-0275-34

Preparer / Date	Checker / Date	Reviewer & Approver / Date	Rev. No.
Michelle Heinz 12-29-03 Michelle Heinz	Lisa Lichtenauer 12/29/03 Lisa Lichtenauer	Rob Keating 12/29/03 Robert Keating	0

QUALITY ASSURANCE DOCUMENT

This document has been prepared, checked, and reviewed/approved in accordance with the Quality Assurance requirements of 10CFR50 Appendix B, as specified in the MPR Quality Assurance Manual.

MPR

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Alexandria, VA 22314

RECORD OF REVISIONS

Calculation No.	Prepared By	Checked By	Page: 2
0108-0275-34	M. Ahmed	L. Lichtenauer	

Revision	Affected Pages	Description
0	All	Initial Issue

Note: *The revision number found on each individual page of the calculation carries the revision level of the calculation in effect at the time that page was last revised.*

Table of Contents

1.0 PURPOSE

The purpose of this calculation is to determine the percent reductions in the capacities of the limiting sections of the Salem spent fuel pool structure due to various levels of concrete degradation in the structure.

2.0 RESULTS

2.1 *Moment Capacities*

Table 2-1 shows the percent reductions in the moment capacities of the limiting sections of each wall of the spent fuel pool at various levels of concrete degradation. Also shown in the table are the total moments, allowable moments, and design margins of the limiting sections of each wall with no concrete degradation.

Table 2-1. Percent Reduction in the Allowable Moment of the Spent Fuel Pool Walls

Wall	No Concrete Degradation			% Reduction in Allowable Moment at Various Concrete Degradation Levels (in inches)				
	Total Moment (kip-ft/ft)*	Allowable Moment (kit-ft/ft)	Limiting Design Margin	1"	2"	3"	4"	5"
North	-148	-154	1.04	1.0	2.0	3.0	4.0	5.0
South	-86	-258	3.00	1.6	3.1	4.7	6.2	7.8
East	-98	-103	1.05	1.5	3.0	4.5	6.0	7.5
West	-294	-299	1.02	0.92	1.8	2.8	3.7	4.6
Slab	-191	-197	1.03	0.79	1.6	2.4	3.2	3.9

*Note that a negative moment indicates compression on the inside (water side) of the pool wall, and tension on the outside of the pool wall (per Appendix C, Page 13 of Reference 1).

The expressions relating the percent reduction in moment capacity and the concrete degradation level are provided below. In each equation, 'x' represents the concrete degradation level, in inches, and 'y' represents the percent reduction in allowable moment.

North Wall: $y = 1.006x + 0.0003$

South Wall: $y = 1.5523x + 0.0013$

East Wall: $y = 1.5042x + 0.0007$

West Wall: $y = 0.9167x + 0.0002$

Slab: $y = 0.7894x + 0.0001$

Note that Table 2-1 only addresses the walls of the spent fuel pool. The walls of the transfer pool, located beside the spent fuel pool, are not expected to show significantly larger reductions in moment capacities.

2.2 Shear Capacities

The spent fuel pool wall design margins for shear are sufficiently high that concrete degradation will not have an impact. The spent fuel pool walls are not evaluated in detail for shear in this calculation.

3.0 METHODOLOGY

The design analysis of the Salem spent fuel pool building was performed in MPR-1863 (Reference 1) based on the requirements specified in the Salem Structural Design Criteria (Reference 2). The spent fuel pool was divided into approximately seventy sections for the evaluation. Loads and design margins were calculated for each section.

This calculation evaluates only the one section of each spent fuel pool wall having the most limiting design margin for moment. These limiting sections are determined from Reference 1. The depth of concrete degradation is varied by increments of 0.25" for each limiting section, and the reduction in moment capacity based on the degraded concrete conditions is calculated. Shear capacities at degraded concrete conditions are not specifically evaluated in this calculation, as discussed in Section 4.

Although all sections of the spent fuel pool were evaluated for various load combinations, load types, and load directions in Reference 1, it is justifiable to evaluate only the most limiting section of each wall from among all the load combinations, load types, and load directions based on the following:

- Capacities are limited by either a multiple of the working stress allowable or the ultimate strength design allowable, depending on the load combination under consideration, per Reference 2, Paragraph 7.2.1. The moment arm of a concrete section is determined by the distance between the centroids of the compression block and tension reinforcement. The compression block is triangular in the working stress design method and rectangular in the ultimate strength design method. Therefore, changes in the overall depth of a concrete section (e.g., resulting from concrete degradation) would have a similar effect on the moment arms resulting from both design methods, and the percent reductions in moment capacities would be comparable. It is noted that all limiting design margins evaluated in this calculation result from the working stress design method. The design margins based on ultimate strength design are much larger than those based on working strength design (limiting design margins from Appendix C of Reference 1 are 1.02 and 1.27 based on working stress and ultimate strength capacities, respectively).

- The load type (horizontal moment, vertical moment, etc.) determines whether the vertical or horizontal rebar carry load. The size and spacing of the rebar in a wall of the structure, which determines the effective area of the rebar, may vary depending on whether the rebar is oriented horizontally or vertically. However, the percent reduction in moment capacity is most significantly influenced by the depth of the concrete section and is relatively insensitive to the effective area of the steel.

- The direction of a load will determine whether the inside rebar (i.e., on the water side of the wall) or the outside rebar are in tension. Although the size and spacing of the rebar in a wall of the structure may vary depending on whether the rebar is on the inside or outside of a wall, the percent reduction in moment capacity is most significantly influenced by the depth of the concrete section and is relatively insensitive to the effective area of the steel. It is noted that all limiting design margins evaluated in this calculation result from compression of concrete on the inside of the structure, and tension of rebar on the outside of the structure.

As stated above, the wall depth has a significant impact on moment capacity. PSEG Nuclear drawings (References 5-13) show that the South wall is the only wall not constant in depth from the bottom to the top. However, it is still justifiable to evaluate only one most limiting section from the South wall as all sections have significantly large design margins, per Appendix C of Reference 1.

4.0 CALCULATION

4.1 Shear

Reference 2, Paragraph 7.2.1, stipulates that shear capacities for the spent fuel pool wall be calculated according to ACI 318-63 (Reference 3). Reference 2 limits normal loads to working stress allowables; operating basis earthquake loads to 1/3 above working stress allowables; and design basis earthquake and tornado related loads to ultimate strength design allowables. Per

Reference 3, Paragraph 1201(a) and Paragraph 1701(a), working stress and ultimate strength design allowables for shear loads are calculated as

$$V = v_c bd$$

where:

V = allowable shear load for working stress or ultimate strength design
v_c = allowable shear stress for unreinforced concrete
$= 1.1\sqrt{f_c'}$ for working stress design (Reference 3, Paragraph 1201(c))
$= 2\phi\sqrt{f_c'}$ for ultimate strength design (Reference 3, Paragraph 1701(c))
b = width of compression face
d = distance from extreme compression fiber to centroid of tension reinforcement
f_c' = concrete compressive strength
ϕ = capacity reduction factor

The above equation shows that shear capacity is a function of the cross-sectional area of the concrete section under consideration, such that the percent reduction in shear capacity is directly proportional to the percent reduction in the depth of the section.

Appendix C of Reference 1 shows that the design margins for shear are large; the most limiting is 1.99. Therefore, the spent fuel pool walls will still be acceptable for shear even under degraded concrete conditions, and the effect of concrete degradation on the shear capacities of the spent fuel pool walls is not specifically evaluated in this calculation.

4.2 Moment

4.2.1 Limiting Spent Fuel Pool Sections

Table 4-1 shows the design margin, load information, and location of the most limiting section of each spent fuel pool wall considering no concrete degradation. All values in the table are from Appendix C of Reference 1.

Table 4-1. Limiting Design Margins of Each Spent Fuel Pool Wall

Location			Load Combination	Load Type	Total Moment (kip-ft/ft)	Capacity (kit-ft/ft)	Margin
Wall	Along Wall	Height					
North	Middle	Bottom	Normal Operation	Vertical Moment	-148	-154	1.04
South	West	Bottom	Normal Operation	Horizontal Moment	-86	-258	3.00
East	Middle	Bottom	Normal Operation	Vertical Moment	-98	-103	1.05
West	Middle	Top	East-West OBE	Horizontal Moment	-294	-299	1.02
Slab	Middle	Middle	Normal Operation	Horizontal Moment	-191	-197	1.03

Table 4-1 shows that the limiting design margins result from only two load combinations – Normal Operation and East-West Operating Basis Earthquake (OBE). Per Reference 2, Paragraph 7.2.1, the moment capacities for the spent fuel pool structure are to be calculated according to Reference 3. Per Reference 3, moment capacities are limited to working stress allowables for normal loads and 1/3 above working stress allowables for OBE loads. Therefore, the working stress design method is used in this calculation to determine the percent reduction in the moment capacities of the limiting spent fuel pool sections when the concrete of those sections is degraded.

4.2.2 Working Stress Design Method

The working stress design method (detailed in Reference 4) assumes that the area of the reinforcing steel in a structure may be thought of as replaced by an equivalent area of concrete, scaled by the ratio of the moduli, n:

$$n = \frac{E_s}{E_c} \qquad (1)$$

where:

E_c = concrete modulus of elasticity
E_s = reinforcing steel modulus of elasticity

This is illustrated in Figure 4-1.

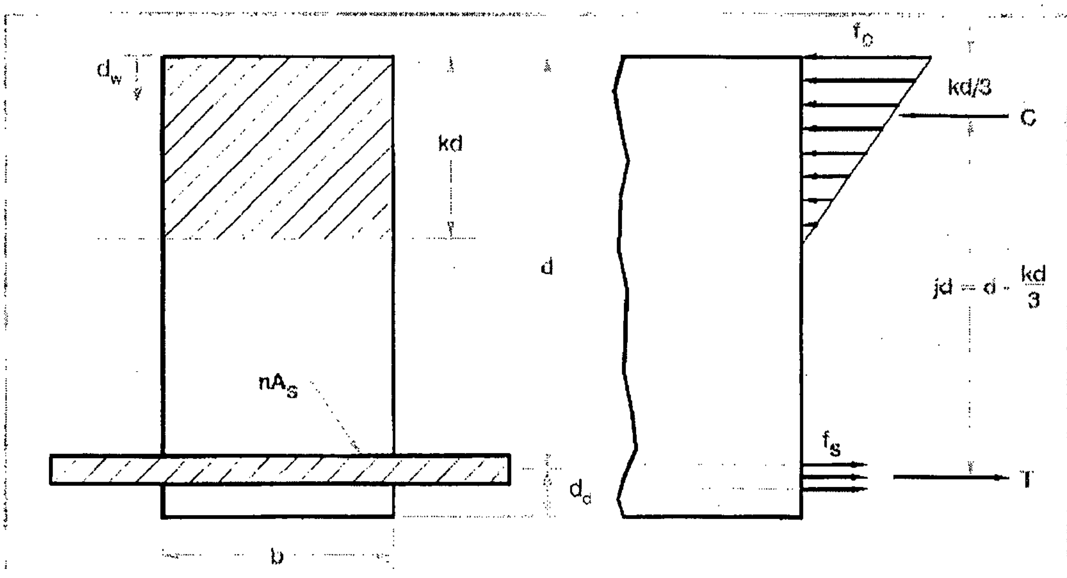

Figure 4-1. Working Stress Design Method

In order to find the neutral axis, designated by 'kd' in Figure 4-1, the moments of the compression and tension areas about the neutral axis are balanced, giving

$$b\frac{(kd)^2}{2} - nA_s(d - kd) = 0$$

where:

A_s = equivalent area of tension reinforcement
d = distance from extreme compression fiber to centroid of tension reinforcement
b = width of concrete section
k = ratio of the distance from the extreme compression fiber to the neutral axis and the distance 'd'

If the reinforcing ratio is

$$\rho = \frac{A_s}{bd} \qquad (2)$$

Then

$$b\frac{(kd)^2}{2} - \rho nbd(d - kd) = 0$$

Solving for k gives

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \qquad (3)$$

If the concrete is cracked and is not supporting tension, the stress profile resembles that shown in Figure 4-1, and the distance between the compressive and tensile forces is

$$jd = d - \frac{kd}{3}$$

Dividing through by 'd' yields

$$j = 1 - \frac{k}{3} \qquad (4)$$

The moment balance requires that the compression force equals the tension force ($C = T$), and that the external bending moment M be equal to $Tjd = Cjd$. If yielding of the steel is limiting, and $T = A_s f_s$, then

$$M = A_s f_s jd \qquad (5)$$

where:

f_s = allowable stress of steel for working stress design

If the concrete limits, then $C = ½f_c'kdb$, and

$$M = ½f_c' k\, jd^2 b$$

Where:

f_c' = concrete compressive strength

Per References 5-13, the walls of the spent fuel pool are very lightly reinforced (reinforcing steel of size #11 or less is used). Therefore, this analysis assumes that yielding of the steel will limit the failure.

4.2.3 Capacities with Concrete Degradation

Equations

The equations used to calculate the reduced moment capacities of the limiting spent fuel pool sections are summarized below. Note that equations (1) through (5) were developed in Section 4.2.

Equation (1) – Modulus Ratio

$$n = \frac{E_s}{E_c}$$

Equation (2) - Reinforcing Ratio

$$\rho = \frac{A_s}{bd}$$

Equation (3) – Neutral Axis Location

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$

Equation (4) – Distance between Compressive and Tensile Forces

$$j = 1 - \frac{k}{3}$$

Equation (5) – Working Stress Design Moment (Allowable Moment)

$$M = A_s f_s jd$$

Equation (6) - Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement

$$d = L - (d_d + d_w)$$

where:

L = full depth of concrete section
d_d = depth of steel from the nearest wall edge (refer to Figure 4-1)
d_w = depth of concrete degradation (refer to Figure 4-1)

Equation (7) - Percent Reduction in Original Moment Capacity

$$\%\,\text{Reduction} = 100\left(1 - \frac{\text{MomentCapacity}_{\text{with-degradation}}}{\text{MomentCapacity}_{\text{no-degradation}}}\right)$$

The results of these calculations for each limiting spent fuel pool section are provided in Appendix A. A plot of the percent reductions in moment capacities versus the concrete degradation levels is provided in Appendix B. Equations relating the percent reductions in moment capacities to the concrete degradation level are shown on the plot.

Material Properties

The material properties to be used in equations (1) through (7) to calculate the reduction in the moment capacities of the limiting wall sections at degraded conditions are listed below, along with references for each.

f_c'	= strength of concrete	= 3,500 psi	(Reference 5)
f_y	= yield strength of steel	= 60,000 psi	(Reference 5)
E_s	= modulus of steel	= 29×10^6 psi	(Reference 3, Paragraph 1100)
f_s	= steel allowable stress	= 24,000 psi	(Reference 3, Paragraph 1003(a))

Note that Paragraph 1003(a) of Reference 3 limits the allowable stress of steel in working stress design, f_s, to 24,000 psi for deformed bars with a yield strength of 60,000 psi or more and in sizes #11 and smaller reinforcing steel. Per References 5-13, all reinforcing steel in the spent fuel pool structure is smaller than #11.

The modulus of concrete, E_c, is calculated as

$E_c = w^{1.5} 33 (f'_c)^{1/2} = 3.41 \times 10^6$ psi (Reference 3, Paragraph 1102(a))

where

$w = 145\ \text{lbs/ft}^3$ (Reference 3, Paragraph 1102(a))

Dimensions

The spent fuel pool wall and reinforcing steel dimensions to be used in equations (1) through (7) to calculate the reductions in moment capacities of the limiting spent fuel pool sections are discussed in this section. Note that the limiting section location information and the reinforcing steel direction and location information provided in Table 4-2 are taken from Appendix C of Reference 1.

The full depth of the limiting sections of the spent fuel pool walls, L, is determined from References 5-13. Because the values of the depths of steel from the nearest wall edges shown in References 5-13 are to the edges of the reinforcement steel, d_d is calculated by summing the values of the depths from References 5-13 and ½ the diameter of the steel reinforcement, taken from Reference 14. The dimensions used in the calculations are summarized in Table 4-2.

The size and spacing of reinforcing steel within a structure determine the equivalent area, A_s, of the steel. The size and spacing of the reinforcing steel in the limiting section of each spent fuel pool wall is provided in Table 4-2. This information was provided by References 5-13. Table 4-2 also shows the equivalent areas of the reinforcing steel, taken from Reference 14.

Table 4-2. Wall and Reinforcing Steel Dimensions

Limiting Section Location			Reinforcing Steel Direction	Reinforcing Steel Location*	Reinforcing Steel Size & Spacing	Reinforcing Steel Diameter (in.)	Full Depth of Wall, L (in.)	Depth of Steel, d_d (in.)	Equivalent Area, A_s (in^2)
Wall	Along Wall	Height							
North	Middle	Bottom	Vertical	Outside	#8@12"	1.00	105	4.0	0.79
South	West	Bottom	Horizontal	Outside	#11@9"	1.41	72	5.705	2.08
East	Middle	Bottom	Vertical	Outside	#8@12"	1.00	72	4.25	0.79
West	Middle	Top	Horizontal	Outside	#8@9"	1.00	115	4.0	1.05
Slab	Middle	Middle	North-South	Outside	#8@12"	1.00	132	3.5	0.79

* 'Inside' refers to the water side of the wall. 'Outside' refers to the side of the wall remote from the water.

The unit width of a concrete section used to calculate moment capacities in this calculation, b, is 12 inches. The depth of concrete degradation, d_w, is varied by increments of 0.25" for each limiting section to determine the result on the moment capacity of each section.

4.2.4 Transfer Pool

The transfer pool is located next to the spent fuel pool, with the two pools separated by the wall previously indicated as the 'south wall' in this calculation. The walls of the transfer pool were not evaluated in Reference 1. While the transfer pool walls are not specifically evaluated in this calculation, a short assessment regarding the effect of concrete degradation on the capacity is made in this section.

The results of Sections 4.2.1 – 4.2.3, in particular the figure in Appendix B, show that the reduction in moment capacity due to concrete degradation is most dependent on the depth of a section; the smaller the section depth, the higher the reduction in moment capacity. References 5-13 show that the transfer pool walls have the following full depths:

Slab: 78"
South Wall: 140"
East Wall: 72" (same as the east wall of the spent fuel pool)
West Wall: 115" (same as the west wall of the spent fuel pool)
North Wall: varies from 48" to 72" (same as the south wall of the spent fuel pool)

Note that none of the transfer pool walls have a smaller depth than any of the spent fuel pool walls (see Section 3.0 for a discussion of the varying depth of the spent fuel pool south wall/transfer pool north wall). Therefore, the percent reduction in moment capacities for the transfer pool walls are not expected to be significantly greater than those for the spent fuel pool walls.

5.0 REFERENCES

1. MPR-1863, "Salem Generating Station Spent Fuel Pool Building Structural Design Analysis," Revision 0.

2. PSEG Nuclear Department Technical Standard SC.DE-TS.ZZ-4201(Q), "Salem Structural Design Criteria," Revision 1.

3. American Concrete Institute Standard ACI 318-63, "Building Code Requirements for Reinforced Concrete," June, 1963.

4. Winter, George and Arthur H. Nilson, "Design of Concrete Structures," McGraw-Hill Book Company: New York, 1979.

5. PSEG Nuclear Drawing No. 201075 A 8706-2, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 78'-0"."

6. PSEG Nuclear Drawing No. 201076 A 8706-4, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 84'-0"."
7. PSEG Nuclear Drawing No. 201077 A 8706-8, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 100'-0" and 116'-0"."
8. PSEG Nuclear Drawing No. 201078 A 8706-4, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Plan at El. 130'-0"."
9. PSEG Nuclear Drawing No. 201079 A 8706-3, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Roof Plan."
10. PSEG Nuclear Drawing No. 201080 A 8706-7, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections A-A & B-B."
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12. PSEG Nuclear Drawing No. 201082 A 8706-5, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Sections F-F & G-G."
13. PSEG Nuclear Drawing No. 201085 A 8706-5, "Salem Nuclear Generating Station, No. 1 Unit - Fuel Handling Area, Elevation P-P & Str. Bar Schedule."
14. Spiegel, Leonard, and George F. Limbrunner, "Reinforced Concrete Design," Prentice-Hall, Inc.: Englewood Cliffs, NJ, 1980.

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Calculation No. 0108-0275-34	Prepared By M. Herzig	Checked By L. Lichtenauer	Page: A-1 Revision: 0

A

Reduction in Moment Capacities Resulting from Concrete Degradation

North Wall, Water Side Compression, #8 Rebar, 8.75' Wall Thickness

Allowable Stress of Steel, Working Stress Design, f_s (psi)	24000
Strength of Concrete, f_c' (psi)	3500
Modulus of Concrete, E_c, (psi)	3.41E+06
Modulus of Steel, E_s, (psi)	2.90E+07
Modulus ratio, $n = E_s/E_c$	8.50E+00
Equivalent Steel Area, A_s (in^2)	0.79
Unit Width/Height of Wall, b (in.)	12
Depth of steel from nearest wall edge, d_d (in.)	4
Full depth of wall, L (in.)	105

Depth of Concrete Degradation, d_w (in.)	Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement, $d = [L-(d_d+d_w)]$ (in.)	Reinforcing Ratio, $\rho = A_s/(bd)$	Neutral Axis Location, k=sqrt[2*ρ*n +(ρ*n)2] - ρ*n	Distance between Compressive and Tensile Forces, j=1-k/3	Moment Capacity, M (ft*kip/ft)	% Reduction in Moment Capacity
0	101.0	6.52E-04	9.99E-02	9.67E-01	154.3	0.00
0.25	100.8	6.53E-04	1.00E-01	9.67E-01	153.9	0.25
0.50	100.5	6.55E-04	1.00E-01	9.67E-01	153.5	0.50
0.75	100.3	6.57E-04	1.00E-01	9.67E-01	153.1	0.75
1.00	100.0	6.58E-04	1.00E-01	9.67E-01	152.7	1.01
1.25	99.8	6.60E-04	1.00E-01	9.67E-01	152.3	1.26
1.50	99.5	6.62E-04	1.01E-01	9.66E-01	151.9	1.51
1.75	99.3	6.63E-04	1.01E-01	9.66E-01	151.5	1.76
2.00	99.0	6.65E-04	1.01E-01	9.66E-01	151.2	2.01
2.25	98.8	6.67E-04	1.01E-01	9.66E-01	150.8	2.26
2.50	98.5	6.68E-04	1.01E-01	9.66E-01	150.4	2.52
2.75	98.3	6.70E-04	1.01E-01	9.66E-01	150.0	2.77
3.00	98.0	6.72E-04	1.01E-01	9.66E-01	149.6	3.02
3.25	97.8	6.73E-04	1.01E-01	9.66E-01	149.2	3.27
3.50	97.5	6.75E-04	1.02E-01	9.66E-01	148.8	3.52
3.75	97.3	6.77E-04	1.02E-01	9.66E-01	148.4	3.77
4.00	97.0	6.79E-04	1.02E-01	9.66E-01	148.1	4.02
4.25	96.8	6.80E-04	1.02E-01	9.66E-01	147.7	4.28
4.50	96.5	6.82E-04	1.02E-01	9.66E-01	147.3	4.53
4.75	96.3	6.84E-04	1.02E-01	9.66E-01	146.9	4.78
5.00	96.0	6.86E-04	1.02E-01	9.66E-01	146.5	5.03
5.25	95.8	6.88E-04	1.02E-01	9.66E-01	146.1	5.28
5.50	95.5	6.89E-04	1.03E-01	9.66E-01	145.7	5.53
5.75	95.3	6.91E-04	1.03E-01	9.66E-01	145.3	5.78
6.00	95.0	6.93E-04	1.03E-01	9.66E-01	145.0	6.04

West Wall, Water Side Compression, #8 Rebar, 9.583' Wall Thickness

Allowable Stress of Steel, Working Stress Design, f_s (psi)	24000
Strength of Concrete, f_c' (psi)	3500
Modulus of Concrete, E_c, (psi)	3.41E+06
Modulus of Steel, E_s, (psi)	2.90E+07
Modulus ratio, $n = E_s/E_c$	8.50E+00
Equivalent Steel Area, A_s (in^2)	1.05
Unit Width/Height of Wall, b (in.)	12
Depth of steel from nearest wall edge, d_d (in.)	4
Full depth of wall, L (in.)	115

Depth of Concrete Degradation, d_w (in.)	Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement, d = [L-(d_d+d_w)] (in.)	Reinforcing Ratio, $\rho = A_s/(bd)$	Neutral Axis Location, k=sqrt[2*ρ*n +(ρ*n)2] - ρ*n	Distance between Compressive and Tensile Forces, j=1-k/3	Moment Capacity*, M (ft*kip/ft)	% Reduction in Moment Capacity
0.00	111.0	7.88E-04	1.09E-01	9.64E-01	224.6	0.00
0.25	110.8	7.90E-04	1.09E-01	9.64E-01	224.1	0.23
0.50	110.5	7.92E-04	1.10E-01	9.63E-01	223.6	0.46
0.75	110.3	7.94E-04	1.10E-01	9.63E-01	223.1	0.69
1.00	110.0	7.95E-04	1.10E-01	9.63E-01	222.5	0.92
1.25	109.8	7.97E-04	1.10E-01	9.63E-01	222.0	1.15
1.50	109.5	7.99E-04	1.10E-01	9.63E-01	221.5	1.38
1.75	109.3	8.01E-04	1.10E-01	9.63E-01	221.0	1.60
2.00	109.0	8.03E-04	1.10E-01	9.63E-01	220.5	1.83
2.25	108.8	8.05E-04	1.10E-01	9.63E-01	220.0	2.06
2.50	108.5	8.06E-04	1.10E-01	9.63E-01	219.5	2.29
2.75	108.3	8.08E-04	1.11E-01	9.63E-01	218.9	2.52
3.00	108.0	8.10E-04	1.11E-01	9.63E-01	218.4	2.75
3.25	107.8	8.12E-04	1.11E-01	9.63E-01	217.9	2.98
3.50	107.5	8.14E-04	1.11E-01	9.63E-01	217.4	3.21
3.75	107.3	8.16E-04	1.11E-01	9.63E-01	216.9	3.44
4.00	107.0	8.18E-04	1.11E-01	9.63E-01	216.4	3.67
4.25	106.8	8.20E-04	1.11E-01	9.63E-01	215.9	3.90
4.50	106.5	8.22E-04	1.11E-01	9.63E-01	215.3	4.13
4.75	106.3	8.24E-04	1.12E-01	9.63E-01	214.8	4.35
5.00	106.0	8.25E-04	1.12E-01	9.63E-01	214.3	4.58
5.25	105.8	8.27E-04	1.12E-01	9.63E-01	213.8	4.81
5.50	105.5	8.29E-04	1.12E-01	9.63E-01	213.3	5.04
5.75	105.3	8.31E-04	1.12E-01	9.63E-01	212.8	5.27
6.00	105.0	8.33E-04	1.12E-01	9.63E-01	212.3	5.50

**Per Table 4-1 in the main body of the calculation, the limiting design margin for the west wall is based on the OBE allowable, which is 1/3 above normal allowables. Although the moment capacities in the above table are normal allowables, the percent reductions for OBE allowables are the same.*

East Wall, Water Side Compression, #8 Rebar, 6' Wall Thickness

Allowable Stress of Steel, Working Stress Design, f_s (psi)	24000
Strength of Concrete, f_c' (psi)	3500
Modulus of Concrete, E_c, (psi)	3.41E+06
Modulus of Steel, E_s, (psi)	2.90E+07
Modulus ratio, $n = E_s/E_c$	8.50E+00
Equivalent Steel Area, A_s (in^2)	0.79
Unit Width/Height of Wall, b (in.)	12
Depth of steel from nearest wall edge, d_d (in.)	4.25
Full depth of wall, L (in.)	72

Depth of Concrete Degradation, d_w (in.)	Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement, $d = [L-(d_d+d_w)]$ (in.)	Reinforcing Ratio, $\rho = A_s/(bd)$	Neutral Axis Location, k=sqrt[2*ρ*n +(ρ*n)2] - ρ*n	Distance between Compressive and Tensile Forces, j=1-k/3	Moment Capacity, M (ft*kip/ft)	% Reduction in Moment Capacity
0.00	67.8	9.72E-04	1.21E-01	9.60E-01	102.7	0.00
0.25	67.5	9.75E-04	1.21E-01	9.60E-01	102.4	0.38
0.50	67.3	9.79E-04	1.21E-01	9.60E-01	102.0	0.75
0.75	67.0	9.83E-04	1.21E-01	9.60E-01	101.6	1.13
1.00	66.8	9.86E-04	1.21E-01	9.60E-01	101.2	1.50
1.25	66.5	9.90E-04	1.22E-01	9.59E-01	100.8	1.88
1.50	66.3	9.94E-04	1.22E-01	9.59E-01	100.4	2.26
1.75	66.0	9.97E-04	1.22E-01	9.59E-01	100.0	2.63
2.00	65.8	1.00E-03	1.22E-01	9.59E-01	99.7	3.01
2.25	65.5	1.01E-03	1.22E-01	9.59E-01	99.3	3.39
2.50	65.3	1.01E-03	1.23E-01	9.59E-01	98.9	3.76
2.75	65.0	1.01E-03	1.23E-01	9.59E-01	98.5	4.14
3.00	64.8	1.02E-03	1.23E-01	9.59E-01	98.1	4.51
3.25	64.5	1.02E-03	1.23E-01	9.59E-01	97.7	4.89
3.50	64.3	1.02E-03	1.24E-01	9.59E-01	97.3	5.27
3.75	64.0	1.03E-03	1.24E-01	9.59E-01	96.9	5.64
4.00	63.8	1.03E-03	1.24E-01	9.59E-01	96.6	6.02
4.25	63.5	1.04E-03	1.24E-01	9.59E-01	96.2	6.39
4.50	63.3	1.04E-03	1.24E-01	9.59E-01	95.8	6.77
4.75	63.0	1.04E-03	1.25E-01	9.58E-01	95.4	7.15
5.00	62.8	1.05E-03	1.25E-01	9.58E-01	95.0	7.52
5.25	62.5	1.05E-03	1.25E-01	9.58E-01	94.6	7.90
5.50	62.3	1.06E-03	1.25E-01	9.58E-01	94.2	8.27
5.75	62.0	1.06E-03	1.26E-01	9.58E-01	93.9	8.65
6.00	61.8	1.07E-03	1.26E-01	9.58E-01	93.5	9.03

Slab, Water Side Compression, #8 Rebar, 11' Slab Thickness

Allowable Stress of Steel, Working Stress Design, f_s (psi)	24000
Strength of Concrete, f_c' (psi)	3500
Modulus of Concrete, E_c, (psi)	3.41E+06
Modulus of Steel, E_s, (psi)	2.90E+07
Modulus ratio, $n = E_s/E_c$	8.50E+00
Equivalent Steel Area, A_s (in^2)	0.79
Unit Width/Height of Wall, b (in.)	12
Depth of steel from nearest wall edge, d_d (in.)	3.5
Full depth of wall, L (in.)	132

Depth of Concrete Degradation, d_w (in.)	Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement, $d = [L-(d_d+d_w)]$ (in.)	Reinforcing Ratio, $\rho = A_s/(bd)$	Neutral Axis Location, $k=sqrt[2*\rho*n+(\rho*n)2] - \rho*n$	Distance between Compressive and Tensile Forces, $j=1-k/3$	Moment Capacity, M (ft*kip/ft)	% Reduction in Moment Capacity
0.00	128.5	5.12E-04	8.91E-02	9.70E-01	197.0	0.00
0.25	128.3	5.13E-04	8.92E-02	9.70E-01	196.6	0.20
0.50	128.0	5.14E-04	8.93E-02	9.70E-01	196.2	0.39
0.75	127.8	5.15E-04	8.93E-02	9.70E-01	195.8	0.59
1.00	127.5	5.16E-04	8.94E-02	9.70E-01	195.4	0.79
1.25	127.3	5.17E-04	8.95E-02	9.70E-01	195.1	0.99
1.50	127.0	5.18E-04	8.96E-02	9.70E-01	194.7	1.18
1.75	126.8	5.19E-04	8.97E-02	9.70E-01	194.3	1.38
2.00	126.5	5.20E-04	8.98E-02	9.70E-01	193.9	1.58
2.25	126.3	5.21E-04	8.98E-02	9.70E-01	193.5	1.78
2.50	126.0	5.22E-04	8.99E-02	9.70E-01	193.1	1.97
2.75	125.8	5.24E-04	9.00E-02	9.70E-01	192.7	2.17
3.00	125.5	5.25E-04	9.01E-02	9.70E-01	192.3	2.37
3.25	125.3	5.26E-04	9.02E-02	9.70E-01	191.9	2.57
3.50	125.0	5.27E-04	9.03E-02	9.70E-01	191.6	2.76
3.75	124.8	5.28E-04	9.04E-02	9.70E-01	191.2	2.96
4.00	124.5	5.29E-04	9.04E-02	9.70E-01	190.8	3.16
4.25	124.3	5.30E-04	9.05E-02	9.70E-01	190.4	3.36
4.50	124.0	5.31E-04	9.06E-02	9.70E-01	190.0	3.55
4.75	123.8	5.32E-04	9.07E-02	9.70E-01	189.6	3.75
5.00	123.5	5.33E-04	9.08E-02	9.70E-01	189.2	3.95
5.25	123.3	5.34E-04	9.09E-02	9.70E-01	188.8	4.14
5.50	123.0	5.35E-04	9.10E-02	9.70E-01	188.4	4.34
5.75	122.8	5.36E-04	9.11E-02	9.70E-01	188.1	4.54
6.00	122.5	5.37E-04	9.11E-02	9.70E-01	187.7	4.74

Bottom South Wall, Water Side Compression, #11 Rebar, 6' Wall Thickness

Allowable Stress of Steel, Working Stress Design, f_s (psi)	24000
Strength of Concrete, f_c' (psi)	3500
Modulus of Concrete, E_c, (psi)	3.41E+06
Modulus of Steel, E_s, (psi)	2.90E+07
Modulus ratio, $n = E_s/E_c$	8.50E+00
Equivalent Steel Area, A_s (in^2)	2.08
Unit Width/Height of Wall, b (in.)	12
Depth of steel from nearest wall edge, d_d (in.)	5.705
Full depth of wall, L (in.)	72

Depth of Concrete Degradation, d_w (in.)	Distance from Extreme Compression Fiber to Centroid of Tension Reinforcement, $d = [L-(d_d+d_w)]$ (in.)	Reinforcing Ratio, $\rho = A_s/(bd)$	Neutral Axis Location, k=sqrt[2*ρ*n +(ρ*n)2] - ρ*n	Distance between Compressive and Tensile Forces, j=1-k/3	Moment Capacity, M (ft*kip/ft)	% Reduction in Moment Capacity
0.00	66.3	2.61E-03	1.90E-01	9.37E-01	258.3	0.00
0.25	66.0	2.62E-03	1.90E-01	9.37E-01	257.3	0.39
0.50	65.8	2.63E-03	1.90E-01	9.37E-01	256.3	0.78
0.75	65.5	2.64E-03	1.91E-01	9.36E-01	255.3	1.17
1.00	65.3	2.65E-03	1.91E-01	9.36E-01	254.3	1.55
1.25	65.0	2.66E-03	1.91E-01	9.36E-01	253.3	1.94
1.50	64.8	2.68E-03	1.92E-01	9.36E-01	252.3	2.33
1.75	64.5	2.69E-03	1.92E-01	9.36E-01	251.3	2.72
2.00	64.3	2.70E-03	1.92E-01	9.36E-01	250.3	3.11
2.25	64.0	2.71E-03	1.93E-01	9.36E-01	249.3	3.50
2.50	63.8	2.72E-03	1.93E-01	9.36E-01	248.3	3.88
2.75	63.5	2.73E-03	1.93E-01	9.36E-01	247.3	4.27
3.00	63.3	2.74E-03	1.94E-01	9.35E-01	246.3	4.66
3.25	63.0	2.75E-03	1.94E-01	9.35E-01	245.3	5.05
3.50	62.8	2.76E-03	1.94E-01	9.35E-01	244.3	5.44
3.75	62.5	2.77E-03	1.95E-01	9.35E-01	243.3	5.82
4.00	62.3	2.78E-03	1.95E-01	9.35E-01	242.3	6.21
4.25	62.0	2.79E-03	1.96E-01	9.35E-01	241.3	6.60
4.50	61.8	2.80E-03	1.96E-01	9.35E-01	240.3	6.99
4.75	61.5	2.82E-03	1.96E-01	9.35E-01	239.3	7.38
5.00	61.3	2.83E-03	1.97E-01	9.34E-01	238.3	7.76
5.25	61.0	2.84E-03	1.97E-01	9.34E-01	237.3	8.15
5.50	60.8	2.85E-03	1.97E-01	9.34E-01	236.3	8.54
5.75	60.5	2.86E-03	1.98E-01	9.34E-01	235.3	8.93
6.00	60.3	2.87E-03	1.98E-01	9.34E-01	234.3	9.32

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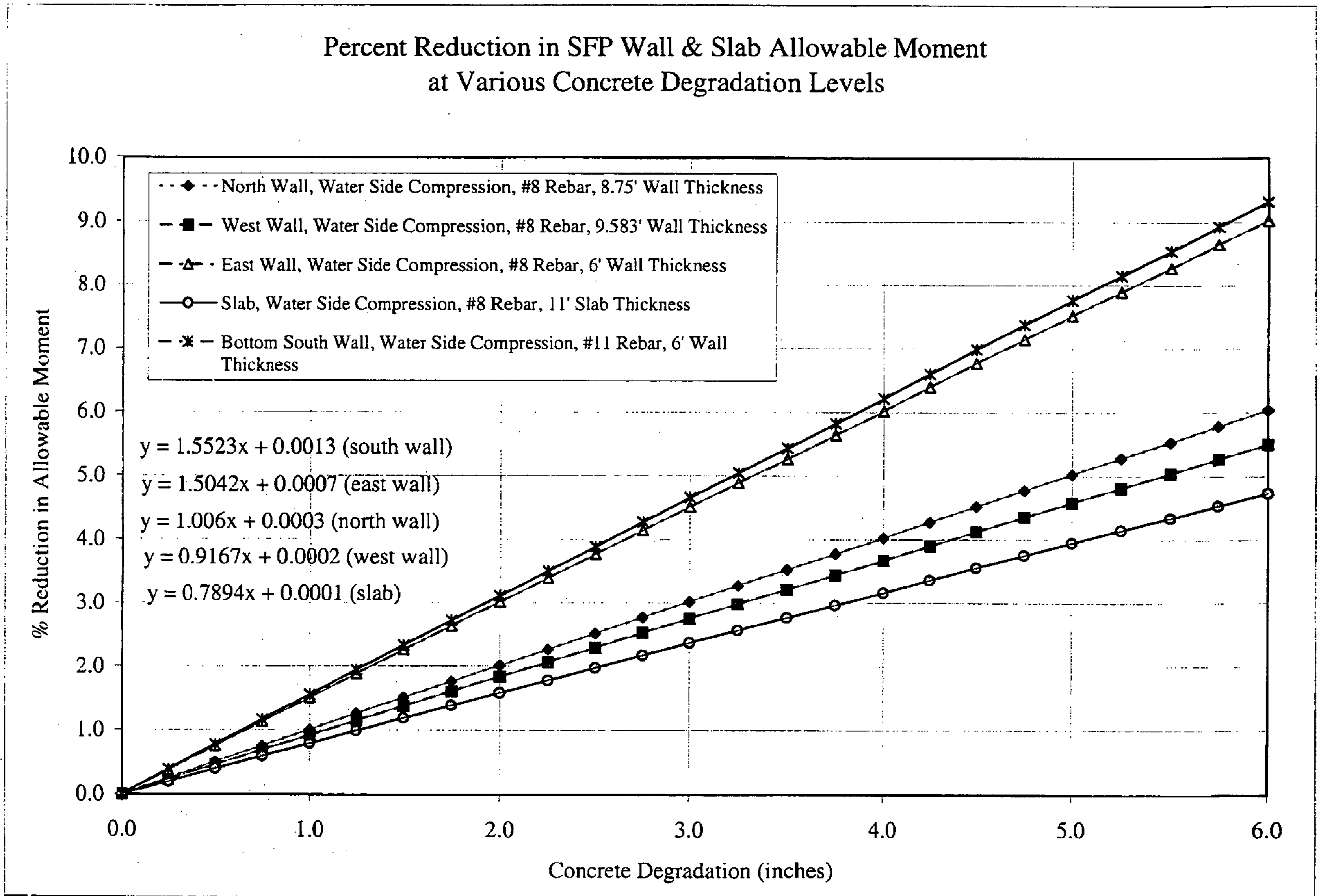
Calculation No.	Prepared By	Checked By	Page: B-1
0108-0275-34	M. Hrizy	L. Lichtenauer	Revision: 0

B
Plot of Spent Fuel Pool Moment Capacity Reductions


Percent Reduction in SFP Wall & Slab Allowable Moment
at Various Concrete Degradation Levels
North Wall, Water Side Compression, #8 Rebar, 8.75' Wall Thickness
West Wall, Water Side Compression, #8 Rebar, 9.583' Wall Thickness
East Wall, Water Side Compression, #8 Rebar, 6' Wall Thickness
Slab, Water Side Compression, #8 Rebar, 11' Slab Thickness
Bottom South Wall, Water Side Compression, #11 Rebar, 6' Wall Thickness
y = 1.5523x + 0.0013 (south wall)
y = 1.5042x + 0.0007 (east wall)
y = 1.006x + 0.0003 (north wall)
y = 0.9167x + 0.0002 (west wall)
y = 0.7894x + 0.0001 (slab)
% Reduction in Allowable Moment
Concrete Degradation (inches)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0