1.3 Excessive Tensioning Hoop Tendons

Description:

The design calls for tensioning the Hoop tendons to 1,635 kips. When considering the number of hoop tendons and the height of the containment building, we can calculate the number of hoop tendons per unit height and the hoop tendon force per unit height. This number can be compared to typical industry standard for this type of containment. And it can be compared with the accident pressure condition that the pre-stress is meant to sustain. The containment pre-stress to accident pressure ratio is an important parameter often referred to in the industry.

Additionally, occasional excessive tensioning, as allowed by normal operation, can damage the concrete by generating localized micro-cracks and/or enhanced creep.

Data to be collected and Analyzed:

1. Calculate loads and stresses generated by hoop tendons (FM 1.3 Exhibit 1);
2. Compare with equivalent numbers from other nuclear reactor containments (FM 1.3 Exhibit 2 and FM 1.3 Exhibit 8);
3. NCR incidents in the past that may have involved over-tensioning of horizontal tendons;
4. Procedure review for writing NCR for overstressing tendons;
5. Interview with experienced and knowledgeable engineers (FM 1.4 Exhibit 3);
6. Review WO 681043-01 and PSC procedures state that the maximum tension is 80% of the yield (FM 1.4 Exhibit 4).

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0/27

Verified Supporting Evidence:

a. The hoop pre-stress at CR3 is generally higher than the pre-stress level at most equivalent nuclear power plant containment buildings (FM 1.3 Exhibit 2, in particular table 2.1 page 20 of 21). Compare 978 kips/ft at CR3 from FM 1.3 Exhibit 1 (section A.1.1) with a maximum of 730 kips/ft from FM 1.3 Exhibit 2 table 2.1 (page 32 of 64) at Bechtel-built plants;

b. Also see PII calculations in FM 1.3 Exhibit 8 that confirms the higher pre-stress to accident stress ratio;

c. And see report from Dr. Orhan Gurbuz pointing out the higher ratio (FM 1.3 Exhibit 5 item 2g);

Verified Refuting Evidence:

a. The higher pre-stress level results in a hoop pre-stress around 2,000 psi from a simplified analysis (ignoring stress in the liner plate and Poisson's effects) (FM 1.3 Exhibit 1). The concrete used at CR3 has a specified 28 days compressive strength of 5,000 psi and a tested 32 years (today) compressive strength of 7,500 psi with a standard deviation of 820 psi. The pre-stress level is therefore only around 40% of the specified f'c and 27% of the measured f'c. This remains too low to damage the material in compression.

b. Tendon tensioning records and interview results show no evidence of over-tensioning (FM 1.3 Exhibit 3);

c. Additionally, the worst case scenario of having tendon force at 80% of GUTS increases the existing radial stress by only 14%;

Discussion:

As seen in FM 1.3 Exhibit 1, the hoop pre-stress at CR3 is around 1.6 times accident pressure of 55 psi. This is higher than seen at most other nuclear containments (see FM 1.3 Exhibit 2 table 2.1 and FM 1.3 Exhibit 8). This fact was pointed out by Orhan Gurbuz, a former Bechtel engineer who did a quick condition assessment of the CR3 delamination (FM 1.3 Exhibit 5 item 2g).

The higher pre-stress level leads to higher creep for a given concrete material and a given specific creep given in terms of μin/in per psi (FM 1.3 Exhibit 6). In particular in the study done by Schupack and Associates, "Report on recommended concrete creep and shrinkage values for computing prestressing losses", the conclusion in given in terms of a specific creep of 0.4 μin/in per psi (FM 1.3

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Exhibit 6, page 6 of 33). The NCRs for over-stressing tendons are contained in the PSC procedure on tendon surveillances (FM 1.3 Exhibit 7). The procedure calls for writing an NCR if the tendon lift-off force exceeds 80% GUTS. It does not call for writing an NCR if the measured lift-off force is higher than the "predicted" tendon force from tendon force loss predictions (see FM 6.5 and FM 6.6 for details).
Conclusions: The pre-stress level of 1.6 times accident pressure is acceptable. Additionally, no excessive tensioning of horizontal tendons has been identified. It is concluded that excessive tensioning of the horizontal tendons did not contribute to the 2009 delamination of the containment concrete.

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Loads and stresses from Hoop tendons

A. Tendon force at 1,635 kips

The loads and stresses generated by the hoop tendon forces are calculated based on the following:

- Hoop Tendon Force — F_{tendon} = 1,635,000 lbf
- Hoop Tendon Centerline Radius — r = 812.375 in
- Wall Inside Radius — r_i = 780 in
- Wall Outside Radius — r_o = 822 in
- Height of Containment Cylinder — h = 157 ft = 1,884 in
- Number of Hoop Tendons Though Wall Section — n_H = 94 tendons

A.1 Radial force per unit height

There are two approaches to calculate the radial force per unit height

A.1.1 Using the containment height and number of tendons

The radial force per unit height is equal to

$$F_h = \frac{F_{tendon}}{H} * \#_{tendon} = 978\ \frac{\text{kips}}{\text{ft}}$$

A.1.2 Using the number of tendons per unit height in that part of the containment wall

On one bay, the "close" tendons are separated by 12.75 in and the "far" tendons are separated by 25.5 in. This is equivalent to 2 tendons per 38.25 in, or 94 tendons per 1,797.75 in, or 94 tendons per 149.8 ft. The difference with 157 ft (and case A.1.1) is that there are fewer tendons at the bottom of the wall.

For our purpose, it is therefore better to calculate the radial force per unit height as

$$F_h = \frac{F_{tendon}}{38.25\ in} * 2\ tendons * 12\ \frac{\text{in}}{\text{ft}} = 1{,}025\ \frac{\text{kips}}{\text{ft}}$$

A.2 Radial force per unit circumferential length

The radial force per unit circumferential length is equal to

$$F_r = \frac{F_{tendon}}{r} = 2{,}012\ \frac{\text{lbf}}{\text{in}}$$

A.3 Average radial pressure

A.3.1 Using full height and number of tendons

The average radial pressure due to the hoop tendon forces is

$$P_r = \frac{F_r}{\frac{h}{n_H}} = \frac{2012}{\frac{1884}{94}} = 100.4 \text{ psi}$$

A.3.2 Using the number of tendons per unit height in that part of the containment wall

Alternately, the average radial pressure can be calculated using

$$P_r = \frac{F_h}{12 * 780\ in} = 109.5 \text{ psi}$$

A.4 Average hoop stress

A.4.1 Using full height and number of tendons

The average hoop stress is then

$$\sigma = \frac{P_r \frac{r_i + r_o}{2}}{r_o - r_i} = \frac{100.4 \frac{780 + 822}{2}}{822 - 780} = 1{,}915 \text{ psi}$$

A.4.2 Using the number of tendons per unit height in that part of the containment wall

The average hoop stress is then

$$\sigma = \frac{P_r \frac{r_i + r_o}{2}}{r_o - r_i} = \frac{109.5 \frac{780 + 822}{2}}{822 - 780} = 2{,}089 \text{ psi}$$

B. <u>End of life tendon force of 1,325 kips</u>

When using the end-of-life tendon force of 1,325 kips, we get the following results

B.1 Radial force per unit height

B.1.1 Using the containment height and number of tendons

The radial force per unit height is equal to

$$F_h = \frac{F_{tendon}}{H} * \#_{tendon} = 793 \ \frac{\text{kips}}{\text{ft}}$$

B.1.2 Using the number of tendons per unit height in that part of the containment wall

$$F_h = \frac{F_{tendon}}{38.25\ in} * 2\ tendons * 12 \ \frac{\text{in}}{\text{ft}} = 831 \ \frac{\text{kips}}{\text{ft}}$$

Note the FSAR specified prestress level of 829 kips / ft.

B.2 Radial force per unit circumferential length

The radial force per unit circumferential length is equal to

$$F_r = \frac{F_{tendon}}{r} = 1{,}632\ \frac{\text{lbf}}{\text{in}}$$

B.3 Average radial pressure

B.3.1 Using full height and number of tendons

The average radial pressure due to the hoop tendon forces is

$$P_r = \frac{F_r}{\frac{h}{n_H}} = \frac{1632}{\frac{1884}{94}} = 81.4\ \text{psi}$$

B.3.2 Using the number of tendons per unit height in that part of the containment wall

Alternately, the average radial pressure can be calculated using

$$P_r = \frac{F_h}{12 * 780\ in} = 88.8\ \text{psi}$$

The accident pressure is 55 psi.

The ratio of P_r over 55 psi is $\frac{88.8}{55} = 1.61$.

B.4 Average hoop stress

B.4.1 Using full height and number of tendons

The average hoop stress is

$$\sigma = \frac{P_r \frac{r_i + r_o}{2}}{r_o - r_i} = \frac{81.4 \frac{780 + 822}{2}}{822 - 780} = 1{,}552\ \text{psi}$$

B.4.2 Using the number of tendons per unit height in that part of the containment wall

The average hoop stress is

$$\sigma = \frac{P_r \frac{r_i + r_o}{2}}{r_o - r_i} = \frac{88.8 \frac{780 + 822}{2}}{822 - 780} = 1{,}694\ \text{psi}$$

C. Conclusion

With the accident pressure at 55 psi, the ratio of P_r over 55 psi is approximately 1.61.

BC-TOP-5-A
Revision 3
FEBRUARY 1975

PRESTRESSED CONCRETE NUCLEAR REACTOR CONTAINMENT STRUCTURES

BECHTEL POWER CORPORATION
SAN FRANCISCO, CALIFORNIA

CAVEAT: THIS REPORT HAS BEEN PREPARED BY AND FOR THE USE OF BECHTEL POWER CORPORATION AND ITS RELATED ENTITIES. ITS USE BY OTHERS IS PERMITTED ONLY ON THE UNDERSTANDING THAT THERE ARE NO REPRESENTATIONS OR WARRANTIES, EXPRESS OR IMPLIED, AS TO THE VALIDITY OF THE INFORMATION OR CONCLUSIONS CONTAINED HEREIN.

TOPICAL REPORT

BC-TOP-5-A

Revision 3

PRESTRESSED CONCRETE NUCLEAR REACTOR CONTAINMENT STRUCTURES

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Bechtel Power Corporation Issue Date: February, 1975

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

MAR 2 8 1975

Mr. John V. Morowski
Vice President-Engineering
Bechtel Power Corporation
Fifty Beale Street
San Francisco, California 94119

Dear Mr. Morowski:

The NRC staff has completed its review of Bechtel Power Corporation's Topical Report BC-TOP-5, Revision 3, dated February 1975 and entitled "Prestressed Concrete Nuclear Reactor Containment Structures". We conclude that BC-TOP-5, Revision 3, is acceptable by reference in the Safety Analysis Reports for the facilities listed in Enclosure 2 to this letter. Enclosure 1 is a summary of our evaluation. BC-TOP-5, Revision 3, may also be acceptable by reference in other applications; however, the NRC staff reserves the right to make this determination on a case-by-case basis.

BC-TOP-5 does not provide all of the pertinent information required by the NRC staff in its review of specific applications. Therefore, the appropriate supplementary information identified in the enclosed Topical Report Evaluation will have to be provided in individual Safety Analysis Reports.

The staff does not intend to repeat its review of BC-TOP-5, Revision 3, when it appears as a reference in the license applications for the facilities identified in Enclosure 2. Should NRC criteria or regulations change, such that our conclusions concerning BC-TOP-5, Revision 3, are invalidated, you will be notified and given the opportunity to revise and resubmit your topical report for review, should you so desire.

AMERICAN REVOLUTION BICENTENNIAL 1776-1976

We request that you reissue BC-TOP-5, Revision 3, dated February 1975 in accordance with the provisions of the "Elements of the Regulatory Staff Topical Report Review Program" which was forwarded to you on August 26, 1974. If you have any questions in this regard, please let us know.

Sincerely,

R. W. Klecker, Technical Coordinator
for Light Water Reactors Group 1
Division of Reactor Licensing

Enclosures:
1. Topical Report Evaluation
2. List of Facilities

MAR 26 1975

Enclosure 1

TOPICAL REPORT EVALUATION

Report No.: BC-TOP-5, Revision 3
Report Title: Prestressed Concrete Nuclear Reactor Containment Structures
Report Date: February 7, 1975
Originating Organization: Bechtel Corporation
Reviewed by: Structural Engineering Branch, Division of Technical Review

Summary of Report

The practices described in this report represent the criteria, analytical tools, adaptation of codes, and the experience in design, construction, testing and surveillance of prestressed concrete containments as designed by Bechtel Power Corporation. Although all current projects addressed in this topical report have utilized pressurized water reactors, the information, procedures and techniques presented therein appear valid for projects utilizing other types of reactors when containment system concepts and conditions are similar. The applicability of the topical report for such projects will be determined at the time of filing their SARs.

In accordance with the Atomic Energy Commission Policy Statement on Standardization of Nuclear Power Plants, April 28, 1972, this report is intended to serve as a reference document for Safety Analysis Reports in the licensing application for nuclear units.

The report is organized to respond to Section 3.8.2 of the "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants" prepared by the AEC staff, February 1972. The design considerations for structures and sub-compartments within the containment, which vary with the reactor type and containment system, are not in the scope of the topical report. Example calculations, analytical results, and designs are presented in the report to illustrate the technical procedures which are used for the design of a containment.

Included in the submittal of Revision 3 of the Topical Report were the responses to the staff's requests for additional information. These requests were made in September, 1973, and December, 1974.

The Bechtel Power Corporation has adopted as design criteria for concrete containments the provisions of Article CC-3000, Design, of Division 2 of the ASME Code, Section III, through Amendment #6, together with the staff's exceptions to this version. The topical report includes the provisions of the above-referenced version of Article CC-3000. For convenience of reference, this version has been reproduced and included in BC-TOP-5 in Appendix C, Design Criteria. Also presented in Appendix C are explanations of the exceptions and changes made to Article CC-3000.

The various sections of the report follow very closely Revision 1 of the "Standard Format and Content of Safety Analyses Reports for Nuclear Power Plants," dated October 1972. In this document Section 3.8.1 instead of 3.8.2 addresses concrete containments. The correspondence by subsections is as follows:

Bechtel Topical		Rev. 1 of St'd Format
2.0 Physical Description	⟷	3.8.1.1 Description of Containment
3.0 Design Bases 4.0 Design Documents	⟷	3.8.1.2 Applicable Codes, Standards and Specifications
5.0 Load Definitions and Combinations	⟷	3.8.1.3 Loads and Loading Combinations
6.0 Design Appendix B 7.0 Analysis Appendix C	⟷	3.8.1.4 Design and Analysis Procedures 3.8.1.5 Structural Acceptance Criteria
-Not Included -	⟷	3.8.1.6 Materials, Quality Control and Special Construction Techniques
9.0 Surveillance, Testing Procedures and Results	⟷	3.8.1.7 Testing and Inservice Surveillance Requirements

The report where necessary, carries through in providing information on the two main types of Bechtel prestressed containments currently being built as well as the various modification that may be used on a given application. The two types of containments include those that use the 90-wire tendons arranged at 120 degrees in a shallow dome and those that use large tendons (strand on wire) arranged at 180 degrees in a hemispherical dome.

Areas which are not included in the scope of this topical are:

1. Exceptions as may be taken to NRC Regulatory guides for a specific case.
2. The use of the various other Bechtel topical reports and the specific revision number that relate to the containment review such as BC-TOP-1, 3, 4, 9, etc.

Summary of Regulatory Evaluation

The subject report provides a basis for several aspects considered by the Regulatory staff when evaluating the licensability of prestressed concrete containments used in the plants listed in Enclosure 2 and, as such, provides time savings in the review process by becoming a standard reference. For these specific plants, it is an acceptable reference for all subsections of Section 3.8.1 of the standard format except subsections 3.8.1.1, Descriptions, and 3.8.1.6, Materials, Quality Control and Special Construction Techniques. For these two subsections, the specific material indicated in the standard format must be provided.

The topical report also states that the information, procedures and techniques presented are valid for projects using types of reactors other than PWR's when the containment system concepts and conditions are similar. It is clear in this case that such action would require a staff determination as to whether BC-TOP-5 is, in fact, relevant. For the present it remains as a reference for dry pressurized water reactors only.

Regulatory Position

BC-TOP-5, Revision 3, provides an acceptable reference for a current group of plants listed in Enclosure 2. However, the requirements that relate to description of the containment, and materials, quality control, and special construction techniques must be provided specifically for each plant.

The report's applicability for future plants will be determined on a case-by-case basis depending on the manner in which the applicant elects to use it as a reference.

Enclosure 2

BC-TOP-5 APPLICABILITY TO CURRENT PLANTS

Bechtel projects whose Safety Analysis Reports are docketed and for which BC-TOP-5, including Appendix C, may be used as reference.

APPLICANT	PROJECT NAME	PSAR or FSAR
Boston Edison Company	Pilgrim 2	PSAR
Portland General Electric Co.	Pebble Springs 1 (Boardman-1)	PSAR
Rochester Gas & Elect. Co. (SNUPPS)	Sterling	PSAR
Union Electric Company (SNUPPS)	Callaway 1& 2	PSAR
Northern States Power Co. (SNUPPS)	Tyrone 1	PSAR
Kansas Gas & Elect. Co. (SNUPPS)	Wolf Creek 1	PSAR
Gulf States Utility Co.	Blue Hills 1 & 2	PSAR
Arizona Public Service Co.	Palo Verde 1, 2 & 3	PSAR
Detroit Edison Company	Greenwood 2 & 3	PSAR

TOPICAL REPORT
BC-TOP-5-A
Revision 3
PRESTRESSED CONCRETE NUCLEAR REACTOR CONTAINMENT STRUCTURES

ABSTRACT

This report contains a discussion of the techniques and procedures used for the design of prestressed concrete nuclear reactor containment structures. A physical description of Bechtel designed containment structures is presented. The design bases and load combinations are given for anticipated conditions of service. Reference design documents which include industry codes, specifications, AEC Regulatory Guides, Bechtel Topical Reports and additional criteria as appropriate to containment design are listed. Stepwise procedures typically followed by Bechtel for design of containments is discussed and design examples are presented. A description of currently used analytical methods and the practical application of these methods for containment design is also presented. The principal containment construction materials are identified and codes of practice pertaining to construction procedures are listed. Preoperational structural testing procedures and post-operational surveillance programs are furnished along with results of tests on completed containment structures.

SECTION 1.0
INTRODUCTION

The practices described in this report represent the criteria, analytical tools, adaptation of codes, and the experience in design, construction, testing and surveillance of prestressed concrete containments as designed by Bechtel Power Corporation. Although all of the existing projects addressed in this topical report have utilized pressurized water reactors, the information, procedures and techniques presented herein are valid for projects utilizing other types of reactors when containment system concepts and conditions are smiliar.

In accordance with the Atomic Energy Commission Policy Statement on Standardization of Nuclear Power Plants, April 28, 1972, this report is intended to serve as a reference document for Safety Analysis Reports in the licensing application for nuclear units.

The report is organized to respond to Section 3.8.2 of the "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants" prepared by the AEC Staff, February 1972. The design considerations for structures and subcompartments within the containment, which vary with the reactor type and containment system, are not in the scope of this topical report. Examples of calculations, analytical results, and designs are presented in the report to illustrate the technical procedures which are used for the design of a containment.

Further, the topical report shows Bechtel accepted design and is intended to serve as a guide to engineers of containment structures; as such, it provides a basis for greater uniformity of containment design.

SECTION 2.0

PHYSICAL DESCRIPTION

2.1 Containment Basic Configuration (Refer to Figure 6-1)

The typical containment structure consists of a vertical cylinder with a convex dome and a basically flat bottom slab. See Figure 2-1 for the configuration of prestressed containments and Table 2-1 for dimensions of containments designed by Bechtel Corporation. The cylindrical and dome portions, as well as localized areas of the base slab where the vertical tendons are anchored, are prestressed concrete, and the base slab is reinforced concrete. The dome and cylinder are nominally reinforced with bonded reinforcing steel for crack control. Additional bonded reinforcing is provided at discontinuities and around openings in the shell. A continuous access gallery below the base slab is provided for access to the post-tensioning system. A welded steel liner plate is attached to the inside face of the concrete shell. Typical base slab and wall details are shown in Figures 2-2 and 2-3.

There are two basic dome configurations which are used for prestressed containments; the Hemispherical dome and the Sphere-Torus dome. See Figure 2-1. The tendon access gallery for containments using a hemispherical dome is large enough to accommodate tendon installation equipment.

2.2 Post-Tensioning System (Refer to Figures 6-2 through 6-6 and Figure 6-18)

Containments designed by Bechtel Corporation use tendons of two size categories; "Small" tendons (about 500 ton capacity) and "Large" tendons (about 1000 ton capacity). The tendon systems which have been developed for containments and are accepted by Bechtel for application to prestressed containments are shown in Figures 2-7, 2-8, and 2-9. They use either high strength wires or strand and are limited to BBRV buttonhead and VSL anchorage techniques. Small tendons use about 90 one-quarter inch diameter wires per tendon. Large tendons use about 180 one-quarter inch diameter wires per tendon or 55 one-half inch strands.

Each tendon consists of wire or strand together with end anchor components consisting of stressing washers and for the VSL system wedges and retainer plates. The tendons transfer load to the structure through shims and a bearing plate.

Tendons are installed in tendon sheaths which form ducts through the concrete between anchorage points. The tendon

sheathing is a galvanized, spiral wrapped, semi-rigid corrugated steel tubing. It is designed to retain its shape and resist construction loads. The inside diameter of the tendon sheathing is sufficiently large to permit installation of the tendons with minimum difficulty. Trumpets, which are enlarged ducts attached to the bearing plate, allow the wires or strands to spread out at the anchorage to suit washer hole spacing. Further, trumpets facilitate field button heading of wires.

An enclosed space surrounds each tendon; it is bounded by tendon sheathing, trumpets and grease caps attached to the bearing plate. The space is filled with a casing filler which provides corrosion protection for the tendon.

2.2.1 Cylindrical Wall Prestressing

Prestressing of the cylindrical wall is achieved by a post-tensioning system consisting of vertical and circumferential (hoop) tendons. Vertical tendons are anchored at the base slab bottom surface and for the sphere-torus dome at the ring girder top surface.

Hoop tendons are anchored at a series of buttresses equally spaced around the cylinder. Containments with "small" tendons normally use six buttresses. In this case, individual hoop tendons are anchored at buttresses 120° apart. Containments with "large" tendons use three buttresses, and each hoop tendon is anchored at buttresses 240° apart. For both three and six buttress containments, the successive hoop tendons are anchored at alternate buttresses. See Figure 2-4 for buttress arrangements and Figure 2-5 for schematic arrangements of hoop tendons.

2.2.2 Dome Prestressing

Prestressing of the sphere-torus dome is achieved by a three-way pattern of tendons that are anchored at both ends at the ring girder. Prestressing of the hemispherical dome is achieved by a two-way pattern of tendons which are an extension of the continuous vertical tendons and are anchored at the base slab. Hoop tendons extend into the hemispherical dome region to provide a two-way pattern up to the 90 degree solid angle of the dome. See Figure 2-6 for schematic arrangements of dome tendons.

2.3 Liner Plate Systems (Refer to Figures 6-21, 6-24 and 6-25)

2.3.1 Liner Plate and Anchors

A steel liner plate covers the entire inside surface of the containment (excluding penetrations). The liner is typically 1/4 inch thick and is thickened locally by reinforcing around penetrations, nozzles and large brackets. It is anchored to the concrete, including the thickened plate areas. Typical details of the liner plate system are shown in Figure 6-24.

2.3.2 Penetration Assemblies

Penetration assemblies consist of steel nozzles, reinforcing plates, and anchors. Penetration nozzles through the liner plate and concrete shell are categorized as large and small. Large penetrations are defined as those which have a nozzle inside diameter equal or greater than two and one-half times the nominal shell (wall) thickness. Small penetrations are those which have diameters less than two and one-half times the nominal shell (wall) thickness.

2.3.2.1 Large Penetrations

A circular equipment hatch and two personnel air lock assemblies penetrate the concrete cylinder walls. They are anchored to the concrete walls and are welded to the steel liner. Hatch and lock doors are provided with double gasketed flanges with provisions for leak testing the flange-gasket combinations. Details are shown in Figure 6-25.

One of two personnel locks normally is for emergency access only. Each personnel lock has a double door and is a welded steel assembly. A quick-acting equalizing valve connects the personnel lock with the interior and exterior of the containment to equalize pressure in the two systems when personnel are entering or leaving the containment.

During plant operation, the two doors of each personnel lock are interlocked to prevent both being opened simultaneously. Remote indicating lights and annunciators in the control room indicate the doors operational status. Provision is made to bypass the interlock system during plant cold shutdown.

2.3.2.2 Small Penetrations

2.3.2.2.1 Process Pipe Penetrations

Single barrier piping penetrations are provided for all piping passing through the containment walls. The closure for process piping to the liner plate is accomplished with a special head welded into the piping system and to the nozzle of the penetration assembly. In the case of piping carrying hot fluid, the pipe may be insulated to prevent excessive concrete temperatures.

Closures to these penetration assemblies are provided by the piping systems which are served by the penetrations. For typical details, see Figure 6-25.

2.3.2.2.2 Electrical Penetrations

Canister type assemblies are used for all electrical penetrations. The canister is a hollow cylinder closed on both ends with an insulated bushing or connector assembly. The canister is provided with a test plug to allow test pressurization of the penetration assembly. Figure 6-25 shows details of electrical penetrations. The method of sealing the canister depends upon the type of cable and connector assembly involved. In general, there are three types used:

Type 1 - High-voltage power, 6900 volts and over

Type 2 - Power and control, 600 volts and below

Type 3 - Instrumentation, thermocouple leads, coaxial, and other special wires

Type 1 penetrations. - Insulated conductors are passed through the header plates and potted to effect a pressure seal. Mechanical splices within the potting compound provide gas stops. High voltage insulating bushings and seals may also be used to provide the barrier.

Type 2 and 3 penetrations. - These cables are connected between the header plate assembly at each end of the penetration canister. Hermetically sealed feedthroughs with external pigtails provide a pressure barrier for each conductor.

2.3.2.2.3 Fuel Transfer Tube Penetration

A transfer tube penetration is provided for refueling. An inner pipe acts as the refueling tube and is fitted with a double-gasketed blind flange in the refueling canal and a standard gate valve in the spent fuel pit. This arrangement prevents leakage through the refueling tube. The outer pipe is welded to the containment liner and provisions are made for test-pressurizing during plant operation. Details are shown in Figure 6-21.

2.3.3 Attachments and Brackets

Attachments to the shell wall include brackets for support of the service crane, electrical conduit and cable tray brackets, piping and platforms. The service crane brackets are structural steel with tension (top) members penetrating the thickened liner plate and anchored in the concrete. For details see Figure 6-24. Floor brackets attached to the wall provide vertical support only.

Attachments to the base slab include anchor bolts for equipment support and reinforcing steel for internal structures support. Attachment is accomplished by connectors welded to both top and bottom of thickened liner plate.

2.4 Shell Discontinuities (Refer to Figures 6-7 through 6-17, 6-19, 6-20, 6-22, 6-23, 6-26, 6-27)

Significant discontinuities in the shell structure are at the dome to wall transition (ring girder) for the sphere-torus dome, the wall to base slab connection, the buttresses, and large penetration openings.

2.4.1 Ring Girder

The cylindrical face of the ring girder provides an anchorage zone for dome tendons and the top surface provides an anchorage zone for vertical tendons. The cylindrical surface may be flush with the outer shell surface or it may be stepped out about 1'-0" as shown in Figure 2-1. The ring girder face may be larger for the smaller tendon system to accommodate the greater number of tendon anchorages.

2.4.2 Wall-Base Slab Joint

Two types of lower wall configuration are used as shown in Figure 2-3, the straight wall to slab joint and the haunched joint. The haunched design is used where large radial shear force cannot be accommodated by the straight wall.

2.4.3 Buttresses

Buttresses project out from the shell surface to provide adequate space for hoop tendon anchorage and tendon stressing equipment. The anchorage surfaces of the buttress are normal to the tangent line of hoop tendons anchored. Details are shown in Figure 2-4.

2.4.4 Large Penetration Openings

The concrete shell around the equipment hatch opening is thickened by one of the methods shown in Figure 2-10.

FACILITY	DESIGN PRESSURE (psig)	FREE VOLUME (cu. ft.)	DOME SHAPE	INSIDE DIA D	INSIDE HEIGHT H	HEIGHT SPRING LINE h	DOME RADII MAJOR R	DOME RADII MINOR r	THICKNESS WALL t_w	THICKNESS SLAB t_s	THICKNESS DOME t_d	EQUIPMENT HATCH DIA.	HAUNCH l x w	FOUNDATION MATERIAL
PALISADES UNIT 1	55	1,640,000	SPHERE-TORUS	116'-0"	190'-6"	150'-6"	87'-6"	34'-0"	3'-6"	8'-6" TO 13'-6" VARIES	3'-0"	12'-0"	11'-6" x 3'-6"	SAND
POINT BEACH UNITS 1 & 2	60	970,000	do	105'-0"	147'-3"	99'-6"	80'-11"	31'-0"	3'-6"	8'-0" TO 11'-6" VARIES	3'-0"	15'-0"	11'-6" x 3'-6"	PILES
TURKEY POINT UNITS 3 & 4	55	1,550,000	do	116'-0"	184'-4"	141'-2"	87'-6"	34'-0"	3'-9"	10'-6"	3'-3"	14'-0"	7'-0" x 3'-6"	OOLITE
OCONEE UNITS 1, 2, & 3	59	1,910,000	do	116'-0"	220'-3"	177'-0"	87'-6"	34'-0"	3'-9"	8'-6"	3'-3"	19'-0"	10'-0" x 3'-6"	ROCK
CALVERT CLIFFS UNITS 1 & 2	50	2,000,000	do	130'-0"	194'-11"	146'-0"	78'-8"	32'-0"	3'-9"	10'-0"	3'-3"	19'-0"	10'-0" x 3'-6"	SAND
ARKANSAS UNIT 1	59	1,850,000	do	116'-0"	208'-6"	168'-6"	87'-6"	34'-6"	3'-9"	9'-0"	3'-3"	15'-6"	11'-6" x 3'-6"	ROCK
ARKANSAS UNIT 2	54	1,850,000	do	do	do	do	do	do	do	do	do	do	do	ROCK
MILLSTONE UNIT 2	54	1,920,000	do	130'-0"	187'-7"	140'-11"	94'-6"	35'-6"	3'-9"	8'-6"	3'-3"	19'-0"	10'-0" x 3'-6"	HARD ROCK
MIDLAND UNITS 1 & 2	67	1,670,000	do	116'-0"	193'-0"	153'-0"	87'-6"	34'-6"	3'-6"	9'-0" TO 13'-6" VARIES	3'-0"	13'-0"	NONE	CLAY (GLACIAL TILL)
FARLEY UNIT 1	54	1,830,000	do	130'-0"	175'-3"	128'-8"	94'-6"	35'-6"	3'-9"	9'-0"	3'-3"	19'-6"	10'-0" x 3'-6"	SOIL
RANCHO SECO UNIT 1	59	1,980,000	do	130'-0"	185'-0"	141'-7"	94'-6"	35'-6"	3'-9"	9'-0"	3'-6"	19'-0"	10'-0" x 3'-6"	SOIL
TROJAN	60	2,000,000	HEMISPHERICAL	124'-0"	205'-9"	143'-9"	62'-0"	– – –	3'-6"	9'-0"	2'-6"	19'-0"	NONE	ROCK

BECHTEL DESIGNED CONTAINMENTS

TABLE 2-1

TABLE 2-1 (cont'd.)

FACILITY	NUMBER OF BUTTRESSES	BUTTRESS DIM.		SLAB DIAM B	RING GIRDER DIAM G	PRESTRESSING SYSTEM	TENDON AREA (sq.in)	ULTIMATE TENDON STRENGTH (kips)	EFFECTIVE PRESTRESS			HOOP CONFIGURATION (Ref. Fig. 2-5 & 2-6)
		FACE	SIDE						HOOP (kips/ft)	VERT (kips/ft)	DOME (kips/ft)	
PALISADES UNIT 1	6	12'-0"	1'-9"	128'-0"	125'-0"	BBRV (RYERSON) 90-¼" DIA. WIRES/TENDON	4.42	1060	665	290	370	CYL. HOOP: 120° WRAP
POINT BEACH UNITS 1 & 2	6	12'-0"	1'-9"	116'-0"	114'-0"	do	do	do	662	290	360	do
TURKEY POINT UNITS 3 & 4	6	11'-11"	2'-1"	126'-6"	125'-6"	do (PRESCON)	do	do	670	294	360	do
OCONEE UNITS 1, 2, & 3	6	12'-0"	2'-4"	131'-0"	125'-6"	do (PRESCON)	do	do	700	300	360	do
CALVERT CLIFFS UNITS 1 & 2	6	11'-11"	2'-2"	144'-0"	139'-6"	do (PRESCON)	do	do	630	300	360	do
ARKANSAS UNIT 1	3	14'-2"	2'-8"	128'-4"	125'-6"	BBRV (PRESCON) 186-¼" DIA. WIRES/TENDON	9.12	2190	730	342	384	CYL. HOOP: 240° WRAP
ARKANSAS UNIT 2	do	do	do	128'-4"	do	do	do	do	670	384	355	do
MILLSTONE UNIT 2	3	14'-0"	2'-8"	145'-0"	139'-6"	do	do	do	630	380	330	do
MIDLAND UNITS 1 & 2	3	11'-11"	2'-6"	127'-10"	123'-0" (FLUSH)	BBRV (RYERSON) 170-¼" DIA. WIRES/TENDON	8.34	2000	687	350	362	do
FARLEY UNIT 1	3	14'-0"	2'-11"	146'-6"	139'-6"	do	8.34	do	620	350	320	do
RANCHO SECO UNIT 1	3	12'-0"	3'-0"	145'-0"	139'-6"	(VSL) 55-½" DIA. 7 WIRE STRANDS/ TENDON	8.52	2300	642	399	344	do
TROJAN	3	12'-0"	2'-7"	134'-6"	– – –	BBRV (PRESCON) 180-¼" DIA. WIRES/TENDON	8.84	2120	CYL. 635 DOME 365	328	328	CYL. HOOP: 240° WRAP EXTENDING 45° INTO DOME

BECHTEL DESIGNED CONTAINMENTS

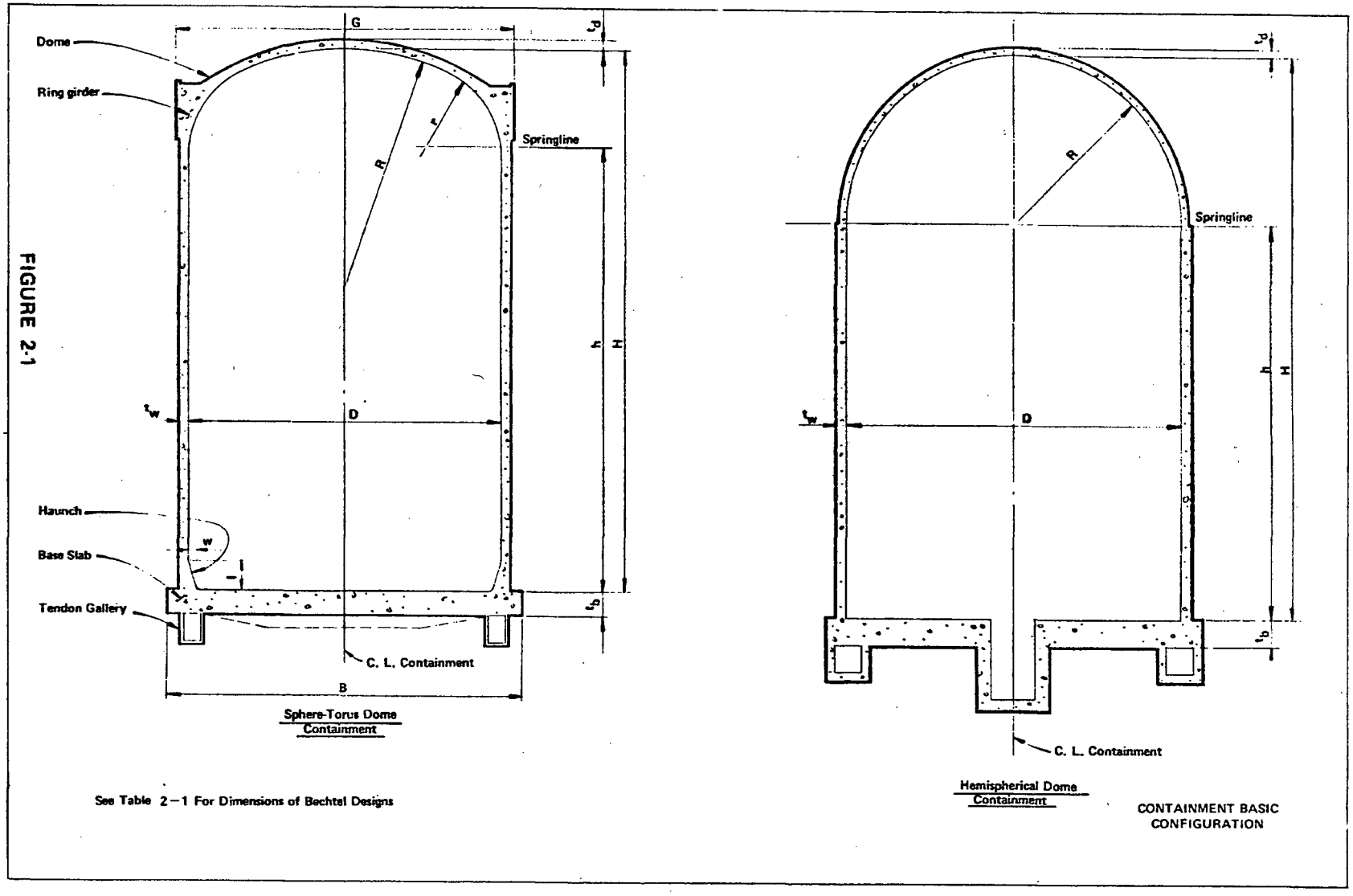

FIGURE 2-1

See Table 2-1 For Dimensions of Bechtel Designs

CONTAINMENT BASIC CONFIGURATION

1/27/2010 Interview with Richard Portmann (Tendon Engineer) for FM 1.3 and 1.4

- Precision Surveillance Corporation (PSC) does tension surveillance for Progress.
- PSC has a separate Quality Program and NCR reporting program. If they over stress a tendon PSC has to write a NCR and Progress does also. Richard Portmann has seen no NCRs on overstressing tendon.
- 80% of tensile will only increase the existing radial stress by 15% and is not sufficient to initiate separation. 80% is the initial tension used for new wires.
- WO 681043-01 contains all work documents, PSC procedures and results. It is about 150 MB and it is in the Progress FM 1.3 file.
- Richard Portmann does pre-job brief for crews and a job safety analysis before they go to work on the surveillance.
- PSC procedure SQ 9.0S covers monitoring of liftoff values and Q 10.0 covers detensioning.
- PSC procedure SP-182 covers the Reactor Building Structural Integrity Tendon Surveillance. SP-182 section 3.3.16 covers the maximum overstress of 80%.
- The surveillance is performed the same way for horizontal and vertical tendons.

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CRYSTAL RIVER UNIT 3

PLANT OPERATING MANUAL

SP-182

REACTOR BUILDING STRUCTURAL INTEGRITY TENDON SURVEILLANCE PROGRAM

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NOTE

These Enclosures are included for reference by the Surveillance Contractors Inspection Manual discussed in Section 4.0, Instructions.

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1.0 PURPOSE

The purpose of the Tendon Surveillance Program is to demonstrate the integrity of the containment prestressing system, including containment tendons, tendon end anchorage hardware, general and adjacent concrete integrity, and evaluation of the corrosion protective (grease) system. Individual inspections of selected tendons, as well as tendon wire and grease sample testing, are to be performed to the requirements of this procedure, to evaluate the overall integrity of the prestressing system.

2.0 REFERENCES

2.1 Developmental References

2.1.1 Technical Specification References

Applicable References	Surveillance Performance During Modes	LCO/Other Requirements During Modes
3.6.1	1 through 6	1, 2, 3, 4
SR 3.6.1.2	1 through 6	N/A
5.6.2.8	1 through 6	N/A
5.7.2	N/A	N/A

Frequency Notes:

Tendon surveillance was required at 1, 3, and 5 years after the Initial Containment Structural Integrity Test (SIT), and shall be performed every 5 years thereafter for the life of the plant. The 10 year and subsequent examinations shall commence not more than 1 year prior to the specified dates and shall be completed not more than 1 year after such dates.

2.1.2 Drawing References

2.1.2.1 Prescon Corporation - Tendons Drawings (series)

2.1.2.2 Swan Manufacturing Company - Conduit Drawings (series)

2.1.2.3 Reactor Building - Concrete Drawings (series)

2.1.3 Procedure References

2.1.3.1 CAP-NGGC-0200, Corrective Action Program

2.1.3.2 MNT-NGGC-0050, Measuring & Test Equipment Calibration Program

2.1.3.3 OPS-NGGC-1305, Operability Determinations

2.1.3.4 Compliance Procedure, CP-151, External Reporting Requirements

2.1.3.5 Administration Instruction, AI-400E, Performance and Transmittal of Procedures

2.1.3.6 Administration Instruction, AI-500, Conduct of Operations - Operations Department Organization and Administration

2.1.3.7 SAF-ESGX-00002, Energy Supply Fall Protection Procedure

2.1.3.8 Administration Instruction, AI-1803, Safety Standard for Ladders, Scaffolds, and Ancillary Equipment

2.1.3.9 Administration Instruction, AI-607, Pre-Job and Post-Job Briefings

2.1.4 Code and Standard References

2.1.4.1 ASTM A421-65, Standard Specification for Uncoated Stress Relieved Wire for Prestressed Concrete

2.1.4.2 ACI 201.1R-68, Revised 1984, Guide for Making a Condition Survey of Concrete Inservice

2.1.4.3 ASTM D512, Standard Test Methods for Chloride Ion in Water

2.1.4.4 ASTM D992, Standard Test Methods for Nitrite-Nitrate in Water

2.1.4.5 APHA 427, Standard Methods for Examination of Water and Waste Water

2.1.4.6 ASTM D95, Standard Test Method for Water in Petroleum Products and Bituminous Materials by Distillation

2.1.4.7 ASTM D974 (Modified), Standard Test Method for Acid and Base Number by Color-Indicator Titration

2.1.5 Other References

2.1.5.1 SAF-PGNF-00065, Accident Prevention Manual

2.1.5.2 Surveillance Contractor operating manuals and calibration charts for hydraulic stressing jack, pumps, and controls.

2.1.5.3 Dome, Vertical and Hoop Tendon History Sheets

2.1.5.4 Precision Surveillance Corporation, Inservice Inspection Manual N1002 for Florida Power Corporation - Crystal River Unit 3 Nuclear Station.

2.1.5.5 SP-182, Reactor Building Structural Integrity Tendon Surveillance Program, Revision 13, and past surveillance results.

2.1.5.6 Information Notice 99-10, Degradation of Prestressing Tendon Systems in Prestressed Concrete Containments.

2.1.6 Regulatory and Code Document References

2.1.6.1 U.S. Regulatory Guide 1.35, Revision 3, July 1990, Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containments

2.1.6.2 Section XI of the ASME Boiler and Pressure Vessel Code, Subsection IWL, Requirements for Class CC Concrete Components of Light-Water Cooled Plants, 1992 Addenda of the 1992 Edition as amended by 10CFR50.55a, Codes and Standards

2.1.7 Design Analysis/Calculation References

2.1.7.1 Calculation S-95-0082, Preparation of Tendon Force Curves for 6th Tendon Surveillance

2.1.7.2 Calculation S-80-0002, Required Minimum Average Tendon Forces

2.1.7.3 Calculation S-01-0019, Preparation of Tendon Force Curves for the 7th Tendon Surveillance at Crystal River.

2.1.7.4 Calculation S-07-0033, Preparation of Tendon Force Curves for the 8th Tendon Surveillance at Crystal River

3.0 PERSONNEL INDOCTRINATION

3.1 Setpoints

None

3.2 Description

The Containment Tendon Surveillance Program defined in this SP-182, including inspection frequencies, and acceptance criteria, complies with the requirements from the 1992 addenda of the 1992 Edition of ASME Section XI, Subsection IWL and the applicable amendments as specified in 10CFR50.55a, Codes and Standards.

The Tendon Surveillance Program is usually conducted during an outage. However, it may be conducted periodically during plant operation. For the surveillance during plant operation, special precautions must be taken to avoid work in hazardous areas resulting from plant operating conditions. This includes the steam vent zone outside and between buttresses 1 and 3, and areas inside the plant where radiological or other safety hazards must be considered. Also, any potential impact or effects on safety related systems and equipment in the immediate vicinity of the surveillance activity must be considered.

For surveillance periods during plant outages, the effects of heavy load drops must be considered for all surveillance work performed over the fuel pool area during the period when fuel pool missile shields have been removed for fuel transfer.

3.2.1 Tendon Selection

Eleven tendons shall be selected for each surveillance period consisting of 5 hoop tendons, 3 vertical tendons, and 3 dome tendons.

Inspections performed at 1, 3, 5, 10, 15, 20 and 25 years after the initial SIT have indicated no abnormal degradation of the post-tensioning system. Therefore, the number of tendons selected for subsequent surveillances complies with ASME Section XI, Subsection IWL, in that at least 2% of the population of each group of tendons (hoop, vertical and dome), or 5 tendons, whichever is less, with a minimum of 3 tendons in each group, will be included.

It will not be necessary to enter the Reactor Building at any time during Tendon Surveillance Program activities. Access to the vertical tendon upper anchorage is from the top of the ring girder and access to the lower anchorage is from the tendon gallery. In instances where a tendon is not accessible or acceptable for lift-off tests, Engineering Technical Services will be notified. If the tendon was identified to be inspected it will be exempted from this surveillance and Engineering will select an alternate tendon as close as possible to the original tendon. The exempted tendon will be inspected per IWL-2524 and IWL-2525 to the extent that the anchorage areas are accessible either during operation or at an outage. If the tendon was an adjacent tendon, Engineering will select the closest neighboring tendon and adjacent (excluding the reduced force tendons, see Enclosure 6: Reduced Force Dome Tendons).

3.2.2 Hold Points

The "Hold Points" identify steps beyond which work will not proceed until an inspection is performed by a qualified Nuclear Quality Control Inspector. The Site Project Manager for the surveillance, or his designee, must notify the Nuclear Quality Control Supervisor, or his designee, at each "Hold Point".

3.2.3 End Cap Removal and Inspection

The end cap will be removed and inspected for any moisture or free water. If free water is present, samples will be taken for later testing. Two grease samples will be collected from each tendon end for the testing laboratory, see Section 3.2.9: Laboratory Testing. If the end cap shows signs of corrosion it will be cleaned of all grease and sent to the Site Paint Shop for cleaning and re-painting by CR3. The tendon anchorage areas, including the bearing plates and shims, will be inspected for corrosion or other signs of deterioration. The anchorage assembly will be inspected for missing buttonheads, missing, broken, and/or damaged wires or other obvious defects. The degree of corrosion will be compared with that noted at the time of installation or previous tendon surveillance inspections. The thickness of the shim pack will also be measured.

The concrete in the area surrounding the tendon anchorage assembly will be inspected for any large spall, severe scaling, D-cracking (as defined in ACI-201.1R-68), other surface deterioration or disintegration, or grease leakage. Engineering will be notified if any cracks are measured to be wider than 0.010 inch.

In addition to the end caps for the selected tendons, all accessible end caps of the remaining tendons shall be visually inspected for significant grease leakage or end cap deformation.

3.2.4 Lift-Off Measurements

Lift-off will be determined by the following method:

The jacking pressure will be applied to the anchor head until lift-off is achieved. (Note: The jacking force will not exceed 1721 kips based on 163 available wires.) After achieving lift-off 2 feeler gauges (0.030 inch) shall be inserted about 180 degrees apart, between the anchor head washer and the shim. The pressure will then be reduced to transfer the load back to the shim stack. Pressure will be applied to the tendon until the feeler gauges can be pulled out with some effort.

3.2.5 Detensioning

Tendons to be detensioned are listed in Enclosure 2: Identification of Surveillance Tendons - Inspection Period 8. The control tendon shall not be detensioned unless required due to failure to meet acceptance criteria. If it must be detensioned a new control tendon, of the same type, will be selected. The jacking pressure will be increased until the shims can be removed, not exceeding a force of 1721 kips (based on 163 available wires). All buttonheads will be visually inspected for off-size, cracked, and missing heads. Also the anchorage assembly will be inspected for missing, broken, and/or damaged wires. The degree of corrosion will be compared with that noted at the time of installation or previous tendon surveillance inspections.

3.2.6 Wire Removal

Tendons selected for a physical condition test are noted in Enclosure 2: Identification of Surveillance Tendons - Inspection Period 8. One random wire will be removed while the tendon is completely detensioned. Any wires identified as broken will be removed and handled the same as the random wire sample. The wire will be measured and inspected for pitting, corrosion, or other signs of deterioration. If there are areas of significant pitting or loss of area on any wire, Engineering will be notified and will determine the course of action. A minimum of 3 samples will be taken for the testing laboratory (see Section 3.2.9: Laboratory Testing). A sample from each end, and one from the middle will be obtained. A fourth sample will be cut from the area of most significant corrosion or pitting, if it exists. The length of each sample shall be the maximum length acceptable for the test apparatus to be used by the testing laboratory.

3.2.7 Retensioning

Tendons to be retensioned shall be stressed to 80% of the minimum ultimate tensile strength of the available wires in the tendon (1868 kips for a 163 available wire tendon). This is performed to minimize the frictional effects on the forces in the overall tendon. During retensioning of the tendon, the elongation of the tendon wires shall be measured and recorded along with the corresponding force at 3 equally spaced intervals. The stressing force will then be reduced to the accepted lift-off or BASE Value (100% value) for final lock-off with a tolerance of -0% to +6%.

At the conclusion of retensioning, all buttonheads will be visually inspected for cracked and/or missing heads. Also, the anchorage assembly will be inspected for broken and/or damaged wires.

3.2.8 Tendon Regreasing

Once all inspections are completed on a tendon the end cap will be replaced with a new 0-ring. If any scratches, nicks, and other sharp depressions are present in the gasket bearing surface, a nonmetallic epoxy will be used to repair them prior to end cap installation. The tendon will then be filled with grease.

The grease will be pumped at a minimum temperature of 130°F, but not greater than 210°F. The pressure will not exceed 55 psi at the filler end of the hoop, dome and upper vertical tendon grease cap. The pressure at the lower vertical grease cap will not exceed 125 psi. Engineering will be notified when the absolute difference between the amount of grease removed and the amount of grease pumped is equal to 5% of the net duct volume for a given tendon. Pumping can continue up to 20 gallons beyond this point under Engineering approval.

3.2.9 Laboratory Testing

Testing laboratories qualified by the Surveillance Contractor's Quality Program will be required to perform the following services for this procedure:

- inspect wires for corrosion and other defects
- perform required tensile tests for yield strength, ultimate tensile strength, and elongation
- test bulk filler grease samples for chlorides, sulfides, nitrates, reserve alkalinity, and water content
- test free water samples for pH level

In the event that the testing laboratory finds a wire with significant pitting or loss of area, Engineering will determine the required course of action. An acceptable course of action would be to remove additional wires from the applicable surveillance tendons to be inspected to determine the extent and cause of change. However, each tendon shall have at least 155 wires which meet the acceptance criteria of Enclosure 8, Criteria and Categories for Rating Degrees of Corrosion on Tendon Anchorage Assemblies and Selected Wires, and Enclosure 9, Criteria for Missing, Broken, and/or Damaged Wires.

One grease sample will be sent to the testing laboratory while the remaining sample is held by the surveillance contractor or CR3 for possible confirmation testing.

3.3 Definitions

3.3.1 Adjacent Tendons - Adjacent or adjoining tendons are usually on each side of a selected tendon. Reduced Force Dome Tendons, Enclosure 6, are not considered adjacent tendons for a normal selected dome tendon. The next immediate normal tendon shall be considered the adjacent tendon. For hoop tendons located at the very bottom or top of a grouping, the next 2 tendons directly above or below, respectively, are considered the adjacent tendons.

3.3.2 Anchor Head (stressing washer) - round flat piece of steel that the tendon buttonheads bear.

3.3.3 Anchorage - components that distribute the forces in the tendon to the concrete containment structure.

3.3.4 BASE or BASE Curve (Predicted) - A plot of the predicted tendon force versus time accounting for prestress losses based on the criteria provided in Regulatory Guide 1.35.1, Revision 0.

3.3.5 Bearing Plate (baseplate) - steel plate embedded in the concrete containment that the anchor head transfer the tendon load to.

3.3.6 Buttonhead - the end of the tendon wire that transfers the tendon load to the anchor head.

3.3.7 Corrosion Protection Medium (grease, filler material) - wax and oil compound that fills the tendon cavity to protect against corrosion.

3.3.8 Elongation - the distance that a tendon wire stretches under load.

3.3.9 End Cap - steel container that protects the tendon anchorage and is bolted to the bearing plate.

3.3.10 Free Water - moisture of a quantity that is observed collected or draining out from the end cap during inspection.

3.3.11 Hold Point - point beyond which work will not proceed until an inspection is performed and sign-off by a qualified Nuclear Quality Control Inspector.

3.3.12 Lift-Off - the force (or pressure) required to separate the anchor head form the shim stack.

3.3.13 Lock-Off - the force (or pressure) when the tendon load is transferred to the shim stack.

3.3.14 Minimum Required Prestress Levels - Average prestress force levels which satisfy the design load conditions for maintaining the structural integrity of the containment. Force levels for this average prestress condition are 1215 kips for the dome tendon group, 1149 kips for the vertical tendon group, and 1252 kips for the hoop tendon group.

3.3.15 Normalization Factor - Factors for each individual tendon which are based upon the original tendon stressing sequence of installation. The normalization factor accounts for the effects of elastic shortening.

3.3.16 Overstress - the force (or pressure) that is approximately the yield strength of the tendon (80% of the minimum ultimate tensile strength).

3.3.17 Pit - An indentation visible to the unaided eye. A pit is caused by corrosion and is not the result of mechanical damage to the wire.

3.3.18 Ram - hydraulic jacking device used to apply force to a tendon.

3.3.19 Sheathing (cavity, conduit, duct, void) - thin-walled pipe in the concrete containment that the tendon wires pass through.

3.3.20 Shims - steel plates inserted between the anchor head and the bearing plate.

3.3.21 Stressing Adaptor (coupler) - threaded device that connects the ram to the anchor head.

3.3.22 Tendon - name for combination of wires and anchor heads.

3.3.23 Tendon Identification Number - the numbering convention for locating the tendons:

Dome: D xxx where:
Vert: y V xx xxx or xx = sequence numbers
Hoop: yz H xx y or z = buttress numbers

3.3.24 Trumpet - the portion of the sheathing connected to the bearing plate (larger diameter then the sheathing).

3.3.25 Wire - 7 mm diameter wire.

3.4 Responsibilities

3.4.1 Surveillance Contractor - This procedure is designed and written for work to be performed by an experienced Surveillance Contractor. The Surveillance Contractor, Precision Surveillance Corporation for the 8th Tendon Surveillance shall be responsible for assuring that all individuals under its supervision are properly trained in the use of this procedure, their inspection manual and associated equipment. The work supervisor is responsible for performance of the step by step instructions of this procedure, and their inspection manual - and for assuring that work is completed satisfactorily and QC is notified for data collection/inspection steps.

3.4.2 Site Project Manager - Responsible for coordinating site support activities for the Surveillance Contractor.

3.4.3 Engineering Technical Services, or its designee - Is responsible for providing technical support and for the evaluation and disposition of problems as identified during the surveillance. This includes responsibility for the general walkdown and inspection of the containment, and for the documentation of the results of this inspection.

3.4.4 Nuclear Quality Control (NQC) Supervisor, or designee - Is to be notified at each "Hold Point" within this procedure and is responsible for providing the required QC inspection personnel.

3.4.5 Nuclear Quality Control Inspector - Is responsible for performing QC inspections as indicated within this procedure, and for the recording and documentation of all inspections. The QC Inspector is responsible for providing the documented approval to proceed at each "Hold Point".

3.4.6 Responsible Engineer (RE) - A Registered Professional Engineer experienced in evaluating the in-service condition of structural concrete. The RE is responsible for development of inspection procedures, approval of examination personnel, evaluation of results, and the preparation of repair procedures.

3.5 Limits & Precautions

3.5.1 General

3.5.1.1 Work must be in compliance with the safety requirements of OSHA, and CR3 procedures and policies.

3.5.1.2 Missile shields shall be in place over Spent Fuel Pool "A" during surveillance activities for applicable tendons, if any, as designated in Enclosure 2: Identification of Surveillance Tendons Inspection Period 8, unless all equipment has been secured/rigged in accordance with NUREG-0612.

3.5.1.3 Missile shields shall be in place when moving any platforms and/or stressing rams over Spent Fuel Pool "A", unless all equipment has been secured/rigged in accordance with NUREG-0612.

3.5.1.4 Dome tendons listed in Enclosure 6, Reduced Force Dome Tendons, are not acceptable for normal lift-off testing; therefore, they shall not be substituted as adjacent tendons.

3.5.1.5 No more than 3 adjacent tendons shall be detensioned at any given time without Engineering concurrence.

3.5.1.6 During plant operation, work cannot be performed in the area between buttress numbers 1 and 3 (see Enclosure 3: Dome Tendon Locations Plan - Inspection Period 8). This is to avoid potential exposure to high pressure steam from the steam vents.

3.5.1.7 For work located in Radiation Controlled areas, due consideration must be given to the ALARA program. This may result in a determination that special preparation and/or precautions are necessary.

3.5.2 Filler Material

3.5.2.1 A certified test report bearing 2 signatures for the grease specified in Step 3.7.1.25 is required. The water soluble chloride and sulfide content must not exceed 2 ppm. The water soluble nitrate content must not exceed 4 ppm.

3.5.2.2 The reuse of filler material in the end cap is acceptable provided a Surveillance Contractor supplied procedure for reuse is accepted by CR3. All grease drained or removed from the anchorage area and tendon cavity shall be disposed of as waste grease.

3.5.2.3 Care must be taken when removing the end cap since the filler material may drop off or drip as a medium viscous liquid.

3.5.2.4 At the lower anchorage, the entire column of filler material may drain from the tendon during end cap removal.

3.5.2.5 Samples of the grease shall be taken prior to cleaning of the anchorage area.

3.5.2.6 Do not use metal implements to remove filler material from around the anchorage. Wooden or plastic instruments may be used.

3.5.2.7 Grease temperatures within the 55-gallon drum must not exceed 210°F.

3.5.3 Equipment

3.5.3.1 Calibration of all equipment used to measure tendon force shall be done in accordance with a calibration procedure prior to the first tendon force measurement and after the final tendon force measurement. Accuracy of the calibration shall be within 1.5% of the specified minimum ultimate strength of the tendon. Calibration of all measuring devices must be signed, dated, and traceable to the National Institute of Standards and Technology (NIST) documentation.

3.5.3.2 Verification of calibration of the stressing jack-pressure gauge systems must be checked daily against a master pressure gauge which is used only for this purpose.

3.5.4 Inspection and Testing

3.5.4.1 "Hold Points" must be verified by a qualified NQC Inspector(s) to comply with the requirements of Appendix B, Criterion X of 10CFR50.

3.5.4.2 Provide sufficient manpower to remove the end caps. The weight of each end cap with grease is about 225 lbs.

3.5.4.3 Once the inspection of a given tendon end has begun, it should be completed as soon as possible to avoid unnecessary exposure of the anchor head.

3.5.4.4 Do not leave the anchorage area and the end cap unprotected when work is not being performed. Tendon anchorages which have been cleaned of grease for an inspection shall be protected by applying a coat of grease to all exposed surfaces prior to the temporary or permanent installation of the end cap.

3.5.4.5 During inclement weather, provide protection to prevent moisture from entering the end anchorage.

3.5.4.6 The Surveillance Contractor shall confirm that the proper stressing adaptor is attached to the anchor head prior to stressing the tendon.

3.5.4.7 Do not stand behind hydraulic jack while stressing a tendon.

3.5.4.8 Exercise extreme caution if fingers or hands are required near the tendon anchor head during testing.

3.5.4.9 Do not exceed a jack force of 1721 kips (based on 163 available wires) during the tendon stressing without NPTS concurrence, except as stated in Step 3.2.7: Retensioning.

3.5.4.10 Do not detension either tendon end until lift-off has been determined for both ends of a hoop or dome tendon (vertical tendons are detensioned at top end only). Lift-off for each end may be determined on separate days.

3.5.4.11 Detensioning of each tendon end may proceed independently of the other end. However, jacks must not be uncoupled until the tendon is completely detensioned. To alleviate holding jacks under pressure for periods of time, it is recommended that both ends of a tendon be detensioned in unison. This should assist in correct positioning of the anchor head washer.

3.5.4.12 Vertical tendons are detensioned from the top end only, which is the shop end of the tendons. All inspection requirements apply to both top and bottom anchorages.

3.5.5 Wire Removal

3.5.5.1 Verify that all lift-off force measurements and detensioning is complete prior to wire removal.

3.5.5.2 Perform necessary measures when removing and handling tendon wires to preclude wire damage such as scratches, kinks, etc., and to preclude the accumulation of dirt or other contaminants on the surface of the wire.

3.5.5.3 The Surveillance Contractor shall carefully clean removed tendon wires just prior to visual inspection and packaging for delivery to the testing laboratory.

3.5.5.4 Do not stand directly in front of either end of the tendon when a wire is to be cut since some tendons may not be completely detensioned.

3.5.6 Retensioning

3.5.6.1 The Surveillance Contractor shall confirm that the proper stressing adaptor is attached to the anchor head prior to stressing the tendon.

3.5.6.2 Both ends of a hoop or dome tendon shall be retensioned simultaneously in a controlled manner. Maintain approximately the same elongation on both ends of the tendon.

3.5.6.3 It is important that when stressing from both ends that the crew's operation in stressing be coordinated so that a rapid stressing unit does not outpace a slower stressing unit. Ideally, the target gauge pressures must meet at the same time.

3.5.6.4 Vertical tendons only require retensioning from the top of the tendon.

3.5.7 End Cap Installation

3.5.7.1 Detrimental foreign matter (if present) such as loose mill scale, loose rust, loose or flaking paint on the gasket bearing surface can be removed with power tools.

3.5.7.2 A thread chaser may be required to clean the grease cap mounting holes.

3.5.7.3 All temporary tubes used for filling the end cap with grease, shall be removed.

3.6 **Acceptance Criteria**

NOTE

Contingency actions for non-compliance with the following acceptance criteria are provided in Section 5.2: Contingencies.

3.6.1 All documentation has been satisfactorily completed, including all required signatures and dates.

3.6.2 Abnormal conditions determined as the result of a visual inspection of the exterior concrete surface of the containment shall be recorded and documented, and investigated by Engineering for possible degradation of the structure.

Cracks found in concrete adjacent to the tendons (within 2 feet of the bearing plate) having widths greater than 0.010 inch shall be recorded and reported to Engineering for evaluation and resolution. Any crack widths greater than 0.050 inch shall be cause for investigation by Engineering to determine the cause and if there is any abnormal degradation of the structural integrity of the containment.

3.6.3 The wire tensile test shall have no failure below the minimum guaranteed ultimate stress of 240,000 psi. The breaking strength of the sample shall be greater than the minimum breaking strength for the wire diameter shown on Enclosure 11: Minimum Wire Break Strengths. Elongation shall meet or exceed the minimum required value for the tendon material. Wire corrosion indicative of metal reduction shall not be present.

3.6.4 Acceptance criteria for tendon force shall be as defined in Steps 3.6.4.1 through 3.6.4.6.

3.6.4.1 IF the measured prestressing force of the selected tendon in a group lies above the BASE value, or is equal to or greater than 95% of the BASE value (as defined in reference 2.1.7.3),
THEN the lift-off test is acceptable for that selected tendon.

3.6.4.2 IF the measured prestressing force of a selected tendon in a group is less than 95% but greater than or equal to 90% of the BASE value,
THEN two adjacent tendons shall be checked for their prestressing forces.
IF the measured prestressing forces of each of the second and third tested tendons are equal to or greater than 95% of their BASE value,
AND the measured prestressing force in all remaining tendons in a group are equal to or greater than 95% of their BASE value,
THEN all three tendons shall be restored to their required BASE value of prestress and the three tendons shall be considered acceptable.

3.6.4.3 IF the measured prestressing force of any two adjoining tendons falls below 95% of their BASE values,
THEN Engineering shall initiate an investigation into the causes of the occurrence. This condition is unacceptable.

3.6.4.4 IF the measured prestressing force of the selected tendon lies below 90% of the BASE value,
THEN the tendon shall be considered defective. Engineering shall initiate an investigation, and determine the extent and cause of such occurrence and the required course of action to be taken. This condition is unacceptable.

3.6.4.5 The normalized average of all measured tendon forces for each tendon group (i.e., dome, vertical, hoop) shall be greater than the minimum required prestress level for that group (as defined in Step 3.3.14). Engineering shall review and disposition any conditions where this is not met.

3.6.4.6 Engineering shall review the results and trends of the measured prestress forces from consecutive surveillances.
IF it is determined that the trend of prestress loss for individual tendons, or for any of the three groups of tendons is larger than expected,
THEN Engineering shall determine the cause and extent of such occurrence.
IF this trend indicates that the resulting prestress forces will be less than the minimum

required prestress forces prior to the next scheduled surveillance, THEN additional testing and evaluation shall be performed prior to the completion of the current surveillance to determine the cause and extent of such occurrence. This condition is unacceptable.

3.6.5 Acceptance criteria for sheathing filler samples shall be as defined in Steps 3.6.5.1 through 3.6.5.6.

3.6.5.1 Water content shall not exceed 10% by weight.

3.6.5.2 Water soluble chlorides, nitrates, and sulfides shall not exceed 10 ppm.

3.6.5.3 Reserve alkalinity (base numbers) shall not be less than 50% of the installed value or less than zero when the installed value was less than 5. If the tendon duct is filled with a mixture of materials having various as-installed base numbers, the lowest number shall govern acceptance.

3.6.5.4 The absolute difference between the amount of grease removed and the amount of grease replaced on a given tendon shall not exceed 10% of the net duct volume.

3.6.5.5 Any grease leakage determined as a result of surveillance inspection activities shall be investigated and repaired using an Engineering approved procedure.

3.6.5.6 Presence of free water shall be measured, sampled and documented and evaluated by Engineering.

3.6.6 During detensioning and retensioning of tendons, if the elongation corresponding to a specific load differs by more than +/- 10% from that recorded during installation an investigation shall be made by Engineering to ensure that the difference is not related to wire failures or wire slips in anchorages. This condition is unacceptable.

3.7 **Prerequisites**

3.7.1 Equipment

The equipment items listed in this section are recommended for implementation of this inspection.

3.7.1.1 Mobile crane with sufficient capacity and boom length to perform all necessary lifts of equipment and materials into position for surveillance activities.

3.7.1.2 All platforms and rigging, scaffolding, hoisting equipment, and accessories for access to the tendons and for performance of all surveillance work.

3.7.1.3 Two hydraulic stressing jacks (ram) having a loading capacity of 1000 tons with stressing pumps, pressure gauges, controls, and adjustable ram support device. The range and divisions of the pressure gages must be such as to permit reading of a 1000 ton force within an accuracy of +/- 2%.

3.7.1.4 Portable metal scaffolds as required.

3.7.1.5 Portable platform or plywood to be used for supporting the tendon end cap when removed (2 feet x 2 feet minimum).

3.7.1.6 Grease pump, transmission lines, various fittings, etc., connected to 55-gallon drum, equipped with heating system to heat grease to a temperature between 130°F and 210°F.

3.7.1.7 Containers, such as 55-gallon drums, to hold bulk filler grease removed from end anchorages during inspection.

NOTE

Permanent 480 volt electrical outlets exist on the top surface of the ring girder. There are five 480V, 3 phase receptacles on top of RB roof. Power provided by vent MCC 3a (MTMC-10) unit 5br located in IB119. See drawings 209-064, 209-131 sh 3 and 215 series for the one labeled RB160 South.

3.7.1.8 Electrical cables or heavy duty extension cords as necessary for lights, hydraulic stressing jack pumps, and other miscellaneous power tools.

3.7.1.9 Portable lights for illuminating tendon anchorage assemblies during check for corrosion and defects.

3.7.1.10 A set of portable communications equipment for communicating between crews at each end of the tendon being inspected.

3.7.1.11 Feeler gages (required range of blade sizes: 0.025 inch to 0.035 inch by 0.001 inch increments).

3.7.1.12 Optical comparators with 0.005 inch accuracy for measuring crack widths in concrete.

3.7.1.13 Grid paper for showing crack patterns.

3.7.1.14 Magnifying glass, 5X.

3.7.1.15 Temperature gauge(s) with a range of 0°F to 300°F and an accuracy of +/- 2% of full scale.

3.7.1.16 Wrenches to remove grease can hold down studs (3/4") and nuts.

3.7.1.17 Extraction tool for removing wires subject to tensile tests.

3.7.1.18 Come-along hoist for applying force to wire extraction tool in Step 3.7.1.17 (or similar device).

3.7.1.19 Torque wrench for end cap replacement, 40 ft-lb through 75 ft-lb minimum capacity.

3.7.1.20 Six-foot diameter wire coiler to coil the removed wire.

3.7.1.21 Small portable power tools (e.g., electric drill, saw, etc.) as required.

3.7.1.22 Wire cutters to cut wires 3/8" diameter (maximum).

3.7.1.23 Wire brush.

3.7.1.24 Steel hand hammer (approximately 24 oz.).

NOTE 1

The quantity is an estimate only; more or less may be required.

NOTE 2

The grease supplier must be acceptable to CR3 and qualified by the Surveillance Contractor's Quality Program for approving subcontractors and material suppliers.

NOTE 3

A certified test report bearing two signatures for the grease is required, indicating the water soluble chloride, sulfide, and nitrate content. The tests must be in accordance with the references specified in Section 2.1.1: Codes and Standard References. The water soluble chloride and sulfide content must not exceed 2 ppm and the water soluble nitrate content must not exceed 4 ppm.

NOTE 4

The reuse of grease from the end cap is acceptable provided a vendor-supplied procedure for reuse is accepted by CR3 in writing.

3.7.1.25 An appropriate quantity of certified filler grease (Visconorust 2090-P4 or latest approved compatible formulation, by Viscosity Oil Company).

NOTE

Chemical composition of solvent must be certified to meet requirements for Viscosity #16.

3.7.1.26 Solvent that can be used for removing grease from around tendon anchorage and cleaning any stained concrete (Viscosity #16 solvent by Viscosity Oil Company, or equal solvent certified to meet the requirements for Viscosity #16, as approved by CR3).

NOTE

Specifications of replacement materials delineated in Steps 3.7.1.27 through 3.7.1.31 shall be the same as those of the original items.

3.7.1.27 New O-rings for hoop, dome, and upper vertical tendon end caps (1/2" dia., 60 durometer, neoprene or nitrile base rubber, 14-1/2" ID x 15-1/2" OD).

3.7.1.28 New O-rings for lower vertical tendon end caps (5/8" dia. 60 durometer, neoprene or nitrile base rubber, 15-3/8" ID x 16-5/8" OD).

3.7.1.29 3/4" dia x 4-1/2", 10 TPI, material ASTM A193 Grade B7 studs, hex nuts (ASTM A194 Grade 2H), and washer assemblies.

3.7.1.30 3/4"-10 thread per inch thread chaser.

NOTE

Quantity is estimate only; more or less may be required.

3.7.1.31 Wire split increment shims of various thicknesses, recommended quantity and sizes: 30 sets 3/16", 25 sets 1/4", 20 sets 3/8", 20 sets 1/2". Material to be ARMCO VNT single normalized ASTM A633, Grade E or equal.

3.7.1.32 Supply of clean rags for cleanup with solvent around anchorages.

3.7.1.33 Wooden or plastic paddles or spatulas to scoop out bulk filler grease from around the anchorage assembly.

3.7.1.34 Carbo Zinc 11, zinc-filled inorganic coating (made by Carboline Co., or equal, as determined by the NPTS and the manufacturer).

3.7.1.35 Non-metallic Epoxy.

3.7.1.36 Any other miscellaneous material and tools as required.

3.7.2 Initial Conditions

3.7.2.1 Perform a Pre-Job Briefing for each new crew/shift in accordance with AI-607, Pre-Job and Post-Job Briefings. This briefing is required once at the start of the work by that crew. Also, notify the Control Room Supervisor (or designee), at the beginning of each work day, that work is to begin on this procedure.

3.7.2.2 Missile shields shall be in place over Spent Fuel Pool "A" during surveillance activities for those tendons indicated in Enclosure 2, Identification of Surveillance Tendons - Inspection Period 8, and as defined in Steps 3.5.1.2 and 3.5.1.3, unless all equipment has been secured/rigged in accordance with NUREG-0612. The Maintenance Manager or designee shall review all equipment setup and rigging to ensure NUREG-0612 requirements have been met prior to performing work activities which could impact the spent fuel pool without the missile shields installed.

3.7.2.3 Read and understand the Limits & Precautions, Section 3.5.

3.7.2.4 Read and understand the Description, Section 3.2.

3.7.2.5 Ensure that all applicable equipment listed in Section 3.7.1, Equipment, is available and ready to be used.

3.7.2.6 Ensure that there is adequate access to all tendon end caps of tendons that are to be tested (see Enclosure 2: Identification of Surveillance Tendons - Inspection Period 8).

3.7.2.7 Ensure that the testing laboratory indicated in Step 3.2.9 is prepared to receive the wires.

3.7.2.8 Ensure that the testing laboratory indicated in Step 3.2.9 is prepared to receive the grease and free water samples.

3.7.2.9 Ensure references listed in Section 2.0, References, are available for use.

3.7.2.10 Ensure that all personnel are familiar with the operating manuals of the equipment to be used during the inspection.

NOTE

Calibrated equipment must be re-calibrated at the end of the tendon surveillance program by the Post-Test Due Date in accordance with the requirements of that item.

3.7.2.11 Verify that stressing jacks, pressure gauges, comparators, and all other measuring devices have been calibrated per Step 3.5.3.1 and are in good working condition AND that calibrations are not expected to expire during the time period it will take to perform this surveillance.
IF the calibration is expected to expire prior to completing the surveillance,
THEN have equipment re-calibrated prior to commencing the surveillance.

3.7.2.12 Notify the Radwaste Organization that waste grease will need proper disposal.

3.7.2.13 Notify the NQC Supervisor or his designee that work is to begin on this procedure.

4.0 INSTRUCTIONS

Refer to Precision Surveillance Corporation's Inservice Inspection Manual N1002 for the step-by-step instructions to be followed for the 8th Tendon Surveillance at Crystal River Unit #3.

Only "controlled" copies of PSC's In-Service Inspection Manual N1002 shall be used for the performance of this surveillance.

5.0 FOLLOW-UP ACTIONS

5.1 Restoration Instructions

5.1.1 System restoration is accomplished during performance of Section 4.0: Instructions.

5.1.2 Repair procedures shall be developed and performed in accordance with IWL-4000.

5.1.3 Post repair system pressure testing requirements shall be developed and performed in accordance with IWL-5000.

5.1.4 Reinstallation and replacement requirements of the post-tensioning system shall be developed and performed in accordance with IWL-7000.

5.2 Contingencies

5.2.1 IF the acceptance criteria of Step 3.6.4.5 is not met,
AND engineering evaluation cannot ascertain acceptability of the containment tendons,
THEN refer immediately to the "Action Statement" of Technical Specification 3.6.1, Containment, and Technical Specification 5.7.2, Special Reports.

5.2.2 IF any of the acceptance criteria of Steps 3.6.2, 3.6.3, 3.6.4.3, 3.6.4.4, 3.6.4.6, 3.6.5, and 3.6.6 are not met,
THEN it shall be evaluated by Engineering per IWL-3300.

5.2.3 IF there is rejectable corrosion or pitting on the wire as defined in Enclosure 8, Criteria and Categories For Rating Degrees of Corrosion on Tendon Anchorage Assemblies and Selected Wires,
OR, if existing broken or unseated wires and/or detached buttonheads were not documented and accepted during a pre-service examination or during previous inspections,
OR, if the number of missing, broken, and/or damaged wires does not meet the criteria of Enclosure 9, Criteria For Missing, Broken, and/or Damaged Wires,
THEN each case shall be evaluated by Engineering per IWL-3300.

5.2.4 IF there is rejectable corrosion, pitting, or cracking, as defined in Enclosure 8, Criteria and Categories For Rating Degrees of Corrosion on Tendon Anchorage Assemblies and Selected Wires, on the tendon anchorage assembly,
THEN it shall be evaluated by Engineering per IWL-3300.

5.2.5 IF the post-test calibration of the equipment used to measure tendon force (Step 3.5.3.1) differs from the pretest calibration by more than the specified accuracy tolerance,
THEN the results of the examination shall be evaluated by Engineering.

5.3 Reports

5.3.1 Surveillance Contractors Report

A written report documenting the inspection results, laboratory test results and surveillance conclusions for each inspection period must be prepared and submitted to CR3 within 30 days of completion of the inspections and tests. The surveillance contractor's approved report and all data must be forwarded to Engineering.

5.3.2 Reportable Conditions

In the event that the acceptance criteria in Section 3.6 is not met, the Surveillance Contractor shall notify Engineering within 1 working day. Engineering shall follow the reporting process defined in CP-151: External Reporting Requirements (Step 2.1.3.5). If Engineering determines that there is abnormal degradation of the containment structure (based on the contingency actions in Section 5.2: Contingencies), CR3 shall submit a report to the NRC within 30 days of the current surveillance completion. This report shall include a description of the degradation, operability determination, root cause determination and the corrective action.

5.3.3 Procedure Closure

Upon completion of each Tendon Surveillance, Engineering is to ensure that SP-182 and Precision Surveillance Corporation Manual N1002 are transmitted to Records Management.

5.3.4 Evaluation Summary Report

A written summary report of the results and conclusions for each inspection period must be prepared within 90 days of the completion of the tests and inspections. The report shall also include items that did not meet the acceptance criteria and were evaluated by Engineering and address the following:

- the cause of the condition which did not meet the acceptance criteria
- the acceptability of the concrete containment without repair of the item
- whether or not repair or replacement is required and, if required, the extent, method, and completion date for the repair or replacement
- extent, nature, and frequency of additional examinations

This report shall be submitted to the NRC as part of the 90 ISI Summary Report.

IDENTIFICATION OF SURVEILLANCE TENDONS FROM PRIOR SURVEILLANCE PERIODS

	1ST SURVEILLANCE 11/77-2/78	2ND SURVEILLANCE 3/80-5/80	3RD SURVEILLANCE 9/81-12/81	4TH SURVEILLANCE 9/87-11/87	5TH SURVEILLANCE 11/93-1/94	6TH SURVEILLANCE 10/97-1/98	7TH SURVEILLANCE 08/01-12/01
DOME TENDONS 123 TOTAL 3 GROUPS OF 41 D100s, D200s, D300s	D139 D215 D221 D D228 D234 D340	D122 D140 D208 D D323 D331	D123 D125 D212 D322 D D329	D105 D D212 D328	D215 D231 D D224	D212 D304 D D113 D115 D311	D212 D339 D D126
VERTICAL TENDONS 144 TOTAL 6 GROUPS OF 24 12, 34, 56, 23, 45, 61	12V19 12V20 12V21 23V15 34V6 45V3 D 56V1	12V12 12V20 23V5 34V1 45V6 56V20 56V1 D	12V1 34V6 34V19 D 45V16 56V11 61V5	12V1 34V4 56V2 D	34V6 56V15 D 61V14	12V1 61V21 D 23V2	12V1 61V08 45V14 D
HORIZONTAL TENDONS 282 TOTAL 6 GROUPS @ 47 HIGH, 13, 24, 35, 46, 51, 62	13H10 13H19 13H37 13H47 51H11 62H9 64H21 64H29 64H37 D 64H46	13H22 13H32 D 13H43 51H10 51H23 51H37 53H24 53H28 53H44 64H42	13H19 13H46 42H20 42H40 51H26 51H45 53H35 53H40 62H34 64H10 D	13H20 13H40 D 51H26 51H41 64H19	34H1 42H1 46H21 46H28 46H29 D 46H30 46H47 D 62H8	51H26 42H18 42H32 42H44 53H2 62H41 D 62H46 53H46 42H49 42H30 42H31 42H33 42H34 42H35	46H21 46H36 53H16 D 62H02 62H13
TENDONS INSP.	23	22	21	11	14	22	11
TOTAL TENDONS = 549							
TOTAL TENDONS INSPECTED UP TO 7TH SURVEILLANCE = 124							

NOTE: For information only. These tendons were completed in past surveillance.
D – Designates a detensioned tendon.

IDENTIFICATION OF SURVEILLANCE TENDONS
INSPECTION PERIOD 8

SELECTED DOME TENDONS	ADJACENT TENDONS
D129 * **	D128, D130
D212 C	D211, D213
D238 D,*	D237, D239

SELECTED VERTICAL TENDONS	ADJACENT TENDONS
12V1 C **	61V24, 12V2
45V20 D	45V19, 45V21
61V17	61V16, 61V18

SELECTED HOOP TENDONS	ADJACENT TENDONS
46H21 C,*	46H20, 46H22
42H46 **	42H45, 42H47
13H36	13H35, 13H37
62H30 * **	62H29, 62H31
51H34D **	51H33, 51H35

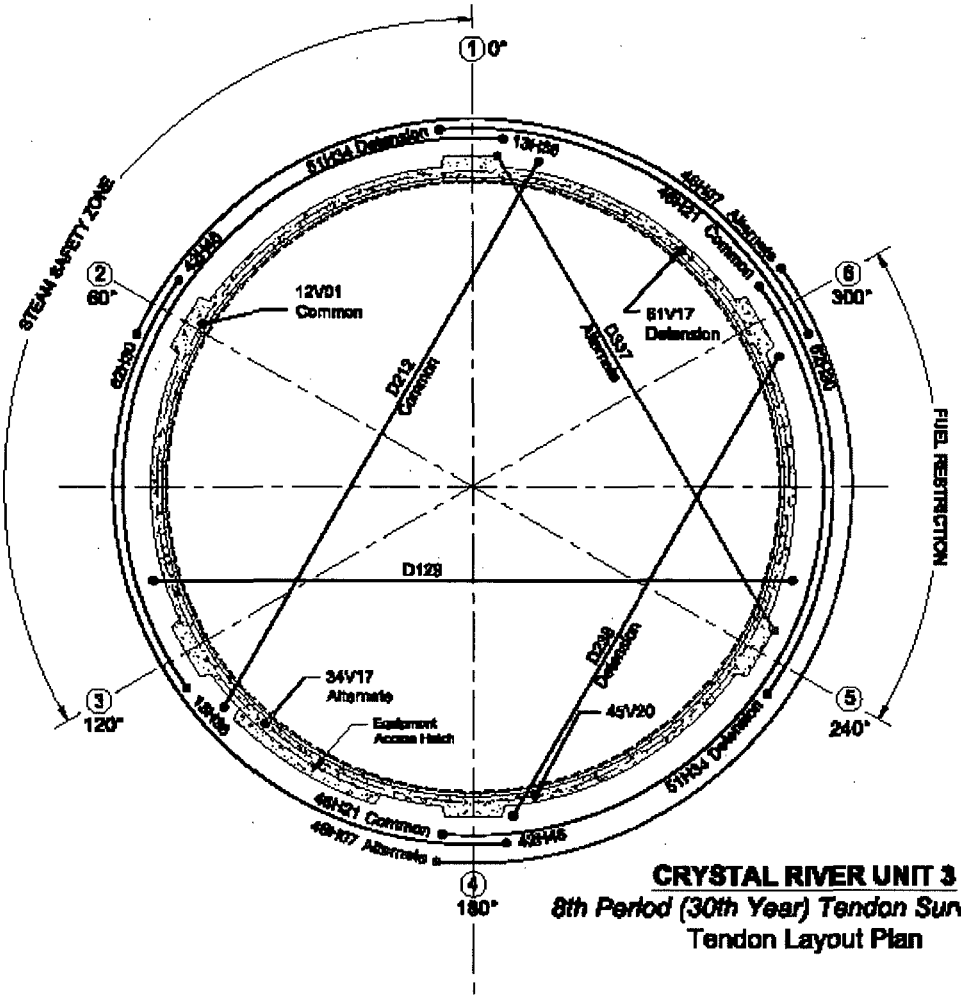
C Denotes selected control tendon.

D Denotes selected tendon for detensioning and wire removal. If removal of one wire from tendon would subject it to rejection per Enclosure 9, Criteria For Missing, Broken, and/or Damaged Wires, contact Engineering for a substitute tendon.

*Denotes tendons, if any, which must have missile shields in-place over Fuel Pool "A", per Step 3.7.2.2, unless all equipment has been secured/rigged in accordance with NUREG-0612.

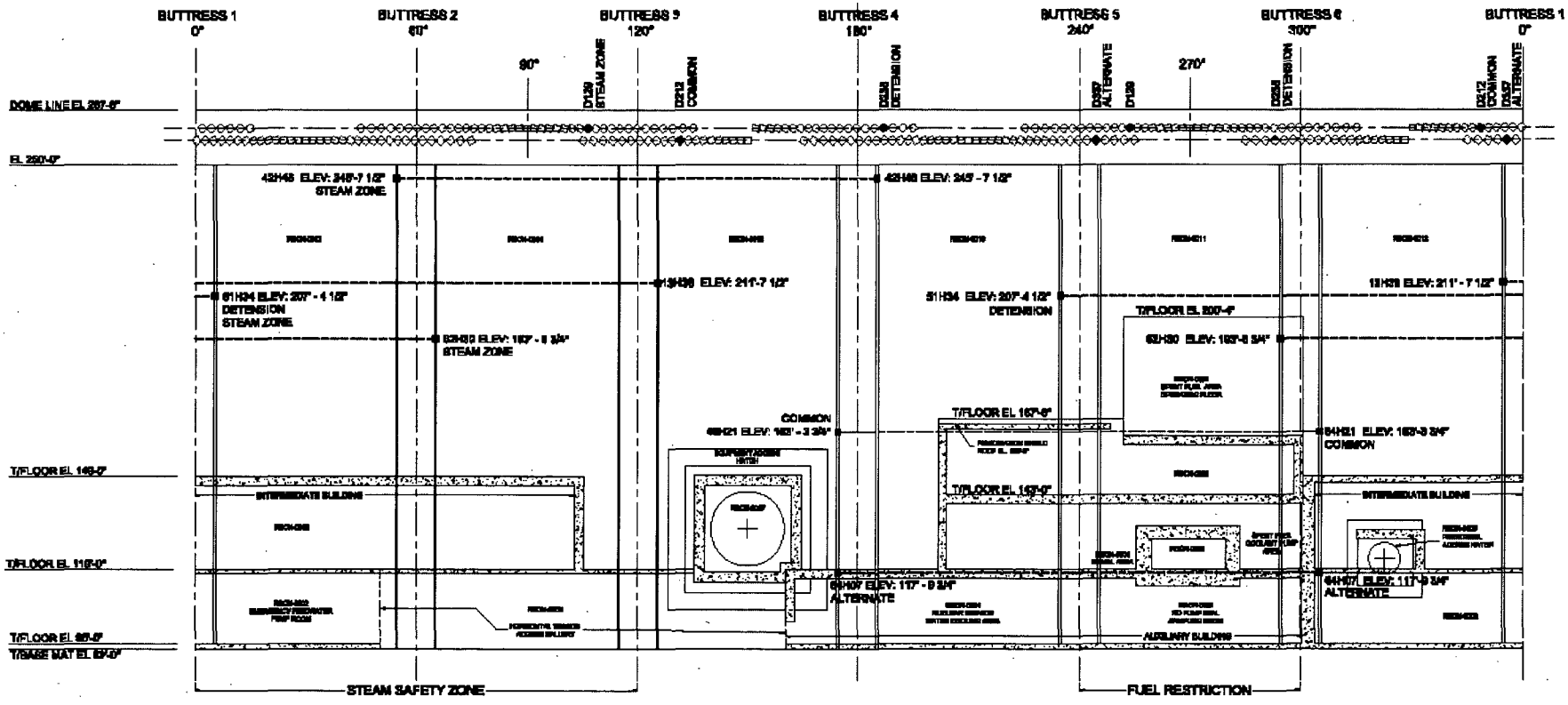
**Denotes tendons, if any, located in the Steam Zone Region.

DOME, VERTICAL, and HOOP TENDON LOCATION PLAN – INSPECTION PERIOD 8



CRYSTAL RIVER UNIT 3
8th Period (30th Year) Tendon Surveillance
Tendon Layout Plan

DOME and HOOP TENDON LOCATION PLAN – INSPECTION PERIOD 8



CRYSTAL RIVER UNIT 3
8th Period (30th Year) Tendon Surveillance
Tendon Layout Elevation

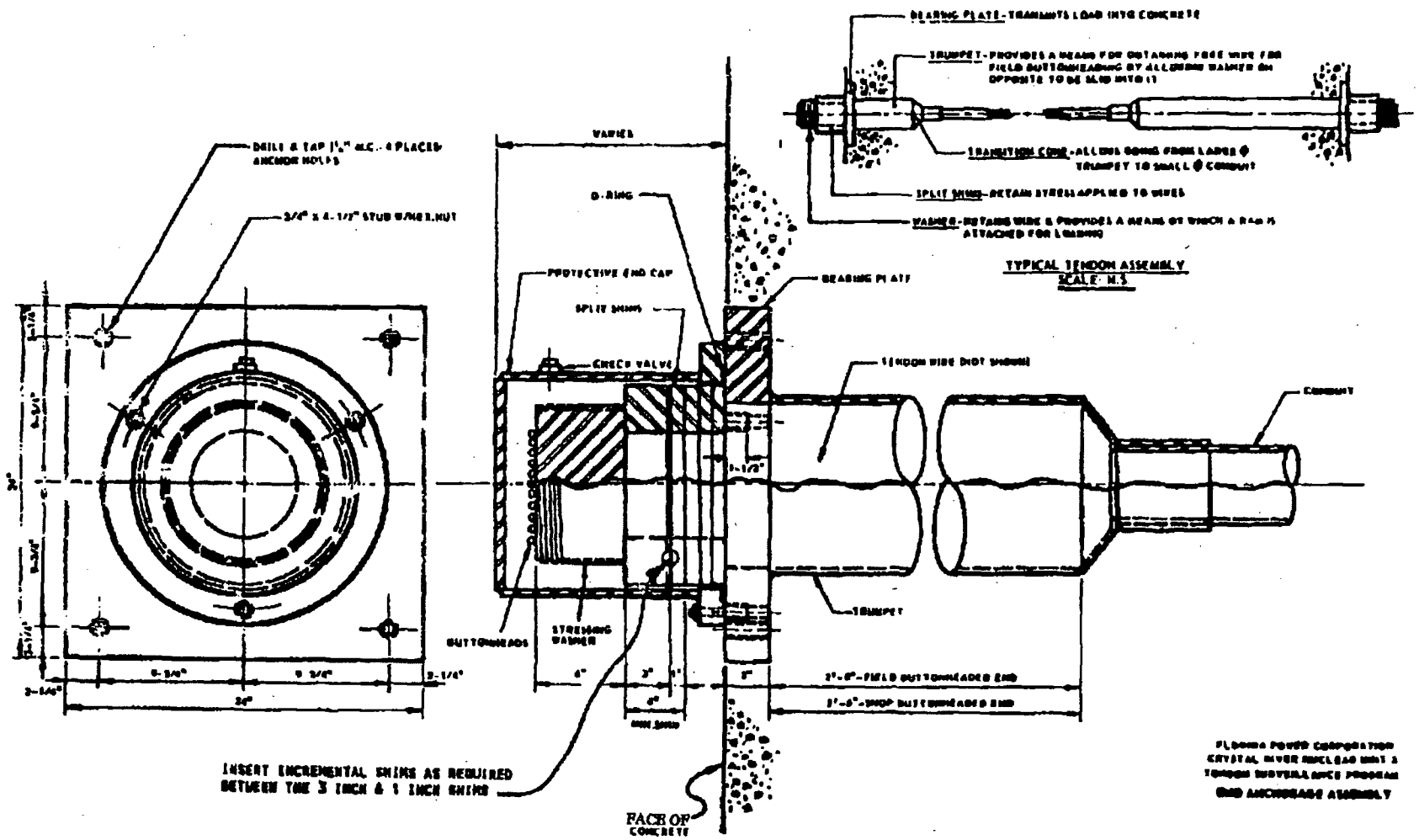
ENCLOSURE 5

REDUCED FORCE DOME TENDONS

The following tendons are not acceptable for normal lift-off testing:

D 101	D 201	D 301
D 109	D 209	D 309
D 117	D 217	D 317
D 125	D 225	D 325
D 133	D 233	D 333
D 141	D 241	D 341

ANCHORAGE ASSEMBLY DETAIL



CRITERIA AND CATEGORIES FOR RATING DEGREES OF CORROSION ON TENDON ANCHORAGE ASSEMBLIES AND SELECTED WIRES

CATEGORIES OF CORROSION

1. Bright metal; no visible oxidation.
2. Metal reddish brown color, no pitting.
3. Metal having patches of red oxide, removable but ready to start pitting.
4. Metal having patches of red oxide, not removable and/or leaving noticeable pits.
5. Metal having heavy rusting, dark red, and about to form an extremely hard crust which when removed leaves very noticeable pitting.
6. Conditions more severe than Category 5.

ACCEPTANCE CRITERIA

Anchorage assembly components in Category 1, 2, or 3 are acceptable.

Wire in Category 1 or 2 is acceptable.

Anchorage assembly components in Category 4, 5, or 6 and/or wires in Category 3, 4, 5, or 6 should be further evaluated by NPTS.

CRITERIA FOR VISUAL INSPECTION OF GREASE

Note that the original Visconorust 2090-P2 grease is no longer available. The new 2090-P4 grease will not be the exact color as the original 2090-P2 grease when it was new. Therefore, color comparisons of old grease against new grease must be made considering this basic difference.

Note if any of the following items are observed during visual inspection of the grease:

1. Extreme discoloration even when considering the above change in grease type.
2. Presence of corrosive particles and/or dirt mixed within the grease, indicating adjacent metal pitting and metal breakdown.
3. Signs of moisture within the bulk filler.
4. Other signs of grease deterioration.

CRITERIA FOR MISSING, BROKEN, AND/OR DAMAGED WIRES

1. Broken wires and unacceptable buttonheads shall not exceed 8 wires per individual tendon, nor more than 2 percent of the total number of wires in that group, nor more than 3 percent in any 10 consecutive tendons in a group. A group shall be defined as follows: Vertical tendons - one stressing sequence quadrant (36 tendons); Dome tendons - one series of layer (41 tendons); Hoop tendons - one side of a buttress (47 tendons).

2. An unseated wire, after stressing, that will move at one end of the tendon but is observed not to move at the other end is to be considered a broken wire.

   CAUTION - Do not strike the buttonhead with any heavy object.

3. If an unseated wire, after stressing, will not move - document and notify Engineering.

NOTE: Missing, broken, and/or damaged wire criteria is based on original quantity of 163 wires per tendon.

ORIGINAL ACCEPTANCE CRITERIA FOR BUTTONHEADS

PRECISION BUTTONHEADS – 0.275" ϕ/ (7mm) WIRE

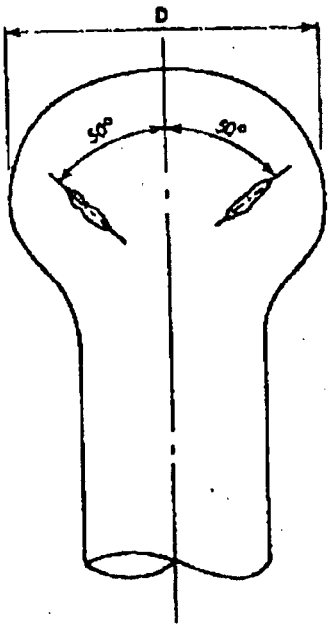
1. DIMENSIONS

Diameter – D	
Minimum	0.410"
Maximum	0.450"

2. SPLITS

Maximum admissible number	4
Maximum admissible width	0.060" Per Split
Maximum admissible length	0.350" Per Split
Maximum admissible angle to axis of wire	50°

NOTE: This is the criteria for new buttonheads and is given in this detail for reference only. The checks called for in Section 4.0, Instructions, of the procedure are not meant to verify these numbers, but they are meant to detect obvious gross deficiencies in any buttonhead.



MINIMUM WIRE BREAK STRENGTHS

(Tinius-Olson Direct Reading - High Scale or Equal)

WIRE DIAMETER (inch)	CROSS SECTIONAL AREA (sq in)	ULTIMATE TENSILE STRENGTH	MINIMUM BREAK (kips) (1,000 lbs)
0.27359 (-)	0.05879	240 ksi	14.110
0.27459 (-)	0.05922	240 ksi	14.213
0.27559 (7 mm)	0.05965	240 ksi	14.316
0.27659 (+)	0.06008	240 ksi	14.316+
0.27759 (+)	0.06052	240 ksi	14.316+

ORIGINAL STRESSING AVERAGE TENDON LIFT-OFF VALUES
VERTICAL TENDONS

TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF
12 V 1	1675	34 V 1	1651	56 V 1*	1784
12 V 2	1699	34 V 2	1588	56 V 2*	1603
12 V 3	1687	34 V 3	1597	56 V 3	1694
12 V 4	1651	34 V 4	1585	56 V 4	1658
12 V 5	1711	34 V 5	1633	56 V 5	1696
12 V 6	1586	34 V 6	1609	56 V 6	1622
12 V 7	1574	34 V 7	1646	56 V 7	1633
12 V 8	1615	34 V 8	1621	56 V 8	1602
12 V 9	1634	34 V 9	1605	56 V 9	1654
12 V 10	1615	34 V 10	1709	56 V 10	1598
12 V 11	1669	34 V 11	1605	56 V 11	1658
12 V 12	1670	34 V 12	1696	56 V 12	1646
12 V 13	1675	34 V 13	1644	56 V 13	1685
12 V 14	1687	34 V 14	1648	56 V 14	1687
12 V 15	1625	34 V 15	1655	56 V 15*	1638
12 V 16	1598	34 V 16	1639	56 V 16	1651
12 V 17	1650	34 V 17	1663	56 V 17	1617
12 V 18	1639	34 V 18	1633	56 V 18	1699
12 V 19	1654	34 V 19*	1573	56 V 19	1664
12 V 20	1598	34 V 20	1637	56 V 20	1687
12 V 21	1638	34 V 21	1660	56 V 21	1661
12 V 22	1655	34 V 22	1624	56 V 22	1665
12 V 23	1638	34 V 23	1646	56 V 23	1651
12 V 24	1624	34 V 24	1648	56 V 24	1675
23 V 1	1711	45 V 1	1686	61 V 1	1629
23 V 2	1598	45 V 2	1627	61 V 2	1629
23 V 3	1661	45 V 3*	1639	61 V 3	1627
23 V 4	1670	45 V 4	1610	61 V 4	1663
23 V 5	1711	45 V 5	1649	61 V 5	1643
23 V 6	1670	45 V 6	1614	61 V 6	1658
23 V 7	1636	45 V 7	1607	61 V 7	1675
23 V 8	1676	45 V 8	1675	61 V 8	1598
23 V 9	1627	45 V 9	1661	61 V 9	1625
23 V 10	1616	45 V 10	1677	61 V 10	1598
23 V 11	1673	45 V 11	1696	61 V 11	1643
23 V 12	1646	45 V 12	1603	61 V 12	1610
23 V 13	1687	45 V 13	1673	61 V 13	1711
23 V 14	1646	45 V 14*	1697	61 V 14	1646
23 V 15	1615	45 V 15	1625	61 V 15	1590
23 V 16	1598	45 V 16	1661	61 V 16	1628
23 V 17	1660	45 V 17	1683	61 V 17	1646
23 V 18	1598	45 V 18	1661	61 V 18	1598
23 V 19	1598	45 V 19	1624	61 V 19	1591
23 V 20	1633	45 V 20	1624	61 V 20	1634
23 V 21	1663	45 V 21	1629	61 V 21*	1622
23 V 22	1622	45 V 22	1672	61 V 22	1622
23 V 23	1639	45 V 23	1634	61 V 23	1663
23 V 24	1634	45 V 24	1684	61 V 24	1616

*Tendons detensioned and/or retensioned during previous inspections.

[All Lift-off Values in kips (1000#)]

ORIGINAL STRESSING AVERAGE TENDON LIFT-OFF VALUES (Cont'd)
DOME TENDONS

TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF
D101	643*	D201	652*	D301	660*
D102	1660	D202	1649	D302	1581
D103	1606	D203	1662	D303	1653
D104	1606	D204	1649	D304**	1610
D105**	1646	D205	1642	D305	1629
D106	1626	D206	1643	D306	1643
D107	1667	D207	1657	D307	1656
D108	1646	D208**	1648	D308	1646
D109	643*	D209	640*	D309	649*
D110	1622	D210	1616	D310	1636
D111	1673	D211	1689	D311	1682
D112	1676	D212	1600	D312	1640
D113	1676	D213	1646	D313	1636
D114	1670	D214	1642	D314	1621
D115	1700	D215	1666	D315	1607
D116	1646	D216	1614	D316	1604
D117	658*	D217	660*	D317	660*
D118	1563	D218	1626	D318	1635
D119	1642	D219	1639	D319	1697
D120	1652	D220	1619	D320	1653
D121	1633	D221**	1670	D321	1668
D122	1664	D222	1649	D322**	1628
D123	1610	D223	1655	D323	1670
D124	1634	D224	1598	D324	1664
D125	634*	D225	649*	D325	660*
D126	1634	D226	1625	D326	1640
D127	1634	D227	1649	D327	1661
D128	1660	D228	1669	D328	1670
D129	1619	D229	1650	D329	1645
D130	1635	D230	1665	D330	1634
D131	1600	D231**	1651	D331	1636
D132	1620	D232	1603	D332	1667
D133	652*	D233	652*	D333	660*
D134	1640	D234	1643	D334	1598
D135	1673	D235	1632	D335	1669
D136	1679	D236	1591	D336	1616
D137	1562	D237	1661	D337	1622
D138	1645	D238	1664	D338	1638
D139	1686	D239	1615	D339*	1639
D140	1669	D240	1615	D340	1634
D141	649*	D241	660*	D341	652*

*Reduced Force Tendons
**Tendons detensioned and/or retensioned during previous inspections.

[All Lift-off Values in kips (1000#)]

ORIGINAL STRESSING AVERAGE TENDON LIFT-OFF VALUES (Cont'd)
HOOP TENDONS

TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF
13 H 1	1629	53 H 1	1640	64 H 1	1642
13 H 2	1640	53 H 2	1555	64 H 2	1701
13 H 3	1630	53 H 3	1598	64 H 3	1649
13 H 4	1640	53 H 4	1645	64 H 4	1628
13 H 5	1643	53 H 5	1606	64 H 5	1579
13 H 6	1658	53 H 6	1627	64 H 6	1628
13 H 7	1630	53 H 7	1622	64 H 7	1643
13 H 8	1634	53 H 8	1673	64 H 8	1646
13 H 9	1593	53 H 9	1591	64 H 9	1623
13 H 10	1604	53 H 10	1650	64 H 10*	1646
13 H 11	1606	53 H 11	1584	64 H 11	1635
13 H 12	1598	53 H 12	1663	64 H 12	1616
13 H 13	1691	53 H 13	1615	64 H 13	1649
13 H 14	1604	53 H 14	1609	64 H 14	1680
13 H 15	1624	53 H 15	1651	64 H 15	1589
13 H 16	1595	53 H 16*	1639	64 H 16	1616
13 H 17	1630	53 H 17	1603	64 H 17	1617
13 H 18	1618	53 H 18	1657	64 H 18	1670
13 H 19	1625	53 H 19	1604	64 H 19	1617
13 H 20	1604	53 H 20	1633	64 H 20	1665
13 H 21	1642	53 H 21	1634	64 H 21	1653
13 H 22	1652	53 H 22	1651	64 H 22	1665
13 H 23	1649	53 H 23	1609	64 H 23	1637
13 H 24	1646	53 H 24	1621	64 H 24	1653
13 H 25	1643	53 H 25	1663	64 H 25	1657
13 H 26	1628	53 H 26	1650	64 H 26	1635
13 H 27	1618	53 H 27	1627	64 H 27	1655
13 H 28	1658	53 H 28	1626	64 H 28*	1690
13 H 29	1607	53 H 29	1669	64 H 29*	1667
13 H 30	1658	53 H 30	1704	64 H 30	1642
13 H 31	1635	53 H 31	1532	64 H 31	1655
13 H 32*	1653	53 H 32	1668	64 H 32	1635
13 H 33	1623	53 H 33	1646	64 H 33	1648
13 H 34	1641	53 H 34	1632	64 H 34	1702
13 H 35	1631	53 H 35	1604	64 H 35	1655
13 H 36	1650	53 H 36	1655	64 H 36	1678
13 H 37	1629	53 H 37	1664	64 H 37*	1617
13 H 38	1612	53 H 38	1643	64 H 38	1645
13 H 39	1671	53 H 39	1616	64 H 39	1665
13 H 40*	1623	53 H 40	1660	64 H 40	1586
13 H 41	1660	53 H 41	1634	64 H 41	1611
13 H 42	1660	53 H 42	1622	64 H 42	1599
13 H 43	1641	53 H 43	1646	64 H 43	1641
13 H 44	1654	53 H 44	1653	64 H 44	1641
13 H 45	1635	53 H 45	1634	64 H 45	1647
13 H 46	1623	53 H 46	1628	64 H 46	1644
13 H 47	1623	53 H 47	1688	64 H 47*	1623

*Tendons detensioned and/or retensioned during previous inspections.

[All Lift-off Values in kips (1000#)]

ORIGINAL STRESSING AVERAGE TENDON LIFT-OFF VALUES (Cont'd)
HOOP TENDONS

TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF	TENDON I.D.	LIFT-OFF
51 H 1	1639	42 H 1	1645	62 H 1	1603
51 H 2	1620	42 H 2	1650	62 H 2	**
51 H 3	1645	42 H 3	1653	62 H 3	1615
51 H 4	1625	42 H 4	1672	62 H 4	1631
51 H 5	1675	42 H 5	1605	62 H 5	1672
51 H 6	1608	42 H 6	1679	62 H 6	1603
51 H 7	1608	42 H 7	1647	62 H 7	1603
51 H 8	1644	42 H 8	1665	62 H 8	1624
51 H 9	1627	42 H 9	1641	62 H 9	1639
51 H 10	1674	42 H 10	1645	62 H 10	1604
51 H 11	1615	42 H 11	1599	62 H 11	1603
51 H 12	1668	42 H 12	1614	62 H 12	1675
51 H 13	1644	42 H 13	1597	62 H 13	1663
51 H 14	1644	42 H 14	1622	62 H 14	1603
51 H 15	1651	42 H 15	**	62 H 15	1681
51 H 16	1649	42 H 16	1635	62 H 16	1627
51 H 17	1663	42 H 17	1661	62 H 17	1675
51 H 18	1608	42 H 18	1664	62 H 18	1609
51 H 19	1669	42 H 19	1647	62 H 19	1640
51 H 20	1668	42 H 20	1662	62 H 20	1610
51 H 21	1669	42 H 21	1641	62 H 21	1640
51 H 22	1638	42 H 22	1668	62 H 22	1663
51 H 23	1609	42 H 23	1617	62 H 23	1639
51 H 24	1644	42 H 24	1617	62 H 24	1639
51 H 25	1639	42 H 25	1647	62 H 25	1627
51 H 26	1661	42 H 26	1614	62 H 26	1663
51 H 27	1612	42 H 27	1635	62 H 27	1669
51 H 28	1668	42 H 28	1632	62 H 28	1633
51 H 29	1629	42 H 29	1629	62 H 29	**
51 H 30	1657	42 H 30	1644	62 H 30	1597
51 H 31	1646	42 H 31	1653	62 H 31	1681
51 H 32	1658	42 H 32	1626	62 H 32	1639
51 H 33	1650	42 H 33	1665	62 H 33	1674
51 H 34	1651	42 H 34	1626	62 H 34	1626
51 H 35	1608	42 H 35	1657	62 H 35	1664
51 H 36	1616	42 H 36	1680	62 H 36	1639
51 H 37	1606	42 H 37	1641	62 H 37	1622
51 H 38	1628	42 H 38	1650	62 H 38	1639
51 H 39	1607	42 H 39	1623	62 H 39	**
51 H 40	1664	42 H 40	1651	62 H 40	1639
51 H 41	1631	42 H 41	1623	62 H 41*	1609
51 H 42	1610	42 H 42	1599	62 H 42	1609
51 H 43	1642	42 H 43	1611	62 H 43	1591
51 H 44	1646	42 H 44	1605	62 H 44	1651
51 H 45	1581	42 H 45	1665	62 H 45	1622
51 H 46	1657	42 H 46	1644	62 H 46	1645
51 H 47	1663	42 H 47	1615	62 H 47	1582

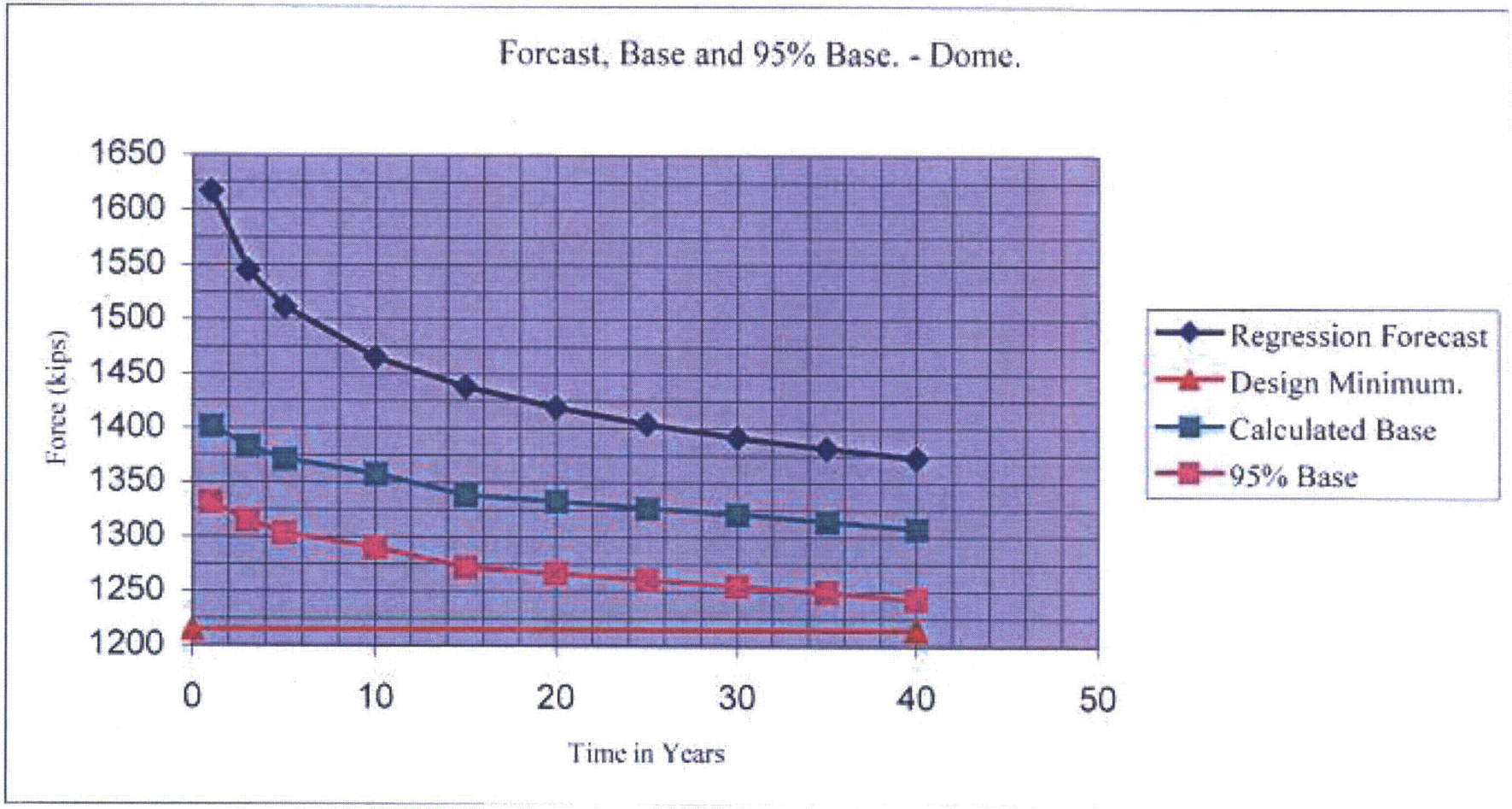
*Tendons detensioned and/or retensioned during previous inspections.
**Not Available.

[All Lift-off Values in kips (1000#)]

TENDON NORMALIZING FACTORS – INSPECTION PERIOD 8

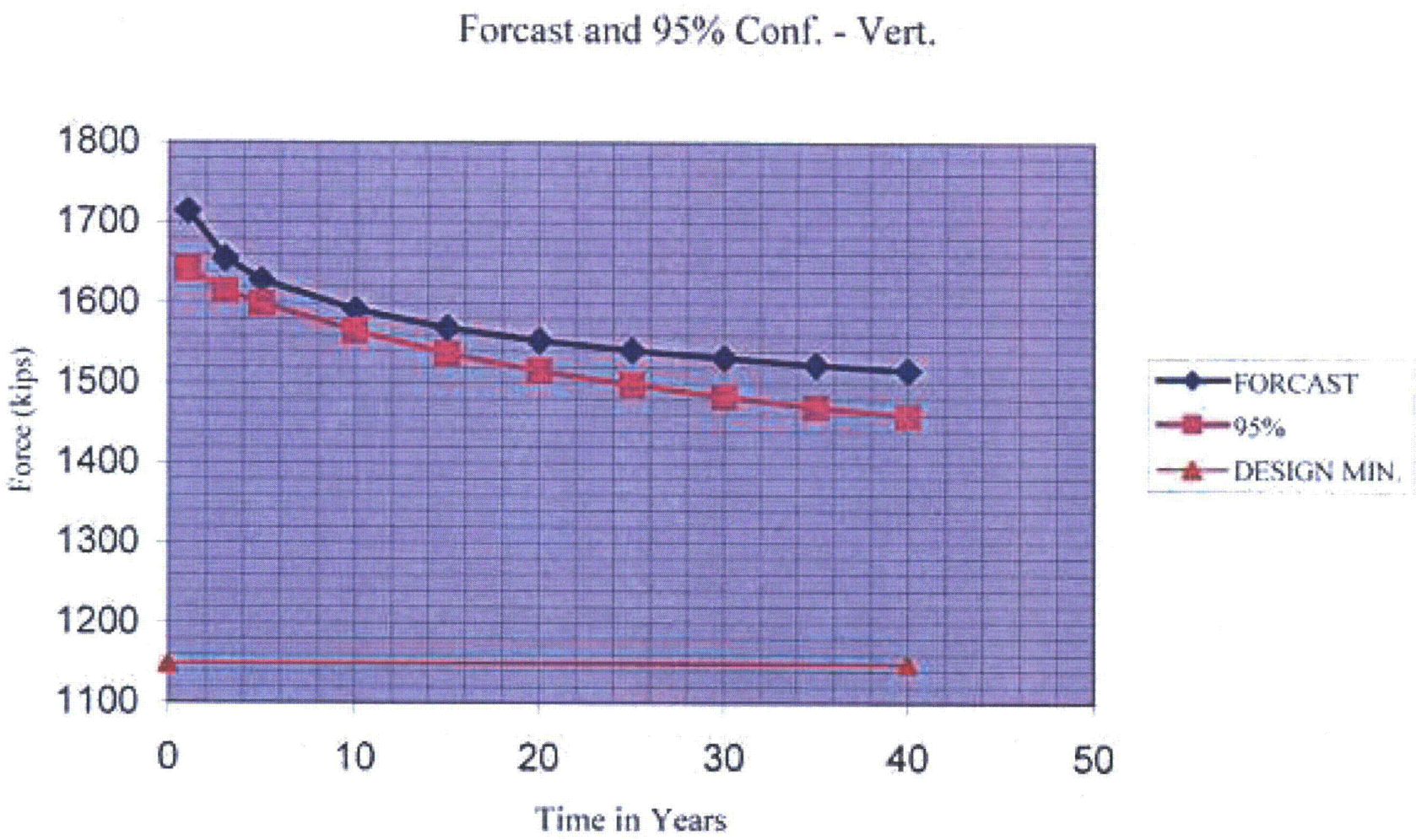
DOME TENDONS		VERTICAL TENDONS		HOOP TENDONS	
TENDON I.D.	NORMALIZING FACTOR	TENDON I.D.	NORMALIZING FACTOR	TENDON I.D.	NORMALIZING FACTOR
D128	53	61V24	-14	46H20	-39
D129	34	12V1	-9	46H21	-12
D130	60	12V2	-80	46H22	-57
D211	18	45V19	31	51H33	36
D212	15	45V20	-61	51H34	-59
D213	10	45V21	-19	51H35	79
D237	-104	61V16	-8	42H45	-44
D238	-26	61V17	17	42H46	-28
D239	-88	61V18	21	42H47	3
				13H35	56
				13H36	-56
				13H37	61
				62H29	7
				62H30	14
				62H31	-46

TENDON HISTORICAL TRENDS
DOME TENDONS

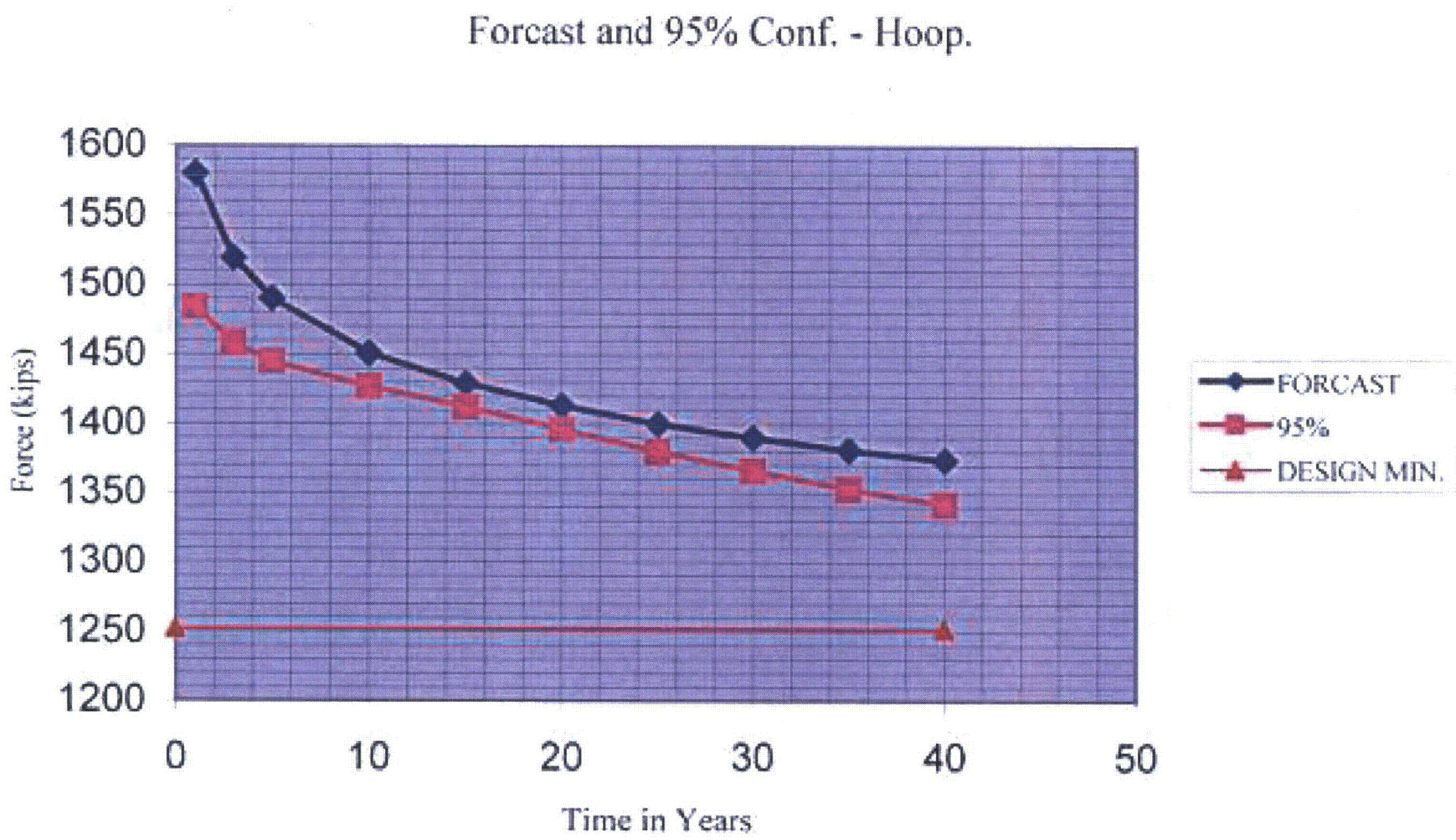


ENCLOSURE 13
(Page 2 of 3)

TENDON HISTORICAL TRENDS
VERTICAL TENDONS



TENDON HISTORICAL TRENDS
HOOP TENDONS



REVISION SUMMARY

Section	Description
ALL	Entire procedure revised for the 8th surveillance. Procedure reformatted, no revision bars used.
3.5.1.2, 3.5.1.3, 3.7.2.2, Enclosure 2	Added the requirement that the equipment needs to be secured/rigged in accordance with NUREG-0612 for work over the spent fuel pool without the missile shields installed. Per EC 64487, Rev. 0
PRR's incorporated	0168617, 222722, 161499, 62724
EC's Incorporated	EC 64487

CRYSTAL RIVER UNIT 3

SGR REPLACEMENT PROGRAM

ASSESSMENT OF THE CONCRETE CRACKS AND PATH FORWARD

By Orhan Gurbuz, Ph. D., P. E.

October 13, 2009

1. INTRODUCTION AND BACKGROUND

Crystal River unit 3 (CR3) plant is in the process of a steam generator replacement (SGR) program. The reactor building is a prestressed concrete containment with a shallow dome, a cylindrical wall, and a flat foundation mat. The cylindrical wall is prestressed with hoop and vertical tendons. The dome is prestressed with a three-way prestressing system. The foundation is a conventionally reinforced slab.

As part of the SGR project, a cut is made in the containment wall above the equipment hatch for removal and replacement of the existing steam generators. The cut is made using the hydro-demolition method, which cuts and removes the concrete with high pressure water jets. Prior to the removal of concrete the tendons in the construction opening are detensioned and removed. During the removal of the concrete at the opening, relatively large cracks were observed near the hoop tendon sheathing.

Since these cracks are larger in size than expected, this author was requested to visit the plant site, inspect the cracks at the construction opening, identify likely causes of cracks, and develop conceptual short and long term restoration alternatives. This report summarizes results of these examinations and evaluations.

2. OBSERVATIONS

Review of the criteria document, discussions with the engineers and field personnel and examination of the conditions at the construction opening provided an insight into the issues faced by the project. Observations from all the data may be summarized as follows:

a. Extensive cracks exist in the construction opening area. Some of these cracks are unusually large, in the order of 1 to 2 inches.

b. The main cracks as observed at the construction opening are in the plane between the horizontal and vertical tendons and parallel to the surface of the wall.

c. Additional cracks were also observed at the opening. These cracks appeared to be local and discontinuous but some of them eventually joined the main cracks.

d. The containment design thickness is 42" and the as-built data is not available. A survey taken around the construction opening showed wall thicknesses ranging from 43.25" to 45.5", including the cracks. If the minimum measured thickness is taken as the as-built condition, the crack widths are calculated to range from 0" to 2.25." Thus, it may be assumed that the outside concrete layer actually moved out, in some locations at least 2".

e. The crack surfaces are rather smooth; the cracks went right through the aggregates, indicating the cement paste may have been stronger than the aggregates. Normally, such cracks would have numerous aggregates protruding from the crack surface, resulting in a jagged interface. Consequently, the limestone aggregate used in the concrete on this project appeared to be weaker than expected;

f. Site engineers stated that there have been two occasions in the past when significant negative internal pressures were inadvertently applied inside the containment. Such negative pressures would exacerbate the width of cracks in the plane of the wall.

g. The design prestress level for this containment is 1.6, meaning that the prestress level is adequate to resist the forces generated by 1.6 times the design pressure. This prestress level is rather high. Although the ASME Code, Section III, Div 2 does not set a prestress level, it is customary to provide 1.2 or 1.25 times the design pressure as the prestress level.

h. As a result of the high prestress level, there is no need for reinforcement in the containment wall at this location. The only reinforcement is near the outer surface which consists of #8 bars at 12 inches, each way, presumably provided for thermal considerations. There is no indication that radial stresses resulting from tendon curvatures or due to Poisson's ratio effects were considered in the original design (see next item).

i. The original design permits concrete tension under extreme loads. The allowable concrete tension is $6(f_c')^{1/2}$. This level of allowable tension would presumably obviate the need for radial ties.

j. The new concrete design for the construction opening incorporates two layers of #11 bars at 11 inches in each direction.

3. POSSIBLE CAUSES OF CRACKS

The containment has been in operation over thirty years. The initial Structural Integrity Test has been successfully completed and numerous Integrated Leak Rate tests have been performed since then. It is stated that no abnormalities have been encountered in its structural behavior (except, it was stated, later tendon lift-offs have shown unusually high prestress losses). As typical of all

concrete structures, there are many discontinuous cracks in the containment wall which are attributed to shrinkage. However, the main cracks observed at the construction opening are more than the expected shrinkage cracks.

From the location and orientation of the cracks two conclusions can be drawn: (1) there is radial tension stress in the wall, and (2) the radial tension has exceeded the tensile strength of concrete in some areas. However, it is extremely difficult to pinpoint the exact root cause of the crack formation and widening. Consequently, the possible reasons for the formation of these cracks can only be hypothesized as discussed below.

a. The majority of these cracks probably existed from the time of initial prestress and resulted from radial tension in the wall which has no radial reinforcement. Radial tension may be generated as a result of the following factors:

    i. As the tendons are stressed, some radial tension develops in the thickness direction as a function of Poisson's ratio for concrete. This radial tension will be essentially uniform over the entire thickness of the wall.

    ii. Furthermore, additional radial stress is generated due to the compressive forces applied on the concrete at the hoop tendon location. These forces will tend to reduce the diameter of the concrete ring between the hoop tendons and the liner while pulling the outer ring inward. These radial stresses will be compressive between the liner and the hoop tendons and tensile outside the hoop tendons, reaching zero at both inside and outside face.

    iii. In summer months, the temperature on the outside part of the wall may be very high. As a result of this temperature increase, the outer layer will tend to grow and pull the inner ring of concrete. The resulting radial stresses will be tensile, with maximum value occurring probably between the hoop tendons and the outside surface, reaching zero at both inside and outside.

    iv. The total radial tension will probably be highest near the hoop tendons. If the maximum radial stress exceeds the modulus of rupture of concrete, a crack plane may form at that interface. The crack plane may be extensive but will eventually be intersected by the buttresses and local areas where radial reinforcement exists. The crack plane will be stopped at the point of these intersections. Maximum width of these cracks is expected to be in the order of 0.25 inches or less, considering the wall displacements under prestress loads.

b. The observed larger crack widths are attributed to widening of existing cracks by the following factors:

i. The concrete removal in the construction opening is performed using hydro-demolition. During this process water jets are used to break the concrete, with jet pressures reaching approximately 20,000 psi (Ref. 1). These are very high pressures. Any inadvertent application of a fraction of the resulting jet force to a small shrinkage crack will tend to widen the crack immediately. Depending on the skill and experience of the workers performing hydro-demolition, some of the cracks may have been accidentally widened by this process.

ii. The SGR program required detensioning of tendons in the construction opening area only prior to cutting of the opening. The tendons adjacent to the opening were not detentioned. Thus, when the outside layer of concrete (with #8 reinforcement at 12 inches, each way) was cut, the restraining force provided by the continuity of this layer was lost. This may have contributed to the increased crack widths adjacent to the opening, which are presumably still under radial tension.

4. SAFETY SIGNIFICANCE

Considering the design bases of this containment, it is concluded that the primary component to resist all loads is the prestressing system which places the entire containment wall and the dome in membrane compression. In addition, the concrete tension is relied on to resist small magnitudes of tension that can be generated under certain loading conditions such as thermal effects. Preliminary evaluations indicate that if the ring of concrete between the cracks and the outside surface is ignored, the remaining concrete and the prestressing system will be able to resist all design loads.

Therefore, these cracks do not impact the safety of the plant.

However, the wall examinations should continue until there is a complete understanding of the causes of cracks and their extent, the cracks should be properly repaired and other activities that are deemed necessary to resolve all the related issues should be completed to assure the integrity of the containment and its functionality.

5. EVALUATIONS

Considering the extent of the issues and the types of restoration activities, the evaluations should be in two categories: (a) Phase 1, and (b) Phase 2. The scope of these evaluations is summarized below.

a. Phase 1 Evaluations

After a review of the documents and discussions with the project engineers, the following Phase 1evaluations are considered essential prior to the refueling of the reactor (the work outlined below is ongoing):

i. Analyze the structure assuming an effective concrete depth and determine its ability to function as intended. For this purpose a field survey was conducted and crack data was collected at the opening. Based on the review of the crack data and considering the fact that concrete inside the hoop tendon sheathing will be in compression, an effective depth was determined. A brief calculation was prepared for the effective depth to be used in the evaluations (Ref.1).

ii. Using the analysis results, evaluate the critical sections around the periphery of the construction opening. This will be a check of the adequacy of the reduced section to resist all design loads within the allowable limits documented in the FSAR.

iii. In these evaluations the actual strength as determined from the core data may be used.

iv. Complete the survey by CTL and incorporate their findings into the evaluations described above.

The expected conclusion of the Phase 1evaluation is that the containment is adequate to function as intended under all design loading conditions. This Phase 1 evaluation is limited to the construction opening and the surrounding areas. Assuming that the expected conclusion is confirmed, refueling can continue as planned and the operations may resume.

b. Phase 2 Evaluations

In the longer term, further evaluations will be needed to determine the need for additional actions to bring the containment structure to its original design condition. These activities may include:

i. Complete the analysis of the entire containment to determine the available margin beyond design basis.

ii. The project has taken important steps to discover the extent of the cracks, particularly in the area of the construction opening. As part of this investigative effort, Concrete Technologies Laboratories (CTL) have been contracted to examine and evaluate the affected containment wall segments using state-of-the-

art techniques. This work should be completed and the results incorporated to the overall restoration program.

iii. If there is an existing Probabilistic Risk Assessment (PRA) report, it may be prudent to update this report.

iv. Study restoration alternatives to bring the containment to its initial design condition. That is, to integrate the outside ring of concrete (which is being ignored in Phase 1) with the inner ring.

v. Based on the above studies, determine on the appropriate action plan.

6. RESTORATION OF THE CONTAINMENT FOR RESTART

Assuming that the large cracks do not extend much beyond the construction opening and that Phase 1 evaluations confirm the integrity of the containment, the plant may be restarted at the completion of the SGR project as planned. The scope of work prior to restart is expected to consist of the following steps:

a. Complete the Phase 1 evaluations.

b. Complete the crack mapping process for the construction opening area. If the crack mapping indicates extensive cracking and large crack widths, the strategy should be re-evaluated.

c. Develop and apply a grouting or epoxy repair method to fill the larger cracks. The crack repair method should not permit application of large pressures in the cracks. This activity may be a part of the closure of the construction opening.

It is estimated the above evaluations can be completed by the end of outage so that the impact on the plant will be minimal.

7. PATH FORWARD

Phase 2 evaluations should be performed once the Phase 1 is successfully completed and the plant is restarted. The goal of these evaluations is to assure that the original plant margin is available. These evaluations should include the following activities:

a. Refine the containment model to be able to capture more detailed stress conditions at or near the discontinuities.

b. Re-analyze the containment structure using a refined model of the structure.

c. Check the structure at critical locations using the FSAR load combinations.

d. Prepare a final report addressing all the questions that have been raised

e. Meet with NRC and present the final conclusions of the report

The estimated time for the final resolution of these issues is at least six months.

8. SUMMARY AND CONCLUSIONS

The construction opening and the cracks around the opening were visually examined. Data on crack width, total wall thickness and distance from the liner to the cracks were collected around the opening. Drawings and criteria documents were reviewed to develop an understanding of the design bases for the Crystal River Unit 3 containment. In addition, discussions were held with the Progress Energy and Bechtel engineers. This author also participated in many telephone calls to discuss the relevant issues.

These examinations, reviews and discussions led to the following conclusions:

a. There are unusually large cracks at the construction opening, in the plane of the hoop tendons. The large cracks appear to close up at a relatively short distance from the opening.

b. These cracks are attributed to radial tension and were probably there from the time of initial prestress, presumably as small cracks and not necessarily continuous.

c. The detensioning procedure and the concrete removal methodology are believed to be the main reasons for enlargement of the cracks during the cutting operations.

d. These cracks are not safety-significant; the containment has sufficient capacity to meet its functional requirements under the design loading conditions.

e. The SGR program can continue as planned, after making the necessary changes in the load transfer procedures to assure the reduced section capacities are not exceeded.

f. The cracks should be repaired by use of epoxy or grout when the construction opening is closed.

g. The construction opening should be closed as planned. However, radial ties should be added to assure that similar cracks will not occur in the new concrete.

h. Because of the of the nature of these cracks and potential unanswered questions, additional work should be performed after the restart, including:

    i. Completion of the survey of the outside surface of the shell to determine the nature and extent of the cracking,

    ii. More refined analysis of the containment to determine the stresses and displacements in more detail, and

iii. If it is deemed necessary, restoration of the wall to its original design condition.

REFERENCES

1. Calculation S09-0050, "Steam Generator Access Opening – Containment Wall Effective Width".

2. Engineering Change Package EC-63016, "Containment Opening for Steam Generator Replacement."

CR3-Unit 3

Reactor Building Shell Calc's

4203-00-212

Design Review

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Material Design

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Florida Power Corporation
Crystal River - Unit No. 3
Reactor Bldg. - Outside Shell
04-4203-000-00212

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APPENDIX 5J

REPORT ON
RECOMMENDED CONCRETE CREEP AND SHRINKAGE VALUES
FOR COMPUTING PRESTRESSING LOSSES

UP TO DATE

12/15/75

Prepared for: Jersey Central Power & Light Co.
Parsippany, New Jersey

Prepared by: Schupack and Associates
Consulting Engineers
Stamford, Connecticut

Dated: June 1968

Project No. (67-535)

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1. SUMMARY

Grouted tendons have been selected for the containment structure at Oyster Creek Nuclear Station - Unit 2. The purpose of this report is to estimate prestress losses for these tendons due to concrete shrinkage and creep over a 50-year period at an average temperature of 85°F ± 10%.

Deformational behavior of controlled concrete similar to this structure is shown to be predictable on the basis of test data on basic creep and shrinkage, and correlation with observations on actual full size beams. For the volume/surface ratio of the concrete in this structure, basic creep is the dominating deformational component and depends primarily on the concrete's specific strength and age at loading. Shrinkage strain is shown to be approximately equal to the lower limit of possible accuracy in estimating total long-term losses.

A uni-axial creep strain of 0.40×10^{-6} inch/inch per psi, and a final shrinkage of 0.0001 inch/inch are recommended for the concrete specified for this structure.

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2. INTRODUCTION

The force initially imparted to prestressing tendons decreases — asymptotically with time due to:

A. Shrinkage of concrete

B. Creep of concrete

C. Relaxation of prestressing steel

Grouted tendons have been selected for the containment structure in the Jersey Central Power and Light Company project. This requires a conservative evaluation of the total losses in prestressing force over a 50-year period in order to ensure the structure's satisfactory function during its entire service life.

3. PURPOSE OF REPORT

It is the object of this report to recommend a conservative estimate of probable shrinkage and creep of concrete in this structure. The evaluation applies specifically to:

A. Steel inside-lined dome and walls with concrete 36" - 42" thick.

B. Concrete with approximately 5,000 psi strength and of normal workability (i.e., six to seven sacks cement per cubic yard, approximately 1" to 4" slump and such normal aggregate that the concrete's modulus of elasticity is approximately equal to the ACI-formula).

C. Average ambient temperature $85^{\circ}F \pm 10\%$

D. Bi-axial compression less than 50% of f'_c

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4. NATURE OF SHRINKAGE AND CREEP

4.1 Shrinkage

Shrinkage is the deformation of concrete due to other physical or chemical causes than applied load and temperature dependent volumetric changes. (i.e., variation in moisture content and hydration of cement). Shrinkage strain is time dependent and influenced by the following factors:

A. Internal properties of the concrete, as affected by:

1. Cement quality
2. Water cement ratio
3. Proportion cement-aggregate
4. Aggregate properties (expansive or permeable aggregate)
5. Admixtures
6. Degree of compaction
7. Reinforcement percentage
8. Curing methods

B. Ambient temperature

C. Ambient humidity as affected by:

1. Size and shape (volume to surface ratio) of element
2. Environment

4.2 Creep

Creep is the time dependent deformation of concrete due to load. Creep strain at any particular time is defined as the difference between total strain and the sum of initial elastic strain, shrinkage strain and strains due to temperature dependent volumetric changes at that time. Creep strain is influenced by the following factors:

A. Internal properties of the concrete (same as for shrinkage, except A-7 above, Reinforcement percentage, that arbitrarily can be considered as an internal property or an external load factor).

B. Ambient temperature (same as for shrinkage)

C. Ambient humidity (same as for shrinkage)

D. External load

1. Time of loading

2. Duration of loading

3. Uni-or bi-axial loading

4.2.1 Evaluation of Creep

For practical evaluation and prediction, creep can be separated into two components:

A. Drying creep results from stress-induced moisture diffusion and consequential shrinkage or swelling during the period of sustained loading. For given stress conditions, drying creep is primarily a function of the humidity factors (C) mentioned above.

B. Basic creep is the creep of concrete not exposed to moisture diffusion. For any given concrete mix, it is a characteristic, predictable function of load and temperature alone [2].

The complexity of variables influencing shrinkage and creep must be viewed realistically. The property-factors, for example, are substantially the same as determine the concrete's strength and workability. For the structure in question, therefore, this set of parameters can be substituted with values typical for the specified concrete.

Similarly, the humidity factors can be limited to a single parameter for size and shape by assuming typical average environmental conditions.

A comprehensive investigation in the range 0 - 95°C (32 - 203°F) of temperature effects on creep and shrinkage by D. J. Hannant [1A], indicates that creep increases approximately linearly with temperature [2]. Applied to this structure, with an average temperature of 85°F ± 10% (110°F inside, 0 to 100°F outside), the influence of the temperature range on creep is negligible. Temperature variations within this range are without measurable influence on the final shrinkage, and can also be neglected.[1B]

The influence of mild reinforcement is to increase the elastic resistance of the composite section and reduce creep and shrinkage effects proportionately. These recommendations will concern the concrete section alone, and give conservative values for the final shrinkage and creep.

Based on the above, a realistic estimate of creep and shrinkage for this structure can be determined by:

A. An evaluation of shrinkage and drying creep for the one significant humidity parameter - the actual volume/surface ratio.

B. A prediction of basic creep for the specified concrete, the given loads, and the concrete age at loading.

5. REVIEW OF LITERATURE AND TEST DATA

5.1 Shrinkage and Creep Research in General

The American Concrete Institute's publication SP-9 [3] presents a summary of the research done on creep up to 1964. A corresponding review of research on shrinkage has been prepared by the California Producers Committee [7] in 1966. Papers presented at the Conference on Prestressed Concrete Pressure Vessels in London in 1967 [1] cover research on the special creep and shrinkage problems arising in prestressed concrete pressure vessels resembling the one in question.

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Most investigations concern effects of a single, or a few, of the factors influencing shrinkage and creep and supply excellent information on the nature of these phenomena, especially for basic creep. The small scale models and laboratory specimens tested, and the comparatively short period of observation (maximum about 4 years), is, however, not representative of 50 years performance of the large concrete structural dimensions in question.

Reported observations on actual structures show the significant variation of combined shrinkage and creep effects with size and shape of member, but suffer from the impossibility of separating shrinkage and creep strain without having unloaded specimens for comparison.

The data reported on a few long-term full scale tests including unloaded companion specimens for shrinkage measurements (*), is insufficient in quantity and homogeneity to support conclusions on shrinkage and creep for this specific structure and extrapolation over 50 years.

All test data can, however, be utilized for this estimate by separating the deformational behavior of the concrete into a primary component - basic creep, and a modifying component - drying creep and shrinkage. The investigations of size and shape influence quoted below show that the volume/surface ratio of this structure completely governs shrinkage in the possible range of prediction, and that drying creep can be neglected. Similarly, the research listed on basic creep shows that it dominates the concrete's deformational behavior on this structure. General test data applies directly to basic creep, and available information in correlation with reported observations on full scale tests is sufficient to warrant regression and extrapolation of this structure's estimated basic creep behavior during its entire service life.

(*) Notably, the AASHO Road Test, Report 4,"Bridge Research" and Technical Report R212 from U. S. Naval Laboratories, Port Hueneme, California

5.2 Research and Test Data on Influence of Size and Shape

A. H. Mattock has investigated shrinkage and creep of precast concrete bridges [4], and Mattock and Torben C. Hansen together [5] have reported data from the first four years of an investigation in progress regarding the influence of size and shape of members on the shrinkage and creep of concrete. Based on the accumulated data and G. Pickett's theory of moisture diffusion [6], Hansen and Mattock have established a relationship between shrinkage and creep, and the volume to surface ratio ($^V/_S$ inch) of a concrete member.

5.2.1 Shrinkage

The following graph shows the variation of final shrinkage with volume-surface ratio for Hansen and Mattock's main tests (Elgin gravel) and an auxiliary test on concrete made from an unusually porous crushed sandstone aggregate:

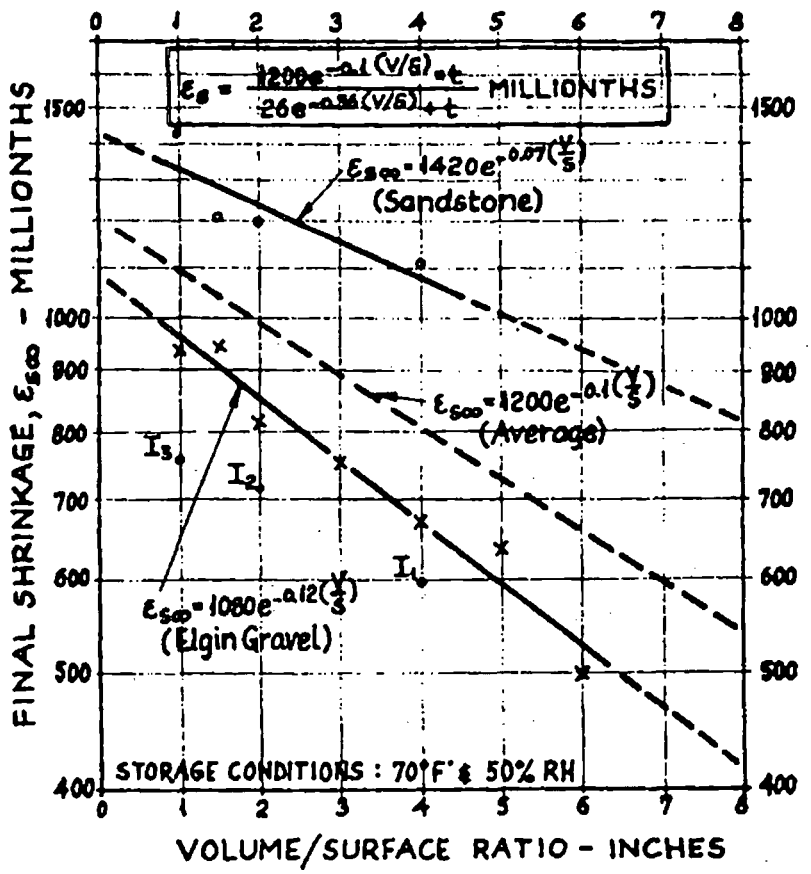

FIG. 1. VARIATION OF FINAL SHRINKAGE WITH VOLUME/SURFACE RATIO

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The plotted average represents a conservative prediction on the concrete specified for the structure in question, and the shrinkage at time "t" may be predicted from the given formula. Based on the underlying general theory of moisture diffusion, extrapolation of this curve is justified up to volume/surface ratios of approximately 24". Further extrapolation results in values outside the scope of accuracy possible in estimating total losses. For a volume/surface ratio of 24", the final shrinkage will be

$$\varepsilon_{s\infty} = 1200 \times e^{-0.10 \times 24} \times 10^{-6} = 0.0001 \text{ inch/inch}$$

All concrete in the present structure has a volume/surface ratio greater than 24", and the above value (0.0001) can be used as a conservative estimate of the final shrinkage strain.

5.2.2 Creep

Hansen and Mattock have established the following relationship between the coefficient for total (= basic + drying) creep ϕ, the coefficient of creep for sealed specimens (= basic creep) ϕ_s, and the volume/surface ratio V/S, valid for concrete similar to the structure in question:

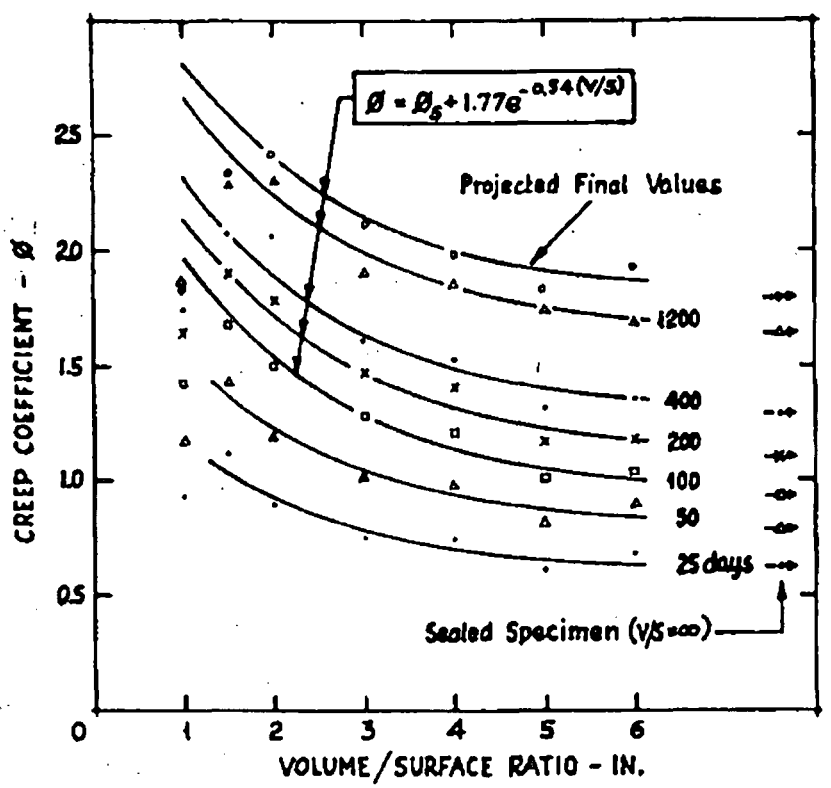

FIG. 2. VARIATION OF CREEP COEFFICIENT WITH VOLUME/SURFACE RATIO AT DIFFERENT AGES. (ELGIN GRAVEL AGGREGATE CONCRETE CYLINDERS)

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Based on the above, and assuming a coefficient of basic creep $\phi_s = 1.5$ (*), the drying creep component for a volume/surface ratio ≥ 6 is found to be:

$$\frac{\Delta\phi}{\phi_s} \leq \frac{1.77}{1.5} \times e^{-.54 \times 6} = 0.046$$

i.e., drying creep is less than 5% of basic creep for volume/surface ratios larger than 6, and can be neglected.

As all volume/surface ratios in the present structure are greater than 6, the size and shape of this structure makes basic creep the only significant creep component.

5.2.3 Research on Basic Creep

A summary of research on basic creep has been published by Torben C. Hansen (2) tabulating all work done up to 1960. Based on Powers and Brownyard's (8), and A. Hummel's (9) investigations of the properties of hardened Portland cement, Hansen has fit the accumulated data to Reiner's general rheological creep equation (10) and established a practical relationship between specific basic creep (i.e., basic creep per unit stress) and time. For given concrete properties the only parameter is concrete age at loading.

This fundamental rheological creep equation has been adapted to the concrete specified for the present structure, and curves for the estimated basic creep have been plotted on a graph, Appendix C, for loading ages 8 and 22 days, 1, 2, 3, 4-1/2 and 6 months. The curves are conservatively designed to over-estimate the probable basic creep by approximately 10%. For the specific conditions of this structure, the percentage deviation on the estimated creep values is expected to be approximately 14%.

Derivation of the basic creep curves and an evaluation of the accuracy of the estimate are presented in Appendix A.

(*) See Basic Creep graph (Appendix C)

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5.2.4 Effects of Bi-axial Stressing

Based on Arthanari and Yu's investigations (11) and the data reported to the London Conference on Prestressed Concrete Pressure Vessels by Browne (1B), Barrett and Murray (1C) and D. J. Hannant (1A), Poisson's ratio for creep under sustained multi-axial load at temperatures 85°F ± 10% can be considered constant and equal to 1/6. The estimated uni-axial creep strains must be evaluated separately for each member of the present structure as the creep effect due to multi-axial loading varies with the different ratios between specific uni-axial loads, and with the degree of restraint due to reinforcement.

6. CORRELATION WITH FULL SCALE TESTS

The estimated shrinkage and creep for the present structure has been checked against data from full scale tests on 22 prestressed I-section beams conducted by the U. S. Naval Engineering Laboratory, Port Hueneme, California. These 42-foot beams had a volume/surface ratio of 2.13" and were constructed with three proprietary systems of prestressing and designed to support a uniform load of 760 lbs. per linear foot. They were tested under different loading conditions and observations made of concrete strains over periods of 6 to 7 years. Eleven additional 21-foot non-loaded specimens were made for shrinkage measurements.

Evaluation of observations relative to the recommended shrinkage and creep strains is presented in Appendix B, and for comparison, creep curves for loading ages of 8 and 22 days derived from these observations are plotted on the graph of estimated basic creep (Appendix C).

For creep, these modified observations (apart from a minor early age discrepancy on the 8-day curve) clearly fall within the recommended values, especially for extrapolated later age conditions.

For shrinkage, the final average value observed is half of the value predicted for a volume/surface ratio of 2.13" by the recommended conservative formula (observed 0.0005, calculated 0.001 inch/inch).

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7. CONCLUSIONS

Shrinkage and creep are complex phenomena affected by a variety of factors reflecting the internal properties of the concrete, ambient temperature and humidity conditions, and the load pattern involved. Attempts to estimate long-term deformation of structures with small volume/surface ratios (say less than 2") result in a broad band of predictions. This is due to the difficulty of evaluating creep behavior when the volume/surface ratios are so small that the drying creep component predominates.

For the temperature range (85°F ± 10%) and volume/surface ratio (larger than 24") of this structure, and the concrete properties and load conditions specified, it is, however, possible to obtain a conservative estimate of long-term deformations. This is done by separating basic creep strains from those due to drying creep and shrinkage.

Basic creep is by far the dominating deformational component and can be predicted with an accuracy of 10 - 15% based on the specified concrete strength and age at load application.

Drying creep is concluded to be negligible for the volume/surface ratio of the concrete in this structure.

Shrinkage is governed by the volume/surface ratio and is conservatively estimated to be at the lower limit of possible accuracy in predictions of total long-term losses.

For bi-axial stress conditions at the considered temperature, Poisson's ratio for creep can be assumed constant throughout the entire evaluation period. Multi-axial creep effects must, however, be evaluated separately for varying ratios of bi-axial stresses.

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8. RECOMMENDATIONS

The following losses due to shrinkage and creep are a reasonably conservative estimate of the values to be expected after 50 years sustained tendon loads at a temperature level of 85°F ± 10% on the Jersey Central Power and Light Company's Prestressed Concrete Containment Structure at Oyster Creek Nuclear Station - Unit 2:

Shrinkage Strain After 50 Years:..... 0.0001 inch/inch

Uni-axial Creep Strain After 50 Years:

Concrete Age at Prestressing Months	Specific Creep Strain inch/inch per psi	Creep Coefficient ϕ_c (E_c 4 x 10^6 psi)
1	$.45 \times 10^{-6}$	1.8
2	$.40 \times 10^{-6}$	1.6
3	$.37 \times 10^{-6}$	1.5
6	$.33 \times 10^{-6}$	1.3

Modification of Uni-axial Creep Strains for Bi-axial Stresses:

Poisson's ratio for creep can be considered constant over 50 years sustained tendon load at temperatures averaging 85°F ± 10%. The recommended value is 1/6.

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REFERENCES

1. Conference on Prestressed Concrete Pressure Vessels
Institution of Civil Engineers Papers #13 - #18 Group C London 1967

A. D. J. Hannant:
"Strain Behavior of Concrete up to 95° Under Compressive Stresses" Paper #17 Group C

B. R. D. Browne:
"Properties of Concrete in Reactor Vessels"
Paper #13 Group C

C. N. T. Barrett and J. T. Murray:
"Creep in Pressure Vessels at Elevated Temperatures"
Paper #14 Group C

2. Torben C. Hansen: "Creep and Stress Relaxation of Concrete"
Cement and Concrete Research Institute Proceedings #31 Stockholm 1960

3. "Symposium on Creep of Concrete", ACI Publication SP-9 1964

4. A. H. Mattock: "Creep and Shrinkage Studies, Precast-Prestressed Concrete Bridges 5"
Journal of the PCA Research and Development Laboratories
Vol. 3, #2, 32 - 66 1961

5. Torben C. Hansen and A. H. Mattock: "Influence of Size and Shape of Member on the Shrinkage and Creep of Concrete"
ACI Journal Proceedings Vol. 63, p 267 February 1966

6. California's Producers Committee on Volume Change,
"Drying Shrinkage of Concrete" 1966

7. Gerald Pickett: "Shrinkage Stresses in Concrete"
PCA Research Department Bulletin #11 1946

"The Effects of Change in Moisture Content on the Creep of Concrete Under a Sustained Load"
PCA Research Department Bulletin #20 1947

"Effects of Aggregate on Shrinkage of Concrete and Hypothesis Concerning Shrinkage"
PCA Research Department Bulletin #66 1956

8. T. C. Powers and T. L.Brownyard: "Studies of the Physical Properties of Hardened Portland Cement Paste"
PCA Research Department Bulletin #22 March 1948

9. A. Hummel: "Das Beton ABC", 5th edition 1942 ?

10. M. Reiner: Chapter on Rheology, "Elasticity and Plasticity"
Encyclopedia of Physics, Vol. VI (Springer Verlag, Berlin) 1958

11. Arthanari and C. W. Yu: "Creep of Concrete Under Uni- and Bi-axial Stress at Elevated Temperatures"
Magazine of Concrete Research #60 1967

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SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX A Basic creep	SHEET NO. A1 OF BY CS DATE 2.5.68 CHK'D DATE

Torben C. Hansen's formulation of Reiner's general rheological creep equation is:

$$\frac{\varepsilon_{creep}}{\sigma} = \beta \frac{\left(0.31 \cdot g(t_0) + \frac{W_0}{C}\right) \cdot V_1}{(N \cdot k_1 + 0.31)\, g(t_0)} \left(1 - e^{-m(t_1 - t_0)}\right) + \alpha_1 \cdot \frac{W_0}{C} \cdot V_1 \cdot \ln\left(\frac{t_1}{t_0}\right)$$

spec. creep — delayed elasticity — viscosity

t_1 = age of concrete at time t_0, in days
t_0 = age of concrete when loaded, in days
$\frac{W_0}{C}$ = water-cement ratio by weight, corrected for bleeding
V_1 = volume concentration of cement paste in concrete
$g(t_0)$ = degree of hydration of cement at time t_0
k_1 = weight ratio of nonevaporable water to cement when all cement is hydrated. k_1 is a function of the proportions of cement components:
$k_1 = 0.187(C_3S) + 0.158(C_2S) + 0.655(C_3A) + 0.213(C_4AF)$
N = $0.75(1+4k)$, where
$k = 0.230(C_3S) - 0.320(C_2S) + 0.317(C_3A) + 0.368(C_4AF)$

α_1, β and m are coefficients to be determined by experiment.

Hansen has fit the results of 42 test series to the equation above to determine α_1, β and m. Powers & Brownyard's values $k = .25$ and $k_1 = .23$ have been used where the cement composition was unknown, as have Hummel's values for degree of hydration with time for standard and rapid hardening Portland cement. Hansen found:

$\beta = 1.76 \times 10^{-6}$, $m = 0.0333$, $\alpha_1 = 5.7 \times 10^{-6}$ (units: specific creep/kg/cm²)

The general equation for basic creep becomes: (units: sp. creep pr psi)

$$\frac{\varepsilon_{creep}}{\sigma} \times 10^6 = 0.189 \cdot \frac{\left(0.31 \cdot g(t_0) + \frac{W_0}{C}\right) \cdot V_1}{g(t_0)} \left(1 - e^{-\frac{t_1 - t_0}{30}}\right) + 0.401 \cdot \frac{W_0}{C} \cdot V_1 \cdot \ln\left(\frac{t_1}{t_0}\right) \quad [R]$$

The average percentage deviation between measured and calculated creep values was 14%, largest (16%) for 50-100 days sustained load, smallest (12%) for 900 days loading. For long term estimates, where the effect of poorly defined early age conditions is minimalized, the accuracy of the equation is conservatively set at ±10%

1.01.7/41

SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX A Basic creep	SHEET NO. A2 OF BY CS DATE 7.5.68 CHK'D. DATE

Values of $r = \frac{W_1}{C}$ and V_1 to be used for the specified concrete in this estimate are as follows:

$r = 0.45, \left(\frac{\Delta r}{r}\right) = 7.5\%$; $V_1 = 0.29, \frac{\Delta V_1}{V_1} = 7.5\%$; $\frac{\Delta g}{g} \sim 10\%$;

$$\frac{\Delta R}{R}(r) = \frac{\frac{dR}{R}}{\frac{dr}{r}}\left(\frac{\Delta r}{r}\right) = \frac{\left(\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g(t_0)} + .4\ln\frac{t_1}{t_0}\right) \times dr}{\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g(t_0)}(.31g(t_0)+r) + .4r\ln\frac{t_1}{t_0}} \times \frac{r}{dr}\left(\frac{\Delta r}{r}\right) = .95 \times \left(\frac{\Delta r}{r}\right) = .95 \times 7.5\%$$

$$\frac{\Delta R}{R}(V_1) = \frac{\frac{dR}{R}}{\frac{dV_1}{V_1}}\left(\frac{\Delta V_1}{V_1}\right) = \frac{dV_1}{V_1} \times \frac{V_1}{dV_1} \times \left(\frac{\Delta V_1}{V_1}\right) \text{ - - - - - - - - - - } = 1.0 \times \left(\frac{\Delta V_1}{V_1}\right) = 1.0 \times 7.5\%$$

$$\frac{\Delta R}{R}(g) = \frac{\frac{dR}{R}}{\frac{dg}{g}}\left(\frac{\Delta g}{g}\right) = \frac{\left(\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g^2} \times r\right) \times dg}{\frac{.189(1-e^{-\frac{t_1-t_0}{30}})}{g}(.31g+r) + .4r\ln\frac{t_1}{t_0}} \times \frac{g}{dg} \times \left(\frac{\Delta g}{g}\right) \cong .09 \times \left(\frac{\Delta g}{g}\right) \approx .10 \times 10\%$$

[computed for: $t_0 \approx 60$ days, $t_1 \approx 18000$ days, $g(t_0) \approx 0.85$, $r \approx 0.45$]

To be on the conservative side, the basic creep curves have been plotted for $r = 0.50$ and $V_1 = 0.30$. According to the above, estimated creep values will be found about $.95 \times 10 + 1.0 \times 3 \approx 13\%$ to the safe side of the probable values, with an average deviation of approximately: $\sqrt{10^2 + 7.5^2 + 7.0^2} \approx 14\%$ ($\frac{\Delta g}{g}$ is already included in the 10%)

As $13\% \approx 14\%$, the curves will represent the upper limit of the confidence interval for equation [R]. The probable overestimation of creep values will be $\sim 2/3 \times 13 \approx 10\%$.

Curves have been plotted for $t_0 = 8, 22, 30, 60, 90, 135$ & 180 days using the corresponding values for $g(t_0)$ taken from Hummel's diagram and $r = 0.50$, $V_1 = 0.30$. The equation is: ?

$$\boxed{\frac{\varepsilon_{creep}}{\sigma} \times 10^n = \frac{0.18\,g(t_0) + .28}{g(t_0)}\left(1 - e^{-\frac{t_1-t_0}{30}}\right) + 0.6\ln\left(\frac{t_1}{t_0}\right)} \qquad [S]$$

After about the first 90 days of loading, delayed elasticity has lost its effect on rate of creep and the viscous properties govern.

1.017/42

SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX B Port Hueneme Test Data	SHEET NO. B1 OF BY CS DATE 20.6.68 CHK'D H.A. DATE 24.6.68

Shrinkage.

The maximal average shrinkage on 11 beams after 2200 days exposure was measured on bottom fibres at 445 μinch/inch ~ 0.0005.

Volume/surface ratio for these specimens was 2.13. Computed by the recommended formula, shrinkage after 2200 days being practically the same as final shrinkage, it is found:

$$\varepsilon_{s,2200} \approx \varepsilon_{s,\infty} = 1200 \cdot e^{-0.2/3} = 970\ \mu\text{inch/inch} \sim 0.0010$$

Comparison of these values shows that the computed estimate is conservative, and that large percentual deviations may be expected on the prediction of small shrinkage values.

Creep.

The recorded creep observations include data on conditions governed by creep recovery, cracked section and tensile stresses, and stress patterns at so low a level as not to be comparable to the creep conditions of the structure in question. The following two cases are considered representative and have been investigated for this comparison:

A. Creep strains at center of gravity of steel (cgs) on 6 beams Load: DL + 0% LL, t_0 = 8 days, E_c(8 days) = 3.36×10^6 psi, σ_c = 1330 psi;

B Differential creep at top fibre, 2×6 beams, Load: (DL + 100% LL) − DL, t_0 = 22 days, E_c(22 days) = 3.36×10^6 psi, $\Delta\sigma$ = 1880 − 50 = 1830 psi;

The reported observations have been averaged for 6 (or 2×6) beams and tabulated below. The following conservative approximations have been made in evaluating the basic creep component from the test data:

1. Mean values of average shrinkage have been used for shrinkage reduction.
2. No reductions have been made for plastic set.
3. The observed values (at cgs) of non linearly distributed overall creep effect (partial load removal at cgs (DL + DL + LL)) have been referred to cgs by being taken into account with 2/3 of the actual values.
4. Volume/surface ratio is 1.55" for top flanges. 1.85" has been used.

Harisen & Mattock's formular determines basic creep from specific creep:

$$\phi_{(v/s)} = \phi_{basic} + 1.77 e^{-0.54(v/s)}; \quad \phi_{basic} = \phi_{2.13} - 0.56 = \phi_{1.85} - 0.65;$$

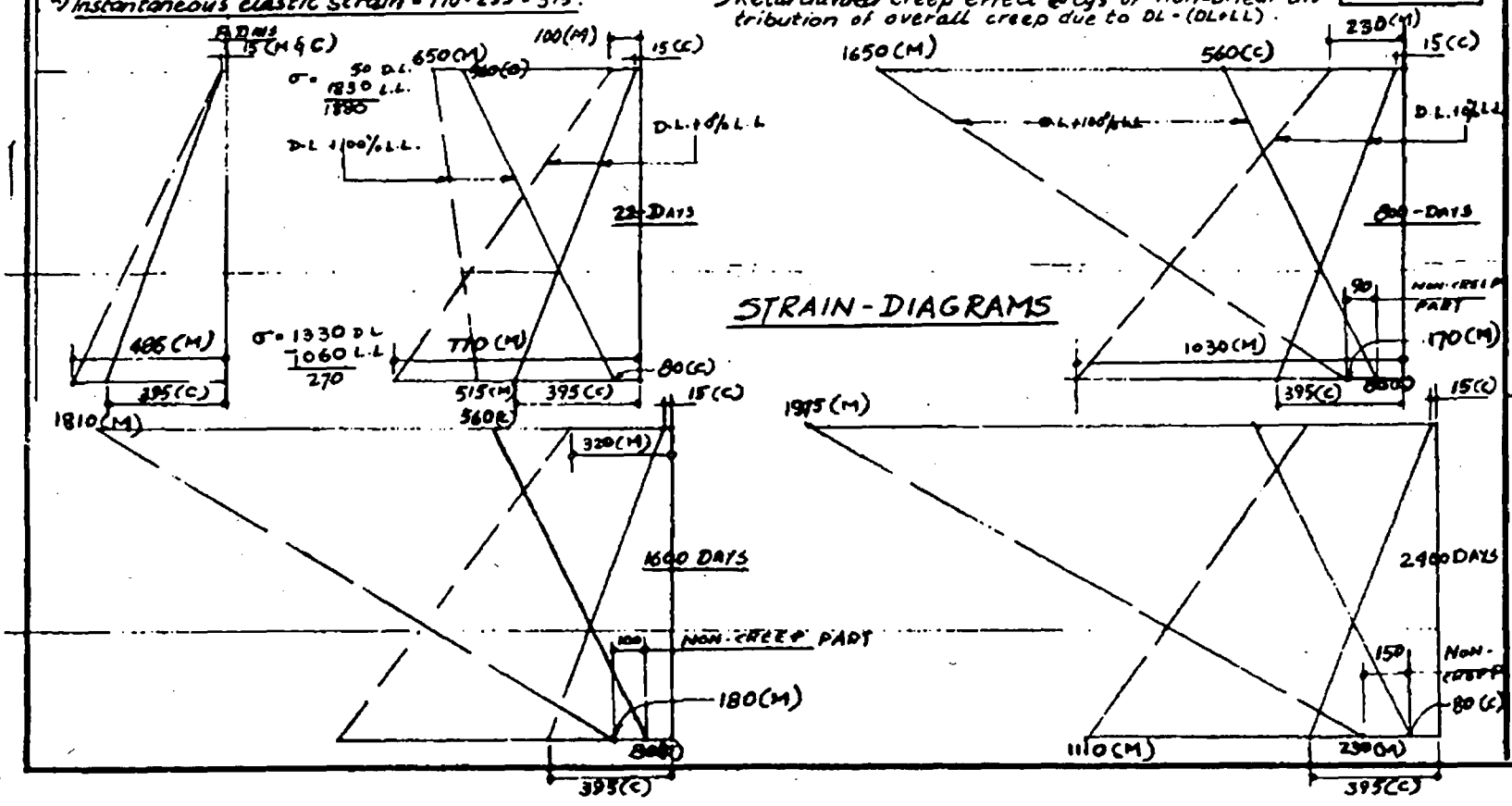
Specific basic creep curves as determined are plotted, see graph, appendix C.

TABULATION OF TEST DATA @ CGS. (DL + 100% LL)

	8 d.	(8-22)	22 days		(22-800)	800 days	(800-1600)	1600 days	(1600-2400)	2400 days	NOTES
	1. DL-M	2. DL-M	3. 1.+2.	4. LL-C	5. DL&LL-M	6. 3.+4.+5.	7. DL&LL-M	8. 6.+7.	9. DL&LL-M	10. 8.+9.	M = measured C = calculated strains in μinch/inch shrinkage values are deducted from M.
PRELOAD (GROUTED)	480	250	730	-270	-280	180	-250	210	-280	180	
PRELOAD (NON-GROUTED)	495	295	790	-240	-280	270	-310	240	-150	400	
ROEBLING (NON-GROUTED)	475	310	795	-250	-470	65	-450	95	-430	115	
AVERAGE VALUES	485	285	770	-255 *)	-345	170 **)		180 **)		230 **)	

*) Instantaneous elastic strain = 770 - 255 = 515.

**) Retarded creep effect @ cgs of non-linear distribution of overall creep due to DL - (DL+LL).



SCHUPACK AND ASSOCIATES
CONSULTING ENGINEERS
STAMFORD, CONN.

APPENDIX B
Port Hueneme Test Data

SHEET NO. B2 OF
BY N.A. DATE 24.6.68
CHK'D CS DATE 25.6.68

1.01.7/43

1.017144

SCHUPACK AND ASSOCIATES CONSULTING ENGINEERS STAMFORD, CONN.	APPENDIX B Port Hueneme Test Data	SHEET NO. 83 OF BY H.A. DATE 24.6.68 CHK'D. CS DATE 25.6.68

Total @ C.G.S., average of 6 beams, V/S = 2.13.

Conc. Age →	8 Days	22 Days	800 Days	1600 Days	2400 Days
$\varepsilon_{TOT} - \varepsilon_{SHRINK}$ $\times 10^6$	485	770	1030	1065	1110
$\varepsilon_{elast.}$ instant. (1300 p.s.i.) $\times 10^6$	395	395	395	395	395
ε_{creep} $\times 10^6$	0 90	375	635	670	715
$\phi_{2.13} = \varepsilon_c / \varepsilon_{elast.}$	0.229	0.948	1.605	1.695	1.810
$\phi_{2.13} - \phi_{BASIC}$	0.56	0.56	0.56	0.56	0.56
Equivalent ϕ_{BASIC}	—	0.338	1.045	1.135	1.250
$\varepsilon_{cr.}/p.s.i. = \phi_{BAS}/E_c$ $\times 10^6$	0	0.115	0.311	0.338	0.372

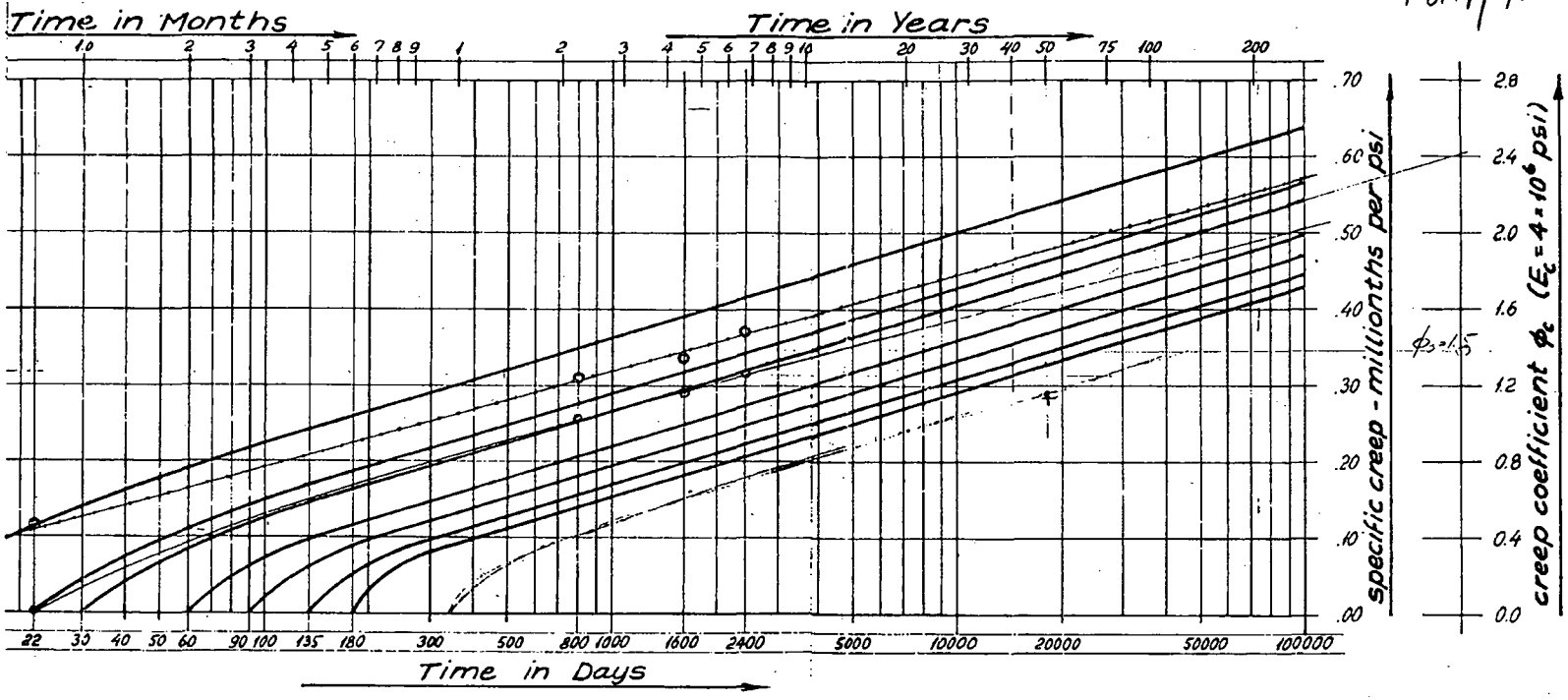
Differential creep @ top fibre, average 2x6 beams, V/S ≈ 1.85

Concrete Age →	22 Days	800 Days	1600 Days	2400 Days
$\varepsilon_{TOT.(D+L)} - \varepsilon_{SHRINK}$ $\times 10^6$	650	1650	1810	1975
$\varepsilon_{TOT.(D)} - \varepsilon_{SHRINK}$ $\times 10^6$	100	230	320	405
$\Delta\varepsilon_{TOTAL}$ (L) $\times 10^6$	550	1420	1490	1570
$\Delta\varepsilon_{elast.}$ (L) 1830 p.s.i. $\times 10^6$	545	545	545	545
$\Delta\varepsilon_{creep}$ (L) $\times 10^6$	5	875	945	1025
Correction for Age-Loading $\times 10^6$	-5	-5	-5	-5
$\Delta\varepsilon_{CREEP}$ (L)	0	870	940	1020
*Other Non-$\varepsilon_{cr(BASIC)}$ in $\Delta\varepsilon_{cr}$ values	—	60	65	100
$\Delta\varepsilon_{CREEP}$ (L) $\times 10^6$	0	810	875	920
$\phi_{1.85} = \Delta\varepsilon_c / \varepsilon_{elast.}$	0	1.490	1.610	1.690
$\phi_{1.85} - \phi_{BASIC}$	—	0.65	0.65	0.65
Equivalent ϕ_{BASIC}	—	0.84	0.96	1.04
$\Delta\varepsilon_c/p.s.i.$ $\times 10^6$		0.250	0.286	0.310

*Overall effect of non-creep part ~ 2/3 effect @ C.G.S. (see strain diag)

Basic Creep for 5000 psi Concrete

1.01.7/45.



Total creep @ cgs ....., Port Hueneme Tests[1]; age @ loading: 8 days ;

Differential creep, top fibre, —"— —"— —"— [1]; —"— @ —"— : 22 —"— ;

Estimated basic creep[2] for loading ages: 8 & 22 days and 1, 2, 3, 4½ & 6 months

SCHUPACK & ASSOCIATES
CONSULTING ENGINEERS - STAMFORD, CONN.
Job: Oyster Creek Unit 2
By: C.S. Date: May 10, 1968

1.01.7/46

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.	
	CHK'D. T.K.		
	SQ. CP.		
REACTOR BUILDING	CP. DPN.	4203-00	23
SCHUPACK & ASSOCIATES REPORT	ENG.	WORK ORDER / SIZE / DRAWING	REV.
ON CREEP AND SHRINKAGE VALUES	REV. CH. APP. DATE	4-24-69	

SUMMARY OF THE "REPORT ON RECOMMENDED CONCRETE CREEP AND SHRINKAGE VALUES FOR COMPUTING PRESTRESSING LOSSES" BY SCHUPACK AND ASSOCIATES, CONSULTING ENGINEERS STAMFORD, CONNECTICUT DATED JUNE 1968

JOB: OYSTER CREEK NUCLEAR STATION-UNIT 2

TYPE TENDONS : GROUTED TENDONS

CONCRETE SHRINKAGE AND CREEP OVER A 50 YEAR PERIOD AT AN AVERAGE TEMPERATURE OF 85°F ± 10°F TO ESTIMATE PRESTRESS LOSSES

SHRINKAGE STRAIN = 0.0001 INCH/INCH

$= 100 \times 10^{-6}$

A UNI-AXIAL CREEP STRAIN $= 0.4 \times 10^{-6}$ /PSI

WALL

STRESS IN MERIDIONAL DIRECTION = -783.8 PSI

STRESS IN HOOP DIRECTION = -1465.0 PSI

NET CREEP STRAIN IN THE MERIDIONAL DIRECTION

$$= 0.4 \times 10^{-6}\left(-783.8 + \frac{1}{6} \times 1465\right)$$

$$= 0.4 \times 10^{-6}(-783.8 + 244.2)$$

$$= 0.4 \times 10^{-6}(-539.6)$$

$$= \underline{216 \times 10^{-6}}$$

NET CREEP STRAIN IN THE HOOP DIRECTION

$$= 0.4 \times 10^{-6}\left(-1465 + \frac{1}{6} \times 783.8\right)$$

$$= \underline{534 \times 10^{-6}}$$

1.0.1.7/47

CRYSTAL RIVER	MADE RS	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.
REACTOR BUILDING	CHK'D.	
SCHUPACK & ASSOCIATES REPORT	SQ. CP.	
CONCREEP AND SHRINKAGE VALUES	CP. DPN.	4203-00-212 / 24
	ENG.	WORK ORDER / SIZE / DRAWING / REV.
	REV. CH. APP. DATE	4-24-69

DOME

STRESS IN THE MERIDIONAL DIRECTION = 1648 PSI

STRESS IN THE HOOP DIRECTION = 1579 PSI

NET CREEP STRAIN IN THE MERIDIONAL DIRECTION

$$= 0.4 \times 10^{-6} (-1648 + \frac{1}{6} \times 1579)$$

$$= 0.4 \times 10^{-6} (-1648 + 263)$$

$$= 0.4 \times 10^{-6} \times 1385$$

$$= 554 \times 10^{-6}$$

NET CREEP STRAIN IN THE HOOP DIRECTION

$$= 0.4 \times 10^{-6} (-1579 + \frac{1}{6} \times 1648)$$

$$= 0.4 \times 10^{-6} (-1579 + 275)$$

$$= 0.4 \times 10^{-6} (-1304)$$

$$= 522 \times 10^{-6}.$$

CREEP LOCATION	GAI SPEC.	SCHUPACK RECOMM.	SCHUPACK RECOMM.*
VERTICAL	200×10^{-6}	216×10^{-6}	162×10^{-6}
HOOP	300×10^{-6}	534×10^{-6}	401×10^{-6}
DOME	350×10^{-6}	554×10^{-6}	416×10^{-6}

TYPE OF STRAIN	GAI SPEC.	SCHUPACK RECOMM.
SHRINKAGE STRAIN	120×10^{-6}	100×10^{-6}

* BASED ON UNI-AXIAL CREEP STRAIN 0.3×10^{-6} INCH/INCH PER PSI (VALUE OBTAINED BASED ON THE ASSUMPTION CONCRETE AGE AT POST TENSIONING IS 1 YEAR)

1.01.7/48

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.			
	CHK'D. F.L.				
REACTOR BUILDING	BD. CF.				
	CF. DFN.	4203-00212			25
REVISED PRESTRESS REQUIREMENTS	ENG.	WORK ORDER	SIZE	DRAWING	REV.
	REV. CH. APP. DATE	4-25-69			

WALL

VERTICAL DIRECTION

TOTAL FORCE REQD = 175,200

NO OF TENDONS USED = 144

FORCE / TENDON = 1216 KIPS

AREA OF STEEL REQD = 1216 / (148.0 (153.6) × 0.95) = 8.337 □"

AREA OF EACH WIRE = 0.0597 □"

∴ NO OF WIRES REQD = 8.337 / 0.0597 = 139.6 WIRES

= 140 WIRES (SAY)

HORIZONTAL DIRECTION

145

FORCE REQD / FT = 824.9 KIPS/FT.

SPACING OF TENDON = 19½"

LET F' FORCE REQD / TENDON

F' / 824.9 × 12 = 19.5

F = 19.5 × 824.9 / 12

= 19.5 × 68.74

= 1341 KIPS

AREA OF STEEL REQD = 1341 / (0.95 × 149.5)

= 9.441 □"

NO OF WIRES REQD = 9.441 / 0.0597

= 158.1 = 159 (SAY)

1.01.7/49

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.		
	CHK'D. W.J.L.			
REACTOR BUILDING	SQ. CF.			
DOME PRESTRESSING	CF. DFN.	4203.00		-26-
	ENG.	WORK ORDER	SIZE DRAWING	REV.
	REV. CH. APP. DATE	7-18-69		

TENDON DESCRIPTION

STABILIZED WIRE

163 - 0.276" ϕ WIRE ; $A_s = 9.73$ □"

f'_s = ULTIMATE STRENGTH OF PRESTRESSING STEEL = 240 KSI

LOSSES

AVERAGE AXIAL COMPRESSIVE STRESS IN DOME = 1.53 KSI (TAKEN FROM LEININGER CALCULATION PAGE#TM1-DT-68)

ELASTIC SHORTENING $= \frac{1.53}{2} \times \frac{29}{4}$

$= 5.55$ KSI

CREEP $= .350 \times 10^{-6} \times 29 \times 10^{6}$

$= 10.15$ KSI

SHRINKAGE $= .12 \times 10^{-6} \times 29 \times 10^{6}$

$= 3.48$ KSI

4% RELAXATION LOSS BASED ON $0.6 f'_s$ $= 0.6 \times 240 \times 0.04$

$= 5.76$ KSI

TOTAL LOSSES $= 5.55 + 10.15 + 3.48 + 5.76$

$= 24.94$ KSI

STRESS IN WIRES AFTER LOSSES (UNDER NORMAL WORKING CONDITION)

$= 0.7 f'_s - 24.94$

$= 0.7 \times 240 - 24.94$

$= 168.0 - 24.94$

$= 143.06$ KSI

$0.6 f'_s = 0.6 \times 240 = 144.0$ KSI

FORCE IN EACH TENDON AFTER LOSSES (UNDER NORMAL WORKING CONDITION)

$= 143.06 \times 9.73$

$= 1392$ KIPS

1.01.7/50

CRYSTAL RIVER	MADE R.S.	GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PENNA.	
	CHK'D. W.J.L.		
REACTOR BUILDING	BD. CP.		
	CP. DFN.	4203-00	27
DOME PRESTRESSING	ENG.	WORK ORDER / SIZE / DRAWING	REV.
	REV. CK. APP. DATE	7-18-69	

UNDER ACCIDENT CONDITION, THE PRESTRESSING FORCE IS INCREASED DUE TO ELASTIC ELONGATION

STRESS BEFORE ACCIDENT = -1630 PSI D+F

STRESS AFTER ACCIDENT = 56 PSI D+F+1.5P+T$_a$

Δ STRESS = 1686 PSI

STRESS IN WIRE DUE TO ELASTIC ELONGATION

$= 1.686 \times \frac{29}{4}$

$= 12.22$ KSI $\leq T_p = T_e$

TOTAL STRESS IN WIRES DURING ACCIDENT

$= 143.06 + 12.22$

$= 155.28$ KSI ✓

ϕ FACTOR $= 0.95$

FINAL FORCE / TENDON $= 0.95 \times 155.28 \times 9.73$

$= 1435$ KIPS ✓

HORIZ. SPACING OF TENDON

(SEE LEININGER PAGE # TMI-DT69)

$= 22.7 \times \frac{1435}{1055}$

$= 30.87''$

$= 30''$ (SAY)

USE 123 TENDONS (41 TENDONS IN EACH OF 3 DIRECTIONS) WITH A CONSTANT HORIZONTAL SPACING OF 30".

℄ OF LAST TENDON IS AT A RADIUS 50'-0"

TANGENT POINT FOR ALL TENDONS IS AT A RADIUS OF 55'-3"

1.01.7/84 -1-

To: Mr. S.N. Dobreff.

FROM: R. SHANMUGASUNDARAM

SUBJECT: CRYSTAL RIVER - REACTOR BUILDING CHANGES REQUIRED BASED ON THE FINAL REVISED PRESTRESS CALCULATIONS. GAI-4203

BASED ON THE FINAL REVISED PRESTRESS CALCULATIONS (PLEASE SEE MY MEMO DATED APRIL 10 '69), THE REVISED FINAL REQUIREMENTS ARE COMPARED WITH THE INITIAL REQUIREMENTS WHICH THE PRESCON, THE PRESTRESS CONTRACTOR HAS FOLLOWED SO FAR INLATING OUT OF THE TENDONS.

PRESTRESS FORCE REQUIRED/FT

CYLINDRICAL WALL	AS SPECIFIED INITIAL REQUIREMENTS	FINAL REQUIREMENTS
VERTICAL	457.0 KIPS 11%	414.6 KIPS
HOOP	829.0 KIPS 5%	824.9 KIPS

(163 - 0.276" Φ STABILISED WIRES)

NO. OF TENDONS REQUIRED

CYLINDRICAL WALL	INITIAL REQUIREMENTS	FINAL REQUIREMENTS
VERTICAL	144 13%	128
HOOP	282 3.2%	273

1.01.7/55 -2-

NO OF WIRES REQUIRED IN EACH TENDON KEEPING THE NUMBER OF TENDONS SAME AS OF THE INITIAL REQUIREMENTS

CYLINDRICAL WALL	INITIAL REQUIREMENTS	FINAL REQUIREMENTS
VERTICAL	163	145
HOOP	163	159

ALL THE ABOVE CALCULATIONS ARE BASED ON THE STABILISED WIRES WITH 4% RELAXATION LOSSES AND SHRINKAGE AND CREEP LOSSES AS GIVEN IN GAI CONTRACT SPECIFICATION. INITIAL EFFECTIVE PRESTRESS IS LIMITED TO 0.6 OF THE ULTIMATE STRENGTH OF PRESTRESSING STEEL AND IS ALLOWED TO INCREASE BEYOND THE ABOVE LIMIT ONLY DURING 1.5 ACC. PRESSURE.

THE FOLLOWING IS THE COMPARISON BETWEEN THE GAI SPECIFICATIONS AND SCHUPACK & ASSOCIATES RECOMMENDATIONS ON LOSSES DUE TO CREEP AND SHRINKAGE

TYPE OF LOSSES	GAI SPEC.	SCHUPACK RECOMM.*
CREEP STRAIN		
VERTICAL	200×10^{-6}	162×10^{-6}
HOOP	300×10^{-6}	401×10^{-6}
DOME	350×10^{-6}	416×10^{-6}
SHRINKAGE STRAIN	120×10^{-6}	100×10^{-6}

1.01.7/56 -

* VALUE OBTAINED BASED ON THE ASSUMPTION CONCRETE AGE AT POST TENSIONING IS 1 YEAR.

INCASE OF DOME PRESTRESSING, THE FINAL SPACING HAS NOT BEEN WORKED OUT FOR CRYSTAL RIVER.

THEREFORE, THE FOLLOWING DECISIONS ARE TO BE MADE.

(1) LOSSES DUE TO, ELASTIC SHORTENING, CREEP AND SHRINKAGE FOR CRYSTAL RIVER CONCRETE.

(2) FINALISATION OF THE NO OF TENDONS IN THE WALL.

R. SHANMUGA SUNDARAM
STRUCTURAL ENGINEER.

C.C. D.K. CRONEBERGER
D.A. SKILTON.

GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS READING, PA.	DEPARTMENT NAME Structural	DEPT. NO. 0414	FILING CODE 1:01.7A/16
	PROJECT NAME Crystal River Unit 3	W.O. NUMBER 04-4203-115	PAGE R1(1-11)

SUBJECT: Tendon Surveillance - Creep rates

ORIGINATOR: D. Bliss

DATE: 10/25/77

VERIFIER:

DATE:

Check creep loss values for 1 year after SIT

Ref. Dome Repair Calc's Book II, section 1

at time of calculation (5/76) average age of concrete was ≅ 700 days

1 year after SIT ≅ 700 days + 1½ year
≅ 1250 days
≅ 3.41 year

from time dependent creep curve
$S_c = 0.17\ \mu''/psi$

for Vertical tendons

$\Delta f_{sv} = (0.17)(997\ psi)(29 \times 10^6\ psi) = 4.9\ ksi$ vs 4.7

for Hoop tendons

$\Delta f_{sH} = (0.17)(1776)(29 \times 10^6) = 8.8\ ksi$ vs 8.4

for Dome tendons

$\Delta f_{sD} = (0.17)(2873)(29 \times 10^6) = 14.2\ ksi$ vs 14.9

∴ values used in development of chart are as accurate as can be predicted with the given input data.

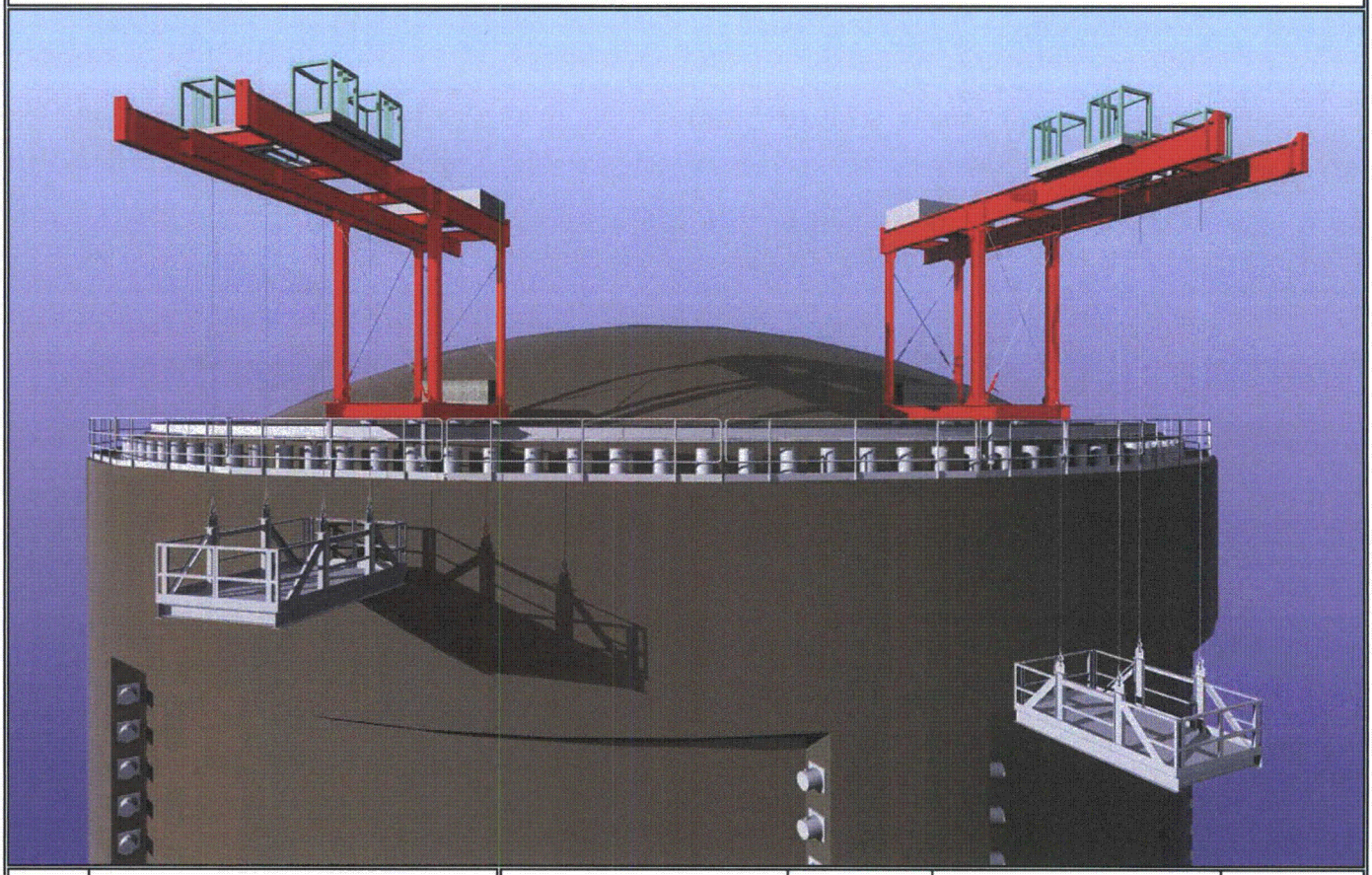
FILING CODE

DOCUMENT NUMBER:	CR-N1002-500	REVISION: 0	PAGE: i
DOCUMENT TITLE:	PRE-SURVEILLANCE ENGINEERING PACKAGE		
PROJECT TITLE:	30TH YEAR TENDON SURVEILLANCE AT CRYSTAL RIVER	DATE: 08/15/07	

DOCUMENT COVER SHEET

Document No: CR-1002-500

Title: PRE–SURVEILLANCE ENGINEERING PACKAGE



No.	Description	Prepared By	Date	Reviewed By	Date
0	Original Issue	*B.A. GIOMETTI*	*08/15/07*	*C.E. COX*	*8/15/07*

PSC SIGN OFF

REVISIONS

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ATTACHMENTS

ATTACHMENT A – ASME SECTION XI, 1992 EDITION WITH 1992 ADDENDA I-14
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ATTACHMENT C – CR-N991-100, "PREDICTED BASE FORCES" I - 142
ATTACHMENT D – CR-N1002-010, "SQ 11.1 RESTRESSING" I
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1.0 PURPOSE

1.1 SURVEILLANCE PURPOSE

1.1.1 The purpose of the Tendon Surveillance Program is to demonstrate the integrity of the containment post-tensioning system, including containment tendons, tendon end anchorage hardware; general and adjacent concrete integrity, and evaluation of the corrosion protective (grease) system. Individual inspections of selected tendons, as well as tendon wire and grease sample testing evaluate the overall integrity of the post-tensioning system.

1.2 DOCUMENT PURPOSE

1.2.1 The purpose of this document is to provide the engineering data and evaluations necessary to perform surveillance related activities. The scope of this document addresses:

1.2.1.1 The scope of work for the 8th period (30th Year) Tendon Surveillance

1.2.1.2 Predicted forces and normalization factors for selected scope tendons as well as the tendons adjacent to the ones selected. The predicted forces are used to accept as-found force levels if they are within the outlined acceptance criteria.

1.2.1.3 Tendon retensioning elongation tendon for all tendons scheduled to be detensioned for wire removal. The retension data is used to compare the elongation data obtained while restoring the tendon's force to the elongation that was observed while originally stressing the tendon in order to assure the force has been properly restored.

1.2.1.4 Grease void volumes for selected surveillance tendons and their respective adjacent tendons. The volumes allow IWL Level II Inspectors to document the percentage of the total void volume that is removed during inspection and replaced upon completion. This documentation is required per IWL-2526. If the absolute difference between the subject amounts exceeds 10% of the net duct volume, the Licensee shall report the conditions as required by 10CFR50.55a.

1.2.1.5 The design of the support frames and work platforms, which will be used to access hoop and dome tendons in order to assure their ability to withstand working and natural loadings.

1.2.1.5.1 The platform and the Upper Support Frame have been designed for both lateral and vertical loads by analyzing various loading conditions with RISA-3D Version 5.5. RISA-3D calculates both the force and stress in each individual member with respect to the member size designation. The entire system has been designed in accordance with AISC 9th Ed. ASD.

2.0 REFERENCES

2.1 REFERENCE DOCUMENTS

2.1.1 Crystal River Unit 3 Final Safety Analysis Report (FSAR), latest revision

2.1.2 American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section XI, "Rules for Inservice Inspection of Nuclear Power Plant Components", 1992 Edition, with 1992 Addenda.

2.1.3 NRC Regulatory Guide 1.35, Revision 3. Dated July 1990, "Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containment"

2.1.4 Code of Federal Regulation 10 CFR 50.55a

2.1.5 AISC Manual of Steel Construction – Allowable Stress Design, 9th Edition.

2.1.6 ASCE Standard SEI/ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures, 2nd Edition.

2.1.7 ASME Standard A120.1-2001 – Safety Requirements for Powered Platforms for Building Maintenance

2.1.8 PSC Engineering Document E-GEN-500 – Rail Clamp Capacity Test

2.1.9 PSC Engineering Document CR-N991-100, "Predicted Base Forces For The 30th Year Containment IWL Inspection"

2.1.10 PSC Engineering Document CR-N1002-010, "SQ 11.1 Restressing"

2.1.11 PSC Engineering Document CR-N1002-010, "SQ 12.1 Grease Void Volumes"

2.1.12 PSC Engineering Document CR-N1002-100, "Upper Support Frame Set Position Design"

2.1.13 PSC Engineering Document CR-N1002-101, "Upper Support Frame Moving Position Design"

2.1.14 PSC Engineering Document CR-N1002-102, "Work Platform Design"

2.2 REFERENCE DRAWINGS

2.2.1 S – 425 – 001 IWE/IWL Inspection Concrete Layout 0° to 180°

2.2.2 S – 425 – 002 IWE/IWL Inspection Concrete Layout 180° to 360°

2.2.3 S – 425 – 003 IWE/IWL Inspection Exterior Dome Layout

2.2.4 S – 425 – 004 IWE/IWL Inspection Vertical Tendon Layout

2.2.5 S – 425 – 005 IWE/IWL Inspection Hoop Tendon "13" Layout

2.2.6 S – 425 – 006 IWE/IWL Inspection Hoop Tendon "42" Layout

2.2.7 S – 425 – 007 IWE/IWL Inspection Hoop Tendon "53" Layout

2.2.8 S – 425 – 008 IWE/IWL Inspection Hoop Tendon "64" Layout

2.2.9 S – 425 – 009 IWE/IWL Inspection Hoop Tendon "51" Layout

2.2.10 S – 425 – 0010 IWE/IWL Inspection Hoop Tendon "62" Layout

2.2.11 S – 425 – 0011 IWE/IWL Inspection Dome Tendons Layout – Plan

2.2.12 S – 425 – 0012 IWE/IWL Inspection Dome Tendons Layout – Elevation

2.2.13 S – 425 – 0020 IWE/IWL Inspection Tendon Detail

2.2.14 SC – 421 – 041 Reactor Building Ring Girder Concrete Outline – Plan and Sections

3.0 BACKGROUND

3.1 CONTAINMENT ARRANGEMENT

3.1.1 The Crystal River Unit 3 containment building is a post-tensioned and reinforced concrete structure comprised of a vertical cylinder with a hemispherical dome roof, and is supported on a conventional reinforced concrete foundation slab.

3.1.2 The containment structure post-tensioning system provides sufficient external pressure load on the containment structure to balance the internal pressure of the structure as well as the design basis accident internal containment pressure.

3.1.3 The containment post-tensioning system consists of:

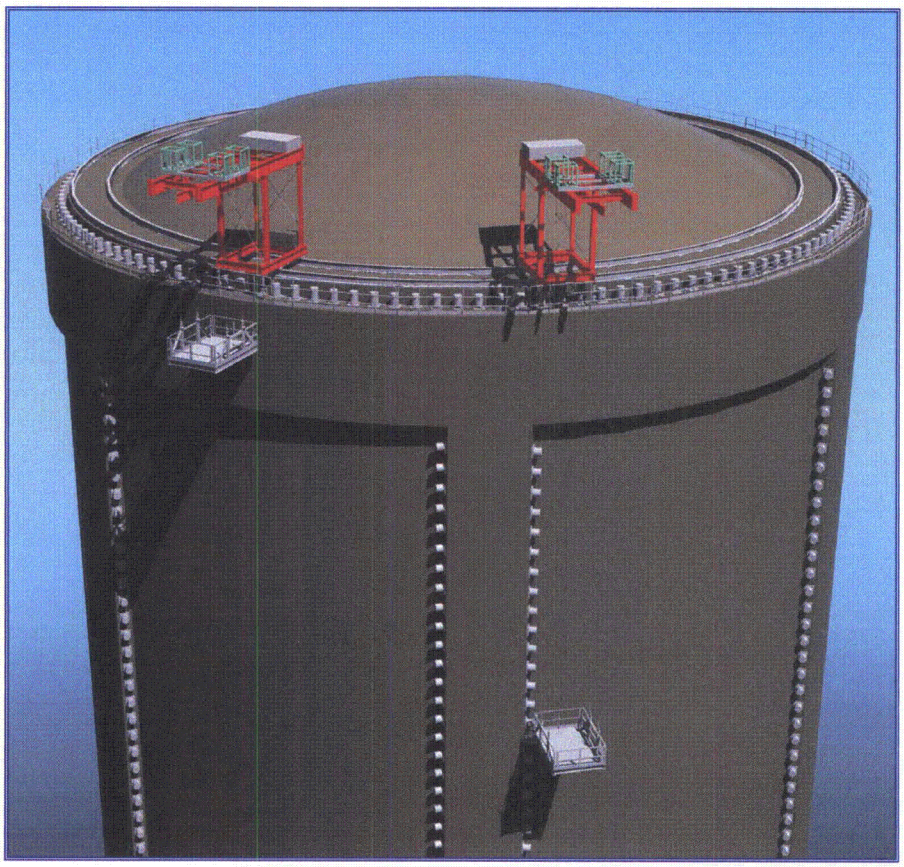
3.1.3.1 Approximately 144 vertical tendons in the cylinder wall anchored at the top surface of the ring girder and at the bottom of the base slab;

3.1.3.2 Approximately 282 hoop tendons in the cylinder wall anchored alternately at 120 degrees to each other. Each tendon encloses 120 degrees of arc and is anchored at two of the six vertical buttresses.

3.1.3.3 Three groups of 41 dome tendons (total 123 tendons) alternately oriented at 120 degrees to each other and anchored at the vertical face of the ring girder;

3.1.4 Each tendon consists of nominally 163 7mm diameter high strength low relaxation wires with buttonhead anchorages. The tendons are housed in individual spirally wrapped, corrugated, thin wall sheet metal sheathing connected to a steel bearing plate and trumplet at each end. The sheathing is cast into the containment structure concrete walls and dome. The tendons are capped at each anchorage with a sheathing filler cap and the tendon sheathing and caps are filled with corrosion preventing grease.

FIGURE 1: TYPICAL SURVEILLANCE SETUP



3.1.5 Inspection Equipment

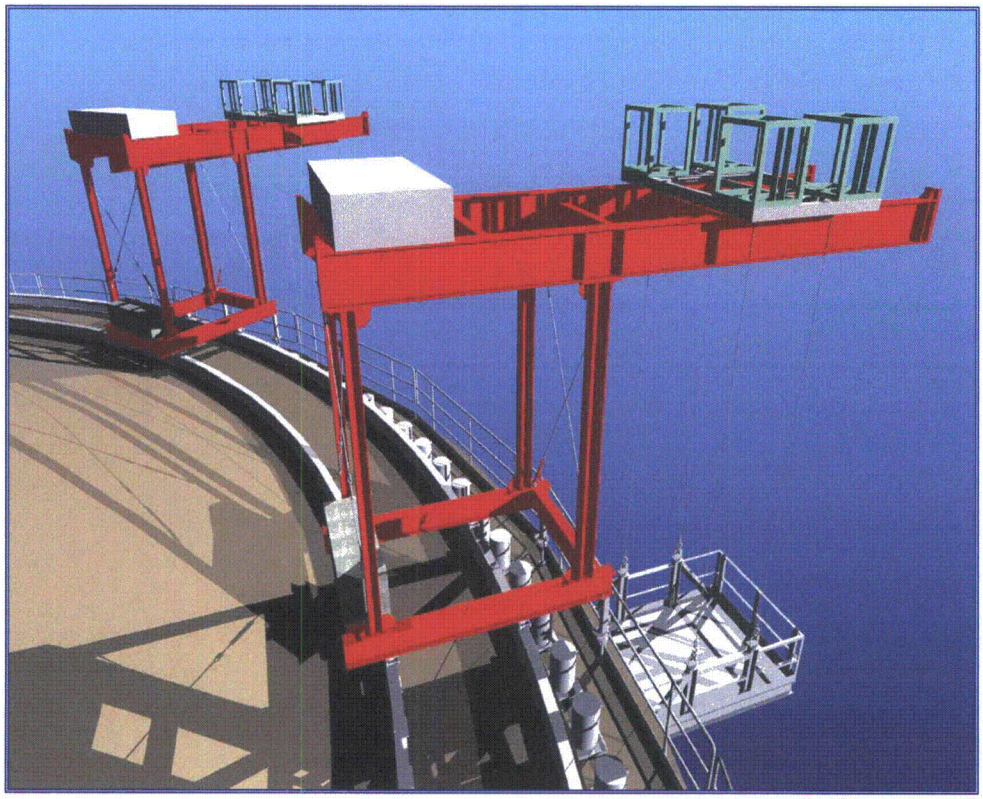
3.1.5.1 In order to perform the work required, PSC will supply the following equipment in order to perform the inspection activities. This list is for information only and equipment may be added or subtracted as needed.

1400 Ton Hydraulic Rams	Heise Digital Hydraulic Gauge	55 gallon empty drums
Jack Chairs	Misc. Electrical Transformers	110 volt electric cable
High Pressure Hydraulic Pumps	Tractel Motor Assemblies	480 volts electric cable
Jack chair extensions	2 Ga. OTC pump	Buttonhead puller
163 wire stressing adapters	Jib supports	Smooth wire puller
Pull rods with nuts	Hydraulic wire puller	Surveillance tool trailer
Inspection platforms (2)	1" diameter guide pins	Grease can plugs
Upper Support Frames (2)	1" diameter eye bolts	Graco grease pump
5 ton electric chain hoists (2)	Trolleys	Grease hose 1½" x 50'
Spider baskets/platforms	1½" diameter brass shut-off valves	55 gallon drum heater
Hydraulic hoses - 20' long	Shop and field pulling caps	Office Trailer
0-10000 psi Hydraulic Gauges	Anchorhead lifting ring	Mobile crane

3.1.6 Access Equipment

3.1.6.1 In order to access the hoop and dome tendons, two Upper Support Frames (hereafter referred to as USF) will be installed on the containment dome rails. Each USF will have a 5-Ton hoist running on a monorail to facilitate the positioning of the hydraulic stressing ram. A work platform will also be hung from each USF attached to four drive motors, which will raise and lower the platform into working position. The entire USF assembly will utilize rollers, which will allow it to travel circumferentially around the dome in order to access all selected tendons.

FIGURE 2: UPPER SUPPORT FRAMES INSTALLED ON CONTAINMENT DOME



3.1.6.2 The setup and design information for the Upper Support Frames and work platforms is included in section 8.0 of this document.

3.1.7 Stressing Equipment

3.1.7.1 In order to perform stressing activities on the selected tendons (i.e. monitoring tendon forces, detensioning, and retensioning) PSC will provide the required 1400-ton hydraulic ram setups. Each setup includes:

3.1.7.1.1 A calibrated hydraulic cylinder used for stressing, whose calibration is traceable to the National Institute of Standards and Technology (NIST).

3.1.7.1.2 A jack chair used to support the cylinder against the bearing plate as well as facilitate the removal and installation of stressing shims.

3.1.7.1.3 A pullrod/nut combination to convert the compressive force of the cylinder to a tensile force being applied to the tendon

3.1.7.1.4 A 163-wire stressing adapter to couple the pulling rod to each selected tendon's anchorhead.

3.1.7.1.5 A two stage high pressure hydraulic pump to power the cylinder

3.1.7.1.6 A calibrated field gauge for reading the pressure applied

3.1.7.2 Extreme caution shall be taken to ensure that the stressing adapter is fully engaged with the anchorhead in order to eliminate the potential for overstressing of the anchorhead threads.

4.0 SCOPE OF WORK

4.1 TENDON SELECTION REQUIREMENTS

4.1.1 The specific requirements for selection of the scope of work as well as specific requirements and acceptance criteria for the performance of the inspection for the 8th period (30th Year) Tendon Surveillance are defined by The Crystal River Technical Specifications, the Code of Federal Regulations 10 CFR 50.55a and ASME Section XI, Sub-Section IWL, define the specific requirements for selection of the inspection tendons.

4.2 TENDON SELECTION

4.2.1 Tendons were selected for the eighth surveillance period by Progress Energy Florida and distributed to Precision Surveillance Corporation. Tendons to be examined are selected on a random basis among the tendons that have not been examined during previous inspections, except for one tendon from each group, which is designated as the common tendon and is examined during each surveillance.

4.2.1.1 Eleven tendons were selected for this surveillance consisting of 5 hoop tendons, 3 vertical tendons, and 3 dome tendons. In addition to the eleven surveillance tendons, one tendon from each group (vertical, hoop, dome) was selected as an alternate. The alternate tendons will only be inspected in the event that one of the surveillance tendons is found to be inaccessible. Table 1 below lists the tendons selected for the current surveillance as well as the inspections that will be performed on each tendon.

4.2.2 In the event that the acceptance criteria are not met for a certain tendon, it may be necessary to inspect either one or both tendons adjacent to the selected tendon. The criteria and need for this inspection are outlined in the applicable governing codes (see references 2.1.2, 2.1.3, and 2.1.4) as well as the surveillance procedure. The adjacent tendons are listed in Table 2.

TABLE 1 : SCOPE OF WORK

CRYSTAL RIVER UNIT 3 – 30TH YEAR SURVEILLANCE SCOPE OF WORK												
		VISUAL					PHYSICAL					
TENDON	END	SQ6.0	SQ6.1	SQ7.0	SQ8.0	SQ8.3	SQ9.0	SQ10.2	SQ10.3	SQ11.0	SQ12.1	COMMENTS
12V01	D & G	●	●	●	●	●	●				●	COMMON, STEAM ZONE @ DOME
45V20	D & G	●	●	●	●	●	●				●	ORIGINAL SCOPE
61V17	D & G	●	●	●	●	●	●	●	●	●	●	DETENSION
34V17	D & G											ALTERNATE
13H36	BT. 1 & 3	●	●	●	●	●	●				●	
42H46	BT. 2 & 4	●	●	●	●	●	●				●	STEAM ZONE @ BT2
46H21	BT. 4 & 6	●	●	●	●	●	●				●	COMMON
51H34	BT. 5 & 1	●	●	●	●	●	●	●	●	●	●	DETENSION, STEAM ZONE @BT1
62H30	BT. 6 & 2	●	●	●	●	●	●				●	STEAM ZONE @BT 2
46H07	BT. 4 & 6											ALTERNATE
D129	BT. 3 & 5	●	●	●	●	●	●				●	STEAM ZONE @BT3
D212	BT. 1 & 3	●	●	●	●	●	●				●	COMMON
D238	BT. 4 & 6	●	●	●	●	●	●	●	●	●	●	DETENSION
D337	BT. 1 & 5											ALTERNATE

LEGEND

SQ 6.0 – GREASE CAP REMOVAL
SQ 6.1 – INSPECTION FOR WATER
SQ 7.0 – AQUIRE GREASE SAMPLES
SQ 8.0 – ANCHORAGE INSPECTION
SQ 8.3 – CONCRETE INSPECTION
SQ 9.0 – MONITOR TENDON FORCE
SQ 10.2 – TENDON WIRE INSPECTION
SQ 10.3 – TESTING TENDON WIRES
SQ 11.0 – RETENSION TENDONS
SQ 12.1 – GREASE REPLACEMENT

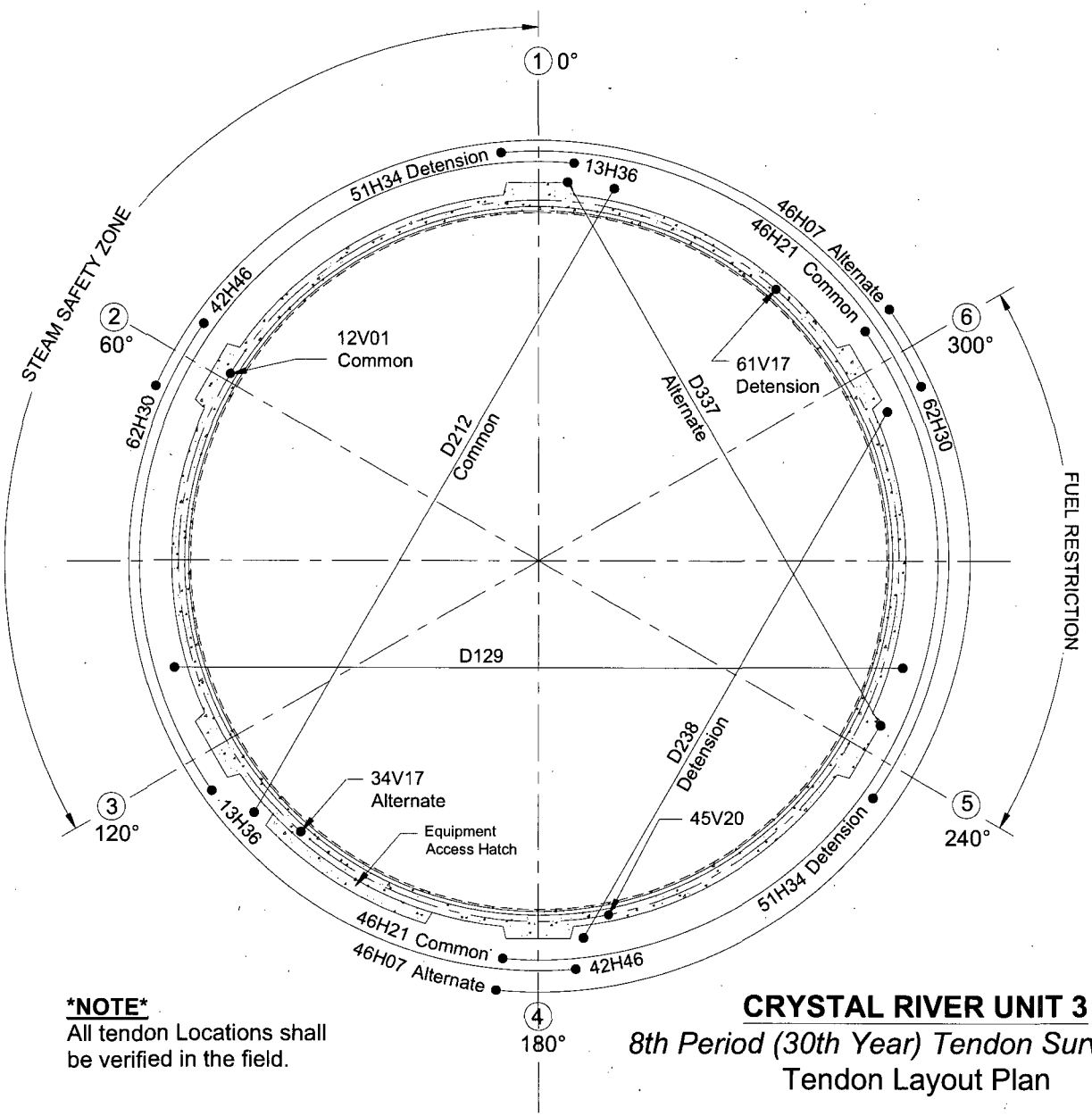
TABLE 2 : ADJACENT TENDONS

CRYSTAL RIVER UNIT 3 – 30TH YEAR SURVEILLANCE ADJACENT TENDONS					
TENDON	END	COMMENTS	TENDON	END	COMMENTS
61V24	D & G	ADJACENT	51H33	BT. 5 & 1	ADJACENT
12V02	D & G	ADJACENT	51H35	BT. 5 & 1	ADJACENT
45V19	D & G	ADJACENT	62H29	BT. 6 & 2	ADJACENT
45V21	D & G	ADJACENT	62H31	BT. 6 & 2	ADJACENT
61V16	D & G	ADJACENT	46H06	BT. 4 & 6	ADJACENT OF ALTERNATE
61V18	D & G	ADJACENT	46H08	BT. 4 & 6	ADJACENT OF ALTERNATE
34V16	D & G	ADJACENT	D128	BT. 3 & 5	ADJACENT
34V18	D & G	ADJACENT	D130	BT. 3 & 5	ADJACENT
13H35	BT. 1 & 3	ADJACENT	D211	BT. 1 & 3	ADJACENT
13H37	BT. 1 & 3	ADJACENT	D213	BT. 1 & 3	ADJACENT
42H45	BT. 2 & 4	ADJACENT	D237	BT. 4 & 6	ADJACENT
42H47	BT. 2 & 4	ADJACENT	D239	BT. 4 & 6	ADJACENT
46H20	BT. 4 & 6	ADJACENT	D336	BT. 1 & 5	ADJACENT
46H22	BT. 4 & 6	ADJACENT	D338	BT. 1 & 5	ADJACENT

4.3 TENDON LAYOUT

4.3.1 In order to properly locate the correct tendon in the field, layouts of the selected surveillance tendons as well as alternates have been performed. There is a plan and elevation view showing the tendons as well as surrounding buildings. The tendon layouts are shown below in Figures 3, 4 and 5.

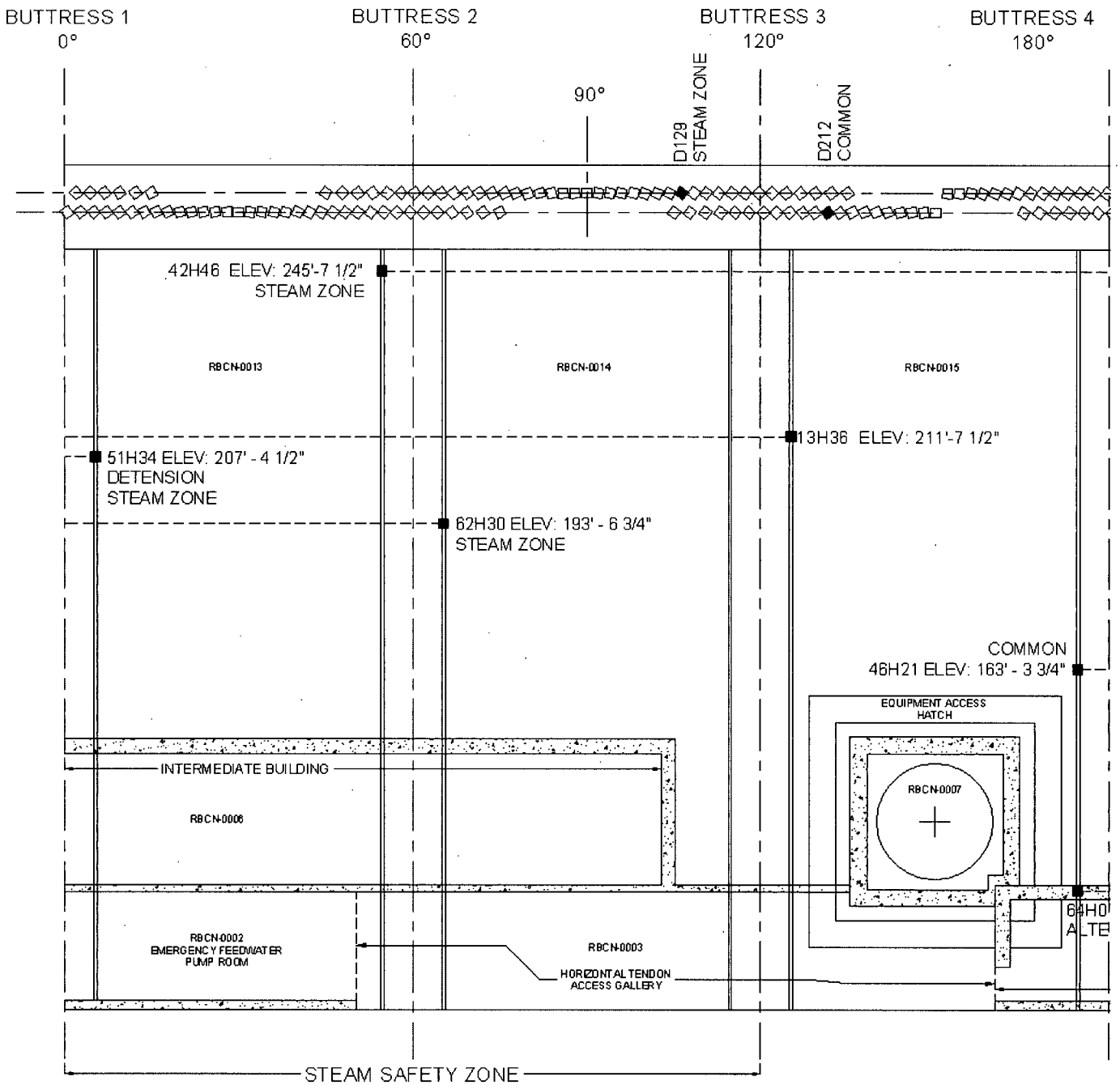
FIGURE 3: CR03 8TH PERIOD SURVEILLANCE – TENDON LAYOUT – PLAN



NOTE
All tendon Locations shall be verified in the field.

CRYSTAL RIVER UNIT 3
8th Period (30th Year) Tendon Surveillance
Tendon Layout Plan

FIGURE 4: CR03 8TH PERIOD SURVEILLANCE – TENDON LAYOUT – ELEVATION 0° TO 180°



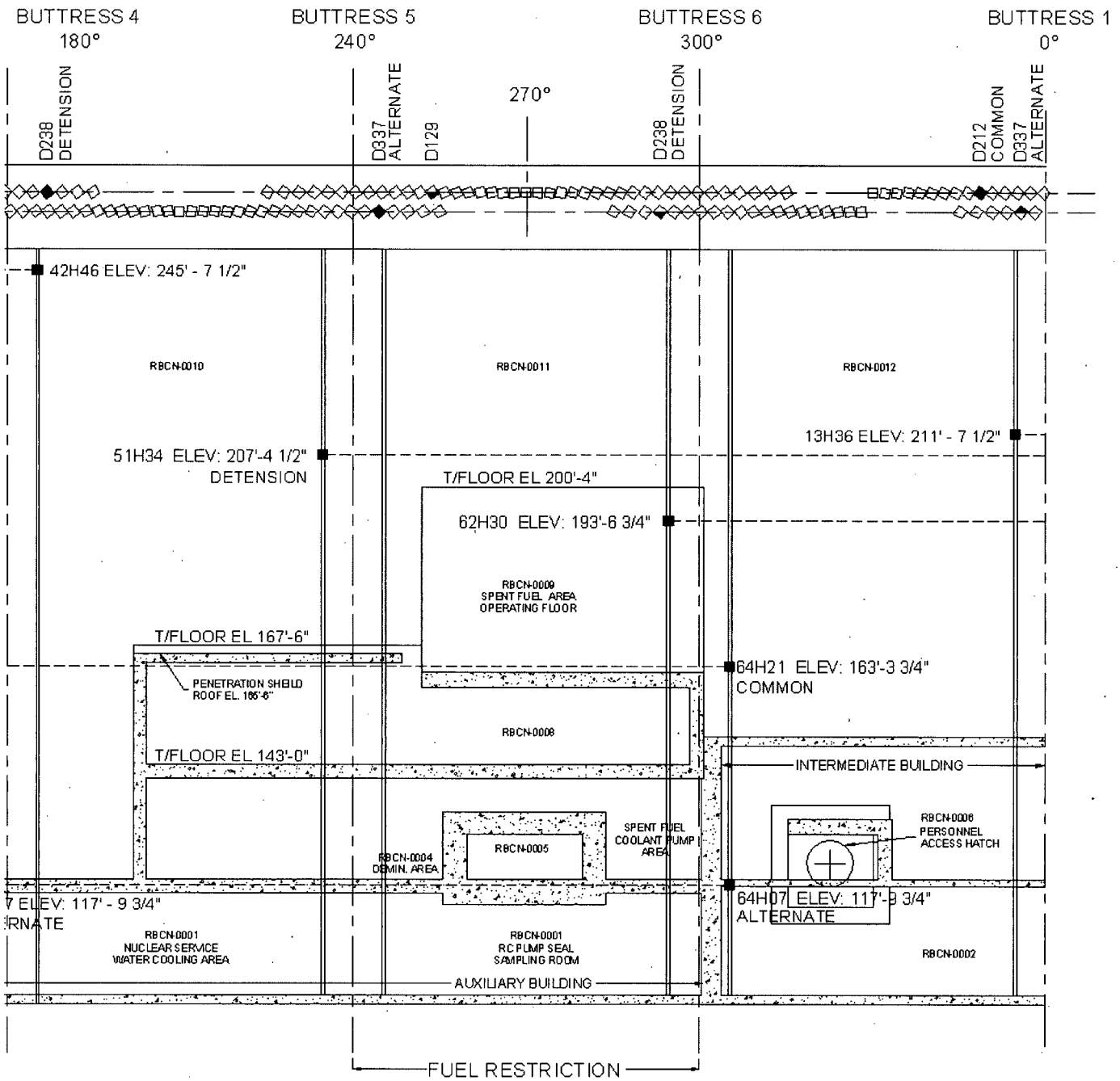
CRYSTAL RIVER UNIT 3

8th Period (30th Year) Tendon Surveillance

Tendon Layout Elevation - 0° to 180°

NOTE
All tendon Locations shall be verified in the field.

FIGURE 5: CR03 8TH PERIOD SURVEILLANCE – TENDON LAYOUT – ELEVATION 180° TO 360°



CRYSTAL RIVER UNIT 3

8th Period (30th Year) Tendon Surveillance

Tendon Layout Elevation - 180° to 360°

NOTE
All tendon Locations shall be verified in the field.

5.0 MONITOR TENDON FORCES

5.1 PERFORMING LIFT-OFF TESTING

5.1.1 In order to check the force being held by each of the selected surveillance tendons a Lift-off test will be performed. This process will be performed in accordance with the PSC supplied In-Service Inspection Procedure. A lift-off is achieved by transferring the tension on the tendon from its shim stack to the hydraulic ram. Once the tension has been transferred, the force is calculated from the internal pressure of the hydraulic ram. The steps involved in performing a liftoff are as follows:

5.1.1.1 The hydraulic pressure will be applied to the ram until lift-off is achieved.

5.1.1.2 After achieving lift-off 2 feeler gauges (0.030 inch) shall be inserted about 180 degrees apart, between the anchor head and the top shim.

5.1.1.3 The pressure will then be reduced to transfer the load back to the shim stack.

5.1.1.4 Pressure will be applied to the tendon until the feeler gauge can be pulled out with some effort.

5.1.1.5 These steps will then be repeated until three consecutive readings within the specified tolerance are achieved.

5.1.1.6 The pressure used to achieve liftoff is then converted to the force for that tendon using the hydraulic ram calibration data, which is traceable to NIST, and the normalization factor and compared to the predicted force for that tendon to verify acceptability.

5.1.2 The predicted forces and normalization factors for each surveillance tendon, as well as the tendons adjacent to it, have been calculated and included in Attachment C and are summarized below in Table 3.

TABLE 3 : SQ 9.1 – PREDICTED FORECS SUMMARY

CRYSTAL RIVER UNIT 3 – SQ 9.1 – PREDICTED FORECS SUMMARY									
TENDON	PREDICTED BASE (kips)	95% PREDICTED BASE (kips)	90% PREDICTED BASE (kips)	NORMALI-ZATION FACOTR	TENDON	PREDICTED BASE (kips)	95% PREDICTED BASE (kips)	90% PREDICTED BASE (kips)	NORMALI-ZATION FACOTR
61V24	1529	1452	1376	-14	13H35	1373	1304	1235	+56
12V01	1525	1449	1372	-9	13H36	1484	1410	1336	-56
12V02	1596	1516	1437	-80	13H37	1368	1299	1231	+61
45V19	1484	1409	1335	+31	62H29	1421	1350	1279	+7
45V20	1507	1432	1357	+7	62H30	1413	1342	1272	+14
45V21	1533	1456	1380	-19	62H31	1475	1401	1328	-46
61V16	1523	1447	1371	-8	D128	1268	1205	1141	+53
61V17	1498	1423	1348	+17	D129	1287	1223	1159	+34
61V18	1493	1418	1344	+21	D130	1261	1198	1135	+60
46H20	1467	1394	1321	-39	D211	1305	1239	1174	+18
46H21	1441	1369	1297	-12	D212	1305	1240	1175	+15
46H22	1486	1412	1337	-57	D213	1312	1246	1180	+10
51H33	1392	1323	1253	+36	D237	1426	1335	1283	-104
51H34	1487	1413	1339	-59	D238	1348	1281	1213	-26
51H35	1348	1281	1213	+79	D239	1409	1338	1268	-88
42H45	1473	1399	1325	-44					
42H46	1456	1383	1310	-28	= SURVEILLANCE TENDON				
42H47	1425	1354	1283	+3	= ADJACENT TENDON				

6.0 RETENSION TENDONS

6.1 DETENSIONING, WIRE REMOVAL, AND RETENSIONING OF TENDONS

6.1.1 One tendon out of each group (vertical, hoop, and dome) is selected to have a wire removed and tested. This process will be performed in accordance with the PSC supplied In-Service Inspection Procedure. In order for this to be accomplished the following will happen:

6.1.1.1 Liftoff-testing will be completed to ensure the tendon's force is at an acceptable level.

6.1.1.2 Once the liftoff has been completed, the force will be transferred to the hydraulic ram and all of the shims will be removed.

6.1.1.3 After the shims have been removed, the pressure will then be released from both ends of the tendon until there is no longer any force on it. This is the detensioning of the tendon.

6.1.1.4 While the tendon is in the detensioned state, a sample wire is removed and scheduled for physical testing as required by IWL-2523.

6.1.1.5 Once the wire has been removed, the tendon is then restored to an acceptable force by pressurizing the ram and replacing the stressing shims.

6.1.2 In order to perform an acceptable restressing operation, the following parameters must be calculated since the tendon has one less effective wire (because of the removed test wire).

6.1.2.1 The new Pre-Tension Force (PTF)

6.1.2.2 The new Overstress Force (OSF) which is 80% of the tendons Gross Ultimate Tensile Strength (GUTS)

6.1.2.3 The new elongations at each of the measurement stops and at OSF

6.1.3 All of the new restressing calculations are included in Attachment D and the parameters are summarized in Table 4 below.

TABLE 4 : SQ 11.1 – RESTRESSING DATA

CRYSTAL RIVER UNIT 3 – SQ 11.1 – RESTRESSING DATA								
	PREVIOUSLY		AT RETENSIONING					
TENDON	ORIGINAL PTF (kips)	ORIGINAL OSF (kips)	NUMBER OF WIRES	NEW PTF (kips)	NEW OSF (kips)	600 (kips)	1200 (kips)	ORIGINAL ELONGATION (in.)
61V17	357	1866	162	354	1855	1.92	6.62	11.75
			161	352	1843	1.95	6.68	
			160	350	1832	1.98	6.74	
51H34	362	1866	162	359	1855	1.65	5.76	10.25
			161	357	1843	1.67	5.81	
			160	355	1832	1.70	5.87	
D238	359	1866	162	357.1	1854.6	1.34	4.64	8.25
			161	354.9	1843.1	1.36	4.68	
			160	352.7	1831.7	1.38	4.73	

7.0 GREASE VOID VOLUMES

7.1 GREASE REPLACEMENT

7.1.1 Once all inspections have concluded, any sheathing filler (grease) that was lost during the inspections must be replaced. Throughout the inspection all grease that is removed is documented. The required methods of refilling, as well as the acceptance requirements for amount refilled, are dependent upon the percentage of the total void that is lost. In order to calculate this percentage, the grease void volume has been calculated for each selected surveillance tendon as well as both of its adjacent tendons. The grease void calculations are included in Attachment E and are summarized below in Table 5.

TABLE 5 : SQ 12.1 – GREASE VOID VOLUMES

CRYSTAL RIVER UNIT 3 – SQ 12.1 – GREASE VOID VOLUMES								
TENDON	NET DUCT VOLUME (GALLONS)	10% DUCT VOLUME (GALLONS)	TENDON	NET DUCT VOLUME (GALLONS)	10% DUCT VOLUME (GALLONS)	TENDON	NET DUCT VOLUME (GALLONS)	10% DUCT VOLUME (GALLONS)
61V24	144.09	14.41	13H35	121.77	12.18	D128	118.31	11.83
12V01	143.97	14.40	13H36	121.67	12.17	D129	116.41	11.64
12V02	144.12	14.41	13H37	122.18	12.22	D130	116.12	11.61
45V19	144.51	14.45	42H45	121.40	12.14	D211	115.58	11.56
45V20	144.47	14.45	42H46	122.57	12.26	D212	115.99	11.60
45V21	144.05	14.40	42H47	121.84	12.18	D213	116.85	11.69
61V16	144.36	14.44	46H20	121.37	12.14	D237	104.93	10.49
61V17	144.74	14.47	46H21	122.06	12.21	D238	102.59	10.26
61V18	144.15	14.42	46H22	122.43	12.24	D239	100.43	10.04
34V16	145.50	14.55	51H33	121.94	12.19	D336	107.62	10.76
34V17	145.67	14.57	51H34	120.88	12.09	D337	106.08	10.61
34V18	144.97	14.50	51H35	122.21	12.22	D338	103.92	10.39
			62H29	121.93	12.19			
			62H30	121.75	12.17			
			62H31	122.13	12.21			
			46H06	122.00	12.20			
			46H07	121.18	12.12	= SURVEILLANCE TENDON		
			46H08	121.46	12.15	= ADJACENT TENDON		

Evaluation of Containment Stresses

Prepared by Dr. Ray Waldo and Dr. Patrick Berbon, PII team

The table below provides a comparison of containment design parameters for several plants similar to Crystal River.

PLANT	Design Pressure (psig)	Cont. Radius (feet)	Design Force (kip/ft)	Tendon Wires	Tendon Force (kip)	% of Ultimate Stress	# of Hoop Tendons	Wall Height (feet)	Effect. Prestress (kip/ft)	Prestress/ Design Ratio
CR-3	55	65	515	163@7 mm	1325	57	94	157'	793	1.54
TMI-1	55	65	515	169@ 0.25"			110	157'		
Oconee-1,2,3	59	58	493	90@ 0.25"	588	55	210	176.5'	700	1.42
TP-3,4	55	58	459	90@ 0.25"	756	71	135	152.3'	670	1.46
Palisades	55	58	493	90@ 0.25"	631	60	167	158.5'	665	1.35
Calvert Cliffs 1,2	50	65	468	90@ 0.25"	605	57	156	149.7'	630	1.35
Farley 1,2	54	65	505	170@ 0.25"	887	46	90	128.8'	620	1.23
ANO-1	59	58	493	186@ 0.25"	1080	49	114	168.7'	730	1.48
ANO-2	54	58	451	186@ 0.25"	1066	49	106	168.7'	670	1.49

The closest plant design to Crystal River is Three Mile Island. They are comparable for the important concrete stress parameters. The other plants have smaller tendons but more of them which results in to similar average stresses but lower peak stresses.

The standard comparison is tendon force per foot of wall height. CR-3 is slightly higher than its peers.

- The average concrete pre-stress is calculated from the tendon force times number of tendons divided by the height of containment. It is expressed in kips/ft;
- The average design force is calculated from the accident pressure times the radius of containment times 0.144 to obtain kips/ft units;
- The pre-stress to design ratio is obtained by dividing the average pre-stress by the average design force;

Note that the pre-stress to design pressure ratio of 1.54 obtained here is slightly different from the ratio of 1.61 obtained in other calculations. The reason for the difference is that

we have here considered the average force distributed on the entire height of containment. In practice, there are more tendons in the higher part of containment (829 kips/ft) and less in the lower 10 ft (415 kips/ft) (see Design Basis Document, Rev 6, page 15 of 44). The ratio 829 kips/ft over 515 kips/ft is equal to 1.61 as found in our other calculation.

Crystal River has slightly higher pre-stresses than are typically seen in the industry. However, they remain well below the physical limitations of the materials used.