6.3 Thermal Effects of Greasing

Description:

The high-strength steel wires that make up the pre-stressing tendons are very sensitive to stress corrosion cracking while under tension. Corrosion protection was initially done by grouting the inside of the tendon sleeves after tendon original stressing. However, NRC Reg1.107 (FM 6.3 Exhibit 1 is NRC RegGuide 1.107 Rev.1 from February 1977) and NRC Reg1.90 (FM 6.3 Exhibit 2 is NRC RegGuide 1.90 Rev.1 from August 1977) moved the industry towards non-grouted tendons in order to fulfil in-service periodic inspections (FM 6.3 Exhibit 3 is a 1982 published paper by H. Ashar and D.J. Naus on the topic). From that time on, grease has been used for corrosion inhibition of the stressed tendons.

The procedure to grease the tendon during original installation is described in FM 6.3 Exhibit 4 (part of the documentation generated after the dome delamination event in 1976) and observed in the examples of FM 6.3 Exhibit 5 (typical pre-stressing field documentation including details of the greasing operation):

1. Install the tendons inside the sleeves (the sleeves themselves were installed as part of the concrete form-work and they are embedded in the concrete);

2. Tension the tendons to lock-off force (the details of the tensioning procedure are investigated in other failure modes);

3. Grease the tendons (basically done by filling the sleeves with grease, all around the tendons themselves);

The grease is injected in the tendon sleeves at a pressure up to 85psi and a temperature around 160oF (FM 6.3 Exhibit 4). The pressure and temperature cause thermal expansion of the duct and of the concrete surrounding the duct. Differences in expansion and/or rate of expansion can induce thermal stresses and possibly cause cracking.

Additionally, the tendons are greased again during surveillance activities after they have been tested (FM 6.3 Exhibit 15).

09/10

Data to be collected and Analyzed:

1. Details of the original greasing procedure (FM 6.3 Exhibit 4 and FM 6.3 Exhibit 5);

2. Materials properties comparison (FM 6.3 Exhibit 6 is pp. 88-89 of P.K. Mehta reference book on concrete "Concrete: Structure, Properties, and Materials" and FM 6.3 Exhibit 7 is the summary of Coefficient of Thermal Expansion (CTE) values and FM 6.3 Exhibit 8 is a summary of Thermal Conductivity values);

3. Calculate the heat transferred from the grease to the sleeve during tendon greasing (FM 6.3 Exhibit 9 is a thermal transfer analysis done by PII);

4. Calculate pressure capability of the tendon sleeves (FM 6.3 Exhibit 10 is a sleeve pipe pressure calculation done by Progress energy);

5. Calculate additional stress created by the grease injection at high temperature and pressure using first principles (FM 6.3 Exhibit 12 is a PII calculation of the stresses generated by the hot grease);

6. Investigate grease additions during surveillance activities (FM 6.3 Exhibit 15);

Discussion:

The files exampled in FM 6.3 Exhibit 5 are from the original installation of the post-tensioning tendons. They are the "Crystal River 3 Reactor Building Pre-Stressing System Tendon History" files. The identification number is the number of the tendon in question.

The first example in FM 6.3 Exhibit 5 is tendon 12V2:

1. Tendon 12V2 is the second (2) Vertical tendon (V) between buttresses 1 and 2 (12), hence 12V2;

2. The tendon was received on-site on 1/16/1974;

3. And installed in the conduit (sleeve) on 7/3/1974;

4. No wires were removed or replaced;

5. The field anchor-head was button-headed on 8/27/1974;

6. And subsequently stressed on 10/14/1974;

7. All the elongation is on the same side as the vertical tendons are accessible only on the dome side for tensioning;

8. The total elongation is 12.5". This is the elongation from 1,500 psi (FM 6.3 Exhibit 5) or "an initial force that will remove all slack" (FM 6.3 Exhibit 13 chapter 1.0 Purpose), which was taken as 360 kips in practice (FM 6.3 Exhibit 13 chapter 3.0 Design Inputs). The theoretical elongation of 14.94 inches (FM 6.3 Exhibit 14) is longer than the actual elongation because of the wobble friction in the wires (FM 6.3 Exhibit 13);

9. The grease is then filled on 10/23/1974;

10. At a pressure of 112 psi and a temperature of 126°F;

An important point to remember is that the issue of differential thermal expansion in this case is NOT associated with different Coefficient of Thermal Expansions (CTE) between the two materials but with different Thermal Conductivity Coefficients between the steel sleeve and the concrete. The sleeve expands much faster initially and this creates a load on the slowly heating and expanding surrounding concrete (see FM 6.3 Exhibit 4 from the dome analysis).

We made the conservative assumption that all the heat from the grease was transferred to the sleeve rather than using the study on heat transferred from the grease to the tendon sleeve (FM 6.3 Exhibit 9).

From observation of the tendon surveillance data, the number of greasing cycles is under 4 for all tendons, and it is 1 for most tendons in the structure. Note that an analysis of potential tensile strength degradation due to thermal effects is included in FM 4.8.

Verified Supporting Evidence:

a. The Thermal Conductivity Coefficient of the steel and concrete are very different so that the sleeve expands much faster than the concrete and this creates a force from the hot sleeve to the still-cold surrounding concrete (concrete 1 btu/ft/h/^{o}F and steel 25 btu/ft/h/^{o}F from FM 6.3 Exhibit 8);

Verified Refuting Evidence:

a. Once the temperature has equilibrated at the sleeve/concrete interface, there is no thermal stress because the coefficient of thermal expansion of the sleeve steel material and of the concrete are very similar (FM 6.3 Exhibit 6);

b. The pressure capability of the tendon sleeves is high enough to support the 85 psi grease injection pressure (FM 6.3 Exhibit 10);

c. Stress analysis demonstrates that the additional stresses added due to thermal effects of greasing in the concrete do not exceed the concrete tensile capability (FM 6.3 Exhibit 12);

Conclusion:

The stresses due to high-temperature greasing did not lead to the delamination.
They may have contributed to localized micro-cracking around the tendon sleeves.

U.S. NUCLEAR REGULATORY COMMISSION

Revision 1
February 1977

REGULATORY GUIDE

OFFICE OF STANDARDS DEVELOPMENT

REGULATORY GUIDE 1.107

QUALIFICATIONS FOR CEMENT GROUTING FOR PRESTRESSING TENDONS[1] IN CONTAINMENT STRUCTURES

A. INTRODUCTION

General Design Criterion 1, "Quality Standards and Records," of Appendix A, "General Design Criteria for Nuclear Power Plants," to 10 CFR Part 50, "Licensing of Production and Utilization Facilities," requires that structures, systems, and components important to safety be designed, fabricated, and erected to quality standards commensurate with the importance of the safety functions to be performed.

The prestessing tendon system of a prestressed concrete containment structure is a principal strength element of the structure. Since the ability of the containment structure to withstand the events postulated to occur during the life of the structure depends on the functional reliability of the structure's principal strength elements, any significant deterioration of the prestressed elements due to corrosion may present potential risk to the public safety. Hence it is important that any system for inhibiting corrosion of the prestressing elements possess a high degree of reliability in performing its intended function.

This guide describes quality standards acceptable to the NRC staff for the use of portland cement grout as the corrosion inhibitor for prestressing tendons in prestressed concrete containment structures. The Advisory Committee on Reactor Safeguards has been consulted concerning this guide and has concurred in the regulatory position.

[1] For the purposes of this guide, a "tendon" is defined as a tensioned steel element consisting of wires, strands, or bars anchored at each end to an end anchorage assembly.

* Lines indicate substantive change from previous issues.

B. DISCUSSION

The recommendations of this guide are applicable when portland cement grout is used as the corrosion inhibitor for the highly stressed tendons of prestressed concrete containment structures. The recommendations of the guide are not intended for use in relation to the grout for foundation anchors.

To date, the staff has evaluated applications proposing grout as the corrosion protection system for both bar tendons and strand tendons. The recommendations of this regulatory guide therefore apply to a grouted tendon system when the tendon is fabricated from either bars or strands. For grouting of wire tendons, a program based on similar quality standards may be developed and submitted to the staff for evaluation.

Unlike greased tendons, grouted tendons are not available for direct inspection after they are grouted. It is therefore essential that the proposed grout and grouting procedure be thoroughly evaluated before it is used in the construction of the containment structure. An advantage of grouting, in addition to providing corrosion protection, is that a well-designed and well-constructed grouted tendon system provides a degree of bond between the tendons and the surrounding concrete. This bond in turn helps the anchorage system to resist the fluctuating stresses that arise after construction of the structure.

Section III, Division 2, "Code for Concrete Reactor Vessels and Containments," of the ASME Boiler and Pressure Vessel Code (Ref. 1) provides some requirements for grout constituents and for the physical and chemical properties of grout. Regulatory Position C.1 of this guide briefly describes minimum quality standards for grout materials, referencing the ASME Code Articles where applicable and acceptable to the NRC staff. The regulatory position also outlines important considerations affecting proper grouting. References 2, 3, and 4, as well as data furnished by applicants who have proposed grout as the corrosion inhibitor for prestressing steel, have been used to arrive at this position.

Appendix A to this guide provides a list of relevant literature that may be used by the applicant to establish procedures and criteria for the specific grouted tendon program. However, the listing of these references does not constitute a blanket endorsement by the staff of their content.

Specific areas of concern that should be given proper attention during the development of a grouted tendon system are discussed in the following paragraphs.

The effectiveness of grout in performing its intended function of inhibiting corrosion depends mainly on two characteristics:

1. The grout (whether freshly mixed or hardened) should not cause chemical attack on the prestressing elements through its interaction with the material of the tendon steel, the material of the anchor hardware, or the material of the duct.[2]

2. The grout should completely fill the tendon duct on hardening.

Various deleterious substances have been reported as potential sources of corrosion of prestressing steel. Most of the reported failures of prestressing elements have been attributed (a) to the presence of chlorides in the atmosphere or in the constituents of grout or (b) to the presence of hydrogen sulfide in the atmosphere (Refs. 5, 6, and 7). Nitrates and sulfates generally found in mixing water have been theorized to be potential sources of stress corrosion of prestressing steel. However, it has been reported (Ref. 8) that, in a concrete environment, oxygenated anions such as sulfates and nitrates do not exhibit intense corrosion properties. It has also been reported (Ref. 3) that most of the chlorides are neutralized during the hydration of portland cement. The threshold values below which these substances will not participate in initiating corrosion have not been established. Hence a safe and prudent approach would be to make sure that these substances are limited to the lowest practical levels in grout constituents. The use of water contaminated with hydrogen sulfide should be prohibited.

The limits recommended for chlorides, nitrates, sulfates, and sulfides in Regulatory Position C.1.e should not be exceeded in the overall composition of the grout. The quantities of these substances in the grout constituents should be determined individually for each of the constituents by the applicable ASTM (American Society of Testing and Materials)[3] methods and expressed in parts per million parts of water in the grout composition.

In general, portland cement conforming to ASTM C150, Type I or Type II, is suitable for the grout. However, grouting under certain climatic or environmental conditions may dictate the use of other types of cements. Chlorides are normally present in cement, but the amount is usually not reported. The determination of chlorides in cement should be a requirement when specifying the cement for grout.

Admixtures should be free of any substance likely to damage the prestressing steel. Use of aluminum powder to produce expansion has been viewed by many engineers as having possible deleterious effects. Under an alkaline environment (pH >9), the aluminum powder generates minute bubbles of hydrogen gas (H_2) that would not endanger the tensioned steel at the prevailing range of pressures and temperatures. However, the potential danger of hydrogen attack on steel does exist if the tensioned steel elements or stressed anchorage components contain surface flaws. The parameters affecting the use of aluminum powder are described in Reference 9.

The protective mechanism of grout is primarily dependent on its ability to provide a continuous alkaline environment around the tensioned steel elements. The natural alkalinity of the primary product of cement hydration (i.e., calcium hydroxide) tends to be at a pH value of 12.5. The effectiveness of the alkaline environment may be reduced by the leaching of alkaline substances with water, by reaction in an acidic or sulfide-containing environment, or by the presence of oxygen and chloride ions. It is reported in Reference 10 that the ability of chloride ions to develop corrosion increases with decreasing alkalinity of the calcium hydroxide solutions. Thus it is advisable to monitor the pH value of the in-place grout under actual field conditions and ensure that it remains above a value where the passivating effect of the grout is not reduced by the available chloride ions in the composition of the grout.

Section CC-2243.2 of the ASME Code (Ref. 1) requires the use of flow cones with the limits on efflux times at the specific quiescent times to ensure adequate fluidity of the grout. However, for certain types of grouts (in particular, one with a thixotropic additive), these requirements may not be appropriate to define their pumpability. Applicants in such cases

[2] For the purposes of this guide, a "duct" is a hole or void provided in the concrete for the post-tensioning tendon. A duct may be provided by embedding metal sheathing in cast-in-place concrete.

[3] A list of relevant ASTM standards is provided in Appendix B of this guide.

should propose alternative means of quality control to accomplish the same objective. Also, a general practice is to limit the pumping pressure during grouting to 300 psi (see Refs. 2 and 3). However, grout with a thixotropic additive may need higher pumping pressures for long vertical tendons. In such cases, it should be demonstrated through tests that high pressures will not deteriorate the quality of grout; damage the duct, duct splices, or surrounding concrete; or deform the containment liner.

In addition to the control on grout materials and on mixing and injecting the grout to ensure the intended protection of the prestressing steel, it is important to take other precautions directly related to the corrosion protection of the prestressing steel:

1. It is necessary that the tendon remain clean, dry, free from deleterious corrosion, and undamaged up to the time it is grouted. Specific protection measures should be provided at coastal sites, at sites having a high moisture level, and at sites near industrial areas.

2. When a preassembled tendon-sheathing assembly is to be placed before concreting, the tendon should be protected against corrosive environment during assembly, handling, storing, transporting, placing, and tensioning.

3. Before placing the tendon in the duct, it is important to ascertain that the duct is free of obstructions, moisture, and other deleterious substances.

4. Ferrous metal sheathing is galvanized to protect it against corrosion before grouting of the tendon. However, the contact surfaces of the tendons and the sheathing are potential areas for the formation of corrosion cells and hydrogen evolution. From the tendon corrosion point of view, this is critical if the time between the tensioning and grouting is long and the duct contains moisture with or without deleterious substances.

5. In general, the period between tensioning and grouting is critical from the standpoint of stress corrosion or hydrogen stress cracking. Steps should be taken to minimize this time period.

Effective corrosion protection of prestressing tendons can be provided by portland cement grout if appropriate precautions are taken to eliminate the potential sources of corrosion. To this end, close quality control is necessary for each constituent of the grout, the tendon material, and the tendon duct material and for the method of mixing and pumping the grout and ensuring that the tendon is surrounded from end to end with qualified grout.

C. REGULATORY POSITION

The following minimum quality standards should be maintained when portland cement grout is to be used for the corrosion protection of prestressing steel.

1. Materials

a. *Portland Cement.* Cement should conform to the requirements of ASTM C150. The type to be selected should be suitable for the intended use.

b. *Fine Aggregate.* Fine aggregate-filler may be used when permitted by the requirements of Article CC-2243.1 of the ASME Boiler and Pressure Vessel Code, Section III, Division 2 (Ref. 1).

c. *Water.* The water should not contain ingredients harmful to the prestressing steel or the grout. Water contaminated (1 ppm) with hydrogen sulfide (sulfide ion) should be prohibited. The water to be used for grouting should be qualified for use by making comparative tests in accordance with the test methods and tolerance levels described in Article CC-2223.2 of the ASME Code.

d. *Admixtures.* Acceptable admixtures may be used if tests have demonstrated that their use improves the properties of grout, e.g., increases workability, reduces bleeding, prevents water separation when pumped at high pressure, entrains air, expands the grout, or reduces shrinkage. The quantities of harmful substances in the admixture should be kept to a minimum. Use of calcium chloride should be prohibited.

e. *Limits on Deleterious Substances and pH.*

(1) The quantity of the following substances (added individually for each constituent and expressed as parts per million parts of water) in the overall grout composition should not exceed the following limits:

Chloride	100 ppm (200 ppm if pH is maintained above 12)
Nitrates	100 ppm
Sulfates[4]	250 ppm
Sulfides	2 ppm (test method of Ref. 15)

(2) The pH value of the grout at inlet and outlet of the duct should be maintained above 11.6 (12 if the allowable chloride content is 200 ppm).

(3) During the grouting period, the amount of deleterious substances in the grout constituents should be checked weekly and whenever the composition of the constituents is changed or is suspected of having changed.

[4] Sulfates in the form of sulfur trioxide as a cement component need not be considered.

2. Physical Properties of the Grout

The physical properties of the cement grout should satisfy the requirements of Article CC-2243.2 of the ASME Code. Adequate tests should be carried out in accordance with the test methods described in that article to demonstrate that the grout satisfies these requirements.

3. Duct

a. The duct size should be adequate to allow the insertion and tensioning of tendons without undue difficulty. The area of the grout that penetrates and surrounds the tendon at any section should be at least equal to the cross-sectional area of the tendon. The duct sheathing and its splices should be of ferrous metal and should be protected to prevent corrosive deterioration prior to the grouting of tendons. The duct sheathing and its splices should be sufficiently tight that a thin cement slurry cannot pass through while the surrounding concrete is being placed. The duct sheathing and its splices, when surrounded by hardened concrete, should be capable of withstanding the maximum grouting pressure without leakage.

b. Vents should be provided at any major changes in section of the duct, as well as at the high points. Drains should be provided at the low points. Vents and drains should be checked for possible obstructions prior to grouting.

4. Equipment for Grouting

a. The grouting equipment should include a mixer that is capable of continuous mechanical mixing and that can produce a grout free of lumps and undispersed cement. To this end, tests should be performed to demonstrate the optimum range of mixing time and the sequence of placing the constituent materials in the mixer under extreme anticipated environmental conditions.

b. The pump should be of the positive displacement type and should be capable of exerting the required maximum pressure. A safety device should be provided to guard against exerting a pressure that could damage the duct, duct splices, or surrounding concrete or deform the containment liner. The pumps should not suck air in with the grout.

c. A screen having clear openings not more than 1/8 inch (1/4 inch for grout with a thixotropic additive) should be provided between the mixed grout and the pump to ensure that the grout does not contain lumps. If an excessive amount of lumps remain on the screen, the batch should be rejected.

5. Grouting

a. Grouting should be carried out immediately after tensioning. The period between tensioning and grouting should be kept below 72 hours. The tendon should be protected from inclement weather and other adverse environmental conditions during this period. If an additional delay is expected, the tendons should be protected by methods or products that would not jeopardize the effectiveness of the grout as a corrosion inhibitor. In any case, the tendon anchorages should be visually examined just prior to grouting to detect any breakage or degradation of prestressing elements as evidenced by the movement or dislocation of anchoring hardware.

b. Flushing before grouting is not recommended. When flushing has to precede the grouting, appropriate measures should be taken to ensure that:

(1) The level of harmful substances in the in-place grout does not increase above that in the designed grout, and

(2) The properties of the injected grout satisfy the recommendations in Regulatory Position C.2.

c. Tests should establish the length of time that the grout can be used after mixing. The tests should verify that:

(1) The intended reaction of such admixtures as expansive agents continues when such a grout is injected in the duct, and

(2) This time is less than that required for the initial set of the grout as determined by the method of ASTM C191.

d. The temperature along the entire length of the tendon duct during grouting should be above 35°F. This temperature should be maintained until the minimum (2-inch cube) strength of the job-cured grout exceeds 800 psi. The grout temperature should not exceed 90°F during mixing and pumping unless it can be established by test that a higher temperature will not adversely affect the grouting operation.

e. The development of the grouting procedure should consider the extremes of anticipated environmental conditions. The procedure should ensure that the ducts will be filled and that the tendon steel will be completely surrounded by grout.

f. All openings, air vents, and drains should be hermetically sealed after grouting to prevent the ingress of water and other corrosive agents.

g. If an applicant chooses to provide permanent protection of the anchor hardware by means of qualified grout or concrete, the protection should be provided on the following bases:

(1) All exposed anchor hardware should be thoroughly examined before being provided with the permanent protection.

(2) The permanent protection should be designed and constructed in a manner that would prevent the intrusion of water and deleterious substances to the anchorage components.

6. Tendon

The tendon should be clean, dry, free from deleterious corrosion, and undamaged up to the time when it is grouted. The preassembled tendon sheathing assembly should be protected against corrosive influences from the time of assembly to the time of grouting.

D. IMPLEMENTATION

The purpose of this section is to provide information to applicants and licensees regarding the NRC staff's plans for using this regulatory guide.

Except in those cases in which the applicant proposes an acceptable alternative method for complying with specified portions of the Commission's regulations, the guide will be used by the NRC staff on the following bases:

1. Submittals in connection with construction permit applications docketed after February 14, 1977, will be evaluated on the basis of this guide.

2. Submittals in connection with operating license applications for plants whose construction permit applications were docketed prior to February 14, 1977, will be evaluated in accordance with the commitment made by the applicant in the construction permit.

APPENDIX A

REFERENCES

1. "Code for Concrete Reactor Vessels and Containments," American Concrete Institute Committee 359 and American Society of Mechanical Engineers Subcommittee on Nuclear Power, 1975. Copies may be obtained from the American Society of Mechanical Engineers, 345 E. 47th St., New York, N.Y. 10017 or from the American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, Mich. 48219.

2. "Recommended Practice for Grouting of Post-Tensioned Concrete," Prestressed Concrete Institute Committee on Post-Tensioning, published in PCI Journal, Nov./Dec. 1972. Copies may be obtained from the Prestressed Concrete Institute, 20 North Wacker Drive, Chicago, Ill. 60606.

3. "Report on Grout and Grouting of Prestressed Concrete," Proceedings of the Seventh Congress of the Federation Internationale de la Precontrainte, 1974. Copies may be obtained from the Federation Internationale de la Precontrainte, Terminal House, Grosvenor Gardens, London SW1W OAU.

4. "Specifications for Structural Concrete for Buildings," American Concrete Institute Committee 301, 1972. Copies may be obtained from the American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, Mich. 48219.

5. Leonhardt, F., *Prestressed Concrete Design and Construction*, Wilhelm Ernst & Sohn, Berlin, Second Edition, 1964.

6. Szilard, R., "Corrosion and Corrosion Protection of Tendons in Prestressed Concrete Bridges," ACI Journal, Jan. 1969. Copies may be obtained from the American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, Mich. 48219.

7. Monfore, G. E., and Verbeck, G. J., "Corrosion of Prestressed Wire in Concrete," ACI Journal, July 1960. Copies may be obtained from the address shown in Reference 6.

8. Scott, G. N., "Corrosion Protection Properties of Portland Cement Concrete," Journal of the American Water Works Association, Vol. 57, No. 8, Aug. 1965. Copies may be obtained from the American Water Works Association, 2 Park Avenue, New York, N.Y. 10016.

9. "Admixtures for Concrete," American Concrete Institute Committee 212. Copies may be obtained from the American Concrete Institute, P.O. Box 19150, Redford Station, Detroit, Mich. 48219.

10. Hausman, D. A., "Steel Corrosion in Concrete," *Materials Protection*, November 1967. Copies may be obtained from the National Association of Corrosion Engineers, 2400 West Loop S., Houston, Texas 77027.

11. Harstead, G. A., et al, "Testing for Large Curved Prestressing Tendons," Proceedings of the American Society of Civil Engineers, Power Division, March 1971. Copies may be obtained from the American Society of Civil Engineers, 345 E. 47th Street, New York, N.Y. 10017.

12. Lange, H., "The Vacuum Process, A New Method for Injecting Prestressing Tendons," paper submitted for the Seventh Congress of the Federation Internationale de la Precontrainte, New York, N.Y. 1974. Copies may be obtained from the Federation Internationale de la Precontrainte, Terminal House, Grosvenor Gardens, London SW1W OAU.

13. Schupack, M., "Development of a Water Retentive Grouting Aid to Control the Bleed in Cement Grout Used for Post-Tensioning," presented at the Seventh Congress of the Federation Internationale de la Precontrainte, New York, N.Y., 1974. Copies may be obtained from the address shown in Reference 12.

14. Kajfasz, S., et al, "Phenomena Associated with Grouting of Large Tendon Ducts and Morphology of Defects," technical contribution to the Seventh Congress of the Federation Internationale de la Precontrainte, New York, N.Y. 1974. Copies may be obtained from the address shown in Reference 12.

15. "Standard Method for the Examination of Water and Waste Water"—1971. Copies may be obtained from American Public Health Association, 1015 18th Street NW., Washington, D.C. 20036.

APPENDIX B

LIST OF RELEVANT ASTM STANDARDS

C109-73, "Standard Method of Test for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. Cube Specimens)"
C150-74, "Standard Specification for Portland Cement Concrete"
C191-74, "Standard Method of Test for Time of Setting Hydraulic Cement by Vicat Needle"
C260-74, "Standard Specifications for Air-Entraining Admixtures for Concrete"
C494-71, "Standard Specification for Chemical Admixtures for Concrete"
D512-67, "Test for Chloride Ion in Industrial Water and Industrial Waste Water"
D992-71, "Test for Nitrate Ion in Water"
D516-74, "Tests for Sulfate Ion in Water"
D596-74, "Reporting Results of Analysis of Water"
D1129-74, "Terms Relating to Water"
D1293-65, "pH of Water and Waste Water"

Revision 1*
August 1977

U.S. NUCLEAR REGULATORY COMMISSION

REGULATORY GUIDE

OFFICE OF STANDARDS DEVELOPMENT

REGULATORY GUIDE 1.90

INSERVICE INSPECTION OF PRESTRESSED CONCRETE CONTAINMENT STRUCTURES WITH GROUTED TENDONS†

A. INTRODUCTION

General Design Criterion 53, "Provisions for Containment Testing and Inspection," of Appendix A, "General Design Criteria for Nuclear Power Plants," to 10 CFR Part 50, "Licensing of Production and Utilization Facilities," requires, in part, that the containment be designed to permit (1) appropriate periodic inspection of all important areas and (2) an appropriate surveillance program. This guide describes bases acceptable to the NRC staff for developing an appropriate surveillance program for prestressed concrete containment structures with grouted tendons. The Advisory Committee on Reactor Safeguards has been consulted concerning this guide and has concurred in the regulatory position

B. DISCUSSION

Inservice inspection of prestressed concrete containment structures with grouted tendons is needed to verify at specific intervals that the safety margins provided in the design of containment structures have not been reduced as a result of operating and environmental conditions. Grouting of tendons to protect them against corrosion is a proven technology in other types of structures. However, there is as yet no real experience to adequately define the long-term characteristics of containment structures with grouted tendons. The major concern in containment structures with grouted tendons is the possibility that widespread corrosion of the tendon steel may occur and remain undetected. The major factors influencing the occurrence of corrosion are (1) the susceptibility of the tendon steel to corrosion, (2) the degree of exposure of the tendon steel to a deleterious environment, (3) the extent of temperature variations, and (4) the quality of the grout and its installation. Following the recommendations of Regulatory Guide 1.107, "Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures," could significantly reduce the danger of widespread corrosion. However, the mechanism of corrosion in all conditions and situations is not fully understood. Because many parameters can influence the development of corrosion or stress corrosion, there is always an area of uncertainty with regard to the corrosion of tendon steel, and it is necessary to monitor the structure in a manner that would reveal the existence of widespread corrosion.

This guide outlines the recommendations for inservice inspection of containments having grouted tendons of sizes up to an ultimate strength of approximately 1300 tons (11,000 kN) and consisting either of parallel wires or of one or several strands. The detailed recommendations of the guide are not directly applicable to grouted tendon containments having bar tendons. However, the inservice inspection program for grouted tendon containments with bar tendons may be developed using the principles in this guide and will be reviewed by the NRC staff on a case-by-case basis. This guide does not address the inservice inspection of prestressing foundation anchors. If they are used, the inservice inspection program will be reviewed by the NRC staff on a case-by-case basis. Inservice inspection of the containment liner and penetrations is also not addressed in this guide.

The simplest means of monitoring these prestressed concrete structures would be to ascertain the amount of prestress at certain strategically located sections in the structure. However, it is generally felt that available instrumentation for concrete, i.e., strain gages, stress meters, and strain meters, is not reliable enough to provide such information. When

* The substantial number of changes in this revision has made it impractical to indicate the changes with lines in the margin.

† For the purpose of this guide, a tendon is defined as a tensioned steel element consisting of wires, strands, or bars anchored at each end to an end anchorage assembly.

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Regulatory Guides are issued to describe and make available to the public methods acceptable to the NRC staff of implementing specific parts of the Commission's regulations, to delineate techniques used by the staff in evaluating specific problems or postulated accidents, or to provide guidance to applicants. Regulatory Guides are not substitutes for regulations, and compliance with them is not required. Methods and solutions different from those set out in the guides will be acceptable if they provide a basis for the findings requisite to the issuance or continuance of a permit or license by the Commission.

Comments and suggestions for improvements in these guides are encouraged at all times, and guides will be revised, as appropriate, to accommodate comments and to reflect new information or experience. This guide was revised as a result of substantive comments received from the public and additional staff review.

Comments should be sent to the Secretary of the Commission, U.S. Nuclear Regulatory Commission, Washington, D.C. 20555, Attention: Docketing and Service Branch.

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instrumentation that either can be recalibrated or replaced in case of a malfunction or is proven to be sufficiently reliable is developed, monitoring the prestress level would be a desirable means of assessing the continuing integrity of prestressed concrete structures with grouted tendons.

Another means of monitoring the functionality of the containment structure would be to subject it to a pressure test and measure its behavior under pressure. Industry comments indicate that an inservice inspection program based on the test of overall functionality is preferable.

This regulatory guide provides two acceptable alternative methods of inspecting containment structures with grouted tendons: (1) an inservice inspection program based on monitoring the prestress level by means of instrumentation, and (2) an inservice inspection program based on pressure-testing the containment structure.

The detailed inspection program outlined in this guide is applicable to a sphere-torus dome containment having cylindrical walls about 130 feet (40 m) in diameter and an overall height of about 200 feet (61 m) with three groups of tendons, i.e., hoop, vertical, and dome. For the purpose of this guide, such a containment is termed the "reference containment." The recommendations in the guide may be used for similar containments with cylindrical walls up to 140 feet (43 m) in diameter and an overall height up to 210 feet (64 m).

For containments that differ from the reference containment or are under a controlled environment, the inservice inspection program may be developed using the concepts evolved in this guide and the guidelines in Appendix A.

The inservice inspection program recommended in this guide consists of:

1. Force monitoring of ungrouted test tendons;
2. Monitoring performance of grouted tendons by
    a. Monitoring of prestress level, or
    b. Monitoring of deformation under pressure; and
3. Visual examination.

1. FORCE MONITORING OF UNGROUTED TEST TENDONS

Some tendons (otherwise identical) are left ungrouted and are protected from corrosion with grease. The changes observed in these tendons are not intended to represent the changes due to environmental or physical effects (with respect to corrosion) in the grouted tendons. Instead, these test tendons will be used as reference tendons to evaluate the extent of concrete creep and shrinkage and relaxation of the tendon steel.

The measurement of forces in ungrouted test tendons would provide a quantitative means of verifying the design assumptions regarding the volumetric changes in concrete and the relaxation of prestressing steel. If some lift-off readings (or load cell readings) indicate values lower than the expected low values, checks should be made to determine if such values are due to corrosion of wires of ungrouted tendons or to underestimation of prestressing losses. The plant need not be shut down or maintained in a shutdown condition during such an evaluation period. These tendons may also serve as an investigative tool for assessing the structural condition after certain incidents that could affect the containment.

2. MONITORING ALTERNATIVES FOR GROUTED TENDONS

a. Monitoring of Prestress Level (Alternative A)

After the application of prestress, the prestressing force in a tendon decreases owing to the interaction of such factors as:

(1) Stress relaxation of the prestressing steel;

(2) Volumetric changes in concrete;

(3) Differential thermal expansion or contraction between the tendon, grout, and concrete; and

(4) Possible reduction in cross section of the wires due to corrosion, including possible fracture of the wires.

In this alternative, the prestress level is monitored at certain strategically located sections in the containment. Thus it is a sampling procedure in which degradation in the vicinity of the instrumented section will be detected by evaluation of the instrumentation readings. However, if corrosion occurs at locations away from the instrumented sections, it would have to produce gross degradation before the instrumentation readings would be affected.

The prestressing force imparted to the structure by a grouted tendon system could be monitored by an appropriate combination of the following methods:

(1) Monitoring the tensile strains in the wires of a tendon;

(2) Evaluating the prestress level at a section in the structure from readings of appropriately located strain gages or strain or stress meters at the section (see Refs. 1 through 7).

Method (1) above is useful for direct monitoring of prestressing force in a tendon. However, the installation of the instrumentation required for this method needs careful attention during installation and grouting of the tendons. Moreover, strain gages installed on the prestressing wires of a tendon will not detect the loss of force due to relaxation of prestressing steel. Allowance for this can be based on relaxation data for the prestressing steel used.

Evaluation of strain gage and stress meter readings requires a full understanding of what makes up the readings, e.g., elastic, creep, and thermal strain or stress components. Strain gage readings will consist of elastic strains corresponding to the prestressing stress in concrete and strains due to creep and shrinkage of concrete. Strains from creep and shrinkage of concrete can vary between 1.5 and 2.5 times the elastic strains in concrete. However, there are methods that can be used to isolate these effects. Three such methods are:

(1) Calculate average creep and shrinkage strains from the time-dependent losses measured on the ungrouted tendons.

(2) Use stress meters at sections where strain gages are used.

(3) Use special strain meters that respond only to volumetric and temperature changes in concrete (Ref. 7).

A sufficient number of temperature sensors installed at the sections where instrumentation is located can be useful in isolating the thermal effects. It is recognized that the raw instrumentation readings can be deceptive, and adjustments may be necessary to account for the calibration constants and temperature effects. The interpretation and evaluation of the results will be simplified if the instrumentation is provided at sections away from structural discontinuities. The applicant should provide sufficient redundancy in the instrumentation to permit the evaluation of anomalous readings and the isolation of a malfunctioning gage. One such combination would be two strain gages and one stress meter at each face of a section.

After appropriate use has been made of the methods and instruments available, an average stress and an average prestressing force at a section can be evaluated. Even though the predicted prestressing force corresponding to a specific time may include adequate consideration for creep of concrete and relaxation of prestressing steel, the chance that the value based on measurements will compare well with the predicted value is small. Hence it is recommended that an applicant establish a band of acceptable prestress level similar to that illustrated in Figure 1. It is also recommended that the bandwidth not exceed 8% of the initial prestressing force at a section after considering the loss due to elastic shortening, anchorage takeup, and friction. The 8% bandwidth would amount to between 40% and 70% of the total time-dependent losses.

Alternative A is based on the use of instrumentation. Many of these instruments have to be built into the structure in such a manner that they can be neither replaced nor recalibrated. It is quite likely that such built-in instrumentation may not remain reliably operable throughout the life of the structure. Recognizing such a possibility, the guide provides for an alternative of pressure testing (Alternative B) when the data obtained from instrumentation readings are found to be questionable.

b. Monitoring of Deformation Under Pressure (Alternative B)

Testing the containment under pressure and evaluating its elastic response has been proposed as a means of assessing the integrity of the containment. The elastic response under pressure testing is primarily a function of the stiffness of the structure. Any significant decrease in the stiffness of the structure due to loss of prestress would be the result of cracking of the structure. Because of the insensitive and indirect relationship between the prestressing force and the elastic response of the structure, such a method cannot be used to establish the existing prestress level at various sections. However, comparison of the condition and deformation of the structure during the ISI (Inservice Inspection) pressure testing with those during the ISIT (Initial Structural Integrity Testing) pressure testing could provide a basis for evaluating the functionality of the structure. This method has been accepted* previously by the NRC staff on the condition that the containment be designed conservatively so that there will be no cracking (or only slight cracking at the discontinuities) under the peak test pressure. Section III, Division 2, of the ASME Code (Ref. 8) allows a 33-1/3% increase in the allowable stress in tensile reinforcement under a test condition. The NRC staff has accepted this allowance on the assumption that it is only a one-time loading (i.e., during the ISIT). However, if such testing is to be performed a number of times during the life of the containment structure, it is prudent not to use this allowance in order to avoid or minimize gradual propagation of cracking during subsequent pressure tests.

The locations for measuring the deformations under pressure should be based on the recommendations of this guide. For a meaningful comparison of the deformations, it is recommended that the locations where the deformations are to be recorded have deformations larger than 0.06 inch (1.5mm) under the calculated peak containment internal pressure associated with the design basis accident and that these

* Three Mile Island Nuclear Power Station Unit 2 and Forked River Nuclear Power Station.

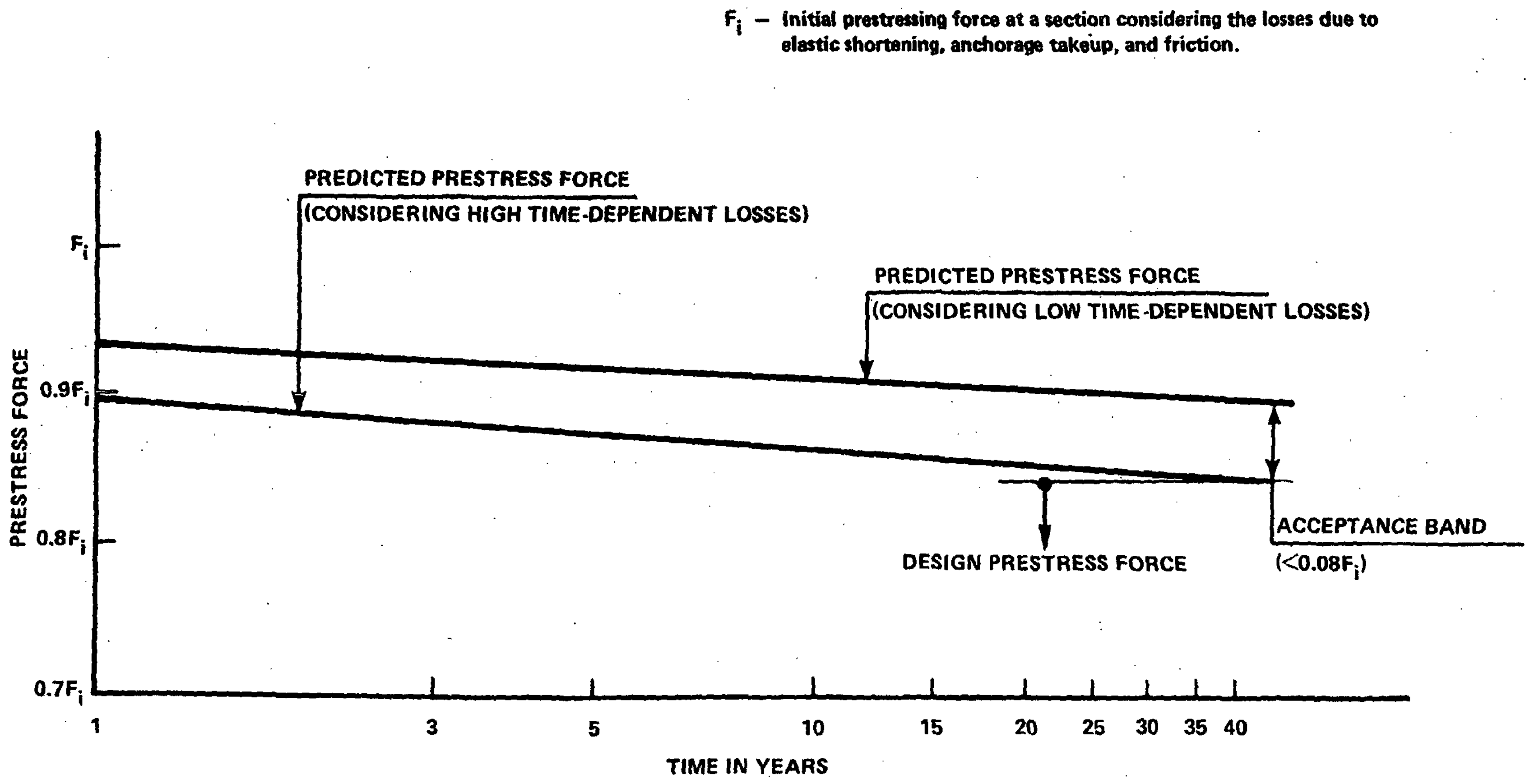


Figure 1. Typical Band of Acceptable Prestress Level

locations be approximately the same during the ISIT and the subsequent ISIs. This will require these locations to be away from the areas of structural discontinuities. Thus the number of locations for measurement of deformations in typical cylinder and dome areas will be in excess of those recommended in Regulatory Guide 1.18, "Structural Acceptance Test for Concrete Primary Reactor Containments."

If an analysis of the effects of such parameters as normal losses in prestressing force, increase in modulus of elasticity of concrete with age, and differences in temperatures during various pressure tests indicates that they could affect the deformations of the selected points, these parameters should be considered in comparing the deformations during various pressure tests.

3. VISUAL EXAMINATION

Visual examination of structurally critical areas consisting of the areas of structural discontinuities and the areas of heavy stress concentration is recommended. Reference 9 provides excellent guidance for reporting the condition of concrete and should be used whenever applicable for reporting the condition of examined areas.

There are numerous examples of the use of pulse velocity technique to obtain information concerning the general quality level of concrete. Based on experience and experimental data (Refs. 10, 11, 12), a pulse velocity of 14,000 ft/sec (4300 m/sec) or higher indicates a good to excellent quality of concrete. For normal weight concrete, a pulse velocity of 11,000 ft/sec (3400 m/sec) or lower indicates concrete of questionable quality. Thus the technique can be used as part of the inspection of concrete containments when the visual examination reveals a high density of wide (> 0.01 in. or 0.25 mm) cracks or otherwise heavy degradation. The detailed procedure and limitations of the techniques are described in Reference 13.

C. REGULATORY POSITION

1. GENERAL

1. All prestressed concrete containment structures with grouted tendons should be subjected to an inservice inspection (ISI) program. The specific guidelines provided herein are for the reference containment described in Section B.

2. For containments that differ from the reference containment, the program described herein should serve as the basis for developing a comparable inservice inspection program. Guidelines for the development of such a program are given in Appendix A to this guide.

3. The inservice inspection program should consist of:

a. Force monitoring of ungrouted test tendons;

b. Periodic reading of instrumentation for determining prestress level (Alternative A) or deformations under pressure (Alternative B) at preestablished sections; and

c. Visual examination.

4. The inservice inspection should be performed at approximately 1, 3, and 5 years after the initial structural integrity test and every 5 years thereafter. However, when an applicant chooses pressure testing (Alternative B) as a part of the inspection, the frequency of inspections should be as indicated in Figure 2.

5. Alternative B may be substituted for Alternative A by the applicant if, at some time during the life of the structure, the inspection based on Alternative A does not provide satisfactory data. The details of such a substitution will be reviewed by the NRC staff on a case-by-case basis.

6. If the containment base mat is prestressed, its proposed inspection program will be evaluated by the NRC staff on a case-by-case basis.

2. UNGROUTED TEST TENDONS

1. The following ungrouted test tendons should be installed in a representative manner:

a. Three vertical tendons,

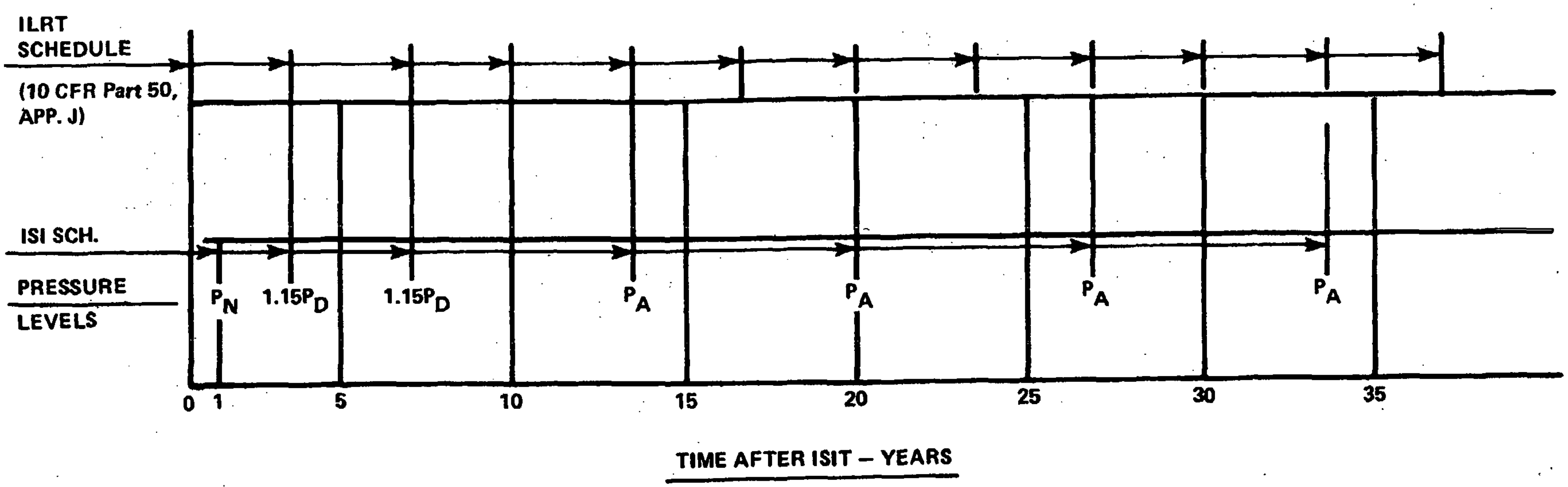
b. Three hoop tendons, and

c. Three dome tendons for the design utilizing three 60° families of tendons.

2. The ungrouted test tendons need not be in addition to the design requirements.

3. The ungrouted test tendons and their anchorage hardware should be identical to the grouted tendons and their hardware.

4. The ungrouted test tendons should be subjected to force measurement by lift-off testing or load cells to assess the effects of concrete shrinkage and creep and relaxation of the tendon steel. These data should be evaluated in conjunction with the overall structural condition of the containment evident from the other examinations.



KEY

P_N – Normal Operating Pressure or Zero

P_D – Containment Design Pressure

P_A – Calculated Peak Internal Pressure Associated with the Design Basis Accident

ILRT – Integrated Leak Rate Testing

ISIT – Initial Structural Integrity Testing

ISI – Inservice Inspection

Figure 2. Schedule for Inservice Inspections (Alternative B)

3. MONITORING ALTERNATIVES FOR GROUTED TENDONS

3.1 Instrumentation for Monitoring the Prestress Level (Alternative A)

3.1.1 Installation

1. The prestressed cylindrical wall and dome should be instrumented. This instrumentation may be either embedded in the concrete or inserted into the structure so that it can be maintained or replaced. Instrument types, locations, and quantities should be selected to provide the best representation of prestress level in the structure. A sufficient number of temperature sensors should be installed to isolate and evaluate the effects of variations in temperature gradients on the instrument readings and observations. Redundancy of the embedded instrumentation should be based on a conservative estimate of the probability of malfunction of the instrumentation to be installed.

2. The instrumentation in the concrete should be arranged and distributed in such a manner as to permit evaluation of the prestressing levels and should be located:

a. At six horizontal planes to measure the hoop prestressing levels;

b. Along three vertical tendons to measure vertical prestress levels;

c. Along three dome tendons for the design using three families of 60° tendons.

3. Sections through the structure should be selected at a minimum of four locations in each horizontal plane, three locations along each vertical tendon, and two locations along each dome tendon (see Figure 3). At these sections, the prestress level should be monitored by (a) a combination of stress meters or strain gages in concrete or on rebar at a minimum of two points through the section or (b) strain gages directly on tendon wires with a minimum of 3% of the tendon wires instrumented.

3.1.2 Characteristics

1. Instrumentation provided for the determination of concrete prestress level should be capable of effective use over the life span of the containment structure within specified operational limits under the following conditions, unless otherwise defined by the designer and approved by the NRC staff:

a. Humidity: 0% to 100%;

b. Temperature: 0°F (-18°C) to 200°F (93°C); and

c. Cyclic loading: 500 cycles of 600 psi (4.2 MPa) stress variation in compression.

2. The instruments should be protected against adverse effects of the expected environment in which they will be located, e.g., electrolytic attack, including the effects of stray electric currents of a magnitude that may be encountered at the particular site and structure. They should be protected against temperature extremes to which they may be exposed while the containment is under construction.

3. The sensitivity of strain gages should be specified; the drift or stability under the conditions in 1 and 2 above should be accounted for in the specified limits, or the gages should be subject to recalibration in service.

4. The stress meters should be able to measure compressive stresses up to 2500 psi (17.2 MPa).

3.1.3 Monitoring Instrumentation Operability

After the installation of the instrumentation, all embedded strain gages and stress meters should be read every two months until the initial structural integrity test (ISIT) is performed. The response of the instrumentation during prestressing and pressure testing (ISIT) should be used to confirm their operability. After the ISIT, the monitoring of the instrumentation should be continued every two months to confirm operability of the instrumentation until the first inservice inspection. The monitoring frequency may be reduced to once every six months thereafter unless local conditions or special circumstances dictate more frequent readouts. The operability of the instrumentation should also be confirmed during subsequent pressure tests. If anomalous readings are obtained, the reason for such readings should be determined. If it is determined that they result from defective gages, the basis for such a determination should be justified.

3.2 Monitoring Deformation Under Pressure (Alternative B)

When it is planned to use this alternative as a part of the total inservice inspection program, it is recommended that the design of the containment structure include the following considerations:

1. Membrane compression should be maintained under the peak pressure expected during the ISI tests.

2. The maximum stress in the tensile reinforcing under the peak pressure expected during the ISI test should not exceed one-half the yield strength of the reinforcing steel ($0.5f_y$).

3.2.1 Pressurization

1. During the first inspection, the containment structure need not be pressurized.

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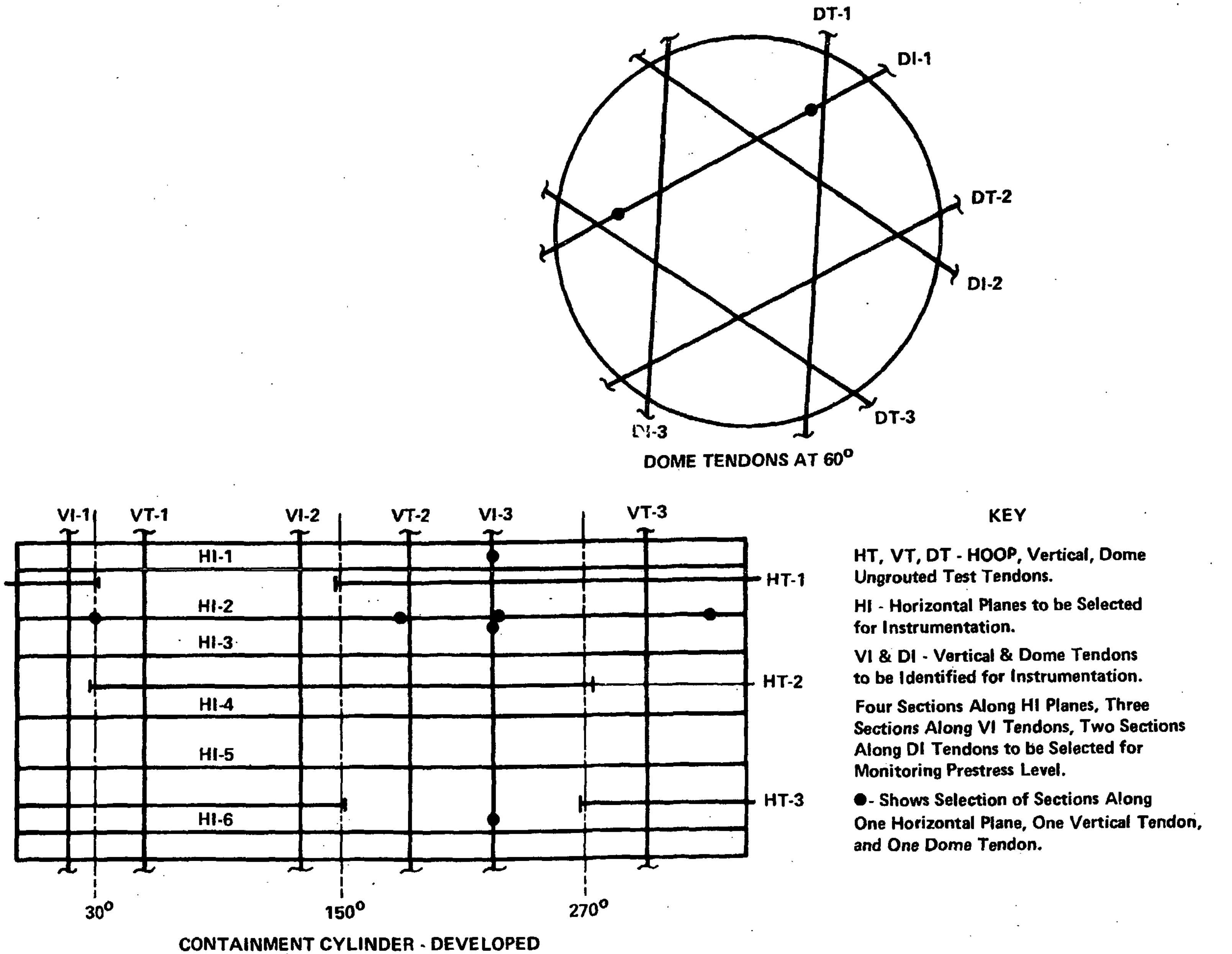

Figure 3. Containment Diagram Showing Typical Locations of Test Tendons and Instrumentation

2. During the second and third inspections, the containment structure should be subjected to a maximum internal pressure of 1.15 times the containment design pressure.

3. During the fourth and subsequent inspections, the containment structure should be subjected to a maximum internal pressure equal to the calculated peak internal pressure associated with the postulated design basis accident.

3.2.2 Instrumentation and Deformations

1. Instrumentation similar to that used during the ISIT should be installed prior to the pressure testing for measurement of overall deformations at the selected points.

2. The limit of accuracy of readings of the instruments to be used should be specified by means of an error band so that a meaningful comparison of deformations measured during the ISIT and ISI can be made.

3. The points to be instrumented for the measurement of radial displacements should be determined in six horizontal planes in the cylindrical portion of the shell, with a minimum of four points in each plane (see Figure 3).

4. The points to be instrumented for the measurement of vertical (or radial) displacements should be determined as follows:

a. At the top of the cylinder relative to the base, at a minimum of four approximately equally spaced azimuths.

b. At the apex of the dome and one intermediate point between the apex and the springline, on at least three equally spaced azimuths.

5. The intermediate pressure levels at which the deformations at the selected points are to be measured should correspond to those for the ISIT.

4. VISUAL EXAMINATION

4.1 Structurally Critical Areas

A visual examination should be performed on the following exposed structurally critical areas:

1. Areas at structural discontinuities (e.g., junction of dome and cylindrical wall or wall and base mat).

2. Areas around large penetrations (e.g., equipment hatch and air locks) or a cluster of small penetrations.

3. Local areas around penetrations that transfer high loads to the containment structure (e.g., around high-energy fluid system lines).

4. Other areas where heavy loads are transferred to the containment structure (crane supports, etc.).

A visual examination of structurally critical areas should be scheduled during all pressure tests while the containment is at its maximum test pressure, even if visual examinations of these areas have been conducted at other times.

4.2 Anchorage Assemblies

Exposed portions of the tendon anchorage assembly hardware or the permanent protection thereon (whether it be concrete, grout, or steel cap) should be visually examined by sampling in the following manner:

1. A minimum of six dome tendons, two located in each 60° group (three families of tendons) and randomly distributed to provide representative sampling,

2. A minimum of five vertical tendons, randomly but representatively distributed,

3. A minimum of ten hoop tendons, randomly but representatively distributed.

For each succeeding examination, the tendon anchorage areas to be examined should be selected on a random but representative basis so that the sample group will change each time.

The inservice inspection program should define the defects the inspector should look for during visual examination of the exposed anchor hardware and protection medium and should establish the corresponding limits and tolerances. Special attention should be given to the concrete supporting the anchor assemblies, and any crack patterns at these points should be observed and analyzed.

5. REPORTABLE CONDITIONS

5.1 Inspection Using Alternative A

If the average prestress force along any tendon falls below the acceptable band (see Figure 1), the condition should be considered as reportable.

If the prestress force determined at any section falls below the design prestress force, the condition should be considered as reportable.

5.2 Inspection Using Alternative B

If the deformation measured under the maximum test pressure at any location is found to have increased by more than 5% of that measured during the ISIT under the same pressure, the condition should be considered as reportable.

5.3 Reportable Conditions for Visual Examinations

If the crack patterns observed at the structurally critical areas indicate a significant decrease in the spacing or an increase in the widths of cracks compared to those observed during the ISIT at zero pressure after depressurization, the condition should be considered as reportable.

If the visual examination of the anchor hardware indicates obvious movements or degradation of the anchor hardware, the condition should be considered as reportable.

If the anchor hardware is covered by permanent protection and the visual examination reveals a degradation (e.g., extensive cracks or corrosion stains) that could bring into question the integrity and effectiveness of the protection medium, the condition should be considered as reportable.

5.4 Reportable Conditions for Ungrouted Test Tendons

When the force monitoring (by liftoff or load cell) of ungrouted test tendons indicates a prestress force below the acceptable band (see Figure 1), the condition should be considered as reportable.

6. REPORTING TO THE COMMISSION

The reportable conditions of Regulatory Position C.5 could be indicative of a possible abnormal degradation of the containment structure (a boundary designed to contain radioactive materials). Any such condition should be reported to the Commission.*

D. IMPLEMENTATION

The purpose of this section is to provide information to applicants and licensees regarding the NRC staff's plans for using this regulatory guide.

Except in those cases in which the applicant proposes an acceptable alternative method for complying with specified portions of the Commission's regulations, the method described herein will be used in the evaluation of submittals in connection with construction permit applications docketed after October 1, 1977.

If an applicant wishes to use this regulatory guide in developing submittals for applications docketed on or before October 1, 1977, the pertinent portions of the application will be evaluated on the basis of this guide.

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*The report to the Commission should be made in accordance with the recommended reporting program of Regulatory Guide 1.16, "Reporting of Operating Information—Appendix A Technical Specifications."

APPENDIX A

GUIDELINES FOR DEVELOPING THE INSERVICE INSPECTION PROGRAM FOR CONTAINMENTS (OTHER THAN REFERENCE CONTAINMENT DISCUSSED IN THE GUIDE) WITH GROUTED TENDONS

Ungrouted Tendons

Three ungrouted tendons should be provided in each group of tendons (e.g.,vertical, hoop, dome, inverted U).

Instrumentation (Alternative A)

The following criteria should be used to determine the number of sections (N) to be monitored for each group of tendons:

$$N = \frac{\text{Actual Area Prestressed by a Group of Tendons}}{\text{K x Area Monitored by a Set of Instruments at a Section (determined as SxL)}}$$

where

S = spacing of tendons in feet (meters)

L = length of a tendon monitored by a set of instruments- may be considered as 12 ft (3.66m)

and K is determined as follows:

For containments under uncontrolled environment and having continuous tendon curvature,

$K \leq 100$

For containments under uncontrolled environment and having essentially straight tendons,

$K \leq 160$

For containments under controlled environment and having either straight or curved tendons.

$K \leq 200$

Monitoring Deformations Under Pressure (Alternative B)

The number of locations (N) to be selected for measuring the deformations under pressure should be determined as follows:

For radial deformations of cylinder,

$$N = \frac{\text{Surface Area of Cylinder in square feet (square meters)}}{2700\ (250)}$$

but not less than 12.

For vertical deformations of cylinder,

N = 4

For radial or vertical deformations of dome,

$$N = \frac{\text{Surface Area of Dome in square feet (square meters)}}{2700\ (250)}$$

but not less than 4

APPENDIX B

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Overview of the Use of Prestressed Concrete in U.S. Nuclear Power Plants

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Abstract

The containment system of a nuclear power plant provides a key part of the overall plant's engineered-safety features. The structure serves as the final barrier against release of any radioactive fission products to the environment and consideration of public safety is one of the primary criteria in providing such a barrier.

Originally the containment was envisioned as a static pressure envelope fabricated of steel and which would adequately contain the fission products released from the primary system during any credible accident scenario. As the size of the nuclear power plants increased, the costs of fabricating containment structures from stress-relieved steel plate became significant and it became advantageous to fabricate the containments of concrete. In addition to economic advantages, the concrete containments could be fabricated in virtually any size (thickness) and shape, they generally utilize indigenous materials for their construction, and they exhibit a ductile mode of failure (leak before break) which is predictable and observable. The paper outlines the extent of the use of prestressed concrete containments in nuclear power plants. However, the accident at Three Mile Island has changed the design parameters associated with the containment. In addition to containing the radioactivity during a postulated maximum LOCA, future containment designs should also provide for pressures generated during degraded core accidents. The change might give a slight edge to the application of prestressing in containment design.

The evolution of large size prestressing systems in the United States and abroad has been the result of the need to resist high pressures with the minimum number of tendons. Furthermore, corrosion inhibiting materials evolved simultaneously with the use of large size prestressing tendons. Cement grout and organic-petrolatum-based compounds needed to be specially formulated to assure thorough penetration through the tendon elements. Early in the development of prestressed concrete containments extensive dialogue occurred between the Nuclear Regulatory Commission (known then as the Atomic Energy Commission) and the industry relative to the use of portland cement grout as a corrosion inhibitor. Concern by the regulators relative to the inability to inspect the prestressing tendons to insure their structural integrity resulted in the issuance of two regulatory guides (RGs) by the NRC: (1) "Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures (RG 1.107)" and (2) "Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons (RG 1.90)." According to some observers this action eventually eliminated any incentives for the use of grouted tendons in prestressed concrete containments.

In the United States it is required that the condition and functional capability of the ungrouted post-tensioning systems of prestressed concrete nuclear power plant containments be periodically assessed. This is accomplished, in part, systematically through an inservice tendon inspection program which must be developed and implemented for each containment. An overview of the essential elements of the inservice inspection requirements is presented and the effectiveness of these requirements is demonstrated through presentation of some of the potential problem areas which have been identified through the periodic assessments of the structural integrity of containments. Also, a summary of major problems which have been encountered with prestressed concrete construction at nuclear power plant containments in the United States is presented; that is, dome delamination, cracking of anchorheads, settlement of bearing plates, etc. The paper will conclude with an assessment of the overall effectiveness of the prestressed concrete containments.

1. Introduction

The principal use of prestressed concrete in the U.S. Nuclear Power Plants is in the construction of their containment structures. The containment structure (or containment) is a vital engineering safety feature of a nuclear power plant. It encloses the entire reactor and reactor coolant system, and serves as the final barrier against release of radioactive fission products to the environment under postulated design basis accident (DBA) conditions. To perform this function it is designed to withstand loadings associated with loss-of-coolant accident (LOCA) resulting from a double-ended rupture of the largest size pipe in the reactor coolant system. The containment is also designed to retain its integrity under low probability ($<10^{-4}$) environmental loadings such as those generated by earthquake, tornado and other site specific environmental events such as floods, seiche, and tsunami. Additionally, it is required to provide biological shielding under both normal and accident conditions, and is required to protect the internal equipment from external missiles, such as tornados or turbine generated missles and aircraft impact (where postulated).

An additional functional requirement for containments has come into play since the accident at Three Mile Island. This requirement consists of maintaining the integrity of the containment under thermal and pressure loads (symmetrical or nonsymmetrical) ensuing from the detonation of hydrogen generated as a result of the metal-water (steam) reaction under degraded core conditions. Dry containments, such as the one at Three Mile Island, which are designed for high LOCA pressures, are not affected by this additional requirement; however, the pressure suppression type containments (PWR ice-condenser, and some BWR containments), designed for low LOCA pressures, are subjected to a thorough evaluation. This requirement may become one of the controlling criteria in the design of future containments.

The functional requirements for containments are satisfied by various types of composite and hybrid steel-concrete constructions. Originally, the containment was envisioned as a static pressure envelope fabricated of steel with a separate radiation shield. As the size of the nuclear power plants increased, the costs of fabricating high pressure containment structures from stress-relieved steel plate became significant, and engineers started looking for alternatives such as steel-lined reinforced concrete which, in addition to economics, had advantages with respect to: improved construction schedules, earlier construction of interior containment structures and erection of equipment, and they can be designed to carry loads other than pressure and temperature (pipe anchors, equipment supports, etc.). Table I presents a distribution of construction types relative to various containment concepts utilized in the United States.

2. Evolution of Containment Configurations and Prestressing Systems

2.1 Containment Configurations

The first prestressed concrete containments were partially prestressed in the vertical direction only with mechanically spliced reinforcing steel in the hoop direction and in the dome. Fully prestressed concrete containments were first built in the late 1960's being cylindrical in shape with shallow dome and resting on a reinforced concrete slab. The dome is prestressed by three sets of tendons at 60° to each other and which are anchored at the side of the thickened dome-cylinder transition (ring girder). The cylinder walls are prestressed with both vertical and hoop tendons. The vertical tendons are anchored at the top to the ring girder and at the bottom of the foundation mat in specially constructed tendon galleries. Anchorage of the hoop tendons is to buttresses protruding from the cylindrical wall. Initial containment designs used six buttresses with subsequent designs utilizing either three or four buttresses. Although anchorage of hoop tendons at three buttresses, as compared to six, increased the length of tendons and friction force, the combination of a low coefficient of friction (μ<0.1) of pre-coated prestressing tendons and the reduced number of buttresses and anchorages produced cost savings. It was for these same reasons that the present-day prestressed concrete containment design evolved; that is, a cylinder with hemispherical dome using inverted-U tendons.

2.2 Prestressing Systems

A posttensioned prestressing system consists of a prestressing tendon in combination with methods of stressing and anchoring the tendon to hardened concrete. Three general categories of prestressing systems exist, depending on the type of tendon utilized: wire, strand or bar. The wire systems utilize a grouping of parallel wires. Strand systems utilize groupings of factory-twisted wire. Bar systems utilize a grouping of high-tensile-strength steel bars. Anchorage is provided by wedges, button-heads, or nuts.

The primary evolution in prestressing systems over the past few years has been with respect to system capacity. Prior to the advent of PCCs the prestressing systems were relatively small size; that is, less than 4.45 MN (500 ton) ultimate capacity. The requirement to withstand high forces resulting from a combination of increased volumes and pressures of the dry pressurized-water reactor (PWR) containments necessitated the development of tendon systems with increased capacity [8.0 to 10.7 MN (900 to 1200 ton]. This development permitted increased spacing of tendons and reduced congestion by almost halving the number of tendons, tendon ducts and anchorages. The large size tendons were developed by using groupings of multi-wire, multi-strand, or bar systems. In the United States, the 8.9 MN (1000 ton) systems approved for use include: (1) BBRV (wire), (2) VSL (strand) and (3) Stressteel S/H (strand).

3. Evolution and Performance of Corrosion Inhibitors for Prestressing Tendons

Prestressed concrete containments essentially are spaced steel structures since their strength is derived from a multitude of steel elements made up of deformed reinforcing bars and prestressing which are present in sufficient quantities to carry imposed tension loads. The prestressing therefore plays a vital role in insuring the structural integrity of the containment throughout its 30- to 40-year design life. However, because the tendons are fabricated from high-strength steels [>1.6 GPa (230 ksi)] in the form of many relatively small-diameter wires or several strands fabricated from small-diameter wires, and the tendons can be subjected to sustained stresses up to 70% of their ultimate tensile strength, they are more susceptible to corrosion than ordinary reinforcing steels and must be protected. Protection of the prestressing steel is generally provided by filling the ducts with portland cement grout or microcrystalline waxes (petrolatums) compounded using organic corrosion inhibitors.

3.1 Grouting

The effectiveness of portland cement grout as a deterrent to corrosion of steel is evidenced by its performance history in prestressed concrete for over 50 years and its use in reinforced concrete construction for over 100 years. Corrosion of steel in correctly formulated concrete (cement) is prevented by the high alkalinity (pH >12.5) of the $Ca(OH)_2$, which produces a passivating gamma iron oxide film on the steel surface [1, 2]. When corrosion does occur it is generally the result of a destruction of the passive layer. This can result from reduction of the alkalinity associated with calcium hydroxide, calcium silicates, and aluminates [3]; from carbonation; or from the presence of high concentrations of chloride, sulfide or nitrate ions. Current grouting materials have evolved over the years to try to ensure that the prestressing materials are completely encapsulated to prevent corrosion; that is, grouts are specially formulated with water reducers and expansive agents to minimize the potentially deleterious effects of water separation and shrinkage.

3.2 Petrolatum-Based Coatings

Although the introduction of petrolatum-based coatings as corrosion protection is much more recent than the use of portland cement grout, the coatings have gained prominence in PCCs in the United States because of their ease of inservice inspections. Additional advantages include: (1) encapsulation provides an approximate 50% reduction in friction factor which permits the use of longer tendons; (2) tendons may be relaxed, retensioned, and replaced as required; and (3) during construction there is the possibility of more efficient scheduling of event sequence because the tendons are protected in the shop.

The petrolatum-based coatings have evolved over the years to better attune the products to the nuclear unbonded tendon containment applications. Initially the product was a casing filler containing polar wetting agents, rust preventative additives, micro-crystalline waxes and proprietary items formulated to be water displacing, self-healing and resistant to electrical conductivity. The next generation of materials were formed by adding a plugging agent to the casing filler to increase the low flow point of the products (∿39°C (100°F)] to keep them from seeking loose sheathing joints and flowing into concrete hairline cracks. A subsequent refinement involved incorporation of a light base number (3 mg KOH/gm of product) to provide alkalinity for improved corrosion protection. Finally, the current generation of materials have evolved through a series of modifications to produce products which have been formulated to: increase the viscosity without sacrificing pumpability, raise the congealing point to 57-63°C (135-145°F), increase the resistance to flow from sheathing joints, improve the water resistance, and raise the base number (35 mg KOH/gm product) to provide higher reserve alkalinity [4].

3.3 Overview of the Performance of Prestressing Tendons [4-8]

Prestressed concrete was first used for nuclear pressure vessels in 1960. As of April 1982, 27 prestressed concrete reactor vessels (PCRVs) were either in operation or scheduled for operation in Europe (France, United Kingdom, Spain and Germany) and the United States. In addition, there are 116 containments for pressurized water reactors (PWRs) and 33 containments for heavy-water reactors (HWRs) commissioned or scheduled for commission throughout the world. Of the 116 containments for PWRs, 62 are in the United States. Reviews of the performance of the prestressing tendons in these structures have revealed that corrosion-related incidents are extremely limited. The evolution of corrosion inhibitors and the use of organic-petrolatum-based compounds designed especially for corrosion protection of prestressing materials have virtually eliminated corrosion of prestressing materials. The few incidences of corrosion that were identified, occurred early in the use of prestressed concrete for containment structures. Where these failures involved tendons coated by petroleum-based materials, the failures generally resulted from the use of off-the-shelf corrosion inhibitors that had not been specially formulated for prestressing materials.

4. Problems and Experiences During Construction of PCCs

In general, the development of the various components of prestressing systems has been substantiated by careful study, testing and thorough evaluations by vendors, engineers and regulators. However, there have been a few occasions, either due to breakdown of the quality control, or due to nonscrutinized construction methods, where significant component failures have occurred. The following is a summary of such reported failures.

At Calvert Cliff nuclear plant (Units 1 and 2) some of the bearing plates under anchor heads of vertical tendons became depressed into the concrete [9]. These depressions ranged in size from 0.8 mm (0.03 in.) to 4.8 mm (0.19 in.) and were generally on the inside edges of the plates. Removal of the plates identified the cause to be inadequate concrete compaction under the plates which produced large size voids. The problem was corrected by detensioning the tendons of affected plates, reinstalling the plates, pressure grouting and retensioning.

Failures occurred in the top anchor heads of 170-wire rock anchor tendons at Bellefonte nuclear plant (Units 1 and 2) [10]. Anchorage of the 12.2 m (40 ft) long tendons to the rock was to be performed using a two stage grouting operation. Initially the tendons were to be grouted over about one-half their length to anchor the bottom heads. This was to be followed by addition of sufficient material to grout the tendons over their remaining length except for the final 0.9 to 1.5 m (3 to 5 ft.). Coupling of the containment vertical tendons to the rock anchors was to be by means of threaded coupling devices. However, during installation of the rock anchorages failures of the top anchor heads were observed just prior to the second stage of grouting. One anchor head failure was observed in which failure of 23 of 170 wires in a tendon occurred. (Figures 1-2 note some of the features of the anchor head cracking and fractures.) In-depth metallographical and fractographical examinations in conjunction with the study of the environment indicated that the failures were the result of stress corrosion cracking of highly stressed AISI 4140 anchor heads in an aqueous environment of varying pH levels. In addition it was noted that during the period between the first and second stage grouting the top anchor heads were covered with grease cans filled with lime water having a pH of 11 to 13.

In November 1979 four anchor heads of 179-wire tendons failed between 1 and 64 days after post-tensioning the Unit 1 containment at the Byron nuclear plant [11]. A thorough study of the chemistry, metallurgy and fracture phenomena indicated that the failure was due to tempered-martensite embrittlement. Failures were time delayed and occurred in a decreasing stress field.

Concrete cracking and grease leakage were noted at various locations on the dome surface, predominately in the southern portion as shown in Fig. 3, after tensioning of approximately two-thirds of the dome tendons at Turkey Point Nuclear Power Plant (Unit 3) [12]. After a thorough examination of the concrete materials, construction method and prestress tensioning sequence, it was concluded that the dome delaminations were caused by the combined action of inadequate concrete consolidation and weakness at construction joints. Some engineers at NRC, however, believe that the delaminations were caused by exceeding the radial tensile strength of "weak" concrete and that well designed radial reinforcing would help prevent the situation from repeating in the domes of similar containments.

In April 1976, surface cracking and voids in the dome concrete at Unit 3 of Crystal River Nuclear Power Plant were discovered (by accident) after the dome had been constructed and fully post-tensioned (Fig. 4) [13]. Primary causes of the delaminations were thought to be the use of low quality coarse aggregate materials accompanied by high radial tension forces above the top tendons, and compression-tension interaction. Other potential contributing factors were tendon misalignment and construction methods. Corrective measures included detensioning of some of the tendons, removal of the delaminated cap, installation of top orthogonal and radial reinforcing, and installation of a new cap concrete.

5. Regulatory Requirements and Effectiveness of Inservice Inspections of Prestressing Tendons

5.1 Background

Early in the development of PCCs extensive dialogue occurred between the Nuclear Regulatory Commission (known then as the Atomic Energy Commission) and industry relative to the use of portland cement grout as a corrosion inhibitor. Extensive tests were conducted to ensure adequate penetration of grout through vertical bar, hoop, and vertical strand tendons [14-16]. However, the regulators were concerned about not being able to positively check the integrity of the prestressing system throughout the life of the structure. As a result of discussions and public meetings, two regulatory guides were developed: (1) "Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures (RG 1.107)" and (2) "Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons (RG 1.90)." This action permits the use of grouted tendons in containments without time consuming meetings and discussions. Though the intent was to thoroughly scrutinize grout material and installation, and to periodically check the status of containment, these actions did not encourage the use of grouted tendons in PCCs.

5.2 Regulatory Requirements

In the United States it is required that the condition and functional capability of the unbonded post-tensioning systems of prestressed concrete nuclear power plants be periodically assessed. This is accomplished, in part, systematically through an inservice tendon inspection program which must be developed and implemented for each containment. The basis for conducting the inspections is presented in Regulatory Guide 1.35 "Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures (Rev. 2)." The intent of RG 1.35 is to provide utilities with a basis for developing inspection programs and to provide reasonable assurance, when properly implemented, that the structural integrity of the containment was being maintained. The NRC does not require periodic reporting of inspection results except when the technical specification requirements (generally based on RG 1.35) of particular nuclear units are not met, or where there are obvious problems with materials, tendon prestress measurements, and/or an appreciable amount of cracking, grease leakage, etc. Because of the variety of factors such as tendon corrosion, anchorage failure, and material defects which can weaken the containment's structural integrity, the Guide has sought to examine all sources of potential problem areas before they become critical. Basic components covered by the Guide include: sample selection, visual inspection, prestress monitoring tests, tendon material tests and inspections, and inspection of the filler grease.

Tendon sample selection criteria are specified for typical prestressed concrete containments having a shallow dome-shaped roof on cylindrical walls. For the shallow-dome roof containment sample selection includes six dome tendons (two from each 60° group or three from each 90° group), five vertical tendons and ten hoop tendons. For the hemispherical dome-shaped roof containment sample selection criteria include 4% of the U-tendon population (not less than four) and 4% of the hoop tendon population (not less than nine) with each result rounded to the nearest integer. If no problems are uncovered during the first three surveillances (scheduled 1, 3, and 5 years after the initial structural integrity test) then the criteria for sample selection are relaxed. For the shallow-dome roof containment the criteria become three dome tendons (one from each 60° group or one from each 90° group plus one additional randomly selected dome tendon), three vertical tendons and three hoop tendons. For the hemispherical-dome roof containment the criteria becomes: (1) 2% of the U-tendon population with results rounded off to the nearest integer, but not less than two; and (2) 2% of the hoop tendon population with the result rounded off to the nearest integer but not less than three. In all cases, the tendons are to be selected on a random but representative basis.

Anchorage assembly hardware of all tendons selected for inspection are to be examined visually. The method used for removing grease in order to permit examination of the stressing washers, shims, wedges, and bearing plates should neither increase the effects of corrosion nor damage the steel. During integrated leak rate testing (ILRT), while the containment is at its maximum test pressure, visual examination of the exterior of the concrete surface is performed to detect areas of widespread concrete cracking, spalling or grease leakage.

Stress levels of each of the tendons in the sample selected for inspection are monitored by performing lift-off or other equivalent tests. These tests include the measurement of the tendon-force level with properly calibrated jacks and the simultaneous measurement of elongations. Allowable elongations, jacking loads, tolerances, and the influences of such variables as temperature are to be predetermined. Acceptance criteria for the results state that the prestress force measured for each tendon should be within the limits predicted for the time of the test. No more than one tendon per sample may be considered defective or a reportable condition occurs, and the cause of the defect must be located and corrected. If only one tendon per sample is defective, then two additional tendons (one on each side of the defective) are tested. If either or both of the two additional tendons are defective, a reportable condition occurs and the cause of the defect is located and corrected. Otherwise, the single defective tendon is considered unique and acceptable.

Previously stressed tendon wires or strands from one tendon of each type are to be removed from the containment for examination over their entire length to determine if there is evidence of corrosion or other deleterious effects. At least three samples are to be cut from each wire or strand (each end and mid-length) and tensile tests conducted. Where either stress cycling is suspected or a potentially corrosive environment is thought to exist, tests simulating these conditions are to be conducted. At successive inspections, samples should be selected from different tendons.

A sample of grease from each tendon in the surveillance is to be analyzed and the results compared to the original grease specification. The original grease specification is subject to the ASME Code which has limits on the amounts of impurities that may be present at the time of installation (10 ppm on the quantity of water-soluble chlorides, nitrates, and sulfides, but no limit is specified for water content). Also the presence of voids in the grease is to be noted. The method for checking the presence of grease is to take into account: (1) minimum grease coverage needed for different parts of the anchorage system; (2) influence of temperatures variations; (3) procedure used to uncover possible voids in grease in trumpet; and (4) requirements imposed by grease specifications, qualification tests and acceptability limits.

5.3 Experiences from Inspections of PCCs [5, 7]

Three instances of tendon force measurements (lift-off tests) have been reported where the force measured was lower than the minimum required prestress level (40 year losses considered). Probably the most frequently found defect is missing buttonheads, but this problem is generally identified during construction or subsequent inservice inspections, and account is also taken in the design for a few non-effective wires in a tendon or group of tendons. Cracking of anchorheads of buttonhead systems made of AISI 4140 steel has also been reported (apparently due to hydrogen stress cracking); but these incidents also have been identified during construction. Two incidences have been reported of grease leakage through cracks to the exterior surface of the containment apparently due to a combination of inadequate duct joints and grease expansion due to thermal effects. There have also been two incidences of grease discoloration due to containments with the probable cause being entry of contaminated rain water into the tendon ducts during construction. Except for one instance in which a significant amount of water was found in several tendon ducts (despite presence of water, corrosion was found to be minor and steps were taken to eliminate recurrence), little water has been found during inspections. Only a few occurrences of wire corrosion have been identified, but these did not result in wire breaks and were so minor that component replacement was not required (it was concluded that the corrosion had occurred prior to filling the ducts with corrosion inhibitor). There have also been a few incidences of incomplete filling of the tendon ducts with corrosion inhibitors, but this has not caused any serious difficulties and has been corrected.

6. Summary

The evolution of containment systems in the United States is presented as well as motivations for changes. Prestressing systems and the mechanisms utilized for providing corrosion protection of these systems are reviewed. A summary of experiences and problems during construction of PCCs is presented. Results obtained indicate that the few construction problems which occurred were identified and remedied prior to a structure being placed in service. A review of regulatory requirements relative to inservice inspections of prestressing tendons is presented. The few incidences of problems or abnormalities that were identified in these inspections were found to be minor in nature and did not threaten the structural integrity of the containments.

In conclusion, the frequency of occurrence of incidences which could lead to a decrease in the functional capability of PCCs is small, especially considering the number of PCCs in service in the United States. Where problems did occur, they generally were the result of construction practices, and were identified and corrected during either the construction phase, the initial structural integrity test, or in subsequent inservice inspections. Thus it can be concluded that the inspections have been effective in achieving their desired objectives of uncovering and correcting potential problem areas.

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Table 3. Summary of operating and future U.S power reactor containment structures*

Commercial U.S. Power Reactors		Concrete: Prestressed Vertical Cylindrical and Dome, Flat Base	Concrete: Reinforced Concrete Hemispherical Dome Vertical Cylindrical Body and Flat Base	Concrete: Other Concrete Structures	Steel: Light Bulb Torus Containment	Steel: Steel Sphere	Steel: Hemispherical Dome Cylindrical Body and Ellipsoidal Base	Steel: Hybrid Steel Steel Dome and Body Reinforced Concrete Base	Number of Structures: Sub-total	Number of Structures: Sub-total
PWR's	Atmospheric Containment Structures Without Pressure Suppression Systems	55	20	2		11	11		99	120
	Sub Atmospheric Containments		11					*	11	
	Ice Condenser Containments		2					8	10	
BWR's	Mark I			2	23				25	60
	Mark II			10					10	
	Mark III		6					15	21	
	Pre-Mark			2		2			4	
Number of Structures	Sub-total	55	39	16	23	13	11	23		
	Sub-total	110			70					
	Total									180

*Washington 2 is not included in the table.

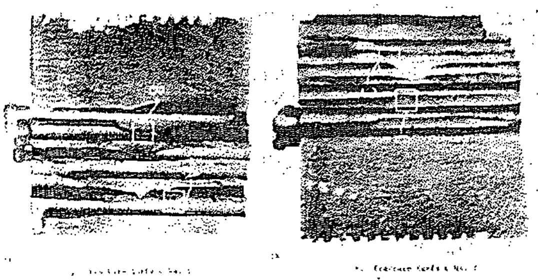


Fig. 1. Appearance of two fracture faces on anchor JA-81-1: Bellefonte.

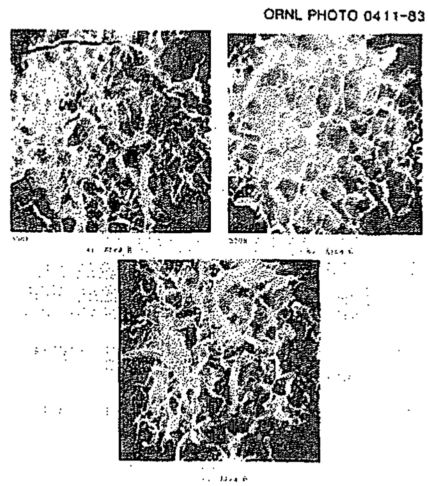


Fig. 2. SEM photographs of areas B, C and D in Fig. 1.

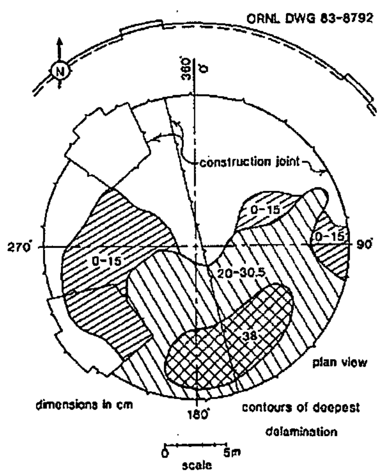


Fig. 3. Extent of dome delamination in Unit 3 containment at Turkey Point.

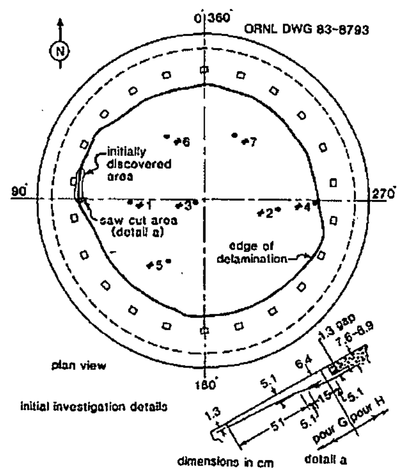


Fig. 4. Extent of dome delamination in Unit 3 containment at Crystal River.

Florida Power
CORPORATION

December 10, 1976

3F1276-10

Mr. John Stolz
Branch Chief
Light Water Reactors Branch I
Division of Project Management
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Crystal River Unit #3
Docket No. 50-302

Dear Mr. Stolz:

We are today supplying you with our final submittal of pages to be incorporated in our interim report Reactor Building Dome Delamination - June 11, 1976.

These pages when properly inserted in the original report will provide you with a single document which will after placement become our final report on the dome delamination.

In addition, we are furnishing a new cover insert to replace the original, so as to more positively identify this document as the Final Report - Reactor Building Dome Delamination - December 10, 1976.

With this submittal, you have in your possession the required "final report regarding the repairs to the containment dome" called for on page 22-1, section 22 - Conclusions of the SER Supplement #2.

Very truly yours,

J T Rodgers

J. T. Rodgers
Asst. Vice President

JTR/iw
Attachment

cc: Mr. Norman C. Moseley, Director w/Att.
Region II, I&C
Atlanta, GA

If the tensile capacity is based upon splitting tensile strength, the resulting value is about $8\sqrt{f'_c}$. The maximum radial tension is around the top tendon layer where the meridional and hoop compression are less than 0.45 f'_c. Since the allowable tensile stress is $3\sqrt{f'_c}$, the design criterion as shown in Figure 3-19 by the dashed line is well within the interaction curve.

3.3.4 Thermal Effects

Two types of thermal effects considered were solar radiation (environmental) and tendon greasing (bulk filling).

a. Solar Radiation

The effect of solar radiation on the surface concrete temperature was calculated. In performing this calculation the initial condition assumed was 60°F throughout the dome thickness. The effect of solar radiation was calculated to heat the dome surface to 152°F. Subsequent to a six hour heat up period, the 6.0 hour gradient shown in Figure 3-20 was calculated. To determine if a thermal shock could have had a significant effect on the stress state in the dome, a sudden cool down due to a thunderstorm was postulated. Therefore, a step function of a six hour quench using a surface temperature of 50°F was assumed.

Figure 3-20 shows the gradients after the initial heat up (0.5 hr), just prior to quench (6.0 hr), after quenching (6.5 hr), and two points along the cooling period (8.0 hr and 12.0 hr).

Using the analytical model described in Section 3.3.2, and the gradients shown in Figure 3-20, the maximum tensile stress at the level of the centerline of the top tendon group was calculated to be 8 psi.

The solar radiational heat also had an affect at construction joint L-M during the three month construction delay. The conduit protruding from the joint had a different temperature than the surrounding concrete. This causes hoop tension around the conduit in the same way as that due to hot grease injection. The temperature gradient is as shown in Figure 3-21 and the maximum tension as calculated by plane strain element of computer program SAP IV (see Appendix D) is 280 psi.

Based upon these studies it is unlikely that the solar effect by itself could have produced other than very limited cracking at the construction joint interface.

$\boxed{\text{b. Tendon Greasing}}$

The field records show that tendon greasing took place in two stages. Eight unstressed tendons were greased prior to stressing the tendons (with the exception of three). The remainder were greased about four months later, after all tendons had been stressed. The second greasing operation was completed in a period of eight days.

The grease was heated prior to injection to reduce its viscosity. According to available field records, the temperature of the grease at the tank outlet was in the range of 150-170°F. It was then pumped via a rubber hose into one tendon conduit at a time. After all the air had been purged from the conduit, pumping ceased and the conduit was sealed at 0 psig.

During hot grease injection, the conduit heated more rapidly than the concrete due to the lower thermal conductivity of concrete. For a step change of temperature from conduit to concrete, the tensile stress in the concrete surrounding the conduit can be calculated from the theory of elasticity. Using the compatibility of radial displacements at the conduit-concrete interface,[11] the tensile stress is determined to be 11 psi/°F. The plane strain element of the SAP IV program as shown in Figure 3-22 yields a tensile stress of 15 psi/°F for the identical condition.

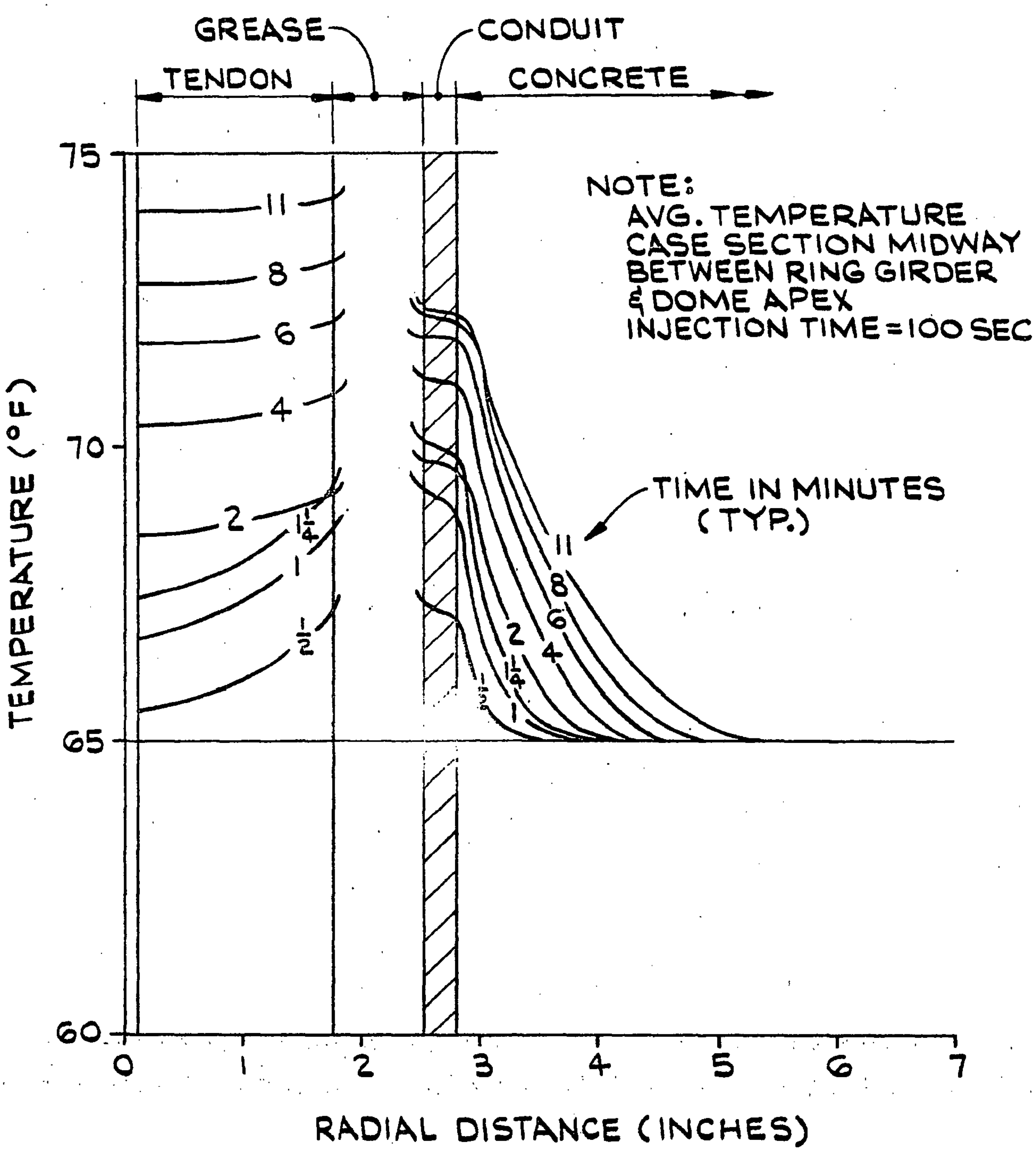
Based upon an averaging of field records of grease temperature in the storage tank and at the conduit outlet, a heat transfer analysis was performed to establish the temperature gradients within the structure due to the greasing operation. The resulting gradients at a point approximately midway between ring girder and dome apex are shown in Figure 3-23. Using the plane strain element in the SAP IV computer program, shown in Figure 3-22, the maximum tension stress is approximately 80 psi, which occurs at the location where the thermal gradient drops to zero. It is recognized that variations in the greasing operation could produce more severe gradients and consequently higher concrete tensile stresses.

The injection pressure of hot grease can also cause tension around the conduit. With the same kind of radial displacement compatibility calculation, the maximum grease pressure reported by the constructor of 85 psi causes 40 psi tension in the concrete by compatibility calculation[11].

The cases studied indicate stresses of sufficient magnitude to cause the delamination only when considered in conjunction with other effects.

3.3.5 Tendon Alignment

To establish the accuracy of the positioning of the tendons within the dome, a survey was conducted to determine the actual dome thickness and the depth from the dome exterior surface to the top of the upper tendon group conduit. The results of the survey are shown in Figures 3-24 through 3-27. This shows that the conduit are high near the periphery and low at the apex and suggests that an increased curvature might exist. A study was performed to determine the significance of



TEMPERATURE GRADIENTS DUE TO TENDON GREASING

FIGURE 3-23

Florida Power Corporation

CRYSTAL RIVER UNIT NO. 3
REACTOR BUILDING PRESTRESSING SYSTEM
TENDON HISTORY

I IDENTIFICATION NUMBER 12V2 CUT LENGTH 188'-6½

SHOP WASHER ID: PC 121 CR 557 FIELD WASHER ID: PC 120 CR 232

1. GAI/QA vendor inspection cover letter number-FPC # 9957 DATE 2/25/74

2. Date tendon received on-site 1-16-74 RMR Number 36036

3. Date installed in conduit 7-3-74 Installation NCR's

Wires removed 0 Wires replaced 0 Total Ineffective wires 0

4. Date buttonheaded 8-27-74 Buttonheading NCR's

Bad wires 0 Accept. Reheads 0 Total Ineffective wires 0

5. Date stressed 10-14-74 Stressing NCR's

Date restressed Restressing NCR's

	SHOP END	FIELD END	TOTAL
Elongation (1500 psi to 80% ult.)-Pred./Act.	12⅛ / 12½	N/A, N/A	12¼ 12½
Lift-Off Pressure - Predicted/Actual	6800 / 7050 PSI	" / "	N/A
Shim Thickness/80% Ultimate Pressure	13" / 7770	4 / "	N/A

Unseated/Broken Wires 0 Total effective wires after stressing 163

6. Date Bulk-filled 10-23-74 Bulk-Filling NCR's

Time since installation 3¼ months Inlet Pressure 112 PSI Outlet Temp. 126°

Date end caps refilled: Shop Field

7. Data compiled by D. Vinlan Organization Salem

Date 4/1/77

8. Additional Comments:

Florida Power CORPORATION

CRYSTAL RIVER UNIT NO. 3
REACTOR BUILDING POST-TENSIONING SYSTEM
TENDON HISTORY

N IDENTIFICATION NUMBER 12 V 3 CUT LENGTH 188'-6 1/2

SHOP WASHER ID: PC 121 CR 587 FIELD WASHER ID: PC 120 CR 176

1. GAI/QA vendor inspection cover letter number-FPC # 9957 DATE 2/25/74
2. Date tendon received on-site 1-16-74 RMR Number 36036
3. Date installed in conduit 7-3-74 Installation NCR's ______

   Wires removed 0 Wires replaced 0 Total Ineffective wires 0
4. Date buttonheaded 7-24-74 Buttonheading NCR's ______

   Bad wires 7 Accept. Reheads 4 Total Ineffective wires 3
5. Date stressed 9-27-74 Stressing NCR's ______

   Date restressed ______ Restressing NCR's ______

	SHOP END	FIELD END	TOTAL
Elongation (1500 psi to 80% ult.)-Pred./Act.	12 1/8 / 12 5/8	N/A / N/A	12 1/8 / 12 5/8
Lift-Off Pressure - Predicted/Actual	6800 / 7000 PSI	" / "	N/A
Shim Thickness/80% Ultimate Pressure	13 3/8 / 7770	4" / "	N/A

Unseated/Broken Wires 0 Total effective wires after stressing 163

6. Date Bulk-filled 10-23-74 Bulk-Filling NCR's ______

   Time since installation 3 3/4 months Inlet Pressure 110 PSI Outlet Temp. 122°

   Date end caps refilled: Shop ______ Field ______
7. Data compiled by D Walker Organization Salem

   Date 4/1/77
8. Additional Comments:

Florida Power Corporation

CRYSTAL RIVER UNIT NO. 3
REACTOR BUILDING PRESTRESSING SYSTEM
TENDON HISTORY

I IDENTIFICATION NUMBER 12 V 4 CUT LENGTH 188'-7

SHOP WASHER ID: PC 121 CR 579 FIELD WASHER ID: PC 120 CR 267

1. GAI/QA vendor inspection cover letter number-FPC # 9957 DATE 2/25/74

2. Date tendon received on-site 1-18-74 RMR Number 36073

3. Date installed in conduit 7-3-74 Installation NCR's

   Wires removed 0 Wires replaced 0 Total Ineffective wires 0

4. Date buttonheaded 8-27-74 Buttonheading NCR's

   Bad wires 5 Accept. Reheads 4 Total Ineffective wires 1

5. Date stressed 10-14-74 Stressing NCR's

   Date restressed Restressing NCR's

	SHOP END	FIELD END	TOTAL
Elongation (1500 psi to 80% ult.)-Pred./Act.	12 1/8 / 12 7/8	N/A / N/A	12 1/8 / 12 7/8
ft-Off Pressure - Predicted/Actual	6760 / 6850 PSI	" / "	N/A
Shim Thickness/80% Ultimate Pressure	14 3/16 / 7720	4" / "	N/A

Unseated/Broken Wires Total effective wires after stressing 162

6. Date Bulk-filled 1-30-75 Bulk-Filling NCR's

   Time since installation 6 months Inlet Pressure 140 LBS Outlet Temp. 122°

   Date end caps refilled: Shop Field

7. Data compiled by D. P. Walker Organization Salem

   Date 4/1/77

8. Additional Comments:

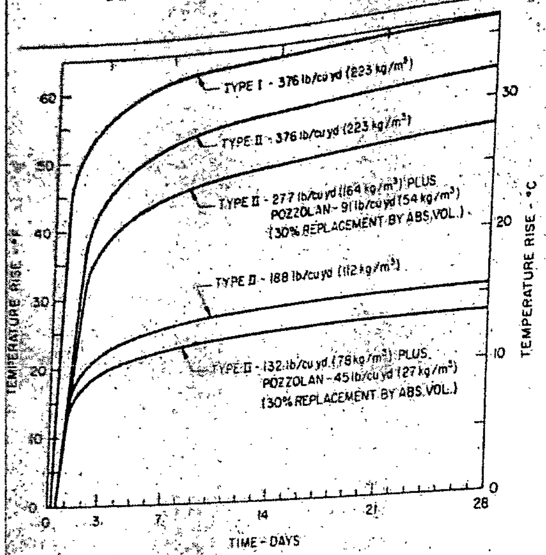


Figure 4-8 Effect of cement and pozzolan contents on temperature rise in concrete. (From R. W. Carlson et al., *J. ACI*, Proc., Vol. 76, No. 7, 1979.)

The use of a low cement content, an ASTM Type II portland cement instead of Type I, and a partial substitution of the portland cement by a pozzolan, are effective means by which the adiabatic temperature rise in mass concrete can be significantly reduced.

while maintaining a satisfactory level of strength. In this way the temperature rise in concrete can be limited to about 16 °C. A partial substitution of the cement by 30 volume percent of pozzolan can further reduce the temperature rise to a mere 13 °C.

Precooling of fresh concrete is another commonly used method of controlling the subsequent temperature drop. Often, chilled aggregates and ice shavings are specified for making mass concrete mixtures in which temperature of fresh concrete is limited to 10 °C or less. During the mixing operation the latent heat needed for fusion of ice is withdrawn from other materials of the concrete mixture, providing a very effective way to lower the temperature.

Selecting an ***aggregate with a low coefficient of thermal expansion*** when it is economically feasible and technologically acceptable may under certain conditions become a critical factor for crack prevention in mass concrete. This is because the thermal shrinkage strain is determined both by the magnitude of the temperature drop and the linear coefficient of thermal expansion of concrete; the latter, in turn, is controlled primarily by the linear coefficient of thermal expansion of the aggregate. The reported values of the linear coefficient of thermal expansion for saturated portland cement pastes of varying water/cement ratios, for mortars containing 1:6 cement/natural silica sand, and for concrete mixtures of different composition are approximately 18, 12, and 6 to 12×10^{-6} per °C, respectively. The coefficient of thermal expansion of commonly used rocks and minerals varies from about 5×10^{-6}

per °C for limestones and gabbros to 10 to 11×10^{-6} per °C for sandstones, natural gravels, and quartzite. Since the coefficient of thermal expansion of concrete can be estimated from the weighted average of the components, assuming 70 to 80 percent aggregate in the concrete mixture, the calculated values of the coefficient for various rock types (both coarse and fine aggregate from the same rock type) are shown in Fig. 4-9. The data in Fig. 4-9 are fairly close to the experimentally measured values of thermal coefficients reported in the published literature for concrete tested in moist conditions, which is representative of the condition of typical mass concrete. Compared to concrete in the moist state, the corresponding concrete in the air-dried state would show a slightly higher coefficient, probably as a result of increase in density.

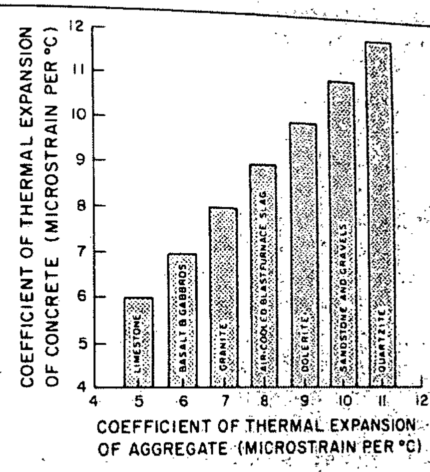


Figure 4-9 Influence of the aggregate type on the coefficient of thermal expansion of concrete.

Since the coefficient of thermal expansion of concrete is directly related to the coefficient of expansion of the aggregate present, in mass concrete the selection of an aggregate with a lower coefficient provides another approach toward lowering the thermal strain.

DRYING SHRINKAGE AND CREEP

Generally, the stress effects from drying shrinkage and creep strains in concrete are not the same (i.e., under conditions of restraint the former is stress inducing while the latter is stress relieving). However, for a variety of reasons it is desirable to discuss both phenomena together. First, both drying shrinkage and creep originate from the same source, the hydrated cement paste; second, the strain–time curves are very similar; third, the factors that influence the drying shrinkage also influence the creep and generally in the same way; fourth, in concrete the microstrain of each, 400 to 1000×10^{-6}, is large and cannot be ignored in structural design; and fifth, both are partially reversible.

Coefficient of Thermal Expansion (CTE) of concrete

From P. Kumar Mehta, “Concrete: Structure, Properties, and Materials” (1986), pages 88 and 89.

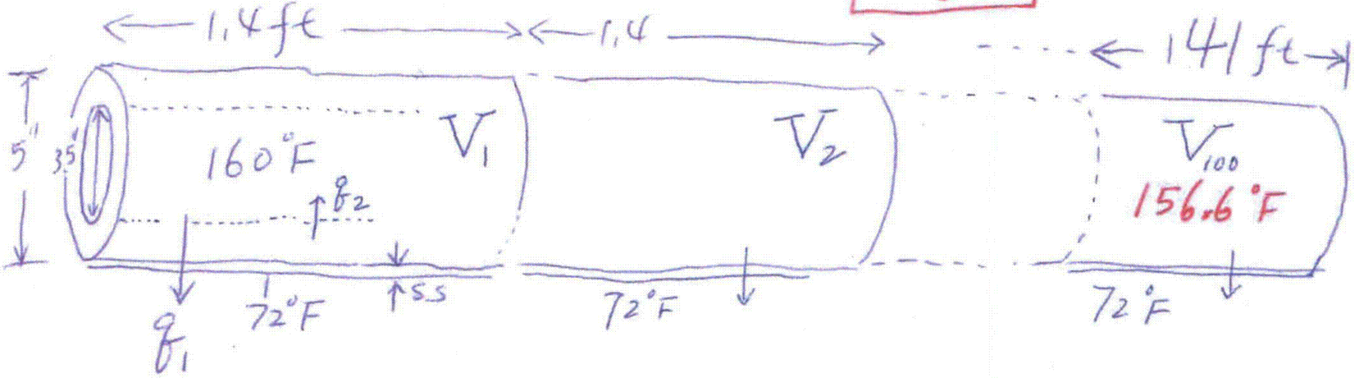
- The coefficient of thermal expansion (CTE) of concrete depends on the CTE of the coarse aggregates;
- It varies in the range 6 to 12 $\times 10^{-6}$ /^{o}C;
- For limestone coarse aggregates, the CTE is close to 6 $\times$ 10^{-6} /^{o}C;

This compares with steel CTE close to 6 $\times$ 10^{-6} /^{o}C.

Thermal Conductivity - *k* - *(W/mK)*			
Material/Substance	Temperature (C)		
	25	125	225
Acetone	0.16		
Acrylic	0.2		
Air	0.024		
Alcohol	0.17		
Aluminum	250	255	250
Aluminum Oxide	30		
Ammonia	0.022		
Antimony	18.5		
Argon	0.016		
Asbestos-cement board	0.744		
Asbestos-cement sheets	0.166		
Asbestos-cement	2.07		
Asbestos, loosely packed	0.15		
Asbestos mill board	0.14		
Asphalt	0.75		
Balsa	0.048		
Bitumen	0.17		
Benzene	0.16		
Beryllium	218		
Brass	109		
Brick dense	1.31		
Brick work	0.69		
Cadmium	92		
Carbon	1.7		
Carbon dioxide	0.0146		
Cement, portland	0.29		
Cement, mortar	1.73		
Chalk	0.09		
Chrome Nickel Steel (18% Cr, 8 % Ni)	16.3		
Clay, dry to moist	0.15 - 1.8		

Concrete, stone	1.7		
Constantan	22		
Copper	401	400	398
Corian (ceramic filled)	1.06		
Corkboard	0.043		
Cork, regranulated	0.044		
Cork	0.07		
Cotton	0.03		
Carbon Steel	54	51	47
Cotton Wool insulation	0.029		
Diatomaceous earth (Sil-o-cel)	0.06		
Earth, dry	1.5		
Ether	0.14		
Epoxy	0.35		
Felt insulation	0.04		
Fiberglass	0.04		
Fiber insulating board	0.048		
Fiber hardboard	0.2		
Fireclay brick 500°C	1.4		
Foam glass	0.045		
Freon 12	0.073		
Gasoline	0.15		
Glass	1.05		
Glass, Pearls, dry	0.18		
Glass, Pearls, saturated	0.76		
Glass, window	0.96		
Glass, wool Insulation	0.04		
Glycerol	0.28		
Gold	310	312	310
Granite	1.7 - 4.0		
Gypsum or plaster board	0.17		
Hairfelt	0.05		
Hardboard high density	0.15		
Hardwoods (oak, maple..)	0.16		
Helium	0.142		
Hydrogen	0.168		
Ice (0°C, 32°F)	2.18		
Insulation materials	0.035 - 0.16		
Iridium	147		
Iron	80	68	60
Iron, wrought	59		
Iron, cast	55		
Kapok insulation	0.034		
Kerosene	0.15		
Lead Pb	35		
Leather, dry	0.14		
Limestone	1.26 - 1.33		
Magnesia insulation (85%)	0.07		
Magnesite	4.15		
Magnesium	156		
Marble	2.08 - 2.94		
Mercury	8		
[illegible]	[illegible]		

Molybdenum	138		
Monel	26		
Nickel	91		
Nitrogen	0.024		
Nylon 6	0.25		
Oil, machine lubricating SAE 50	0.15		
Olive oil	0.17		
Oxygen	0.024		
Paper	0.05		
Paraffin Wax	0.25		
Perlite, atmospheric pressure	0.031		
Perlite, vacuum	0.00137		
Plaster, gypsum	0.48		
Plaster, metal lath	0.47		
Plaster, wood lath	0.28		
Plastics, foamed (insulation materials)	0.03		
Plastics, solid			
Platinum	70	71	72
Plywood	0.13		
Polyethylene HD	0.42 - 0.51		
Polypropylene	0.1 - 0.22		
Polystyrene expanded	0.03		
Porcelain	1.5		
PTFE	0.25		
PVC	0.19		
Pyrex glass	1.005		
Quartz mineral	3		
Rock, solid	2 - 7		
Rock, porous volcanic (Tuff)	0.5 - 2.5		
Rock Wool insulation	0.045		
Sand, dry	0.15 - 0.25		
Sand, moist	0.25 - 2		
Sand, saturated	2 - 4		
Sandstone	1.7		
Sawdust	0.08		
Silica aerogel	0.02		
Silicone oil	0.1		
Silver	429		
Snow (temp < 0^oC)	0.05 - 0.25		
Sodium	84		
Softwoods (fir, pine ..)	0.12		
Soil, with organic matter	0.15 - 2		
Soil, saturated	0.6 - 4		
Steel, Carbon 1%	43		
Stainless Steel	16	17	19
Straw insulation	0.09		
Styrofoam	0.033		
Tin Sn	67		
Zinc Zn	116		
Urethane foam	0.021		
Vermiculite	0.058		
Vinyl ester	0.25		



As stated in Figure, it (I assume) took 100 Sec to fill the conduit. For a total length of 141 ft, I assume the flow velocity = V = 1.41 ft/s

For 1 Second, the grease travels and occupies 1.4 ft worth of volume $V = (1.4)\pi(D_{conduit}^2 - D_{Ten}^2) = 1.4\frac{\pi}{144}(5^2 - 3.5^2) = 0.389\,ft^3$

$D_i = 5"$, $D_o = 5.25"$

$d_H = \frac{4A}{P}$

For grease, $K_g = 0.5\ Btu/hr \cdot ft \cdot F$

$C_p = 0.5\ Btu/lb_m \cdot F$

Hydraulic Dia $d_H = 0.5\,ft$

Flow cross section $A = 0.278\,ft^2$

grease wall film H.T. coefficient $h_g = 4.4\ Btu/hr \cdot ft \cdot F$

$\rho = 50\ lbm/ft^3$

$D_T = 3.5"$

Thermal conductivity of S.S. $K_{S.S.} = 11\ Btu/hr \cdot ft \cdot F$

grease Bulk Temperature at inlet = $T_B = 160F$

Conduit Surface Temp. = $T_w = 72F$

There are 2 components of heat transfered out of grease,

q_1 from grease through conduit wall to concrete

q_2 from grease to tendon.

when grease travels from Volume V_1 to V_2, temperature drops due to heat transfer.

Temperature drop $\Delta T_{1\text{-}2} = \frac{q}{\rho_m V C_p}$

$q = q_1 + q_2$

For $t = 1$ Sec. $L = 1.4\,ft$

$$q_1 = (T_B - T_W)\left[\frac{\pi D_i L}{\frac{D_i}{K_{ss}}\ln\left(\frac{D_o}{D_i}\right) + \frac{1}{h_g}}\right]$$

$$= \frac{(160-72)}{3600}\left[\frac{(3.14)(\frac{5}{12})(1.4)}{\frac{(\frac{5}{12})}{11}\ln\left(\frac{5.25}{5}\right) + \frac{1}{4.4}}\right] = 0.1956\ Btu/F$$

$$q_2 = (T_B - T_W)\, h\, \pi D_T L = \frac{(160-72)}{3600}(4.4)(3.14)\left(\frac{3.5}{12}\right)(1.4) = 0.137\ Btu/F$$

$$q_t = q_1 + q_2 = 0.3326\ Btu/F$$

Temperature drop from Volume 1 $[V_1]$ to volume 2 $[V_2]$

$$\Delta T_{1-2} = \frac{q}{\rho_m V C_p} = \frac{0.3326}{(50)\,0.389\,(0.5)} = 0.0342\,F$$

This means for grease traveling 1.4 ft (in One second) the temperature drops only 0.0342 °F. [Which is not much.]

This also helps the calculation for temperature drop from Volume 2 to Volume 3.

$$\Delta T_{2-3} = (0.0342)\frac{[(160-0.0342)-72]}{160-72} = 0.0342\ °F$$

I can therefore estimate that when grease reaches the end of the Conduit, the bulk temperature will be

$160 - 0.0342 \times (100) = 156.6\,F$

The hot spot therefore occurs at the entrance and at the beginning. [Thermal Shock.]

The temperature drop in the S.S. conduit wall $\Delta T_w \Rightarrow$ will become:

$$q_1 = K_{ss} \, S \cdot \Delta T_w \Rightarrow \frac{\pi L \Delta T_w K_{ss}}{\ln\left(\frac{D_o}{D_i}\right)} \Rightarrow$$

S: Shape factor

$$0.1956 = \frac{3.14\,(1.4)}{0.04879} \cdot \frac{11}{3600} \cdot \Delta T_w \Rightarrow T_w = 0.71\,F$$

The total temperature drop from grease bulk (160F) to conduit wall surface (72F) = 160 - 72 = 88 °F

The bulk of the temperature drop is in the grease wall film = 88 - 0.71 = 87.3 F

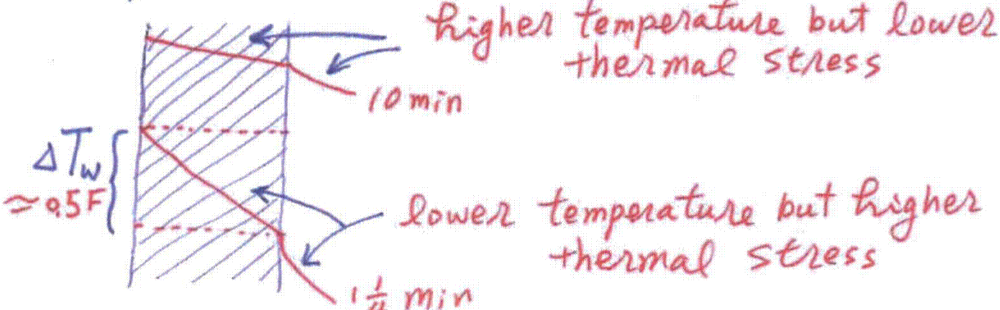
This is consitent with Figure 3-23.
What I demonstrated above is based on a semi-Steady State approach, which is good for scoping study.
For thermal stress analysis, transient effects should be considered. The good news is that the magnitude of the temperature drop in S.S. wall is so small, with the transient effect considered, the effect on thermal stress is still not significant.

As the grease injection takes place for 100 Seconds (See note in Figure 3-23), the heating source will be Kept for 100 Seconds
100 S = $1\frac{2}{3}$ minutes. After 100 s, the bulk of grease will cool down, Yet, heat transfered into Solid will have minutes delayed. (10 minutes later, bulk wall temperature still goes up.)

These are what we see in the temperature transient profiles in conduit wall and are consistent with my calculations and conclusions. [See Figure 3-23, profiles in wall for 1/2, 1, $1\frac{1}{4}$, 2, 4 minutes, etc.]

A higher thermal stress is experienced at higher SLOPE of a temperature profile. The relatively higher slopes occur at 1, $1\frac{1}{4}$ minutes and before 2 minutes. After 2 minutes, slope becomes "mellowed down."

When the slope is near the highest, the temperature drop across the wall (Eye-ball it for the 1 minute curve) is $\Delta T_W \simeq 0.5\,°F$, which is in the ball park of my calculated number, 0.71 F.



My Conclusions are

1. As grease reaches the end of conduit, the grease bulk temperature would drop by roughly 3 °F
2. The temperature drop across the conduit wall is less than 1 °F, steady state or transient wise.
3. The highest thermal stress should occur close to 2 minutes from the starting of grease injection near entrance.

Jason Chao 01/02/2010

Supporting Document for Data Request #205 (Grease injection at 85 psi while tendon sleeves can handle 10 psi)

It was reported in the dome delamination report that tendon grease was injected at up to 85 psi. The Requirement Outline for the tendon sleeves (RO-3040) identifies the ability of the sleeves to handle a hydrostatic pressure of 10 psig without leaking water as a performance requirement. This appears to be a discrepancy, especially considering that the hydrostatic pressure in the vertical tendon sleeves will exceed 10 psig due to the column of grease that normally exists. This evaluation will determine if the tendon sleeves could reasonably accommodate the reported injection pressure without failure.

The vertical tendons extend from elevation 80'-6" to 267'-6" per Prescon Drawings 5EX-003-P-03 and 5EX-003-P-40. The total elevation change is 187'.

The tendon grease is Visconorust 2090P-4. Per the MSDS, the specific gravity is 0.885. The density of the grease is 0.885 * 62.4 lb/ft^3 = 55.2 lb/ft^3. The pressure at the bottom of the tendon sleeve is therefore 55.2 lb/ft^3 * 1 ft^2/144 in^2 = 0.384 psi per foot of elevation of grease. For a full tendon sleeve, the pressure is 187 ft * 0.384 psi/ft = <u>72 psi</u>.

The tendon sleeves are either a rigid or flexible type. From a pressure retaining standpoint, the thinner-wall flexible sleeves are more limiting. From RO-3040, the flexible sleeves are 5-1/4" OD, minimum 22 gauge (0.028) corrugated galvanized steel. The equation for minimum wall thickness from USAS B31.1 can be used to determine the pressure capability of the sleeves:

$$P = [2*S*E*t_m]/[D_o-2*y*t_m]$$

Where S = maximum allowable stress, 10,600 psi [1]
E = joint efficiency, 0.8[2]
t_m = minimum wall thickness, 0.028 in
D_o = outside diameter, 5.25 in
y = constant, 0.4 for temperatures below 900F

$$P = [2*10,600*0.8*0.028]/[5.25-2*0.4*0.028]$$

P = 91 psig internal pressure

The above calculation shows that the tendon sleeves could handle the reported injection pressure as well as the normal hydrostatic pressure, even when considering conservative material and joint efficiency properties. When considering more realistic properties and the safety factor inherent in the allowable stress value, there exists considerable margin in the sleeve material.

Prepared by Craig Miller 1/09/10

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[1] Tendon sleeve material is unknown. The minimum allowable stress for non-butt welded carbon steel, as given in Table A-1 of B31.1, is 10,600 psi at 200F.
[2] B31.1 lists joint efficiency from 0.80 to 1.0 for various longitudinal welds (excluding furnace butt welds, which would not be practical for corrugated pipes). The lowest value of 0.80 is selected for conservatism.

CALCULATION SHEET

ROUGH ESTIMATE OF THE IMPACT OF TENDON GREASE INJECTION ON CONCRETE TENSILE STRESSES

A. DISCUSSION

After installation and tensioning of the containment building tendons, hot grease is injected at moderate pressure to seal the void in the tendon conduit. The injection temperature was typically about 140 F with a limit of 200 F, and pressure was typically about 40 psi with a limit of 150 psi. Both temperature and pressure have the potential for producing a tensile load on the concrete surrounding the tendon conduit. This calculation sheet places an upper limit on that potential impact.

B. REFERENCED PARAMETERS

- Tendon strand diameter: 7mm
- # of strands per tendon: 163
- Conduit OD: 5.25"
- Conduit wall thickness: 1/16"
- Max. grease temperature: 200 F
- Max. grease pressure: 150 psi
- Assumed linear coefficient of thermal expansion for steel and concrete: 7 E-6 /F
- Assumed pressure coefficient for steel and concrete: 30 E6 psi
- Assumed concrete ambient temperature: 70 F
- Containment wall thickness: 42 in
- Conduit/ft = 157 ft/ 94 tendons = 0.60 tendons/ft vertical

C. CONDUIT RADIAL EXPANSION DUE TO PRESSURE

dR/R = 1/(30 E 6 psi) * 150 psi = 0.000005

D. CONDUIT RADIAL EXPANSION DUE TO TEMPERATURE

dR/R = 7 E-6 /F *(200 F – 70 F) = 0.0009

Cross-sectional area of one strand is 0.06 sq in
Total cross-sectional area of 163 strands is 9.7 sq in
Total cross-sectional area of conduit is 21.6 sq in
Grease fraction is 45%

E. CONCRETE TENSILE STRESS DUE TO EXPANDED CONDUIT

Conduit expansion pressure is 0.0009 * 30 E6 psi = 27 ksi
Expansion force per linear inch of conduit is 27 ksi * 1/16 in = 1700 lbf/in

Average tensile pressure on concrete is 0.60 conduit/ft * 1 ft/ 12 in *1700 lbf/in = 85 psi

F. CONCLUSIONS

- Radial expansion due to pressure is negligible relative to temperature
- Radial expansion due to temperature (assuming conduit goes immediately to 200F) produces an average concrete tensile stress of 85 psi
- The thermal expansion impact is actually significantly less than estimated here since actual outlet temperatures were about 140 F instead of the limit of 200 F and the heat loss that occurred by 45% volume grease surrounded by 55% volume steel strands. Actual stress is more like 20 psi.

By Ray Waldo, PII team

Systems	MX
Calc. Sub-Type	-
Priority Code	3
Quality Class	Safety-Related

NUCLEAR GENERATION GROUP

ANALYSIS / CALCULATION

S09-0044
(Calculation #)

Predicted Tendon Elongations for Restoration of SGR Access Opening
(Title including structures, systems, components)

☐ BNP UNIT ______

☒ CR3 ☐ HNP ☐ RNP ☐ NES ☐ ALL

APPROVAL ☐ Electronically Approved

Rev #	Prepared By	Reviewed By	Supervisor
0	Signature Name Amir Moid Date 9/14/09	Signature Chris Sward Name Chris Sward Date 09/14/09	Signature Chris Sward Name Chris Sward Date 09/14/09

(For Vendor Calculations)

Vendor Sargent & Lundy LLC Project No. 11550-048 Vendor Document No. N/A

Owner's Review By John Holliday Date 9/15/09

9/15/09

1.0 Purpose

The purpose of this calculation is to determine the theoretical (predicted) elongation of containment post-tensioning tendons that are either to be replaced or re-stressed due to Steam Generator Replacement (SGR). The elongation to be predicted is the immediate elongation that will occur during stressing from an initial force that will remove all slack to the maximum force at 80% of ultimate capacity.

A description of the opening in the containment shell, as well as the tendons to be removed or de-tensioned for the SGR, is described in calculations S06-0002 and S06-0005 (Refs. 4 and 5).

2.0 References

1) DBD11, Design Basis Document for the Containment, Rev. 6.
2) 5EX7-003 P-10A, Rev. 1, Horizontal Tendon Detail, (Prescon).
3) 5EX7-003 A-07, Rev. 2, Anchor Detail at Buttress, (Prescon).
4) Calculation No. S06-0002, Rev. 1, "Containment Shell Analysis for Steam Generator Replacement - Design Criteria".
5) Calculation No. S06-0005, Rev. 1, "Containment Shell Analysis for Steam Generator Replacement - Shell Evaluation During Replacement Activities".
6) 5EX7-003 P-2, Rev. 3, Vertical Tendon Placement, (Prescon).
7) 5EX7-003 P-3, Rev. 3, Vertical Tendon Placement, (Prescon).
8) 5EX7-003 P-4, Rev. 3, Vertical Tendon Placement, (Prescon).
9) 5EX7-003 P-15, Rev. 0, Vertical Tendon Placement, (Prescon).
10) 5EX7-003 P-16, Rev. 1, Vertical Tendon Placement, (Prescon).
11) 5EX7-003 P-17, Rev. 0, Vertical Tendon Placement, (Prescon).
12) 5EX7-003 P-33, Rev. 0, Vertical Tendon Placement, (Prescon).
13) 5EX7-003 P-34, Rev. 0, Vertical Tendon Placement, (Prescon).
14) 5EX7-003 P-35, Rev. 0, Vertical Tendon Placement, (Prescon).
15) 5EX7-003 P-40, Rev. 2, Vertical Tendon Placement, (Prescon).
16) 5EX7-003 P-41, Rev. 4, Vertical Tendon Placement, (Prescon).
17) 5EX7-003 P-21, Rev. 2, Hoop Tendon Placement, (Prescon).
18) 5EX7-003 P-22, Rev. 1, Hoop Tendon Placement, (Prescon).
19) 5EX7-003 P-23, Rev. 1, Hoop Tendon Placement, (Prescon).
20) 5EX7-003 P-27, Rev. 1, Hoop Tendon Placement, (Prescon).
21) 5EX7-003 P-28, Rev. 1, Hoop Tendon Placement, (Prescon).
22) 5EX7-003 P-29, Rev. 0, Hoop Tendon Placement, (Prescon).
23) ACI 318-63, "Building Code Requirements for Structural Concrete".
24) EC 63016, Rev. 5, “Containment Cooling”.

3.0 Design Inputs

The following design inputs are used in this calculation:

- Tendon properties and design parameters from DBD11 (Ref. 1):

  Curvature friction coefficient, μ=0.16
  Wobble coefficient, K=.0003
  Number of tendon wires, n_w=163
  Effective area of tendons, A_t=9.723 in^2
  Tendon ultimate stress, σ_{ult}=240 ksi

- Modulus of elasticity for carbon steel; E=29000 ksi
- Hoop and Vertical tendon layout from Prescon Dwgs. Refs. 2 and 3
- Vertical tendon profile geometry from Prescon Dwgs. Refs. 6-16
- Hoop tendon profile geometry from Prescon Dwgs. Refs. 17-22
- The initial tendon stress to remove slack to be 360 kips, from EC 63016 (Ref. 24).

4.0 Assumptions

There are no unverified assumptions in this evaluation.

Tendon 23V01

E=	29000	ksi	Modulus of elasticity of steel wires
A_{wire}=	0.05965	sq in	Area of pre-stressing tendon wire (Area of 163 wire tendon=9.723 in^2, per Pg. 6, DBD11, Ref. 1)
μ=	0.16		Curvature friction coefficient, per Pg. 27, DBD11 (Ref. 1)
K=	0.0003		Wobble coefficient, per Pg. 27, Ref. 1
r=	67.28	ft	Radius of vertical tendons about center of containment shell, per Prescon Dwg. P10-A (Ref. 2).
n_w=	**163**		Number of effective wires, per Pg. 6, Ref. 1 and Att. 1
A_t=	9.72	sq in	Effective area of tendon
T_i=	**360**	kips	Initial force in tendon to remove slack
T_f=	**1867**	kips	80% of ultimate tendon force (tendon ultimate stress=240 ksi, per Pg. 5, Ref. 1)
T_o=	1507	kips	Net force applied to tendon

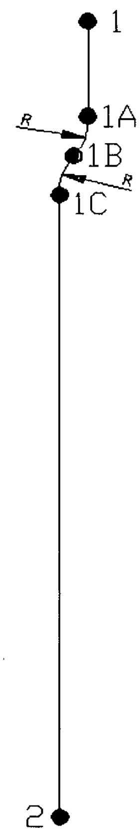
Point	Elevation	Azimuth	y	A°	c	d	R	B	ΣB	ΔL	L	T_x	T_{ave}	δ
1	268.00	117.58										1507.00		
1A	250.00	117.58	18.00	0.00	0.00	18.00	0	0.00	0.00	18.00	18.00	1498.88	1502.94	1.15
1B	242.54	118.17	7.46	0.58	0.68	7.49	41	0.18	0.18	7.51	25.51	1452.35	1475.62	0.47
1C	235.03	118.75	7.51	0.58	0.69	7.54	41	0.18	0.37	7.55	33.05	1407.02	1429.68	0.46
2	79.67	118.75	155.36	0.00	0.00	155.36	0	0.00	0.37	155.36	188.41	1342.94	1374.98	9.09

Predicted Elongation: **11.17** in

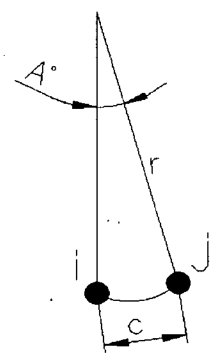
Total Tendon Length= **188.41** ft

δ_{nf}= 12.08 in

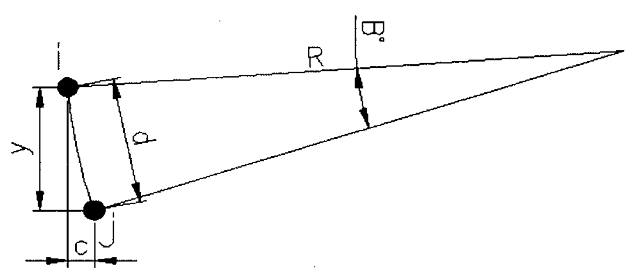
y	Vertical distance between points i and j (ft)
A°	Circumferential change from point i to j (degrees)
c	Cicumferential distance between points i and j (ft)
d	Absolute distance between points i and j ($d^2=c^2+y^2$), (ft)
R	Radius of curvature of tendon between points i and j (ft)
B	Angular change between points i and j (radians)
ΣB	Total angular change from tensioning end to point j (radians)
ΔL	Length of tendon between points i and j (ft)
L	Total length of tendon from tensioning end to point j (ft)
T_x	Force in tendon at point j (kips)
T_{ave}	Average force in tendon segment between points i and j (kips)
δ	Elongation of tendon segment between points i and j (in)
δ_{nf}	Predicted elongation of tendon assuming no friction loss $\delta_{nf}=T_o*L_2/(E*A_t)$



Tendon Profile



Circumferential Offset

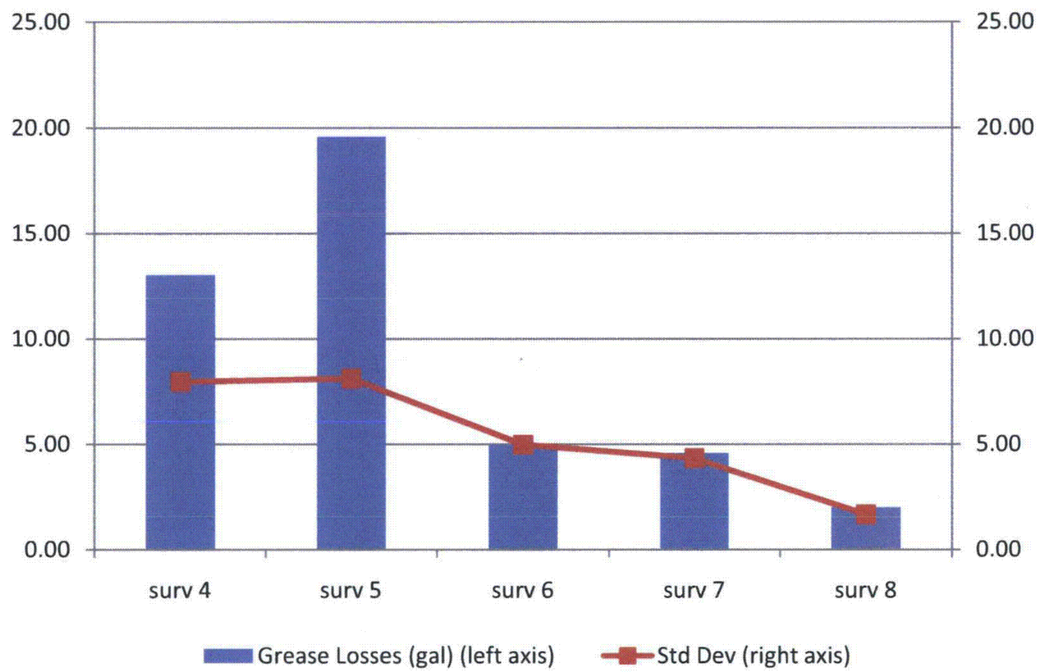


Angular Change

Calculation Wire Elongation

Young's Modulus, E	2.90E+07	psi	
Guaranteed UTS	240	ksi	
Wire Diameter	0.276	in	(7 mm)
Wire Area	0.0596	in^2	
Force per wire	11,443	psi	(80% x 240 ksi x 0.0596 in^2)
Wire length	2,256	in	(188 ft for vertical tendons)
Elongation, Δ=FL/AE	14.94	in	
Correction for 360 kips			
Tendon force 80% GUTS	1,865,242	lbs	(11,443 psi x 163 wires)
Reduced tendon force	1,505,242	lbs	(1,865 kips - 360 kips)
Reduced percentage	19.30%		(1,865 - 1,505) kips / 1,865 kips
Reduced elongation	12.05	in	(14.94 * (1 - 0.193))

Grease losses from tendon sleeves



Average and standard deviation of grease losses for each surveillance

GREASE REMOVAL AND REPLACEMENT

TABLE 4

Crystal River
Unit 3

TENDON NUMBER	END DESIGNATION	GREASE REMOVED (gals.)	GREASE REPLACED (gals.)	NET DIFFERENCE :
12V1	Field + Shop	29.9	34.0	4.1 Gallons
34V4	Filed + Shop	12.9	17.9	5.0
56V2	Field + Shop	89.9	92.9	3.0
D105	Field + Shop	11.0	34.0	23.0
D212	Field + Shop	4.0	16.0	12.0
D328	Field + Shop	8.0	16.0	8.0
13H20	Field + Shop	2.0	21.1	19.1
13H40	Field + Shop	4.0	24.5	20.5
H26	Field + Shop	2.0	21.1	19.1
51H27 *	Field + Shop	1.0	5.0	4.0
51H41	Field + Shop	2.0	25.6	23.6
64H19	Field + Shop	2.0	16.8	14.8

* Unsealed in error.

TABLE 5: SUMMARY OF GREASE REMOVAL AND REPLACEMENT

TENDON	GREASE REMOVED (GALLONS)	GREASE REPLACED (GALLONS)	NET DIFFERENCE *
34V6	30	71.5	41.5
56V15	96	116	20
61V14	64	82	18
D215	19	51	32
D224	38	51	13
D231	40	51	11
35H1	02	22	20
42H1	02	22	20
46H21	04	19	15
46H28	3.5	19	15.5
46H29	07	22	15
46H30	04	17	13
46H47	04	24	20
62H8	02	22	20

* Acceptance Criteria: Net difference shall not exceed 4 gallons. Please refer to the applicable NCR where it is noted that final evaluation and disposition of acceptance criteria is deferred to Gilbert Commonwealth (to be addressed in their engineering report).

20TH YEAR SURVEILLANCE OF THE POST-TENSIONING SYSTEM AT THE CRYSTAL RIVER NUCLEAR PLANT UNIT 3

TABLE XII: SUMMARY OF DATA SHEETS SQ 12.1
GREASE LOSS Vs GREASE REPLACEMENT

TENDON	GREASE REMOVED			GREASE REPLACED			DIFF.	NET	%
	SHOP	FIELD	TOTAL (GAL.)	SHOP	FIELD	TOTAL (GAL.)	(GAL.)	VOLUME	
12V1	2.00	36.75	38.75	40.75	0.75	41.50	+2.75	143.46	1.91
23V2	0.75	9.75	10.50	11.50	0.00	11.50	+1.00	142.52	0.70
61V21	0.75	88.50	89.25	20.25	72.50	92.75	+3.50	144.03	2.43
43V04	0.50	0.00	0.50	7.50	0.00	7.50	+7.00	N/A	N/A
D113	4.50	4.00	8.50	6.20	3.50	9.70	+1.20	115.11	1.04
D115	4.50	4.50	9.00	7.00	4.00	11.00	+2.00	117.17	1.71
D212	6.50	4.00	10.50	20.00	4.50	24.50	+14.00	115.55	12.12♦
D304	12.00	7.25	19.25	38.20	0.00	38.20	+18.95	103.68	18.30♦
D311	24.00	4.00	28.00	46.50	3.50	50.00	+22.00	115.12	19.10♦

♦ SEE NCR No. FN604-018, 019, 020

Precision Surveillance Corporation

20TH YEAR SURVEILLANCE OF THE POST-TENSIONING SYSTEM AT THE CRYSTAL RIVER NUCLEAR PLANT UNIT 3

Florida Power CORPORATION

TABLE XII: SUMMARY OF DATA SHEETS SQ 12.1 GREASE LOSS Vs GREASE REPLACEMENT

TENDON	GREASE REMOVED			GREASE REPLACED			DIFF.	NET	%
	SHOP	FIELD	TOTAL (GAL.)	SHOP	FIELD	TOTAL (GAL.)	(GAL.)	VOLUME	
42H18	3.75	2.00	5.75	5.25	5.25	10.50	+4.75	121.40	3.91
42H29	3.50	3.25	6.75	5.25	6.25	11.50	+4.75	121.48	3.91
42H30	3.00	3.00	6.00	7.00	7.00	14.00	+8.00	121.59	6.58 ♦
42H31	2.50	3.00	5.50	5.25	5.75	11.00	+5.50	121.84	4.51
42H32	3.75	2.50	6.25	6.25	4.75	11.00	+4.75	121.36	3.90
42H33	3.00	3.00	6.00	6.25	4.75	11.00	+5.00	120.38	4.15
42H34	3.00	3.00	6.00	6.50	6.25	12.75	+6.75	121.99	5.53 ♦
42H35	3.25	4.50	7.75	4.90	5.30	10.20	+2.45	121.27	2.02
42H36	4.00	4.00	8.00	4.00	5.75	9.75	+1.75	121.62	1.44
42H37	4.50	4.50	9.00	6.20	5.00	11.20	+2.20	120.44	1.83
42H44	4.75	3.50	8.25	5.30	4.90	10.20	+1.95	121.08	1.61
51H25 *	3.50	3.50	7.00	8.00	4.50	12.50	+5.50	120.73	4.56
51H25 **	3.00	4.00	7.00	4.50	4.00	8.50	+1.50	120.73	1.24
51H26	4.50	3.75	8.25	5.25	4.50	9.75	+1.50	121.43	1.24
51H27	3.50	3.25	6.75	5.30	5.70	11.00	+4.25	121.60	3.50
51H28 *	3.50	4.00	7.50	4.50	4.75	9.25	+1.75	120.46	1.45
51H28 **	4.00	4.00	8.00	4.50	4.00	8.50	+0.50	120.46	0.42
53H2	4.50	4.00	8.50	5.75	6.25	12.00	+3.50	121.32	2.88
53H46	3.25	3.50	6.75	6.00	5.25	11.25	+5.25	121.60	4.32
62H41	4.00	4.00	8.00	7.10	5.30	12.40	+4.40	121.37	3.63
62H46	3.50	4.50	8.00	6.25	6.50	12.75	+4.75	121.11	3.92
51H41	N/A	3.50	3.50	N/A	4.00	4.00	+0.50	N/A	N/A

* GREASED 11/18/97
** GREASED 12/18/97

♦ SEE NCR No. FN604-021

25TH YEAR SURVEILLANCE OF THE POST-TENSIONING SYSTEM AT THE CRYSTAL RIVER NUCLEAR PLANT UNIT 3

Precision Surveillance Corporation

Florida Power
A Progress Energy Company

TABLE XI: SUMMARY OF DATA SHEETS SQ 12.1
GREASE LOSS Vs GREASE REPLACEMENT

TENDON	GREASE REMOVED			GREASE REPLACED			DIFF.	NET	%
	SHOP	FIELD	TOTAL (GAL.)	SHOP	FIELD	TOTAL (GAL.)	(GAL.)	VOLUME	
12V01	2.25	105.75	108.00	0.00	115.00	115.00	7.00	139.43	+5.02
12V02	3.00	0.00	3.00	3.50	0.00	3.50	0.50	139.78	+0.36
23V02	2.75	0.00	2.75	4.75	0.00	4.75	2.00	139.85	+1.43
45V14	7.00	102.50	109.50	0.00	118.50	118.50	9.00	140.34	+6.41
61V08	7.00	102.75	109.75	0.00	112.50	112.50	2.75	139.78	+1.97
46H21	3.25	3.25	6.50	3.50	4.00	7.50	1.00	119.96	+0.83
46H29	1.75	1.75	3.50	3.50	2.50	6.00	2.50	119.73	+2.09
46H30	1.75	1.50	3.25	4.50	2.25	6.75	3.50	119.73	+2.92
46H31	1.75	1.75	3.50	3.25	4.50	7.25	3.75	119.73	+3.13
46H32	1.75	1.75	3.50	2.25	3.50	5.75	2.25	119.73	+1.88
46H33	1.75	1.75	3.50	4.50	3.50	8.00	4.50	119.73	+3.76
46H34	1.75	1.75	3.50	2.50	3.50	6.00	2.50	119.73	+2.09
46H35	1.75	1.75	3.50	3.50	2.50	6.00	2.50	119.09	+2.10
46H36	2.25	1.00	3.25	2.50	4.50	7.00	3.75	119.25	+3.14
46H37	1.75	1.00	2.75	3.50	4.00	7.50	4.75	119.93	+3.96
46H38	1.50	1.50	3.00	2.50	4.00	6.50	3.50	119.73	+2.92
46H39	1.75	3.50	5.25	3.00	3.50	6.50	1.25	119.73	+1.04
56H16	2.50	2.00	4.50	3.00	3.50	6.50	2.00	119.53	+1.67
62H02	2.25	2.50	4.75	4.50	5.25	9.75	5.00	119.33	+4.19
62H09	3.00	2.00	5.00	3.50	5.25	8.75	3.75	119.73	+3.13
62H13	2.50	1.50	4.00	4.75	4.50	9.25	5.25	120.59	+4.53
D126	14.50	25.75	40.25	0.00	42.75	42.75	2.50	115.97	+2.16
D212	24.75	22.00	46.75	0.00	62.25	62.25	15.50	113.86	+13.73 *
D339	40.00	15.00	55.00	73.75	0.00	73.75	18.75	100.18	+18.72 **

* ADDRESSED IN FN 750-007

** ADDRESSED IN FN 750-006

10.2 TENDON CAP RESEALING AND GREASING

10.2.1 After completion of all inspections, the anchorage components were hand coated with cold grease to ensure complete coverage. The caps were reinstalled with new gaskets and the results of the grease cap replacement were recorded on Data Sheet SQ 12.0 and are summarized in Tables 50 thru 55.

10.2.2 Upon acceptable cap replacement, the necessary amount of sheathing filler (grease) was added. All of the inspected tendons were refilled within the acceptable limits as stated in the PSC Procedure SQ12.1. The results of the grease replacement were recorded on Data Sheet SQ 12.1 and are summarized in Tables 56 thru 60.

10.2.2.1 The absolute difference between the amount of grease removed/lost and the amount of grease replaced in the subject tendon shall not exceed 10% of the net duct volume per PSC Procedure SQ 12.1. No tendon accepted above 10% of the net duct volume more than was lost, and all refills were acceptable.

TABLE 50: VERTICALS – SQ12.0 – GREASE CAP REPLACEMENT

TENDON	END	BEARING PLATE SURFACE CLEAN AND ACCEPTABLE	ANCHORAGE ASSEMBLY COATED W/ GREASE	CAP FLANGE AND GASKET SEALING SURFACE IS CLEAN AND DRY	NEW GASKET IS BEING USED FOR FINAL INSTALLATION OF GREASE CAP	ANCHORAGE NUTS EVENLY TIGHTENED AND GASKET COMPRESSED	TOUCH-UP OF HARDWARE, CAP OR BEARING PLATE REQUIRED	ACCEPTABLE ?
12V01	TOP	YES	YES	YES	YES	YES	NO	YES
	BOT	YES	YES	YES	YES	YES	NO	YES
45V20	TOP	YES	YES	YES	YES	YES	NO	YES
	BOT	YES	YES	YES	YES	YES	NO	YES
61V08	TOP	YES	YES	YES	YES	YES	NO	YES
	BOT	YES	YES	YES	YES	YES	NO	YES
61V17	TOP	YES	YES	YES	YES	YES	NO	YES
	BOT	YES	YES	YES	YES	YES	NO	YES

TABLE 51: DOMES – SQ12.0 – GREASE CAP REPLACEMENT

TENDON	END	BEARING PLATE SURFACE CLEAN AND ACCEPTABLE	ANCHORAGE ASSEMBLY COATED W/ GREASE	CAP FLANGE AND GASKET SEALING SURFACE IS CLEAN AND DRY	NEW GASKET IS BEING USED FOR FINAL INSTALLATION OF GREASE CAP	ANCHORAGE NUTS EVENLY TIGHTENED AND GASKET COMPRESSED	TOUCH-UP OF HARDWARE, CAP OR BEARING PLATE REQUIRED	ACCEPTABLE ?
D129	BT 3	YES	YES	YES	YES	YES	NO	YES
	BT 5	YES	YES	YES	YES	YES	NO	YES
D212	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES
D238	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES

TABLE 52: HOOPS – SQ12.0 – GREASE CAP REPLACEMENT

TENDON	END	BEARING PLATE SURFACE CLEAN AND ACCEPTABLE	ANCHORAGE ASSEMBLY COATED W/ GREASE	CAP FLANGE AND GASKET SEALING SURFACE IS CLEAN AND DRY	NEW GASKET IS BEING USED FOR FINAL INSTALLATION OF GREASE CAP	ANCHORAGE NUTS EVENLY TIGHTENED AND GASKET COMPRESSED	TOUCH-UP OF HARDWARE, CAP OR BEARING PLATE REQUIRED	ACCEPTABLE ?
13H36	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES
42H46	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 4	YES	YES	YES	YES	YES	NO	YES
46H21	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
51H34	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 5	YES	YES	YES	YES	YES	NO	YES
62H30	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES

TABLE 53: 13H36 ADJACENTS – SQ12.0 – GREASE CAP REPLACEMENT

TENDON	END	BEARING PLATE SURFACE CLEAN AND ACCEPTABLE	ANCHORAGE ASSEMBLY COATED W/ GREASE	CAP FLANGE AND GASKET SEALING SURFACE IS CLEAN AND DRY	NEW GASKET IS BEING USED FOR FINAL INSTALLATION OF GREASE CAP	ANCHORAGE NUTS EVENLY TIGHTENED AND GASKET COMPRESSED	TOUCH-UP OF HARDWARE, CAP OR BEARING PLATE REQUIRED	ACCEPTABLE ?
13H33	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES
13H34	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES
13H35	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES
13H37	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES
13H38	BT 1	YES	YES	YES	YES	YES	NO	YES
	BT 3	YES	YES	YES	YES	YES	NO	YES

TABLE 54: 46H21 ADJACENTS – SQ12.0 – GREASE CAP REPLACEMENT

TENDON	END	BEARING PLATE SURFACE CLEAN AND ACCEPTABLE	ANCHORAGE ASSEMBLY COATED W/ GREASE	CAP FLANGE AND GASKET SEALING SURFACE IS CLEAN AND DRY	NEW GASKET IS BEING USED FOR FINAL INSTALLATION OF GREASE CAP	ANCHORAGE NUTS EVENLY TIGHTENED AND GASKET COMPRESSED	TOUCH-UP OF HARDWARE, CAP OR BEARING PLATE REQUIRED	ACCEPTABLE ?
46H19	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
46H20	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
46H22	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
46H23	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
46H24	BT 4	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES

TABLE 55: 62H30 ADJACENTS – SQ12.0 – GREASE CAP REPLACEMENT

TENDON	END	BEARING PLATE SURFACE CLEAN AND ACCEPTABLE	ANCHORAGE ASSEMBLY COATED W/ GREASE	CAP FLANGE AND GASKET SEALING SURFACE IS CLEAN AND DRY	NEW GASKET IS BEING USED FOR FINAL INSTALLATION OF GREASE CAP	ANCHORAGE NUTS EVENLY TIGHTENED AND GASKET COMPRESSED	TOUCH-UP OF HARDWARE, CAP OR BEARING PLATE REQUIRED	ACCEPTABLE ?
62H29	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
62H31	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
62H32	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
62H33	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES
62H34	BT 2	YES	YES	YES	YES	YES	NO	YES
	BT 6	YES	YES	YES	YES	YES	NO	YES

TABLE 56: VERTICALS – SQ12.1 – GREASE LOSS vs. GREASE REPLACEMENT

TENDON	END	GREASE REMOVED (GALLONS)		GREASE REPLACED (GALLONS)		DIFF. (GAL.)	DUCT VOLUME (GAL.)	% DIFF.	ACCEPT
		END	TOTAL	END	TOTAL				
12V01	TOP	2.5	78.5	0	82.19	3.69	143.97	2.56	YES
	BOT	76		82.19					
45V20	TOP	3	88	0	91.04	3.04	144.47	2.10	YES
	BOT	85		91.04					
61V08	TOP	2.5	55	7.08	56.64	1.64	144.12	1.13	YES
	BOT	52.5		49.56					
61V17	TOP	2.5	98.5	1	107.2	8.7	144.74	6.01	YES
	BOT	96		106.2					

Depending on the amount of grease loss during the inspection, grease can be replaced by pouring it back into the grease cap.

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TABLE 57: HOOPS – SQ12.1 – GREASE LOSS vs. GREASE REPLACEMENT

TENDON	END	GREASE REMOVED (GALLONS)		GREASE REPLACED (GALLONS)		DIFF. (GAL.)	DUCT VOLUME (GAL.)	% DIFF.	ACCEPT
		END	TOTAL	END	TOTAL				
13H36	BT 1	2.5	5	3.54	7.08	2.08	121.67	1.70	YES
	BT 3	2.5		3.54					
42H46	BT 2	2.5	5 5	3.09	5.74	0.24	122.57	0.19	YES
	BT 4	3.0		2.65					
46H21	BT 4	2.5	5	3.54	7.08	2.08	122.06	1.70	YES
	BT 6	2.5		3.54					
51H34	BT 1	2.5	5	2.65	6.19	1.19	120.88	0.98	YES
	BT 5	2.5		3.54					
62H30	BT 2	2.5	5.	3.54	7.08	2.08	121.75	1.70	YES
	BT 6	2.5		3.54					

TABLE 58: DOMES – SQ12.1 – GREASE LOSS vs. GREASE REPLACEMENT

TENDON	END	GREASE REMOVED (GALLONS)		GREASE REPLACED (GALLONS)		DIFF. (GAL.)	DUCT VOLUME (GAL.)	% DIFF.	ACCEPT
		END	TOTAL	END	TOTAL				
D129	BT 3	13.5	36.5	0	39.71	3.21	116.41	2.75	YES
	BT 5	23		39.71					
D212	BT 1	2.5	20	0	23.78	3.78	115.99	3.25	YES
	BT 3	17.5		23.78					
D238	BT 4	62.5	69.5	73.34	73.34	3.84	102.59	3.74	YES
	BT 6	7		0					

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DOCUMENT TITLE:	FINAL REPORT FOR THE 30TH YEAR CONTAINMENT IWL INSPECTION
PROJECT TITLE:	30TH YEAR TENDON SURVEILLANCE AT CRYSTAL RIVER DATE: 01/24/08

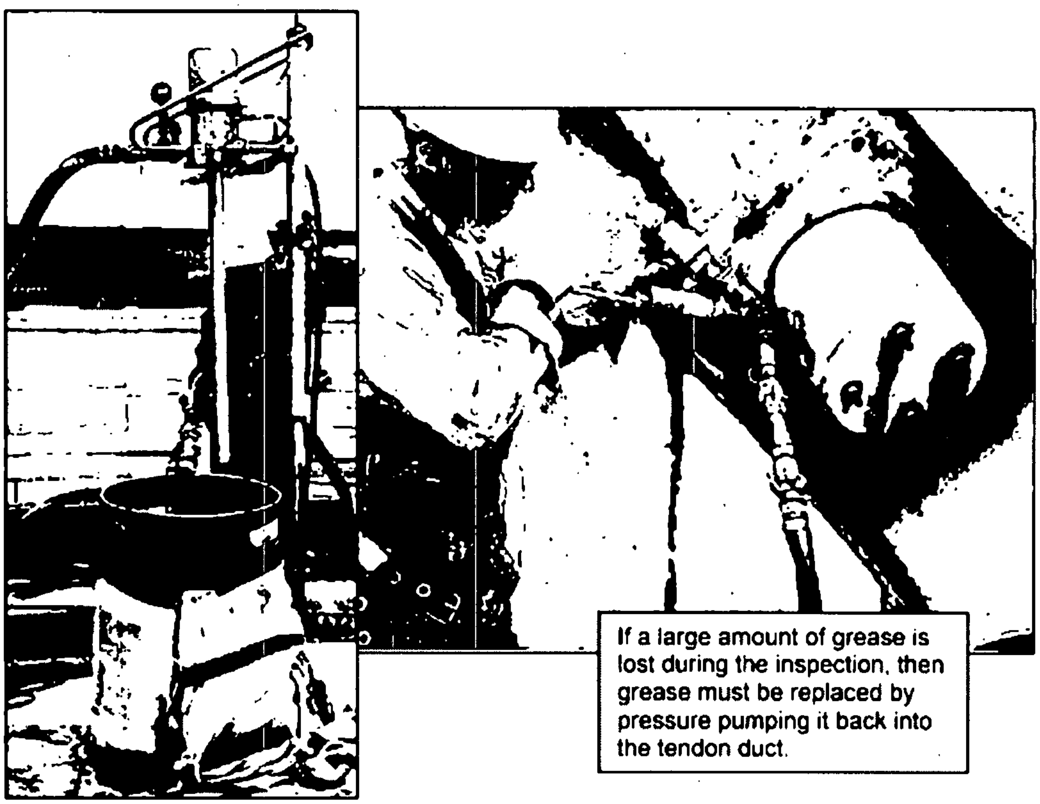
Progress Energy

TABLE 59: ADJACENTS - SQ121 - GREASE LOSS vs. GREASE REPLACEMENT

TENDON	END	GREASE REMOVED (GALLONS)		GREASE REPLACED (GALLONS)		DIFF. (GAL.)	DUCT VOLUME (GAL.)	% DIFF.	ACCEPT
		END	TOTAL	END	TOTAL				
13H33	BT 1	2	4	2.65	5.3	1.3	121.57	1.06	YES
	BT 3	2		2.65					
13H34	BT 1	2	4	2.65	5.3	1.3	121.25	1.07	YES
	BT 3	2		2.65					
13H35	BT 1	2	4	2.65	5.3	1.3	121.77	1.06	YES
	BT 3	2		2.65					
13H37	BT 1	2	4	2.65	5.3	1.3	122.18	1.06	YES
	BT 3	2		2.65					
13H38	BT 1	2	4	2.65	5.3	1.3	121.53	1.06	YES
	BT 3	2		2.65					
46H19	BT 4	2	4	2.21	4.86	0.86	121.25	0.70	YES
	BT 6	2		2.65					
46H20	BT 4	2	4	2.65	5.3	1.3	121.37	1.07	YES
	BT 6	2		2.65					
46H22	BT 4	2	4	2.65	5.74	1.74	122.43	1.42	YES
	BT	2		3.09					
46H23	BT 4	2	4	2 65	5.3	1.3	122.63	1.06	YES
	BT 6	2		2 65					
46H24	BT 4	2	4	2.65	5.74	1.74	121.57	1.43	YES
	BT 6	2		3.09					

TABLE 60 ADJACENTS – SQ121 – GREASE LOSS vs. GREASE REPLACEMENT

TENDON	END	GREASE REMOVED (GALLONS)		GREASE REPLACED (GALLONS)		DIFF. (GAL.)	DUCT VOLUME (GAL.)	% DIFF.	ACCEPT
		END	TOTAL	END	TOTAL				
62H29	BT 2	2	4	2.65	5.3	1.3	121.93	1.06	YES
	BT 6	2		2.65					
62H31	BT 2	2	2	2.65	2.65	0.65	122.13	0.53	YES
	BT 6	0		0					
62H32	BT 2	2	2	2.65	2.65	0.65	121.45	0.53	YES
	BT 6	0		0					
62H33	BT 2	2	4.5	2.65	5.3	0.80	122.07	0.65	YES
	BT 6	2.5		2.65					
62H34	BT 2	2	4	2.65	5.3	1.3	121.47	1.07	YES
	BT 6	2		2.65					



If a large amount of grease is lost during the inspection, then grease must be replaced by pressure pumping it back into the tendon duct.

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