AGING MANAGEMENT OF NUCLEAR PRESTRESSED CONCRETE CONTAINMENTS

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Aging Management of Nuclear Prestressed Concrete Containments

U.S. Experience

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Foreword

This study was initiated to retrieve information on the practice in United States for aging management, inspection, and surveillance of nuclear containment structures with post tensioned prestressed tendons. The project is part of the Energiforsk Nuclear Concrete research program, with the aim to initiate research and development that will contribute to a safe and cost effective long term operation of Swedish and Finnish nuclear power. The program is financed by Vattenfall, E.ON, Fortum, Skellefteå Kraft, Karlstads Energi, Strålsäkerhetsmyndigheten (SSM) and Teollisuuden Voima Oy (TVO).

Sammanfattning

Det främsta syftet med denna rapport är att beskriva praxis i USA för hantering av åldrande, inspektion och övervakning av reaktorinneslutningar med förspända spännkablar. Rapporten ger information om föreskrifter och vägledningar från USA:s Nuclear Regulatory Commission (USNRC) samt American Society of Mechanical Engineers (ASME) kod för konstruktion, besiktning och övervakning av förspända reaktorinneslutningar. Rapporten ger också detaljerad information kring hur efterspända betongkonstruktioner ska övervakas och utvärderas, inklusive tolkning och bedömning av värden från periodisk övervakning vad gäller uppmätta krafter och förskjutningar. Vidare är nuvarande branschpraxis för läckagetester för att påvisa täthet och strukturell integritet inkluderad i denna rapport samt drifterfarenheter. Rapporten omfattar också metoder för analys av befintliga betonginneslutningar för att reducera den konservatism som råder vid konstruktion av inneslutningar, med hjälp av nya datoriserade analysverktyg och realistiska materialegenskaper.

De kärnkraftverk som är i drift i Sverige och i Finland har varit i gång i över 30 år, och kräver kontinuerlig övervakning och åldringshantering. Denna rapport kommer att vara ett värdefullt underlag för svenska och finska tillsynsmyndigheter och ingenjörer som arbetar med hantering av åldrande, kontroll, och övervakning av reaktorinneslutningar. Några av de erfarenheter, lärdomar, och tekniker som används för hantering av åldrande av 100 verksamma kärnkraftverk i USA kan vara användbara och kan införlivas i svenska och finska anläggningar.

Summary

The main objective of this report is to describe the practice in United States for aging management, inspection, and surveillance of nuclear containment structures with post tensioned prestressed tendons. The report provides information about the United States Nuclear Regulatory Commission's (USNRC) regulations and guidance documents and American Society of Mechanical Engineers (ASME) Code for containment design, inspection, and surveillance requirements for post tensioned concrete containments. This report also provides detailed information on how to review and evaluate post tensioned concrete containments inspection and surveillance results, including interpreting and assessment of the measured forces, displacements obtained during periodic surveillance activities. The current industry practice for containment leakage rate testing to demonstrate leak tightness and structural integrity is also included in this report. In addition, the report delineates operating experience in United States for concrete containments. The report also include methods for reanalysis of the containments to remove conservatism from the design using new computer codes and realistic material properties.

The operating nuclear power plants in Sweden and Finland have been operating for more than 30 years and require continuous aging management and inspection. This report will be a useful reference for Swedish and Finnish regulators and utility engineers engaged in aging management, inspection, and surveillance nuclear containment structures. Some of the experiences, lessons learned, and techniques used for aging management of 100 operating nuclear power plants in the United States may be useful and can be incorporated for the Swedish and Finnish plants.

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1 Purpose

Purpose of this report is to identify the practice for aging management of nuclear power plant containments with post tensioned prestressing tendons in United States (USA). This report includes United States Nuclear Regulatory Commission (USNRC) and American Society of Mechanical Engineers (ASME) requirements and guidance for the design of prestressing system, including calculation of loss of prestress during construction as well as over the long term. The report also describe the procedures for inspection and evaluation of prestressing system as well as concrete for prestressed concrete containments (PCCs) with grouted and ungrouted (unbonded) tendons. In addition, the operating experience of concrete containments, and the current practice in USA for re-analysis of the PCCs to remove conservatism from the design is also described in this report.

2 Introduction/Background

The United States has 100 operating nuclear power plants. These plants have two types of reactors, Pressurized Water Reactor (PWR) or Boiling Water Reactor (BWR). There are 65 PWR and 35 BWR plants. The reactors in all of these plants are housed inside a containment structure that acts as a final barrier against release of radioactive fission products to the environment under various accident conditions. The BWR containments are mostly a steel structure in the form of an inverted light bulb which is enclosed by reinforced concrete to provide biological shielding. The PWR containments are constructed either of steel or reinforced concrete or prestressed concrete. US has 38 PWR prestressed concrete containments (PCCs).

PCCs containments consists of a vertical cylinder with a shallow or hemispherical dome and a flat foundation mat. The inside surface of the PCCs are lined with a steel plate to act as a leak tight membrane. The prestressing tendons in the containments are arranged so that the pre-compression imparted to the concrete is adequate to withstand the tensile stresses produced by the internal pressure during a postulated design-basis accident (DBA) without significant cracking of the containment structure. The installation process and time-dependent characteristics of the containment concrete and prestressing steel affect prestressing forces in the tendons after their installation.

The first prestressed concrete containments in US, at R.E. Ginna and H.B. Robinson Unit 2 nuclear stations, were partially prestressed in the vertical direction only with non prestressed reinforcing in the circumferential (hoop) direction.

Fully prestressed concrete containments were first built in the late 1960's being cylindrical in shape with shallow dome and resting on a reinforced concrete slab. The dome is prestressed by three sets of tendons at 60° to each other and which are anchored at the side of the thickened dome-cylinder transition (ring girder). The cylinder walls are pre-stressed with both vertical and hoop tendons. The vertical tendons are anchored at the top to the ring girder and at the bottom of the foundation mat in specially constructed tendon galleries. Anchorage of the hoop tendons is to buttresses protruding from the cylindrical wall. First generation of the PCCs had six or eight buttresses.

Because of the number of tendons, which was very labor intensive to fabricate, install, tension, and make resistant to corrosion, the second generation of fully PCCs designs with three or four buttresses were evolved. In the third generation of PCCs, a hemispherical dome replaced the hallow dome, the ring girder was eliminated, and inverted U-shaped tendons replaced the dome and vertical tendons to facilitate construction. This configuration eliminated potential corrosion of prestressing anchors and cables at the ring beam due to environment. The inverted-U shaped tendons were divided into two sets of tendons oriented at 90 degrees to one another. Figure 1 shows prestressing systems for generations 2 and 3 of US PCCs.

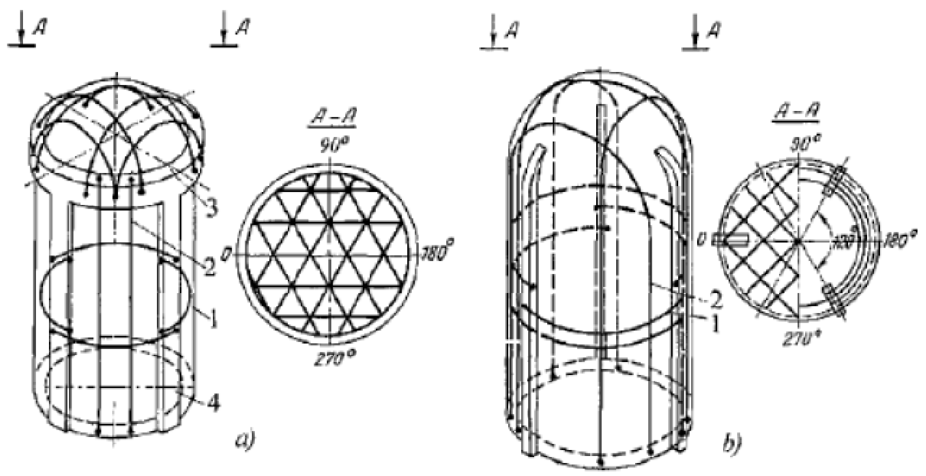

Figure 1

1 Design of Prestressed Containments

2.1 DESIGN CONSIDERATIONS

The earlier US PCCs were designed and constructed using the provisions of American Concrete Institute (ACI) 318 [1], supplemented by the specific loads and load combinations stipulated by the NRC. However, since 1975, Section III, Division 2 of the ASME Boiler and Pressure Vessel Code [2] has been used for the design of concrete containments. United States Nuclear Regulatory Commission (USNRC) Regulatory Guide RG 1.136, Rev. 3 [3], "Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments, and Section 3.8.1 of NUREG-0800, "Standard Review Plan for Safety Analysis Reports," Rev. 6 [4], are the current documents that supplemental criteria or endorsement of the ASME Code. NUREG-0800, Rev. 0 was issued in November 1975 while Revision 1 of the RG 1.136 was issued in 1978.

Prestressing force in the PCCs is designed to counteract the internal design pressure in the containment during the accident. In absence of prestressing force, the amount of rebars (non-prestressed reinforcement) required in containment is excessive and makes construction and placement of concrete extremely difficult. Prestressing force required in each direction (hoop, vertical and or dome) is normally estimated to neutralize the tensile force generated by 1.25 times the containment design accident pressure; however, in some US containments, the prestressing force equivalent to 1.5 times the design accident pressure has been used. The initial tensile average stress over the length in the prestressing tendons is limited to at 70% of the guaranteed ultimate strength of the tendons. This force is reduced by 15 to 25% for the preliminary design for initial and time dependent losses.

2.2 LOSS OF PRESTRESS

USNRC RG 1.35.1 [5], " Determining Prestressing Forces for Inspection of Prestressed Containments", and ASME Section III, Division 2 [2] provide discussion and detailed guidance for determining loss of prestress in containments. According RG 1.35.1, loss in prestress after the application of the force can be classified as follows:

a. Initial losses caused by
   - i. Slip at anchorage
   - ii Elastic shortening of concrete and effect of sequence of stressing various tendons
   - iii Loss due to friction between the tendon and tendon duct.

b. Time dependent losses caused by
   - i. Shrinkage of concrete
   - ii. Creep of concrete
   - iii. Relaxation of prestressing steel

c. Other losses caused by:
   - i. Failure of tendon elements from corrosion or material deficiency
   - ii. Effects in variation temperature

2.2.1 Anchorage Slip Loss

Loss of prestress due to anchorage is determined based on prior experience and testing history of the prestressing system used. For some plants, anchorage slip is accounted for in the lift off forces recorded during initial tensioning and is not required to be considered for calculations for prediction of forces in the long-term.

2.2.2 Elastic Shortening of Concrete and Sequence of Prestressing

PCC has tendons installed in different directions and all the tendons are not stressed simultaneously. If the tendons are stressed simultaneously the loss of prestress due to elastic shortening of concrete F_{LES} will be:

$F_{LES} = F_o E_p A_p/(A_{CN} E_C + A_S E_S + A_P E_P + A_L E_L + A_d E_d)$

Where

F_o is the initial seating force

A_{CN} is the net concrete area

A_S, A_P, A_L, A_d are areas of reinforcing steel, prestressing steel, liner plate, and duct respectively

E_C, E_S, E_P, E_L, E_d are the moduli of elasticity of concrete, reinforcing steel, prestressing steel, liner plate, and duct respectively.

The first tendons that are tensioned undergo a full loss from the subsequent elastic shortening of the PCC structure, while the tendons that are tensioned last undergo, almost no loss of prestress due to elastic shortening. Therefore, RG 1.35.1 states that for all practical purposes, the loss of prestress can be estimated and accounted for by using the following linear relationship:

$F^n_{LES} = n_r F_{LES}/N$

Where N represents the total number of tendons in a particular direction, n represents the sequential number of a randomly selected tendon to be tensioned after the n^{th} tendon i.e. $n_r = N-n$

If the sequence of tensioning tendons in different directions are intermingled, as is the case usually, the stresses produced in one direction by the tendons tensioned in other direction must be considered.

For instance, if for a specific plant, half of the vertical tendons are stressed first, followed by stressing of all hoop tendons, and ending with stressing the remaining half of the vertical tendons, the correction to loss in prestress due to elastic shortening will be considered as follows:

The impact by Poisson's ratio effect on first half set of vertical tendons will be a decrease of prestress force due to tensioning of the hoop tendons as follows:

$\Delta Y = A_h H_{cont}/ (E_c \times T_{cont})$

where

ΔY = Elongation in half of vertical tendons stressed prior to tensioning of hoop tendons

A_h = Average force in hoop tendons

H_{cont} = Height of vertical wall

E_c = Modulus of elasticity of concrete

T_{cont} = Vertical wall thickness

Similarly poissons effect on the hoop tendons due to half of vertical tendons stressed after dome the dome tendons are stressed:

$\Delta R = A_v H_{length}/ 2(E_c \times T_{cont})$

Where

ΔR = Elongation in hoop tendons

H_{length} = Length of hoop tendon

Appendix A of this report shows detailed calculation for loss of prestress in a PCC due to elastic shortening of concrete.

2.2.3 Loss Due to Friction Between the Tendon and the Tendon Duct

Loss in prestress in tendons due to friction between the tendon and tendon duct depends upon wobble and curvature coefficients. RG 1.35.1 [5] and ASME Section III, Division 2 [3], Article CC-3542 recommend that these coefficients shall be experimentally determined and verified during prestressing operations. The friction losses are calculated as follows;

$P_s = P_x e^{(Kl+\mu\alpha)}$

When $(Kl+\mu\alpha)$ is not greater than 0.3, then

$P_s = P_x(1+Kl+\mu\alpha)$

Where

Ps = Stress in tendon at length l from anchorage

Px = Stress at transfer at anchorage

K= Wobble coefficient per foot of prestressed tendon

μ= Curvature frictional coefficient

α= Total angular change of prestressing tendon profile in radians

l = Length of prestressed tendon from jacking end in meters

In USA, the value of K for unbonded tendons is usually between 0.0009 and 0.0060, and μ is usually between 0.05 and 0.15 based on data available from prestressed tendon supplier's previous experience. For straight tendons $\mu = 0$. French EPR Technical Code for Civil Works, AFCEN ETC-C-2012, specify the friction coefficients as follows:

$\mu = 0.18$ for steel corrugated steel ducts and 0.16 for steel ducts

K = 0.0009 for straight tendons and 0.0016 for curved tendons

In order to reduce the losses due to friction, the tendons in PCCs are stressed from ends. The resulting loss in prestress at mid-point of tendon length is 50% less because the value of l is half of the total length.

2.2.4 Time Dependent Loss of Prestress Due to Concrete Shrinkage

According to USNRC RG 1.35.1 [5], the schedule of construction of a typical PCC is such that a substantial portion of the long term shrinkage takes place before the structure is prestressed. Hanson et al. [6] presents formulas for predicting the long term shrinkage based on the assumption that shrinkage approximately follows the laws of diffusion and supports the formulas by experimental investigation. An appropriate extrapolation of these formulas (for the volume-to-surface of the structure in excess of 60 cm, and contributing shrinkage as that occurring 100 days after the average time of construction of the structure) would yield a value 100×10^{-6}, which is considered to be reasonable value at a temperature of 21°C and a relative humidity of 50%. USNRC RG 1.35.1 recommendation for variation of shrinkage strain with relative humidity as shown in Table 1 below.

Mean Daily Relative Humidity, Annual %	**40 year Shrinkage Strain**
Under 40%	130×10^{-6}
40% to 80%	100×10^{-6}
Above 80%	50×10^{-6}

Table 1 - Variation of Shrinkage Strain with Relative Humidity [5]

The shrinkage strain of concrete is subject to variation due to field conditions and material properties of concrete; therefore RG 1.35.1 [5] recommends variation of shrinkage by ±20% as shown in the Table 2 below. The shrinkage strain strains at any time between the time of prestressing (consider zero shrinkage at 10 days) and 40 years can be estimated by shrinkage strain to vary linearly the logarithm of time as shown in Figure 2.

Shrinkage at 70% Relative Humidity Variation ± 20%	**Base Value @ 40 Years**	**1 Year**		**40 Years**	
		High	**Low**	**High**	**Low**
	100×10^{-6}	72×10^{-6}	50×10^{-6}	120×10^{-6}	80×10^{-6}

Table 2 – Variation of Shrinkage Strain with Time

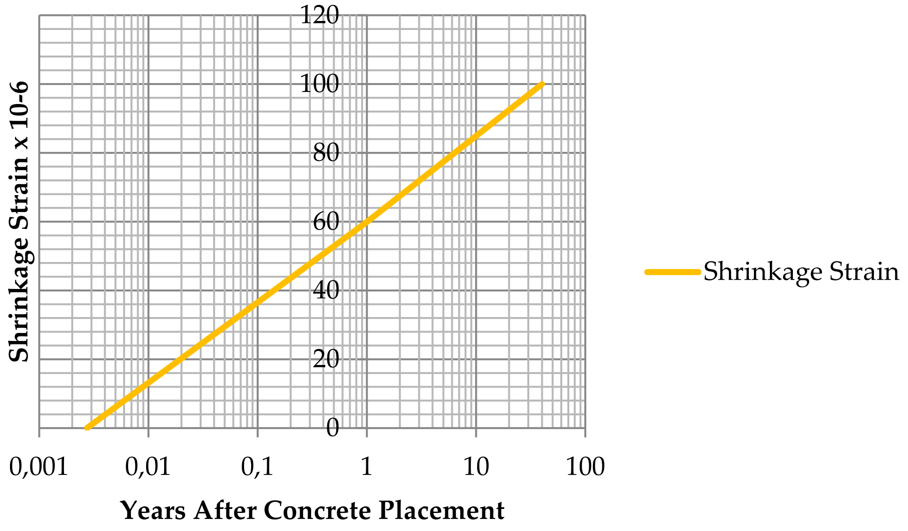

Figure 2 Variation of Shrinkage Strain with Time (logarithmic scale)

Based on the above information, loss of prestress due to shrinkage of concrete can be calculated as follows:

Consider the shrinkage strain at 40 years = 100×10^{-6}

Tendon Steel Modulus of elasticity = 210 GPa

Loss in prestress due to shrinkage = $100 \times 10^{-6} \times 210$ = 21MPa

Ultimate tensile strength of prestress steel tendon = 1680 MPa

Assume average initial stress in prestress steel tendon

= 70% of ultimate strength

= 0.70 x 1680 = 1176 MPa

Loss in prestress due to shrinkage = 21 x 100/1176 = 1.78% of prestress

Considering 20% variation in shrinkage strain

High loss in prestress = 1.78 x 1.2 = 2.14%

Low loss in prestress = 1.78 x 0.8 = 1.42%

The above calculation is based on shrinkage strain of 100 x 10^{-6}. However, this loss can be reduced if the tendon are stressed one year after concrete placement. In addition, most nuclear plants confirm the value of shrinkage strain by performing test for autogenous shrinkage test on concrete used in construction of PCC.

2.2.5 Time Dependent Loss Due to Concrete Creep

The influence of creep is one of the most significant and variable factor in calculating the time-dependent losses in prestress in PCC structures. Creep is of two kinds: basic creep and drying creep. Basic creep is due to slow compression of concrete due to compressive and sustained loads due to prestress. Drying creep is due to exchange of moisture between the PCC structure and its environment. Its characteristics are considered to be similar to of shrinkage, except they represent an additional moisture movement due to stressed condition of the PCC. However, for PCC structure having a volume to surface ratio of more than 24 (wall thickness more than 60 cm), drying creep is considered to be negligible.

Four parameters that influence basic creep are:

1. Concrete mix design - proportion of cement, water, and aggregates; and influence of admixtures
2. Age of loading – degree of hydration taken place before the tendons in PCC structure are stressed
3. Magnitude of sustained compressive stress in concrete due to prestress
4. Temperature

Investigations by Hanson et al. [6] support the assumption that basic creep varies linearly with the intensity of sustained compressive stress, as long as the compressive stress does not exceed 40% of the ultimate strength of concrete. This is normally the case for all PCC structures. Appendix A of RG 1.35.1 [5] describes a method for calculating creep in PCC structures that is acceptable to the USNRC. This method is based on predicting long term creep based on short term laboratory tests on concrete samples used for construction of PCCs.

According to Appendix A of RG 1.35.1:

$$€c/fc = A\,[1- e^{-(t-to)/30}] + B \log_{10} (t/to)$$

Where

t = time (after average time of concrete placement) when creep value is desired in days

to = time of loading after average time of concrete placement in days

fc = average concrete compressive stress

€c = creep strain at time t when age of concrete at loading is "to"

A and B are constants to be determined from tests

An example below demonstrate the use of the above equation

For a plant in USA, short term creep tests were performed with the following results:

to = 181 days	€o = 0.0 in/in	fc =14.7 N/mm^2 (2100 psi)
t1 = 249 days	€1 = -122 x 10^{-6} in/in	
t2 = 307 days	€2 = -155 x 10^{-6} in/in	
t3 = 357 days	€3 = -168 x 10^{-6} in/in	

Using the above data, three equation are formed for t1, t2, and t3

From these three equations, three sets of values of constants A and B are determined by solving once for t1 and t2 equations, then for t2 and t3 equations, and finally for t3 and t1 equations.

These values are

t1 and t2 equations   A= 4.3700 x 10^{-8} and B = 1.2818 x 10^{-7}

t2 and t3 equations   A= 4.6547 x 10^{-8} and B = 1.0920 x 10^{-7}

t3 and t1 equations   A= 5.3434 x 10^{-8} and B = 8.5808 x 10^{-8}

Therefore average values of A and B are: A= 4.7894 x 10^{-8} and B = 1.0753 x 10^{-7}

These values of A and B are used then to determine creep strain in concrete and resulting loss of stress in prestressed tendons. For instance for PCC in which prestressed tendons were tensioned after 2 years 8 months concrete placement.

to = 2 years and 8 months = 970 days (this the average mean time)

t1year = year after tendons were stressed = 970 + 1 year 10 months = 1640 days

t40year = 1640 + 39 x 365 = 15875 days

Average prestress in PCC in hoop direction = 12.55 N/mm^2 (1793 psi)

Therefore:

€1year = {{4.7894 x 10^{-8} [1 – $e^{-(1640-970)/30}$} + 1.0753 x 10^{-7} x log 1640/970} x 1793

= 0.000130

Similarly €40year = 0.000320

Loss of stress in hoop tendons due to concrete at

1 year = [0.000130 x 28.4 x 10^3/ 168]100 = 2.2%

40 year = [0.000320 x 28.4 x 10^3/ 168]100 = 5.41%

To allow for the associated uncertainty in the creep values, RG 1.35.1 requires a variation of +25% and -15%.

Therefore loss of prestress in hoop tendons due to creep can be summarized as follows

SURVEILLANCE	BASE	LOW LOSS	HIGH LOSS
1 Year after prestress	2.20%	1.87%	2.75%
40 year after prestress	5.41%	4.60%	6.76%

Table 3- Loss of Prestress in Hoop Tendons Due to Creep

There has been substantial research since RG 1.35.1 was revised. The current state of the art guidance for creep and shrinkage is provided in ACI 209R-2008 [7]. Equation 2.8 of ACI 209R with adjustments for concrete material properties, humidity, age of loading, and magnitude of prestress can be used to estimate the creep strain. This requires concrete creep tests over a period of one year instead of three months as has been done in the past using RG 1.35.1. However, this approach has not been approved for use by the USNRC at present time. Use of ACI 209R approach tend to reduce the creep strain over the long term and is less conservative than RG 1.35.1.

Loss of prestress in vertical tendons can be calculated similarly. Loss will not be identical because the level of average compressive stress in concrete due to vertical tendons is usually different.

2.2.6 Time Dependent Loss Due to Relaxation of Prestressing Tendon Steel

The stress relaxation properties prestressing tendon steel vary with its chemical and thermal mechanical treatment. Manufacturer/supplier provide data on the long term loss of prestressing tendon steel due to relaxation. There are two types of prestressing steel used for PCCs in USA. In older PCCs prestressing steel with loss due to relaxation at 70^{o}F of up to 8.0 % have been used. Newer PCCs constructed since 1980s have used prestressed steel with low relaxation of about 2.0 % at 21°C. ASME Section III, Division 2, Article CC-2424.2 [3] require that a minimum of three relaxation tests of 1000 hours shall be performed and reported to document adequately that relaxation losses. In addition, each of these three tests should have sufficient number of data points to extrapolate the 1000-hour pure relaxation data to the planned useful life of the structure.

The test data provided by the manufacturer for loss due to relaxation is usually conducted at 21°C. However, the PCC internal temperature during operation is around 100°F; therefore, it is important to ask the manufacturer to provide test data about loss of stress due to relaxation of steel at higher temperatures.

To allow for uncertainty in extrapolating the relaxation data, RG 1.35.1 [5] recommends consideration of ±15.0 % in the test data for the loss of prestress due to relaxation of steel.

2.2.7 Losses Caused by Tendon Degradation

During the design of prestressing system for most of the PCCs, an allowance for breakage of a wire in wire systems or a wire of a strand in a strand system on an overall basis as well as localized basis is included. This eliminates the need of replacing the tendon during construction or during subsequent surveillance of tendons during the life of the plant. However, there is no specific guidance on the extent or number of broken wires to be considered in the USA codes or USNRC regulations or regulatory guides. RG 1.35.1[5] recommends that care should be taken not to overstress intact wires to bring the tendon to a prescribed value. Instead, tendon is stressed to a lower force based on the net area of the intact tendons (generally 70% of the guaranteed ultimate strength).

2.2.8 Effects of Variation in Temperature

RG 1.35.1 [5] recommends that the effect of differences between the average temperature of the structure during installation and subsequent in-service inspections. Localized hot spots and temperature variations along the length of the tendons can cause variation of force along the length of tendon. The differences between coefficients of expansion of steel and concrete can also cause modification of tendon force. In practice, this activity is performed by accounting for difference in ambient temperatures during actual installation and subsequent in-service inspections.

3 USNRC and ASME Regulations and Guidance

USNRC regulations and guidance for surveillance of PCCs have evolved over time, and is different for PCCs with grouted tendons, and for greased/unbonded tendons.

3.1 PCCS WITH GREASED OR UNBONDED TENDONS

3.1.1 USNRC Regulatory Guides and ASME Code

There are 38 PCCs currently operating in USA. Of this 37 PCCs have greased or unbonded tendons. Surveillance or in-service inspection of the earlier PCCs (those licensed before 1973) were developed by the plant licensees on an individual basis and later approved by the USNRC. In general, the inspections are performed a certain period after the structural integrity test (SIT) of the containment and consists of the following. The SIT requirements are specified in Section CC-6000 of ASME Section III, Division 2 Code [2]. The SIT is prerequisite for Code acceptance and stamping before start of operation. The test is performed at 1.15 times the containment design pressure to evaluate design compliance and quality of construction.

- Monitoring of selected 9 tendons for prestressing force, grease, and condition of anchor heads
- Visual examination of containment concrete external surface

The inspections were performed more frequently in the early years and less during the later years. This approach was revised after USNRC issued regulatory guide RG 1.35, Rev.0 [8] in 1973. With the advent of PCCs with inverted U vertical tendons that eliminated ring beam at the spring line if the PCC dome, Revision 2 of the RG 1.35 was issued in 1976. The RG 1.35, Rev. 3 [9] was issued again 1 in 1990 to update and clarify the guidance on the basis of experience obtained in prior inspections. Various interpretations by the utilities of the acceptability of measured prestressing force in tendons concerned USNRC staff enough to issue a companion guide RG 1.35.1 [5] to determine prestressing forces for inspection of PCCs. In August 1996, USNRC staff issued an amendment to the Title 10 of US Code of Federal Regulations (10 CFR 50) [10]. Section 50.55a (b)(viii) of 10CFR 50, Examination of Concrete Containments, endorses Subsection IWL of the ASME Code [11] with some additional requirements, covers the in-service inspection of reinforced and PCCs, and incorporates the provision of RG 1.35, Rev. 3 [9].

A comparison of the requirements in the different documents for inspection and surveillance of PCCs is shown in Table 4.

Item	RG 1.35, Rev. 0	RG 1.35, Rev.2	RG 1.35, Rev.3	ASME Subsection IWL
Applicability	PCCs with shallow dome and cylindrical walls	1. PCCs with shallow dome and cylindrical walls. 2. PCCs with hemispherical dome shaped roof on cylindrical walls	1. PCCs with shallow dome and cylindrical walls. 2. PCCs with hemispherical dome shaped roof on cylindrical walls	1. PCCs with shallow dome and cylindrical walls. 2. PCCs with hemispherical dome shaped roof on cylindrical walls
Inspection interval for concrete surface	1, 3, 5, years after SIT and every 5 years thereafter.	1, 3, 5, years after SIT and every 5 years thereafter.	1, 3, 5, years after SIT and every 5 years thereafter.	1, 3, 5, years after SIT and every 5 years thereafter.
Inspection interval for tendons surveillance	1, 3, 5, years after SIT and every 5 years thereafter. However, for a site with identical containments, only visual examination of tendons for	1, 3, 5, years after SIT and every 5 years thereafter. However, for a site with identical containments, only visual examination of	1, 3, 5, years after SIT and every 5 years thereafter. However, for a site with identical containments only visual examination of	1, 3, 5, years after SIT and every 5 years thereafter. At a site with identical containments, first unit inspected 1, 3, and 10 years, and thereafter every 10

	second unit is required.	tendons for second unit is required.	tendons for second unit is required.	years, and second unit inspected 1, 5, 15 years and every 10 years thereafter.
No. of tendons to be inspected	6- dome, 5- vertical 10- hoop	For shallow dome containments: 6- dome, 5-vertical 10- hoop For spherical dome containments: 4% U tendons 4% hoop tendons but minimum of 4 each. After 5 years, No. of tendons may be reduced by 50%	For inspections at 1, 3, 5 years, 4% of the each group (vertical, hoop, dome, and inverted U) with a minimum of 4 and maximum of 10 tendons from each group. After 5 years, the number of tendons may be reduced by 50% with a maximum of 5 tendons.	For inspections at 1, 3, 5 years, 4% of the each group (vertical, hoop, dome, and inverted U) with a minimum of 4 and maximum of 10 tendons from each group. After 5 years, the No. of tendons may be reduced by 50% with a minimum of 3 and maximum of 5 tendons.
Item	**RG 1.35, Rev. 0**	**RG 1.35, Rev. 2**	**RG 1.35, Rev. 3**	**ASME Subsection IWL**
Tendon Selection	Randomly selected but distributed representatively	Randomly selected but distributed representatively	Randomly selected. One control tendon in each group does not change	Randomly selected. One control tendon in each group does not change
Prestressed Tendon Examination	A measurement of the prestress force tendon tested with acceptable limits being defined as not less than the predicted lower bound nor greater than the predicted upper bound forces at the time of test. An allowable limit of not more than one defective tendon out of the sample population. If one sample tendon is defective, an adjacent tendon each side of defective tendon should be checked. If both of these tendons are acceptable, then the surveillance proceed considering the single efficiency as unoque and acceptable. However, if either of the adjacent tendon is defective or if more than one tendon of the original sample is defective, abnormal degradation of structure is indicated.		The prestressing force in all inspection sample tendons shall be measured by lift-off or an equivalent test. Tendon forces and elongation are acceptable if the following conditions are met: *(a)* The average of all measured tendon forces, including those measured in item (b)(2) below, for each type of tendon is equal to or greater than the minimum required prestress specified at the anchorage for that type of tendon. *(b)* The measured force in each individual tendon is not less than 95% of the predicted force unless the following conditions are satisfied. *(1)* The measured force in not more than one tendon is between 90% and 95% of the predicted force. *(2)* The measured forces in two tendons located adjacent to the tendon described in item (b)(1) above are not less than 95% of the predicted forces. *(3)* For tendons requiring augmented examination, the measured forces in two like tendons located nearest to but on opposite sides of the tendon, with measured force between 90% and 95% of the predicted force, are not less than 95% of the predicted forces. *(4)* The measured forces in all the remaining sample tendons are not less than 95% of the predicted force. *(c)* The prestressing forces for each type of tendon measured, and the measurement from the previous examination, indicate a prestress loss such that predicted tendon forces meet	

			the minimum design prestress forces at the next scheduled examination. *(d)* The measured tendon elongation varies from the last measurement, adjusted for effective wires or strands, by less than 10%	
Item	**RG 1.35, Rev. 0**	**RG 1.35, Rev. 2**	**RG 1.35, Rev. 3**	**ASME Subsection IWL**
Tendon detensioning	None	None	One sample tendon of each type shall be detensioned during each inspection to identify broken or damaged wire	
Tendon material tests and inspections for corrosion or damage	Wires or strand from one dome tendon and two wall tendons be removed to check for corrosion. At each successive inspection sample removed from different tendon.	Wires or strands from one tendon of each type (dome, vertical or inverted U, and hoop), removed to check for corrosion. At each successive inspection sample removed from different tendon.	Wires or strands from one tendon of each group to be removed to check for evidence of corrosion. At each successive inspection sample removed from different tendon.	Wires or strands from one tendon of each group to be removed to check for evidence of corrosion. At each successive inspection sample removed from different tendon.
Tensile tests on wire or strand	Tensile tests to be performed on at least three samples from each removed wire or strand. Tensile strength below guaranteed ultimate strength to be considered abnormal condition.		Tensile tests to be performed on at least three samples from each removed wire or strand to determine yield strength, ultimate strength, and elongation. Failure at less than minimum requirements to be considered abnormal condition.	
Anchorage Hardware	Tendon anchorage assemblies of all selected surveillance tendons should be inspected	Tendon anchorage assemblies of all selected surveillance tendons should be inspected	Tendon anchorage assemblies of all selected surveillance tendons should be inspected	Detailed visual examination of surveillance tendons anchorage assemblies to be performed. Evidence of cracking, broken wires/strand and presence of water not acceptable
Inspection of Concrete surface around anchorage	Concrete surrounding the tendon anchorages should be checked for indications of abnormal material behavior		Concrete surrounding visually inspected tendon the anchorages should be checked for indications of abnormal material behavior	Detailed visual examination of concrete 60 cm from edge of bearing plate be performed and cracks greater than 0.25 mm width not acceptable and shall be documented.
Item	**RG 1.35, Rev. 0**	**RG 1.35, Rev. 2**	**RG 1.35, Rev. 3**	**ASME Subsection IWL**
Inspection of concrete surface in accessible areas	Concrete surface should be checked visually for indications of abnormal material	Concrete surface should be checked visually for indications of abnormal material	The exterior surface of the containment should be visually examined to detect areas of large spall,	Concrete surface areas that are accessible, including coated surfaces be visually examined for

	behavior. If the entire containment is pressurized for leak testing purposes, the visual inspection should be scheduled, if possible, to coincide with the leak test.	behavior. The visual examination of concrete should be scheduled during integrated leakage testing while containment is its maximum test pressure.	severe scaling, D-cracking in an area of 25 square feet or more, other surface deterioration or disintegration, or grease leakage.	evidence of damage or degradation such as ACI 201.1R and ACI 349.3R. Areas not meeting the criteria shall be subjected to detailed visual examination to determine magnitude and extent of deterioration and distress, including that in reinforced steel.
Inspection of concrete surface in inaccessible areas	No specific requirement	No specific requirement	No specific requirement	Concrete surfaces exposed to foundation soil, backfill, or ground water shall be evaluated to determine susceptibility of the concrete to deterioration and the ability to perform the intended design function under conditions anticipated until the structure no longer is required to fulfill its intended design function.
Item	**RG 1.35, Rev. 0**	**RG 1.35, Rev. 2**	**RG 1.35, Rev. 3**	**ASME Subsection IWL**
Corrosion protection medium (grease)	Method used for checking the presence of grease should account for (1) minimum grease coverage, (2) influence of temperature, (3) procedure used to uncover voids, (4) requirements imposed by grease specifications,	Method used for checking the presence of grease should account for (1) minimum grease coverage, (2) influence of temperature, (3) procedure used to uncover voids, (4) requirements imposed by grease specifications,	Samples of grease, and free water (if any) from each end of each tendon shall be collected and examined. Grease condition acceptable if: Water content<10% Chlorides or nitrates or sulfides <10ppm, and reserve alkalinity >50% of installed	Samples of grease, and free water (if any) from each end of each tendon shall be collected and examined. Grease condition acceptable if: Water content<10% Chlorides or nitrates or sulfides <10ppm, and reserve alkalinity >50% of installed

	qualification tests, and acceptability tolerances.	qualification tests, and acceptability tolerances.	value; however if installed value <5, reserved alkalinity value should be >0	value; however if installed value <5, reserved alkalinity value should be >0

Table 4 comparison of the requirements in the different documents for inspection and surveillance of PCCs

3.1.2 Current US Practice

Nuclear power plants in USA perform in-service inspection of PCCs concrete and prestressing systems according to the licensing commitments in their Final Safety Analysis Report (FSAR) and Technical Specifications. Some plants less than 40 years in operation can and still use different editions of RG 1.35 for performing inspections. However, those plants that have opted to use ASME Section XI, Subsection IWL [11] for performing inspection of PCC concrete and prestressing system, must comply with following additional requirements as specified in 10CFR55.55a(b)(viii):

> (viii) *Examination of concrete containments*. Applicants or licensees applying Subsection IWL, 1992 Edition with the 1992 Addenda, shall apply paragraphs (b)(2)(viii)(A) through (b)(2)(viii)(E) of this section. Applicants or licensees applying Subsection IWL, 1995 Edition with the 1996 Addenda, shall apply paragraphs (b)(2)(viii)(A), (b)(2)(viii)(D)(*3*), and (b)(2)(viii)(E) of this section. Applicants or licensees applying Subsection IWL, 1998 Edition through the 2000 Addenda shall apply paragraphs (b)(2)(viii)(E) and (b)(2)(viii)(F) of this section. Applicants or licensees applying Subsection IWL, 2001 Edition through the 2004 Edition, up to and including the 2006 Addenda, shall apply paragraphs (b)(2)(viii)(E) through (b)(2)(viii)(G) of this section. Applicants or licensees applying Subsection IWL, 2007 Edition through the latest edition and addenda incorporated by reference in paragraph (b)(2) of this section, shall apply paragraph (b)(2)(viii)(E) of this section.
>
> (A) Grease caps that are accessible must be visually examined to detect grease leakage or grease cap deformations. Grease caps must be removed for this examination when there is evidence of grease cap deformation that indicates deterioration of anchorage hardware.
>
> (B) When evaluation of consecutive surveillances of prestressing forces for the same tendon or tendons in a group indicates a trend of prestress loss such that the tendon force(s) would be less than the minimum design prestress requirements before the next inspection interval, an evaluation must be performed and reported in the Engineering Evaluation Report as prescribed in IWL–3300.
>
> (C) When the elongation corresponding to a specific load (adjusted for effective wires or strands) during retensioning of tendons differs by more than 10 percent from that recorded during the last measurement, an evaluation must be performed to determine whether the difference is related to wire failures or slip of wires in anchorage. A difference of more than 10 percent must be identified in the ISI Summary Report required by IWA–6000.
>
> (D) The applicant or licensee shall report the following conditions, if they occur, in the ISI Summary Report required by IWA–6000:
>
> (*1*) The sampled sheathing filler grease contains chemically combined water exceeding 10 percent by weight or the presence of free water;
>
> (*2*) The absolute difference between the amount removed and the amount replaced exceeds 10 percent of the tendon net duct volume;
>
> (*3*) Grease leakage is detected during general visual examination of the containment surface.

(E) For Class CC applications, the applicant or licensee shall evaluate the acceptability of inaccessible areas when conditions exist in accessible areas that could indicate the presence of or result in degradation to such inaccessible areas. For each inaccessible area identified, the applicant or licensee shall provide the following in the ISI Summary Report required by IWA–6000:

(*1*) A description of the type and estimated extent of degradation, and the conditions that led to the degradation;

(*2*) An evaluation of each area, and the result of the evaluation, and;

(*3*) A description of necessary corrective actions.

(F) Personnel that examine containment concrete surfaces and tendon hardware, wires, or strands must meet the qualification provisions in IWA–2300. The "owner-defined" personnel qualification provisions in IWL–2310(d) are not approved for use.

(G) Corrosion protection material must be restored following concrete containment post-tensioning system repair and replacement activities in accordance with the quality assurance program requirements specified in IWA–1400.

3.1.3 Guidance for Plants Operating for More Than 40 Years

USNRC issued Generic Aging Lessons Learned (GALL) report [12] to provide guidance to licensees applying for license renewal to extend period of operation beyond the initial limit of 40 years. GALL report lists generic aging management reviews (AMRs) of systems, structures, and components (SSCs) that may be in the scope of license renewal applications (LRAs) and identifies aging management programs (AMPs) that were determined to be acceptable to manage aging effects of SSCs in the scope of license renewal, as required by 10 CFR Part 54, "Requirements for Renewal of Operating Licenses for Nuclear Power Plants." For inspection and managing inspection of PCCs concrete and prestressing system, GALL report has included aging management program (AMP) XI.S2, "ASME Section XI, Subsection IWL." This program complies with the requirements of ASME Section XI, Subsection IWL as endorsed and amended by 10CFR50.55a(b)(Viii). A copy of this AMP is included as Appendix B of this report.

3.2 PCCS WITH GROUTED TENDONS

In the USA, only one plant uses bonded tendons. The prestress is provided only in the vertical direction in containment shell using bar tendons. These tendons are protected by cement grout which creates an alkaline environment that inhibits corrosion and prevents ingress and circulation of corrosive fluids around the steel tendons. The main reason for not using grouted tendons in PCCs in USA is the inability to inspect the tendons over the long term to assess the structural integrity of the PCC. However, grouted tendons are used in PCCs for plants in France, Belgium, Canada, Korea, China, Sweden, and other nations. USNRC issued regulatory guide RG 1.90, Rev. 0, in 1974. Revisions 1 and 2 of this guide was revised and issued in 1977 and 2012 [13] respectively. None of the plants in USA has used RG 1.90 for monitoring prestress in grouted tendons. However, Marneffe et al. reports the successful use of RG 1.90, Revision 1 in a number of PCCs in Belgium [14].

RG 1.90, Rev. 2 recommends an in-service inspection program for PCC with grouted tendons consisting of the following three elements.

1. Force monitoring of ungrouted test tendons
2. Monitoring the prestress level using instrumentation and pressure testing or monitoring deformation under pressure
3. Visual examination

3.2.1 Force Monitoring of Ungrouted Test Tendons

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

   (1) For PCCs with shallow dome install three tendons each in vertical and hoop directions, and three tendons in the dome.

   (2) For PCCs with spherical dome install three hoop and four inverted U tendons.

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

c. The ungrouted test tendons should be subjected to force measurement by lift-off testing and inspection of concrete in accordance with ASME Section XI, Subsection IWL.

3.2.2 Monitoring Alternatives for Performance of Grouted Tendons

RG 1.90 provides two distinct alternatives for monitoring of grouted tendons. The first requires monitoring of prestress level in the PCC with strategically located instrumentation such as strain gauges, stress meters, load indicators. The second alternative requires the monitoring of PCC deformations at critical locations under prescribed pressures.

3.2.3 Monitoring Using Instrumentation and Pressure Testing (Option A)

This option is applicable if the instrumentation such as strain gauges and other devices are installed during the construction phase. Approximately 250 to 300 instruments are required. Initial baseline of strains and deformations in the PCC are recorded during structural integrity test (SIT) as well as during pre-operational integrated leak rate test (ILRT). After SIT, the instruments are monitored periodically at a frequency of 2 to six months. In addition, the containment performance, under design accident pressure, is verified periodically by at 1, 3, 5 year intervals. The frequency is relaxed to 10 years if the installed instrumentation is effective at monitoring prestress levels in concrete.

3.2.4 Monitoring Containment Deformation under Pressure Test

In this option, the PCC is subjected to design accident pressure and deformations are recorded to measure displacements in horizontal direction as well as vertical directions as follows:

a. Measure radial displacements in six horizontal planes in the cylindrical portion with a minimum of four locations in each plane

b. Vertical displacement between the base and top of cylinder at four locations.

c. At the apex of the dome and one intermediate location between the apex and springline on at least three locations.

None of the nuclear operating plants in USA has used RG 1.90 for monitoring prestress in grouted tendons. The RG 1.90 may be applied for new reactors such EPR reactor that is currently in the initial licensing process. However, Marneffe et al. reports the successful use of RG 1.90, Revision 1 in a number of PCCs in Belgium [14]. PCC of the H. B. Robinson Nuclear Plant, is the only plant grouted prestressed tendons. The grouted tendons are only used in the vertical direction between the base slab and dome spring line. Containment integrity tests were performed in 1970, 1974, and 1992. To provide additional assurance during the license renewal for up to 60 years operation, the licensee committed to the USNRC to perform pressure testing, at integrated leak rate test pressure, similar to structural integrity test performed in 1992 to coincide with integrated leak rate testing every 10 years. During the test deformations and cracking associated with the vertical prestressed tendons and will not include radial or axial monitoring. The results of the pressure tests will be used in conjunction with the analytical determination of tendon prestress, the established corrosion resistance of embedded tendons, and previously completed structural integrity tests, and ongoing inspection of concrete.

4 Trending Prestressing Forces

As discussed above in Section 4.1.2 above, USA Regulations in 10CFR50.55a(b)[10] require that, ”When evaluation of consecutive surveillances of prestressing forces for the same tendon or tendons in a group indicates a trend of prestress loss such that the tendon force(s) would be less than the minimum design prestress requirements before the next inspection interval, an evaluation must be performed and reported in the Engineering Evaluation Report as prescribed in IWL–3300.” In addition, ASME Section XI [11], Article IWL-3221.1 require that average of all measured tendon forces, those measured in IWL-3221.1(b)(2), for each type tendon is equal to or greater than the minimum required prestress, and the measured force in each individual tendon is not less than the 95% of the predicted force unless additional conditions are satisfied.

To comply with above listed requirements, USNRC has issued guidance in RG 1.35.1[5]. Essentially the RG 1.35.1 requires constructing the upper and lower bound of the prestressing forces, based on variation of in the time dependent loss of prestressing forces due to creep, shrinkage, and relaxation of steel as described in Section 4.0 above, and comparing it to actual lift off forces recorded during tendon surveillances over time. This process called of trending of prestressing forces.

To start the trending process, following data is collected:

1. Tendons selected for surveillance or inspection from a group (hoop or vertical or inverted U or dome)
2. Predicted prestress force in a group of surveillance tendons at one year and 40 years after SIT, calculated in accordance with procedure described in Section 3.2 above.
3. Predicted lower bound prestress force in a group of surveillance tendons at one year and 40 years after SIT, calculated in accordance with procedure described in Section 3.2 above.
4. Predicted upper bound prestress force in a group of surveillance tendons at one year and 40 years after SIT, calculated in accordance with procedure described in Section 3.2 above.
5. Average of lift off force from two ends in tendon observed during surveillance or inspection.

From the data collected in items 2 thru 4, determine the average predicted prestress force, average predicted lower bound prestress force, average predicted upper bound prestress force in a group of selected tendons at one year and 40 years after SIT.

This data collected for a PCC is shown in Table 5 below.

Time (Years) After SIT T	**Minimum Required Value (MRV)**	**Predicted Prestress LOWER BOUND**	**Predicted Prestress AVERAGE**	**Predicted Prestress UPPER BOUND**	**Lift off Force**
1.00	1167.00	1404.00	1470.40	1529.80	☐
3.00	1167.00	1380.13	1449.17	1511.36	☐
4.18	1167.00	1372.92	1442.77	1505.80	1495.00
4.18	1167.00	1372.92	1442.77	1505.80	1453.00
4.18	1167.00	1372.92	1442.77	1505.80	1473.50
5.00	1167.00	1369.03	1439.30	1502.79	☐
6.19	1167.00	1364.37	1435.16	1499.20	1535.00
6.19	1167.00	1364.37	1435.16	1499.20	1495.00
6.19	1167.00	1364.37	1435.16	1499.20	1501.00
7.77	1167.00	1359.44	1430.78	1495.39	1500.00
7.77	1167.00	1359.44	1430.78	1495.39	1470.00
7.77	1167.00	1359.44	1430.78	1495.39	1484.00

10.00	1167.00	1353.97	1425.91	1491.16	☐
18.80	1167.00	1340.25	1413.71	1480.56	1470.00
18.80	1167.00	1340.25	1413.71	1480.56	1452.00
18.80	1167.00	1340.25	1413.71	1480.56	1465.00
20.00	1167.00	1338.90	1412.52	1479.53	☐
30.00	1167.00	1330.09	1404.68	1472.72	☐
40.00	1167.00	1323.84	1399.12	1467.89	☐
50.00	1167.00	1318.99	1394.81	1464.15	☐
60.00	1167.00	1315.03	1391.29	1461.09	☐

Table 5 average predicted prestress force, average predicted lower bound prestress force, average predicted upper bound prestress force in a group of selected tendons at one year and 40 years after SIT.

The values in table 5 are plotted in Figure 3 to develop trend lines based on measured lift-off forces during surveillances. Based on extensive database of 11 nuclear plants in United Kingdom, Irving et al. established a relationship between the prestressing force and time [15]. This conclusion is used to perform linear regression analysis based on the sum of least square method as recommended in USNRC Information Notice 99-10 [16]. This process can be conveniently accomplished using Excel program. Figure 3 shows the trend line using linear regression analysis using Excel program.

A review of Figure 3 indicates that the following:

1. Lift off forces measured during inspections/surveillances are greater than the minimum require value (MRV), and comply with ASME Section XI, IWL-3221.1(a)
2. Lift off forces measured during inspections/surveillances are greater than 95% of the predicted force, and comply with ASME Section XI, IWL-3221.1(b).
3. Prestress force trend line obtained from regression analysis indicate that the predicted force in the tendon will be greater than the MRV at the next scheduled outage, and comply with ASME Section XI, IWL-3221.1(b).
4. Prestress force trend line obtained from regression analysis indicate that the predicted force in the tendon in the tendon will be greater than the MRV until the next inspection interval or approximately 24 years after SIT. Therefore, an evaluation report as required by 10CFR50.55a(b)(viii)(B) is not required.

Based on the above, it can be concluded that tendon forces identified in group of surveillance tendons meets ASME Section XI, Subsection IWL and 10CFR50.55a requirements and regulations.

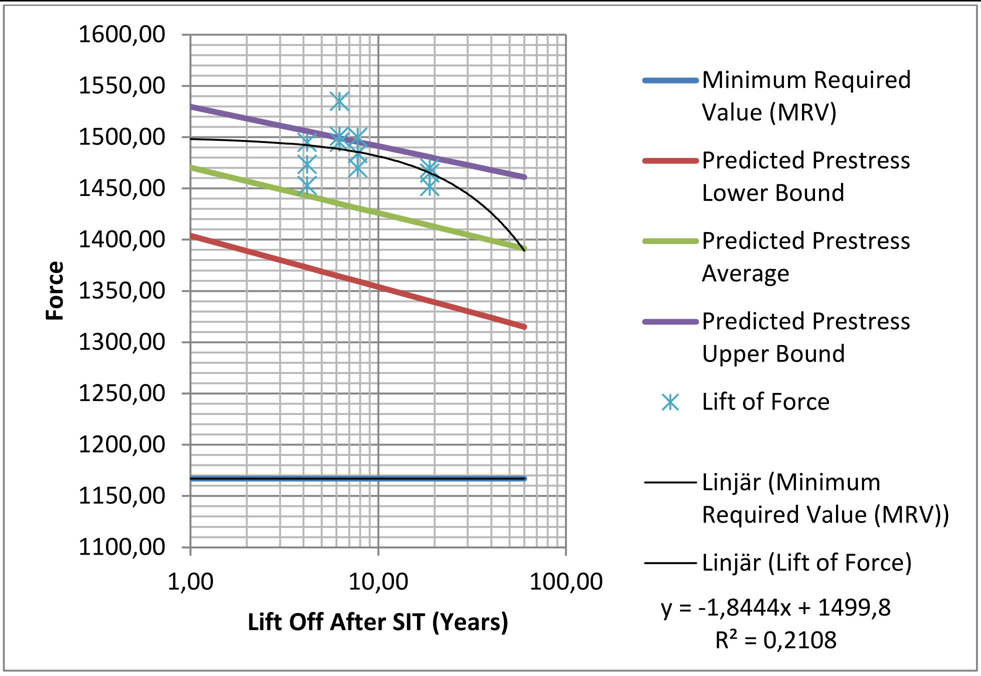

Figure 3 Lift off force versus time in logarithmic scale.

5 Leak Rate Testing of Containment

5.1 US REGULATIONS FOR LEAK RATE TESTING

General Design Criteria (GDC) for the USA nuclear power plants are specified in 10CFR50, Appendix A. GDC 16, 52, 53, and 54 have basic requirements for containment design and testing.

GDC 16 states, "*Containment design*. Reactor containment and associated systems shall be provided to establish an essentially leak-tight barrier against the uncontrolled release of radioactivity to the environment and to assure that the containment design conditions important to safety are not exceeded for as long as postulated accident conditions require."

GDC 52 states, "*Capability for containment leakage rate testing*. The reactor containment and other equipment which may be subjected to containment test conditions shall be designed so that periodic integrated leakage rate testing can be conducted at containment design pressure."

GDC 53 states, "*Provisions for containment testing and inspection*. The reactor containment shall be designed to permit (1) appropriate periodic inspection of all important areas, such as penetrations, (2) an appropriate surveillance program, and (3) periodic testing at containment design pressure of the leak tightness of penetrations which have resilient seals and expansion bellows."

GDC 54 states," *Piping systems penetrating containment*. Piping systems penetrating primary reactor containment shall be provided with leak detection, isolation, and containment capabilities having redundancy, reliability, and performance capabilities which reflect the importance to safety of isolating these piping systems. Such piping systems shall be designed with a capability to test periodically the operability of the isolation valves and associated apparatus and to determine if valve leakage is within acceptable limits."

Appendix J of 10CFR50, "Primary Reactor Containment Leakage Testing of Water-Cooled Power Reactors," establishes the testing requirements and acceptance criteria for preoperational and periodic tests to verify the leak tight integrity of the primary containments, including systems and components that penetrate the containments. All USA plants perform leak rate testing in accordance with Appendix J of 10CFR50 to confirm reliability and performance capability to comply with GDC. Leak tightness tests of containment ensure that containment acts as a leak tight barrier during the normal operating and environmental stressors such as temperature and pressure variation, earthquake, and windstorms. The containment testing also confirm that the materials used in construction of containments such as concrete, steel, prestressing system, electrical and piping penetrations, and personal and equipment air locks have not degraded to an extent that the containment will not be able to perform its leak tight integrity during an accident. Furthermore, these tests can also identify degradations in containment structure so that proper maintenance and repairs are made during the service life of the containment, and systems and components penetrating primary containment.

Appendix J have two options for containment leak rate tests. Option A –deterministic, and Option B – performance based. Option A require containment leakage tests to be performed at a definite frequency. Option B allows licensees, with a satisfactory performance history (two consecutive successful tests), to reduce the test frequency. For instance, after two successful integrated leak rate tests (lIRT), the test frequency is reduced from three tests in 10 years to one test in 15 years. The frequency requirements for different type of tests for both options are described below. Both options require performance of three types of tests:

- Type A - Containment integrated leak rate test (ILRT)
- Type B – Containment penetrations leak rate test
- Type C – Containment isolation valve leak rate test

Type B and C tests are also called as local leak rate tests (LLRT)

5.1.1 Explanation of Terms

Appendix J has many terms that are used to describe the requirements for different tests. Prior to describing the technical and frequency of different tests, it is necessary to be familiar with these terms. These terms from 10CFR 50, Appendix J are copied below:

" II. Explanation of Terms

A. "Primary reactor containment" means the structure or vessel that encloses the components of the reactor coolant pressure boundary, as defined in § 50.2(v), and serves as an essentially leak-tight barrier against the uncontrolled release of radioactivity to the environment.

B. "Containment isolation valve" means any valve which is relied upon to perform a containment isolation function.

C. "Reactor containment leakage test program" includes the performance of Type A, Type B, and Type C tests, described in II.F, II.G, and II.H, respectively.

D. "Leakage rate" for test purposes is that leakage which occurs in a unit of time, stated as a percentage of weight of the original content of containment air at the leakage rate test pressure that escapes to the outside atmosphere during a 24-hour test period.

E. "Overall integrated leakage rate" means that leakage rate which obtains from a summation of leakage through all potential leakage paths including containment welds, valves, fittings, and components which penetrate containment.

F. "Type A Tests" means tests intended to measure the primary reactor containment overall integrated leakage rate (1) after the containment has been completed and is ready for operation, and (2) at periodic intervals thereafter.

G. "Type B Tests" means tests intended to detect local leaks and to measure leakage across each pressure-containing or leakage-limiting boundary for the following primary reactor containment penetrations:

1. Containment penetrations whose design incorporates resilient seals, gaskets, or sealant compounds, piping penetrations fitted with expansion bellows, and electrical penetrations fitted with flexible metal seal assemblies.
2. Air lock door seals, including door operating mechanism penetrations which are part of the containment pressure boundary.
3. Doors with resilient seals or gaskets except for seal-welded doors.
4. Components other than those listed in II.G.1, II.G.2, or II.G.3 which must meet the acceptance criteria in III.B.3.

H. "Type C Tests" means tests intended to measure containment isolation valve leakage rates. The containment isolation valves included are those that:

1. Provide a direct connection between the inside and outside atmospheres of the primary reactor containment under normal operation, such as purge and ventilation, vacuum relief, and instrument valves;
2. Are required to close automatically upon receipt of a containment isolation signal in response to controls intended to effect containment isolation;
3. Are required to operate intermittently under post-accident conditions; and
4. Are in main steam and feed water piping and other systems which penetrate containment of direct-cycle boiling water power reactors.

I. Pa means the calculated peak containment internal pressure related to the design basis accident and specified either in the technical specification or associated bases.

J. Pt means the containment vessel reduced test pressure selected to measure the integrated leakage rate during periodic Type A tests.

K. La (percent/24 hours) means the maximum allowable leakage rate at pressure Pa as specified for preoperational tests in the technical specifications or associated bases, and as specified for periodic tests in the operating license or combined license, including the technical specifications in any referenced design certification or manufactured reactor used at the facility.

L. Ld (percent/24 hours) means the design leakage rate at pressure, Pa, as specified in the technical specifications or associated bases.

M. Lt (percent/24 hours) means the maximum allowable leakage rate at pressure Pt derived from the preoperational test data as specified in III.A.4.(a)(iii).

N. Lam, Ltm (percent/24 hours) means the total measured containment leakage rates at pressure Pa and Pt, respectively, obtained from testing the containment with components and systems in the state as close as practical to that which would exist under design basis accident conditions (e.g., vented, drained, flooded or pressurized).
O. "Acceptance criteria" means the standard against which test results are to be compared for establishing the functional acceptability of the containment as a leakage limiting boundary."

5.1.2 Option A Testing

Nuclear power plants initially used Option A for conducting containment leak rate tests. However, since Option A testing is prescriptive, most of the nuclear power plants switched to Option B testing after the USNRC approved its use. The frequency of testing for option B is normally lower than that for Option A testing. The frequency of testing and acceptance criteria for Option A testing is noted below.

6.1.2.1 Type A Test

10CFR50, Appendix J requires that All Type A tests shall be conducted in accordance with the provisions of the American National Standards N45.4-1972, "Leakage Rate Testing of Containment Structures for Nuclear Reactors," March 16, 1972 [17]. In addition to the Total time and Point-to-Point methods described in that standard, the Mass Point Method, when used with a test duration of at least 24 hours, is an acceptable method to use to calculate leakage rates. A typical description of the Mass Point method can be found in the American National Standard ANSI/ANS 56.8-1987, "Containment System Leakage Testing Requirements." [18].

Frequency: After the preoperational leakage rate tests, three Type A tests be performed at approximately equal intervals during each 10 year service period.

Test Pressure: Test shall be conducted at Pa.

Acceptance Criteria: The leakage rate Lam shall be less than 0.75 La. If local leakage measurements are taken to effect repairs in order to meet the acceptance criteria, these measurements shall be taken at a test pressure Pa.

6.1.2.2 Type B Test

Frequency: Type B tests to be performed during each reactor shutdown or refueling, but in no case at interval greater than 2 years. For containments employing continuous leakage monitoring, Option A requires that type B tests to be performed every other reactor shutdown for refueling or every three years, whichever is less. Air locks are required to be tested every 6 months.

Test Pressure: Type B tests shall be performed by local pneumatic pressurization of the containment penetrations, either individually or in groups, at a pressure not less than Pa.

Acceptance Criteria: The combined leakage rate of all penetrations and valves subject to Type B and C tests shall be less than 0.60 La.

6.1.2.3 Type C Test

Frequency: Type C testing to be performed every refueling outage or every 2 years, whichever is less.

Test Pressure: Valves, unless pressurized with fluid (e.g., water, nitrogen) from a seal system, shall be pressurized with air or nitrogen at a pressure of Pa. Valves, which are sealed with fluid from a seal system shall be pressurized with that fluid to a pressure not less than 1.10 Pa.

Acceptance Criteria:	The combined leakage rate for all penetrations and valves subject to Type B and C tests shall be less than 0.60 La. Leakage from containment isolation valves that are sealed with fluid from a seal system may be excluded when determining the combined leakage rate under certain conditions.

6.1.2.4 Containment Modification

10 CFR Appendix J require that any major modification, replacement of a component which is part of the primary reactor containment boundary, or resealing a seal-welded door, performed after the preoperational leakage rate test shall be followed by either a Type A, Type B, or Type C test, as applicable for the area affected by the modification. ASME Section XI, Subsection IWL require that the pressure test, after major modification, such as steam generator replacement, shall be conducted at the design basis accident pressure, Pa.

5.1.3 Option B Testing

As described above, all nuclear power plants, except one or two, have revised their Technical Specifications and now use Option B for containment testing because it is less prescriptive and results in reduced number and frequency of tests. 10CFR50, Appendix J has the basic requirements for Type A, B, and C tests. The detailed requirements for implementing Option B testing are provided in Nuclear Energy Institute NEI 94-01, Rev. 3 [19] and endorsed by USNRC in Regulatory Guide RG 1.163 [20] and supplemented by USNRC's staff safety evaluation reports.

6.1.3.1 Appendix J Requirements

Appendix J has following requirements for Option B testing:

A. Type A Test

Type A tests to measure the containment system overall integrated leakage rate must be conducted under conditions representing design basis loss-of-coolant accident containment peak pressure. A Type A test must be conducted (1) after the containment system has been completed and is ready for operation and (2) at a periodic interval based on the historical performance of the overall containment system as a barrier to fission product releases to reduce the risk from reactor accidents. A general visual inspection of the accessible interior and exterior surfaces of the containment system for structural deterioration which may affect the containment leak-tight integrity must be conducted prior to each test, and at a periodic interval between tests based on the performance of the containment system. The leakage rate must not exceed the allowable leakage rate (La) with margin, as specified in the Technical Specifications. The test results must be compared with previous results to examine the performance history of the overall containment system to limit leakage.

B. Type B and C Tests

Type B pneumatic tests to detect and measure local leakage rates across pressure retaining, leakage-limiting boundaries, and Type C pneumatic tests to measure containment isolation valve leakage rates, must be conducted (1) prior to initial criticality, and (2) periodically thereafter at intervals based on the safety significance and historical performance of each boundary and isolation valve to ensure the integrity of the overall containment system as a barrier to fission product release to reduce the risk from reactor accidents. The performance-based testing program must contain a performance criterion for Type B and C tests, consideration of leakage-rate limits and factors that are indicative of or affect performance, when establishing test intervals, evaluations of performance of containment system components, and comparison to previous test results to examine the performance history of the overall containment system to limit leakage. The tests must demonstrate that the sum of the leakage rates at accident pressure of Type B tests, and pathway leakage rates from Type C tests, is less than the performance criterion (La) with margin, as specified in the Technical Specification."

6.1.3.2 NEI 94-01 Guidance

NEI 94-01, describes an acceptable approach for implementing the optional performance-based requirements of Option B to 10 CFR 50, Appendix J; includes provisions for extending Type A ILRT intervals to up to fifteen years and incorporates the regulatory positions stated in Regulatory Guide 1.163 [20]. It delineates a performance-based approach for determining Type A, Type B, and Type C containment leakage rate surveillance testing frequencies. Justification of extending test intervals is based on the performance history and risk insights.

NEI 94-01 provides guidance for determining frequency of type A, B, and C tests only. This document does not address how to perform the tests. The details for performing the test are described in ANSI/ANS-56.8 [18].

According to NEI 94-01, the interval for Type A test can be increased to 15 years after two consecutive successful Type A in which leakage rate is less than 1.0La and some other additional requirements such as additional visual examination and plant specific risk assessment. In practice, nuclear power plant operators have found that leakage rate requirement of 1.0La and additional requirements controlling the test frequency easy to comply. Therefore, most of the nuclear power plants have changed or in the process of getting approval for changing their Technical Specifications to increase test interval period to 15 years.

Extensions of Type B and Type C test intervals are also allowed in NEI 94-01 based upon completion of two consecutive periodic as-found tests where the results of each test are within a licensee's allowable administrative limits. Intervals may be increased from 30 months up to a maximum of 120 months for Type B tests (except for containment airlocks) and up to a maximum of 75 months for Type C tests. If the Type B and Type C test results are not acceptable; the test frequency has to be set at the initial test intervals. However, once the cause determination and corrective actions have been completed, acceptable performance may be reestablished and the testing frequency returned to the extended intervals as specified in this document. Containment airlock(s) are required to be tested at an internal pressure of not less than Pa prior to a preoperational Type A test.

Subsequent periodic tests to be performed at a frequency of at least once per 30 months. When containment integrity is required, airlock door seals should be tested within 7 days after each containment entry. NEI 94-01, Rev.3, has detailed requirements for implementing Option B testing.

6 US Containments Operating Experience

The performance of the concrete (reinforced and prestressed) containments in USA has been very good. However, there has been several incidences reported about degradations due to environmental effects and are described below.

6.1 POST TENSIONING SYSTEM

6.1.1 Tendon Anchor Head Corrosion

NRC issued Information Notice 99-10 [16] to inform the licensees of the potential degradation of prestressed tendon at the anchor heads of the Calvert Cliffs and Joseph M. Farley Nuclear Plants. In addition, in 2012, anchor head of one the hoop tendons broke and fell out of the grease cap at Joseph M. Farley Nuclear Plant.

Degradation at Calvert Cliffs, Unit 1 plant, was found during the 20th-year surveillance of the prestressing system in June-July 1997 while investigating the cause of low lift-off force as compared to predicted value for one of the three randomly selected vertical tendons. The low liftoff value was attributed to the uneven shim stack heights on the two opposite sides of the anchor-head. In accordance with the requirement in the plant's Technical Specifications (TSs), the licensee tested two additional vertical tendons adjacent to this tendon. However, during the liftoff testing of one of these tendons, noises were heard that indicated that some of the tendon wires might have broken. A visual examination of the tendon showed that three wires had broken at 12-17 centimeters (5-7 inches) below the bottom of the button-heads. Further examination of the wires at the top of other vertical tendons revealed additional wire breakage. The licensee expanded the liftoff testing and visual examination to 100 percent of the vertical tendons. Similar degradation of other vertical tendons was found. As a part of its corrective action, the licensee replaced 63 of the 202 vertical tendons in Unit 1 and 64 of the 204 vertical tendons in Unit 2. Licensee engineering evaluation determined that root cause of the failure was brittle hydrogen induced cracking. All of the brittle fractures were preceded by severe corrosion that was caused due to ingress of water into the tendon grease caps at the top of the containment. The profile of the concrete at the top was such that it allows accumulation of standing water at the top adjacent to the grease caps.
NRC Information Notice 85-10 [21], and its supplement of March 1985, "Post-Tensioned Containment Tendon Anchor-Head Failure," described prestressing tendon anchor-head failures at both units of the Joseph M. Farley Nuclear Plant. The root cause analysis of that event indicated that there were several factors contributing to it, such as free water in the grease caps at the bottom of the vertical tendons, very hard anchorage material, and high stresses in the anchor-heads. The failures had resulted from hydrogen embrittlement of the anchor-head material. The free water in the bottom grease caps of the vertical tendons may have accumulated (over a number of years) from the poorly drained top anchorage ledge of the vertical tendons (similar to the condition at the Calvert Cliffs containments). However, at Farley, wires failures did not occur.

On May 3, 2012, a loud "bang" noise was heard in the control room of the Unit 1 Farley Nuclear Plant. After further inspection, the licensee found one of the containment tendon end caps had blown out and the tendon is coming out. The licensee determined that a horizontal hoop tendon had broken and the relaxation force dislodged the tendon cover and the tendon sprung out of the containment wall into the auxiliary building (Figure 4). The component that failed was the field-end anchor head of the horizontal hoop tendon.

The failed tendon is part of the post-tensioning system for the Unit 1 Farley Nuclear Plant containment building. The post-tensioning system consists of horizontal, dome, and vertical tendons. A total of one hundred thirty-five (135) horizontal tendons are anchored at three vertical buttresses. Three groups of dome tendons, for a total of ninety-three (93) tendons, are anchored at the vertical face of the dome ring girder. One hundred thirty (130) vertical tendons are anchored at the top surface of the ring girder and at the bottom of the base slab. The number of tendons is the same for both Units 1 and 2 containment building, except one horizontal tendon was not installed on Unit 2 during construction; therefore, there are only one hundred thirty-four (134) horizontal tendons in Unit 2 containment building. The Licensee's root cause investigation, performed due to the anchor head failure, called for the replacement of one (1) failed tendon with its field anchor head and the removal, testing, and replacement of fourteen (14) additional field anchor heads. A total of fifteen (15) different tendons from Units 1 and 2 were affected by the anchor head replacement activities. The failed tendon in Unit 1 was completely replaced. The other fourteen (14) tendons were temporarily de-tensioned, the field end anchor head replaced, and subsequently re-tensioned. The root cause of the failure was hydrogenization which sets up a stress cracking corrosion in the anchor head.

One of the vertical prestressed tendon's anchor at Bellefonte Unit 1 containment also failed with a loud noise similar to Farley containment. The root cause again identified stress corrosion cracking of high strength steel of the anchor.

6.1.2 High Tendon Wire Relaxation

During containment prestress tendon surveillance activities in 1977 at R.E. Ginna Nuclear Power Plant [22], it was discovered that the average compressive lift-off force of the surveillance tendons (7 tendons out of a total of 160 tendons or 4 percent of the total) had decreased to a value marginally above the design requirement of 636 kips. A 10-year retest was performed in 1979 and the marginal force values confirmed. In 1980, a total of 137 tendons were retensioned. The other 23 tendons were re-tensioned previously in 1969. Subsequent surveillance testing has demonstrated that all tendons have met operability criteria. Investigation of the cause of the loss of prestress was undertaken at the Fritz Engineering Laboratory of Lehigh University. An extensive testing program was conducted with two primary objectives. The first objective was to determine the root cause of the loss of prestress, and the second was to determine the effect of retensioning at various times after initial stressing on subsequent loss of prestress. The results of the testing program determined that the principal cause of the loss of prestress in the wall tendons was stress relaxation. An increase in temperature from ambient conditions to operating conditions significantly increases the amount of stress relaxation over time. For example, at a temperature of 104°F after 40 years the stress relaxation in the tendon would be expected to be as high as 21% as opposed to 12% as originally predicted and based on laboratory tests performed at 70°F. The retensioned tendons exhibit considerably less stress relaxation than initially tensioned tendons.

At the Virgil C. Summer Nuclear Station [23], the test results from the first three tendon surveillance's (1982, 1983, 1985) indicated that the wire relaxation force losses in the tendon system were greater than that which were predicted during design. Consequently, in June 1988, the predicted wire relaxation force losses were increased from 8.5 percent to 12.8 percent. The fourth period (10th year) tendon surveillance was performed during

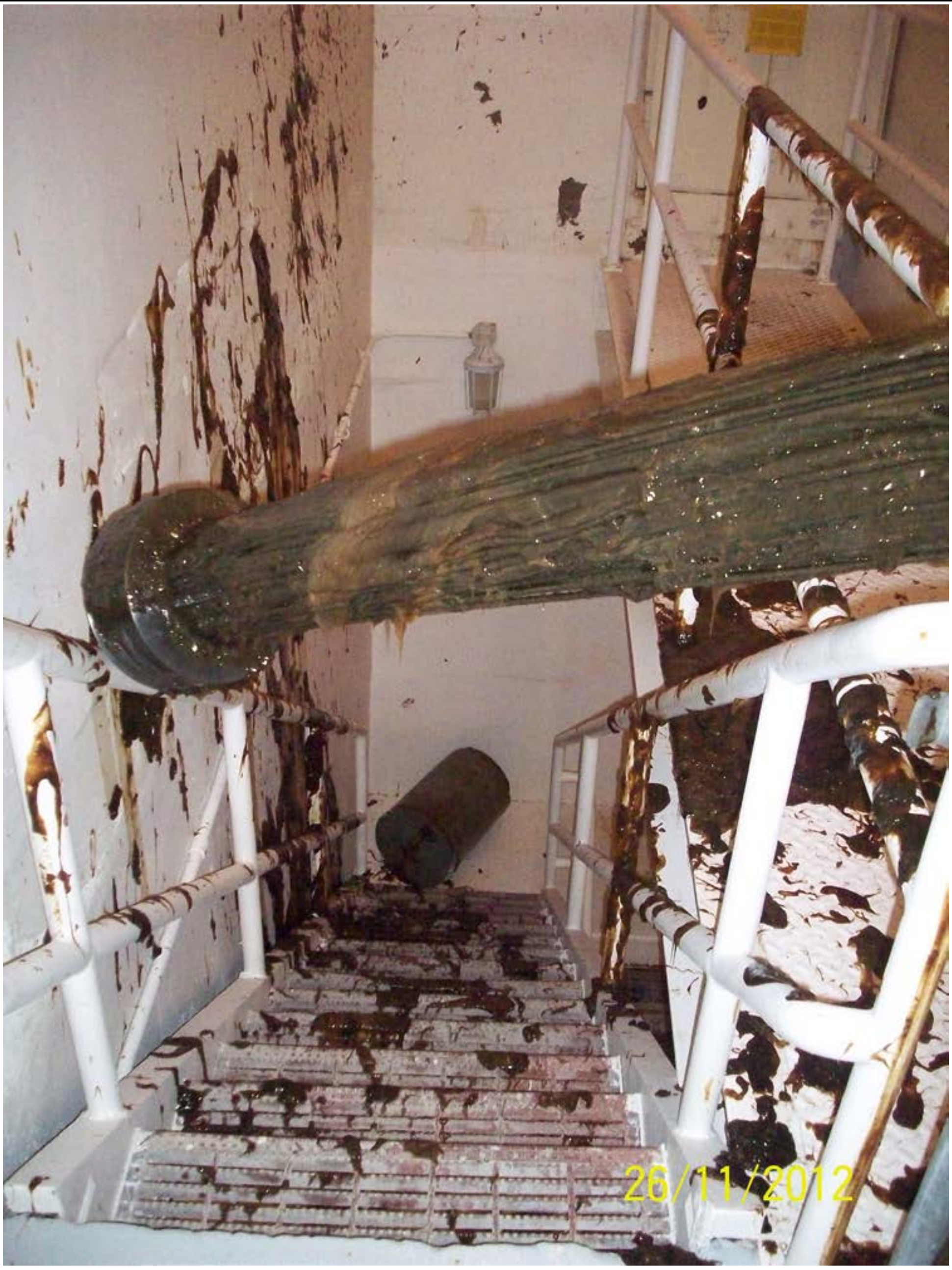

Figure 4 – Farley Tendon Failure

January-April 1990. In addition, the vertical tendons were retensioned because the previous surveillance data indicated that the vertical tendon forces would be below the technical specifications minimum prior to the fifth period surveillance. The reports of the next three surveillance periods in 1990, 1996, and 2000 have each concluded that no abnormal degradation of the post-tensioning system has occurred at the Virgil C. Summer Nuclear Station.

Similarly, at the Turkey Point Nuclear Plant Units 3 and 4 [24], the measured lift-off forces for a number of randomly selected surveillance tendons were below the predicted lower limit. Condition Reports and a Licensee Event Report were issued. In accordance with the Technical Specifications, engineering evaluations were prepared and concluded that the lower than expected tendon lift-off forces were caused by greater than expected tendon wire relaxation losses due to average tendon temperatures higher than originally considered. To accommodate the increased prestress losses, a license amendment was submitted and approved to reduce the containment design pressure from 0.41 MPa (59 psig) to 0.38 MPa (55 psig), and a containment reanalysis was performed to determine the new minimum required prestress forces to maintain Turkey Point licensing-basis requirements. The Crystal River Unit 3 plant had also higher than predicted loss due to relaxation of steel.

All the four nuclear plants, R.E. Ginna, Virgil C. Summer, Turkey Point, and Crystal River are older plants and used prestressing wires fabricated from steel with high relaxation. The plants built later on have used prestressing wires or tendons with low relaxation of steel and/or have considered elevated temperature effects during the design of containment prestressing system.

6.1.3 Grease Leakage

Grease has leaked from the prestressed concrete tendon ducts at several plants during the initial greasing operation as well as during plant operation in USA as well as other countries. At one plant, during the greasing operation, a large volume of grease leaked from the tendon duct through the concrete into the auxiliary building until it was detected and greasing operation was suspended. The concrete around the leakage area was chipped and the tendon duct and leakage was found to be from a tendon duct joint. The tendon duct joint was repaired, concrete replaced, and greasing operation was successfully completed.

At another plant grease leaked from one duct into an adjacent duct during greasing operation conducted after steam generator replacement activities. To resolve, this situation, grease was simultaneously into the two ducts. Grease has also been detected at the surface of containment concrete during plant operations at several locations, especially for plants located in warmer climate. Grease leakage is also common from the vertical tendon's grease caps in the containment tendon gallery due to failure of failure of the grease caps gaskets.

During tendon surveillance activities, at some plants, the absolute difference between the amount of grease removed and the amount replaced exceeds more than 5%. USNRC RG 1.35 [9] requires that licensee shall report to the commission if the difference is greater than 5%; however, 10CFR50.55a [10] and ASME Section XI, Article IWL-3221 allows this difference to be up to 10% of the tendon net duct volume. This justified because the physical characteristics of the grease material and industry standard installation techniques, voids up to approximately 15 percent could be expected after the initial filling operation. Voids in the tendon sheathing may be attributed to a number of factors:

1. Visconorust 2090P-4 (grease filler material used in the tendon) has a coefficient of expansion which yields a contraction of about 1 percent per every 200 Fahrenheit. Initial filling temperatures of the filler material averaged 1600 Fahrenheit. Cold weather conditions can cool the filler material to 400 Fahrenheit, giving a contraction of 6% of the net duct volume.

2. Voids between the wires that comprise the tendon bundle and in other areas, such as where wires are in contact with the sheathing, may yield about 7% percent, or greater, of the net duct volume.

3. Characteristics of the initial filling method may induce air entrapment into the filler material. Pumping operations can introduce air into filler material, and may add up to as much as 2 percent of the net duct volume.

4. Studies conducted by the licensees have concluded that small amount of grease leaked from the tendon duct does not adversely affect the concrete containment integrity.

6.2 CONCRETE DEGRADATION

Inspection of containment concrete external surface is performed in accordance with ASME Section XI, Subsection IWL(9) at 1, 3, and 5 years following the completion of the containment Structural Integrity Test and every 5 years thereafter. Nominal cracking and surface discoloration due to exposure of reinforcement, especially at Cadweld splice sleeves, have been observed at some US nuclear plant reinforced concrete containments. However, no plant in USA has so far identified concrete cracking due to carbonation. The depth of carbonation has not exceeded more than 20 mm (3/4 inch). The only plant that has severe problems due to carbonation is Koeberg nuclear station in South Africa.

Normal cracking of concrete and surface discoloration due to corrosion of reinforcement is not usually encountered at prestressed concrete containments because concrete is under compression and volume of non-prestressed reinforcement is comparatively small. Detailed guidance provided American Concrete Institute (ACI) standard 349, "Evaluation of Existing Nuclear Safety Related Concrete Structures," [25] is used by the licensees to identify, disposition, and repair irregularities in the containment concrete.

In the last few years severe degradation of concrete has been identified at two nuclear power containment structures. Delamination of containment concrete cylindrical shell occurred at Crystal River 3 Nuclear Power Plant and alkali silica reaction (ASR) has been observed at Seabrook Station.

To facilitate replacement of steam generator a temporary construction opening was created in the Crystal River 3 containment. This required removal of prestressed tendons in the area opening and removal of concrete using hydro-demolition process. Following removal of concrete to form the opening, plant personnel identified a delamination adjacent to the construction opening (Figure 5). After extensive analyses and testing, the root cause of the delamination was due to combination and interplay of lack of radial reinforcement, type of concrete used with low tensile strength, and acts of de-tensioning and cutting of containment concrete. The immediate technical root cause was redistribution of stresses, as a result of the containment opening activities, resulting in additional stresses beyond original design of the containment.

The plant removed concrete from a large area (Figure 6), detensioned large number of tendons around the opening, poured new concrete, and devised an elaborate process for retensioning the tendons. However, this effort was not successful. Chunks of concrete fell during the re-tensioning operations. The Crystal River 3 plant has been now shutdown permanently.

As a part of license renewal activities and its assessment of plant structures at Seabrook Station, the licensee performed inspections of safety related structures and presence of groundwater and visual indications of mapped pattern of cracking that was indicative of alkali silica reactions in concrete (Figures 7 and 8). This pattern of cracking was also observed in the outer containment wall. The occurrence of alkali silica reaction was confirmed by petrographic examination. The plant has performed extensive walkdowns, analyses and large scale testing of concrete prototype structures to identify the impact of alkali silica reaction on the structural integrity of the structures, including containment building. This work is still in progress. The root cause was determined to be slow reacting coarse alkali aggregates which could not detected by the tests conducted and approved for use during the time of construction.

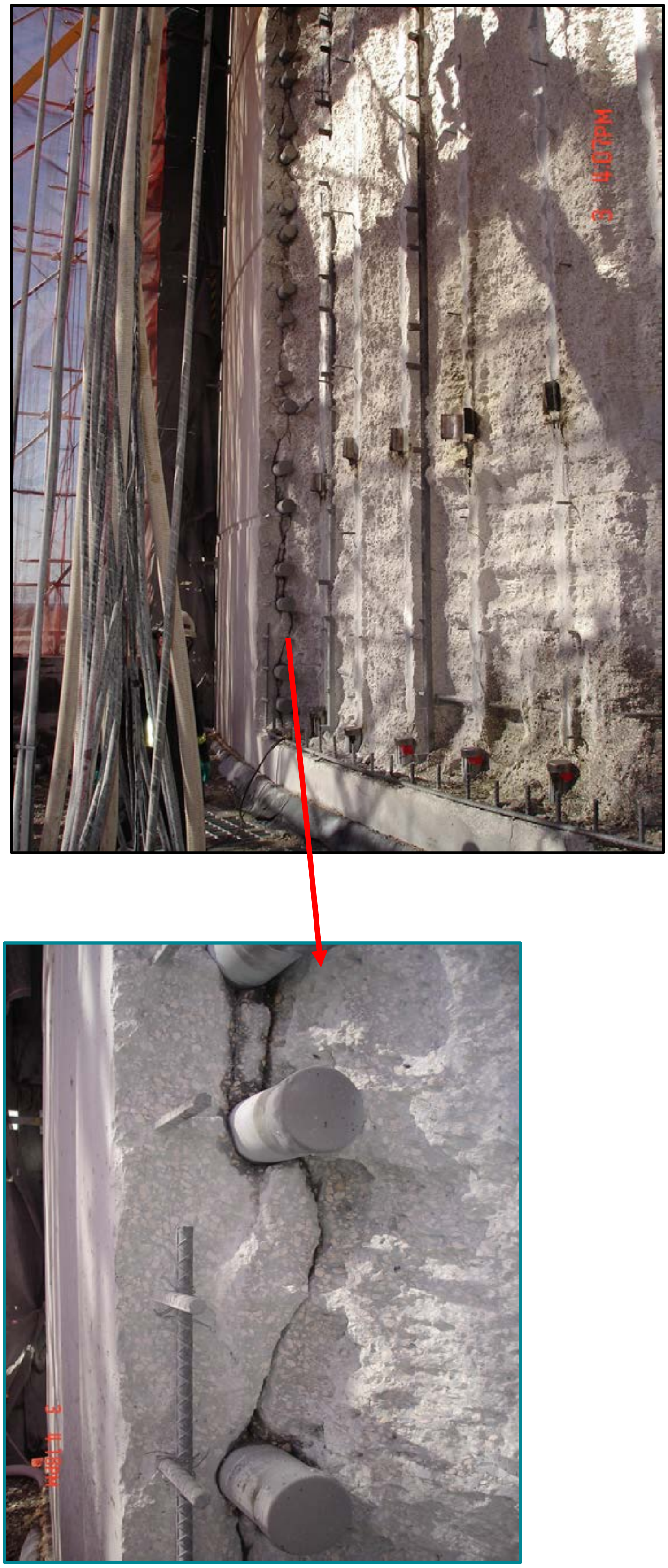

Figure 5 Delamination damage in Crystal River 3

Figure 6 Crystal River 3 attempt to repair damage

Figure 7 - Alkali Silica Reaction of Concrete at Seabrook Station

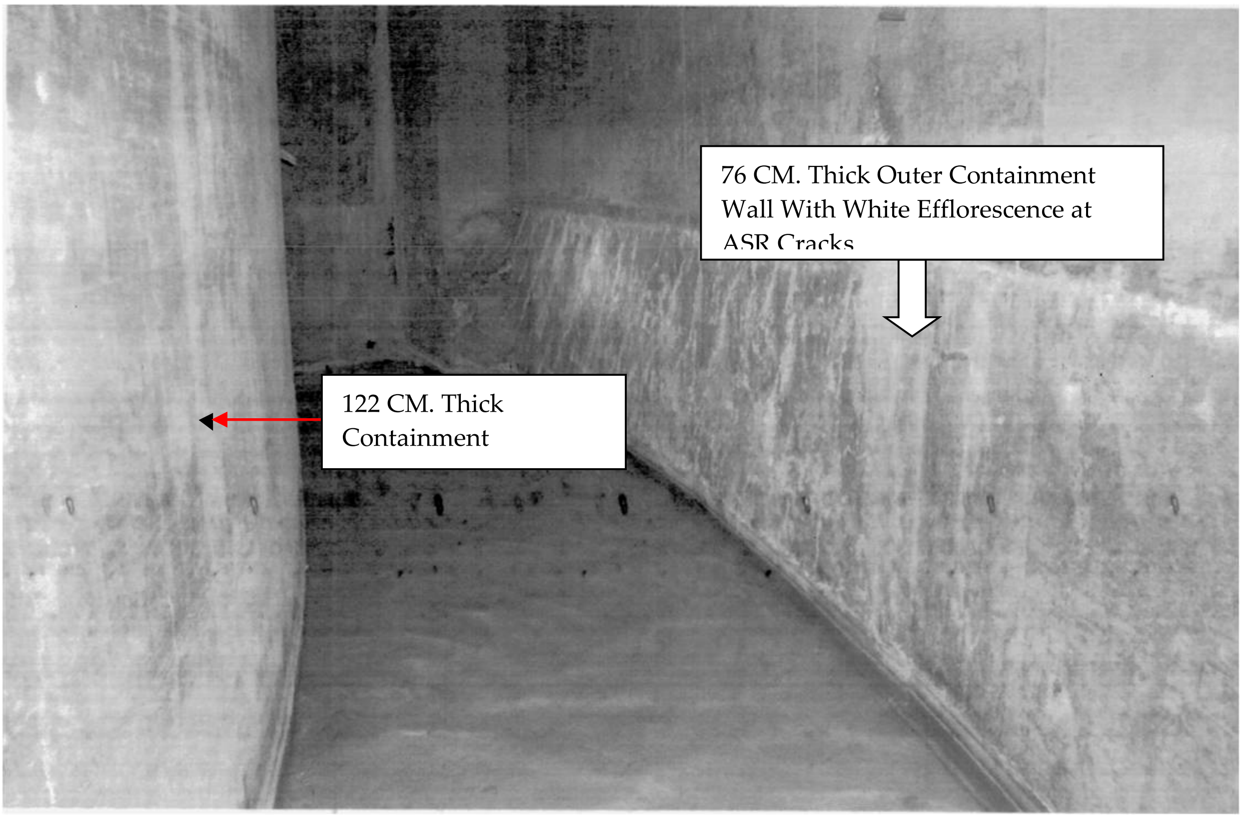

Figure 8 – Alkali Silica Reaction in Outer Containment Wall

6.3 DEGRADATION OF CONTAINMENT LINER PLATE

Corrosion in the concrete containment steel liner plate has been observed at various plants in USA at the concrete/steel interface where the liner becomes embedded in the concrete floor. The licensee removes the corrosion and silicone joint filler at the base of the liner, repaint the liner, and perform ultrasonic testing examination to determine the liner thickness. Since the liner is designed as a leak tight membrane and not as a load carrying member, loss of small thickness due to corrosion is normally acceptable.

At two plants in USA (Salem and Robinson), the containment liner plate is covered by insulated package of a layer of steel metal, a layer of plastic sheeting, and a layer of insulation. Heavy corrosion was found in the liner plate behind the insulation at these plants (Figure 9). The licensee had to remove and replace the heavily corroded parts of the liner, including leak chase channels at the base of liner. In addition, the licensee agreed to enhance the inspection in this region of the liner plate.

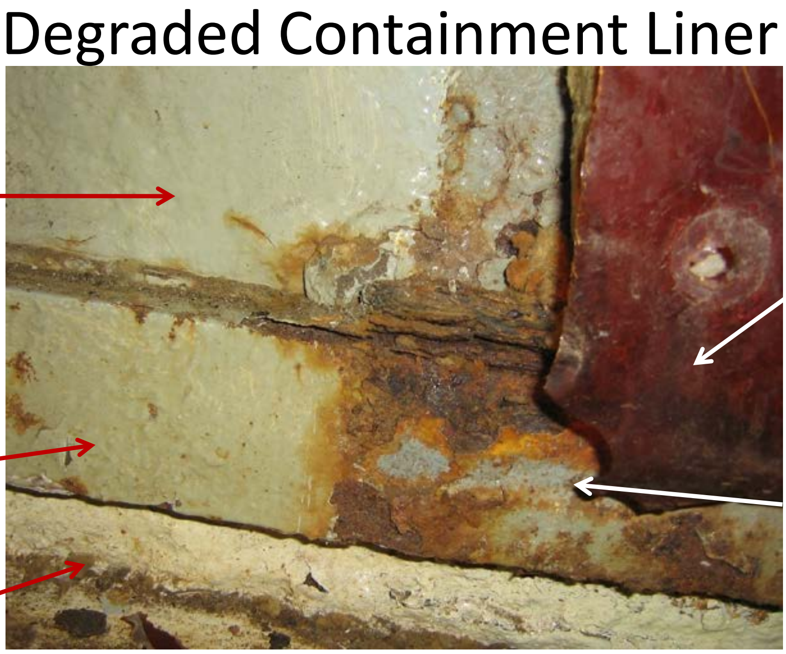

Figure 9 – Containment Liner Plate Corrosion

During a refueling outage at Beaver Valley Power Station Unit 1, the licensee performed a visual examination of the interior containment building liner plate [26]. During this examination, the licensee identified an area approximately 75 mm diameter that exhibited blistered paint. Collapse and cleaning of the blister during further examination revealed an area of 25 mm by 10 mm that penetrated thru the entire thickness of the liner plate (Figures 10 and 11). The licensee removed the corroded section of the liner plate and discovered a partially decomposed piece of wood approximately 50 mm by 100 mm. The wood was used inappropriately during construction as a spacer for the rebars. The licensee as well as an independent group of consultants hired by the USNRC concluded that the root cause was pitting corrosion originating from the concrete side caused by the piece of wood that was in contact with the steel liner. Corrective actions included removal of the wood, grouting of concrete area and replacement of the steel liner in the affected area. The licensee also committed to perform ultrasonic examination of the liner plate at random and non-random locations of the liner plate to detect corrosion at other places in the liner. The results were all negative and corrosion of liner on the concrete side has not been identified at any other place by the ultrasonic testing.

Figure 10 –Beaver Valley - Piece of Wood behind the Liner Plate

Figure 11 – Perforation of Liner at Beaver Valley

7 Prestressed Concrete Containments Reanalysis

The NUREG-0800, Standard Review Plan (SRP), Section 3.7.2 [4] provides general outline for the required analysis approach. In general, elastic structural finite element analysis is used for containment analysis. The analysis should include cracking that is likely to soften the structure and change its dynamic characteristics and damping. In case of post-tensioned containments, it is possible that cracking may not be significant due to level of prestress present during a safe shutdown earthquake. In the past, cracking was not normally considered in the structural seismic analysis. However, cracking of concrete is likely due to forces and moments due to thermal loads during normal operation and or during abnormal accident condition.

In the older designs, cracking was not normally considered in the seismic analysis but during detailed designs of the containment cracking due to thermal loads was used to determine the amount of non-prestressed reinforcement. However, in the new reactor designs', cracking is considered in the seismic analysis of the containments. Cracking of concrete reduces the stiffness of the structural elements and results in corresponding reduction in the soil structure interaction (SSI) frequencies, and can increase seismic demand.

The severity and extent of cracking depends on state of stress of structural elements for various design loading combinations. Therefore, cracking is not uniformly distributed throughout the structure but, rather, is most significant in regions of high stresses. Therefore, it is difficult and realistically impractical to include the cracking effects in seismic analysis.

NUREG-0800 [4] provides the following guidance to account for reduction in stiffness due to cracking:

> "Modeling of the appropriate stiffness and damping for the various structural elements in the mathematical model is essential to obtain realistic seismic responses (e.g., ISRS, building accelerations, member forces, and displacements). For reinforced concrete structures, the stiffness used in the model depends on the degree of concrete cracking which is a function of the level of stress due to the most critical load combination. The effects of concrete cracking on membrane, bending, and shear stiffness should be considered as appropriate in the mathematical model. Because the effect of cracking on the stiffness of concrete members is complex and depends on a number of factors, the approach used should be shown to be conservative. One approach for considering the cracked concrete properties is to reduce the stiffness properties of the uncracked members by a reduction factor. Acceptable stiffness reduction factors for cracked concrete members are given in American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) 43-05 [27] (e.g., 0.5 for cracked walls for flexure and shear)."

Based on the above guidance, and as shown in Table 4.3-1 of ASCE/SEI 43-05 [27], the usual practice is to account for cracking globally by reducing the modulus of elasticity of all structural elements in the seismic analysis model by assuming full modulus of elasticity for one analysis and then using the same model with reduced modulus of elasticity of 50% of the full value. An alternative approach is to use a definite reduction in modulus of elasticity such as 75% in the analysis model and then verify this assumption based on detailed analysis. Both of these approaches has been acceptable, and performed using linear analysis, and conform to the guidance provided in the NUREG-0800 [4]. None of the US codes and regulations requires non-linear analysis of the containment. Either of the two approaches can be used for re-analysis of an operating nuclear power plant but may require a license amendment and change in current licensing basis, if the original analyses did not consider reduction in stiffness due to cracking.

In the current operating nuclear power plants, the structural analysis was based on step approach. The seismic analysis was performed using SASSI program that included detailed soil impedance and embedment effects. The linear elastic static model analysis was performed using commercial finite element codes such as ANSYS or STRUDL with soil springs based on simplified assumptions of soil boundary effects. To ensure that both the SASSI and ANSYS/STRUDL models have the same global and dynamic characteristics, a correlation study assuming fixed base boundary conditions for both models is performed.

The detailed structural and seismic analyses are seldom performed for existing nuclear power containment in USA to avoid cost and long review process. Instead, in case such as low lift–off forces in the tendon are detected, following approaches have been used to show that containment structure is able of withstand the design loads:

1. Use lower value of prestress force and check if the existing non-prestressed reinforcement have some design margin and can withstand higher forces and moments.

2. Review the containment mechanical design to see if the magnitude of the containment design accident pressure reactor can be reduced by refined analysis.

3. Retension the prestress tendons to a higher value forces if they are lower than 0.7 times the ultimate strength of the material.

4. Review the conservatism in the loss of prestress calculations due to creep and shrinkage. The coefficients for the loss of prestress due to creep and shrinkage can be revised based on lift-off forces measured during previous surveillances. This may require approval of appropriate regulatory authority.

5. Refined static finite element analysis of a section of containment using finer mesh around the critical stress areas such as large penetrations and equipment hatch, in order to get an accurate picture at the discontinuities.

6. Modeling of prestressed tendons as a change in temperature and accounting for additional prestress due to pressurization of containment during an accident. Internal pressure during an accident will cause expansion of containment and will result in strain in both concrete and prestressing tendons. This will result in additional force in the prestressing tendons because modulus of elasticity of prestressing steel tendons is about 7-8 times higher than that of concrete.

7. More refined discrete modeling of the prestressed tendons using springs to account for friction losses.

There is no guidance in the ASME Code, Section III, Division 2 [2] on how to treat thermal effects. Guidance provided ACI 349-06 [28] may be used for Guidance. Section E.3 of ACI 349 states that:

> "**E.3.1** The effects of the gradient temperature distribution and the difference between mean temperature distribution and base temperature during normal operation or accident conditions shall be considered.
> **E.3.2** Time-dependent variations of temperature distributions shall be considered in evaluating thermal strains for both normal operating conditions and accident conditions.
> **E.3.3** Thermal stress shall be evaluated considering the stiffness characteristics and the degree of restraint of the structure. The evaluation may be based on cracked section properties, provided the following conditions are met:
> > (a) The tensile stress for any section exceeds the tensile stress at which the section is considered cracked;
> > (b) Redistribution of internal forces and strains due to cracking are included;
> > (c) All concurrent loads are considered; and
> > (d) The coefficient of thermal expansion of concrete may be taken as 5.5×10^{-6} per degree Fahrenheit unless other values are substantiated by "tests."
>
> **E.3.4** Thermal force is not allowed to reduce the design forces due to other loads unless the following are considered:
> > (a) The effect of cracking in the tensile zone of flexural members on reduction of the flexural rigidity and on the redistribution of stress;
> > (b) The reduction of long-term stresses due to relaxation and creep."

To use the guidance in ACI 349 noted above, an iterative process is used to determine the loss in forces and moments due to thermal gradients and cracking during the containment accident pressure loading condition. During the reinforcement design process, the forces and moments due to mechanical (primary loads) and thermal (secondary loads) are converted into stresses first assuming that section is un-cracked and has full thickness. If the tensile stress at the section is found to be greater than tensile strength of concrete (0.4√fc in SI units), the thickness of the section is reduced resulting in reduced thermal forces and moments. This iterative process is continued until the tensile stress is equal to or less than tensile strength of concrete. The reinforcement is then calculated for the revised forces and moments from primary and secondary loads. In addition, the reinforcement is also calculated for primary loads based on uncracked full thickness. The reinforcement that is greater of the two calculations in then used in the design. This helps in removing the conservatism in design. This iterative approach requires a suitable post processor program to handle large number of finite elements. Most design organizations develop and use in-house post processor programs. These programs are not available commercially.

The approaches described above are normally used for reanalysis of prestressed concrete containment to remove the conservatism in the design. USNRC regulations and regulatory guides and ASME Code [2] do not allow the use of non-linear re-analysis of prestressed concrete containments to remove conservatism from the design.

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Appendix A: Loss Of Prestress Due To Elastic Shortening of Concrete

In this appendix detailed calculations are performed to determine the loss of prestress due elastic shortening of concrete in a prestressed concrete containment.

Containment Dimensions

Cylinder Inside diameter = 21.336 meters

Height of cylinder = 48.387 meters

Vertical cylinder wall thickness = 114.3 cm

Number of hoop tendons = 168

Number of Inverted U vertical tendons = 74 or 148 vertical tendons

Average hoop tendon lift off force = 7139 KN/tendon

Average vertical tendon lift off force = 7192 KN/tendon

If the tendons are stressed simultaneouly the loss of prestress due to elastic shortening of concrete F_{LES} will be:

$F_{LES} = F_o\, E_p\, A_p/(A_{CN}\, E_C + A_S\, E_S + A_P\, E_P + A_L\, E_L + A_d\, E_d\,)$

Where

F_o is the initial seating force

A_{CN} is the net concrete area

A_S, A_P, A_L, A_d are areas of reinforcing steel, prestressing steel, liner plate, and duct respectively

E_C, E_S, E_P, E_L, E_d are the moduli of elasticity of concrete, reinforcing steel, prestressing steel, liner plate, and duct respectively.

Vertical reinforcement used = #18 bars @ 30.5 cm C/C each face of cylinder

= 84.66 cm^2/meter EF (Total 169.33 cm/meter)

A_S = 166.37 cm^2/meter along the circumference of cylinder

Tendon cross sectional area = 182.58 cm^2

A_P (per meter in vertical direction) = 148 x 182.58 / 2 x ∏ x21.95

= 195.97 cm^2/meter

A_L (1/4 inch or 6 mm thick liner plate) = 100 x 2.54/4 = 63.5 cm^2/meter

A_d = ∏ x duct diameter x thickness x 148 / 2 x ∏ x 72 = 2.96 cm^2/meter

A_{CNV} = 100 (114.3) – 195.97-1524 – 166.33 = 9543 cm^2/meter

$$F_{LESVERT} = \frac{Fo \times 198.8 \times 10^3 \times 58.2}{[9543(29.64 \times 10^3) + 166.37(203 \times 10^3) + 58.2\,(198.8 \times 10^3) + 63.5\,(203 \times 10^3) + 2.96\,(203 \times 10^3)]}$$

$= 0.0339\, F_o$ for vertical direction

Similarly F_{LESHOR} for horizontal or hoop direction is calculated and is equal to

F_{LESHOR} = 0.0631 F_o for horizontal direction

The tensioning sequence for this containment is such that first half of the vertical tendons are tensioned. Then the hoop tendons are tensioned. Finally the remaining vertical tendons are tensioned. The impact of intermingling this kind of sequence is Poisson's ratio effect on tendons. The Poisson ratio effect produced by tensioning the hoop tendons will reduce the losses in the first set of verticals (Poisson ratio effect causes elongation in the opposite direction). Likewise, the Poisson ratio effect produced by tensioning the second set of vertical tendons will reduce the losses in the hoop tendons.

Poisson ratio effect due to prestressing of hoop tendons is elongation of vertical tendons

$\Delta y = \Omega_{hoop}$ x Hcont x v / Et

Where:
Δy = elongation in vertical direction
Ω_{hoop} = Average hoop/horizontal tendon force per/ft of wall
Hcont = Height of containment to spring line = 48.387 meter
v = Dynamic Poisson's Ratio of concrete = 0.25
E = Modulus of elasticity of concrete = 29.638 GPa
t = Containment thickness = 114.3 cm

Average hoop tension force per tendon = 7139 KN/Tendon

Spacing of hoop tendons = 49.53 cm

Therefore, Ω_{hoop} = 7139 x 100/49.53 = 14414 KN/meter

Using the above calculated values Δy = 5.2324 mm

Equivalent force due to elongation of 5.2324 mm

$P = \Delta y \times A_p \times E_p/L$

Using actual and appropriate values of prestressing tendon area and modulus of elasticity

P = 25.8 Kips

Therefore calculated losses due to elastic shortening of concrete in the half of the vertical tendons will be reduced by 114.75 KN

Similarly calculations are performed for loss in hoop tendon force due to prestressing the second half of vertical tendons is calculated and found to be 30.25 KN. Therefore, the calculated losses in the hoop tendons due to elastic shortening will be reduced by 30.25 KN

Based on the above loss calculations, forces in a representative sample of vertical(inverted U) and hoop tendons are calculated on the following pages (Tables 1, 2, and 3)

LOSS DUE TO ELASTIC SHORTENING IN HOOP TENDONS (Forces are in KN)

Number of hoop tendons:			N =	168		
			FLESHOR =	0.063 Fo		
			FnLESHOR =	nr x FLESHOR/N		
TENDON #	END	SEQUENCE n	nr	Fo	FnLESHOR - 30.5	FINITIAL
14	1	7	161	7419	418	7001
14	2	7	161	7388	416	6972
39	3	20	148	7401	381	7021
39	2	20	148	7526	388	7138
45	2	23	145	7357	370	6987
45	3	23	145	7566	382	7184
99	2	50	118	7121	285	6836
99	3	50	118	7259	291	6968
5	2	86	82	7259	193	7066
5	1	86	82	6859	181	6678
6	2	84	84	7313	200	7112
6	3	84	84	7281	199	7082
58	1	111	57	6921	118	6803
58	3	111	57	7308	126	7182
131	2	149	19	7121	20	7101
131	1	149	19	7032	20	7013

Table 1

LOSS DUE TO ELASTIC SHORTENING IN VERTICAL TENDONS -TENSIONED FIRST BEFORE HOOP TENDONS
(Forces are in KN)

Number of vertical tendons:		N =	74		
		FLESVERT =	0.0339 Fo		
		FnLESVERT =	nr x FLESVERT/N		
TENDON #	SEQUENCE n	nr	Fo	FnLESVERT - 114.75	FINITIAL
18	2	72	6832	111	6722
94	2	72	6943	114	6829
24	4	70	7642	130	7511
88	4	70	7317	120	7197
26	6	68	7019	104	6915
86	6	68	7255	111	7143
10	9	65	7143	98	7046
102	9	65	7317	103	7214
4	16	58	7019	72	6947
108	16	58	7068	73	6995
59	23	51	6957	48	6909
127	23	51	6881	46	6835
61	24	50	6770	40	6730
125	24	50	7006	46	6960

TABLE 2

LOSS DUE TO ELASTIC SHORTENING IN VERTICAL TENDONS
TENSIONED AFTER ALL HOOP TENDONS TENSIONED
Forces are in KN

Number of vertical tendons:		N =	74		
		FLESVERT =	0.0339 Fo		
		FnLESVERT =	nr x FLESVERT/N		
TENDON #	SEQUENCE n	nr	Fo	FnLESVERT	FINITIAL
7	52	22	7170	72	7098
105	52	22	7023	71	6953
13	44	30	6983	96	6887
99	44	30	6988	96	6892
15	40	34	6730	105	6625
97	40	34	6992	109	6883
21	38	36	6983	115	6868
91	38	36	7050	116	6934
42	70	4	6921	13	6908
144	70	4	6988	13	6975
54	62	12	7108	39	7069
132	62	12	6899	38	6861
58	57	17	6743	53	6691
128	57	17	7112	55	7057
70	71	3	7290	10	7280
116	71	3	7019	10	7009

TABLE 3

Appendix B: GALL Report AMP XI.S2, ASME Section XI, Subsection IWL

I. XI.S2 ASME SECTION XI, SUBSECTION IWL

1.I.1 PROGRAM DESCRIPTION

10 CFR 50.55a imposes the examination requirements of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel (B&PV) Code, Section XI, Subsection IWL, for reinforced and prestressed concrete containments (Class CC). The scope of IWL includes reinforced concrete and unbonded post-tensioning systems. This evaluation covers the 2004[1] edition of the ASME Code, Section XI, as approved in 10 CFR 50.55a. ASME Code, Section XI, Subsection IWL and the additional requirements specified in 10 CFR 50.55a(b)(2) constitute an existing mandated program applicable to managing aging of containment reinforced concrete and unbonded post-tensioning systems for license renewal.

The primary inspection method specified in IWL-2500 is visual examination, supplemented by testing. For prestressed containments, tendon wires are tested for yield strength, ultimate tensile strength, and elongation. Tendon corrosion protection medium is analyzed for alkalinity, water content, and soluble ion concentrations. The quantity of free water contained in the anchorage end cap and any free water that drains from tendons during the examination is documented. Samples of free water are analyzed for pH. Prestressing forces are measured in selected sample tendons. IWL specifies acceptance criteria, corrective actions, and expansion of the inspection scope when degradation exceeding the acceptance criteria is found.

The 2004 edition of the Code specifies augmented examination requirements following post-tensioning system repair/replacement activities. The post-tensioning system repair/replacement activities are to be in accordance with the requirements of the 2004 edition of the Code.

The evaluation of 10 CFR 55.55a and Subsection IWL as an aging management program (AMP) for license renewal is provided below.

1.I.2 EVALUATION AND TECHNICAL BASIS

Scope of Program: Subsection IWL-1000 specifies the components of concrete containments within its scope. The components within the scope of Subsection IWL are reinforced concrete and unbonded post-tensioning systems of Class CC containments, as defined by CC-1000. The program also includes testing of the tendon corrosion protection medium and the pH of free water. Subsection IWL exempts from examination portions of the concrete containment that are inaccessible (e.g., concrete covered by liner, foundation

[1] Refer to the GALL Report, Chapter I, for applicability of other editions of the ASME Code, Section XI.

material, or backfill or obstructed by adjacent structures or other components).

10 CFR 50.55a(b)(2)(viii) specifies additional requirements for inaccessible areas. It states that the licensee is to evaluate the acceptability of concrete in inaccessible areas when conditions exist in accessible areas that could indicate the presence of or result in degradation to such inaccessible areas. Steel liners for concrete containments and their integral attachments are not within the scope of Subsection IWL but are included within the scope of Subsection IWE. Subsection IWE is evaluated in AMP XI.S1.

Preventive Action: ASME Code Section XI, Subsection IWL is a condition monitoring program. However, the program includes actions to prevent or minimize corrosion of the prestressing tendons by maintaining corrosion protection medium chemistry within acceptable limits specified in IWL.

Parameters Monitored or Inspected: Table IWL-2500-1 specifies two categories for examination of concrete surfaces: Category L-A for all accessible concrete surfaces and Category L-B for concrete surfaces surrounding anchorages of tendons selected for testing in accordance with IWL-2521. Both of these categories rely on visual examination methods. Concrete surfaces are examined for evidence of damage or degradation, such as concrete cracks. IWL-2510 specifies that concrete surfaces are examined for conditions indicative of degradation, such as those defined in ACI 201.1R and ACI 349.3R. Table IWL-2500-1 also specifies Category L-B for test and examination requirements for unbonded post tensioning systems. The number of tendons selected for examination is in accordance with Table IWL-2521-1. Additional augmented examination requirements for post-tensioning system repair/replacement activities are to be in accordance with Table IWL-2521-2. Tendon anchorage and wires or strands are visually examined for cracks, corrosion, and mechanical damage. Tendon wires or strands are also tested for yield strength, ultimate tensile strength, and elongation. The tendon corrosion protection medium is tested by analysis for alkalinity, water content, and soluble ion concentrations. The pH of free water samples is analyzed.

Detection of Aging Effects: The frequency and scope of examinations specified in 10 CFR 50.55a and Subsection IWL ensure that aging effects would be detected before they would compromise the design-basis requirements. The frequency of inspection is specified in IWL-2400. Concrete inspections are performed in accordance with Examination Category L-A. Under Subsection IWL, in-service inspections of concrete and unbonded post-tensioning systems are required at 1, 3, and 5 years following the initial structural integrity test. Thereafter, inspections are performed at 5-year intervals. For sites with multiple plants, the schedule for in-service inspection is provided in IWL-2421. In the case of tendons, only a sample of the tendons of each tendon type requires examination during each inspection.

The tendons to be examined during an inspection are selected on a random basis. Regarding detection methods for aging effects, all accessible concrete surfaces receive General Visual examination (as defined by the ASME Code). Selected areas, such as those that indicate suspect conditions and concrete surface areas surrounding tendon anchorages (Category L-B), receive a more rigorous Detailed Visual examination (as defined by the ASME Code). Prestressing forces in sample tendons are measured. In addition, one sample tendon of each type is detensioned. A single wire or strand is removed from each detensioned tendon for examination and testing. These visual examination methods and testing would identify the aging effects of accessible concrete components and prestressing systems in concrete containments. Examination of corrosion protection medium and free water are tested for each examined tendon as specified in Table IWL-2525-1.

Monitoring and Trending: Except in inaccessible areas, all concrete surfaces are monitored on a regular basis by virtue of the examination requirements. For prestressed containments, trending of prestressing forces in tendons is required in accordance with paragraph (b)(2)(viii) of 10 CFR 50.55a. In addition to the random sampling used for tendon examination, one tendon of each type is selected from the first-year inspection sample and designated as a common tendon. Each common tendon is then examined during each inspection. Corrosion protection medium chemistry and free water pH are monitored for each examined tendon. This procedure provides monitoring and trending information over the life of the plant. 10 CFR 50.55a and Subsection IWL also require that prestressing forces in all inspection sample tendons be measured by lift-off tests and compared with acceptance standards based on the predicted force for that type of tendon over its life.

Acceptance Criteria: IWL-3000 provides acceptance criteria for concrete containments. For concrete surfaces, the acceptance criteria rely on the determination of the "Responsible Engineer" (as defined by the ASME Code) regarding whether there is any evidence of damage or degradation sufficient to warrant further evaluation or repair. The acceptance criteria are qualitative; guidance is provided in IWL-2510, which references ACI 201.1R and ACI 349.3R for identification of concrete degradation. IWL-2320 requires that the Responsible Engineer be a registered professional engineer experienced in evaluating the in-service condition of structural concrete and knowledgeable of the design and construction codes and other criteria used in design and construction of concrete containments. Quantitative acceptance criteria based on the "Evaluation Criteria" provided in Chapter 5 of ACI 349.3R also may be used to augment the qualitative assessment of the Responsible Engineer.

The acceptance standards for the unbonded post-tensioning system are quantitative in nature. For the post-tensioning system, quantitative acceptance criteria are given for tendon force and elongation, tendon wire or strand samples, and corrosion protection medium. Free water in the tendon anchorage areas is not acceptable, as specified in IWL-3221.3. If free water is found, the recommendations in Table IWL-2525-1 are followed. 10 CFR 50.55a and Subsection IWL do not define the method for calculating predicted tendon prestressing forces for comparison to the measured tendon lift-off forces. The predicted tendon forces are calculated in accordance with Regulatory Guide 1.35.1, which provides an acceptable methodology for use through the period of extended operation.

Corrective Actions: Subsection IWL specifies that items for which examination results do not meet the acceptance standards are to be evaluated in accordance with IWL-3300, "Evaluation," and described in an engineering evaluation report. The report is to include an evaluation of whether the concrete containment is acceptable without repair of the item and, if repair is required, the extent, method, and completion date of the repair or replacement. The report also identifies the cause of the condition and the extent, nature, and frequency of additional examinations. Subsection IWL also provides repair procedures to follow in IWL-4000. This includes requirements for the concrete repair, repair of reinforcing steel, and repair of the post-tensioning system. As discussed in the Appendix for GALL, the staff finds the requirements of 10 CFR Part 50, Appendix B, acceptable to address the corrective actions.

Confirmation Process: As discussed in the Appendix for GALL, the staff finds the requirements of 10 CFR Part 50, Appendix B, acceptable to address the confirmation process.

Administrative Controls: IWA-1400 specifies the preparation of plans, schedules, and in-service inspection summary reports. In addition, written examination instructions and procedures, verification of qualification level of personnel who perform the examinations, and documentation of a quality assurance program are specified. IWA-6000 specifically covers the preparation, submittal, and retention of records and reports. As discussed in the Appendix for GALL, the staff finds the requirements of 10 CFR Part 50, Appendix B, acceptable to address the administrative controls.

Operating Experience: ASME Section XI, Subsection IWL was incorporated into 10 CFR 50.55a in 1996. Prior to this time, the prestressing tendon inspections were performed in accordance with the guidance provided in Regulatory Guide 1.35. Operating experience pertaining to degradation of reinforced concrete in concrete containments was gained through the inspections required by 10 CFR Part 50, Appendix J, and ad hoc inspections conducted by licensees and the Nuclear Regulatory Commission (NRC). NUREG-1522 described instances of cracked, spalled, and degraded concrete for reinforced and prestressed concrete containments. The NUREG also described cracked anchor heads for the prestressing tendons at three prestressed concrete containments. NRC Information Notice 99-10 described occurrences of degradation in prestressing systems. The program is to consider the degradation concerns described in these generic communications. Implementation of Subsection IWL, in accordance with 10 CFR 50.55a, is a necessary element of aging management for concrete containments through the period of extended operation.

1.I.3 REFERENCES

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Aging Management of Nuclear Prestressed Concrete Containments

This report describes the practice in United States for aging management, inspection, and surveillance of nuclear containment structures with post tensioned prestressed tendons. It can be used by authorities and engineers involved in review and evaluation of post tensioned concrete containments inspection and surveillance results, including interpretation and assessment of measured forces and displacements obtained during periodic surveillance activities.