Nuclear Safety NEA/
CSNI/R(2015)5
June 2015
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Bonded or Unbonded Technologies for Nuclear Reactor Prestressed Concrete Containments

OECD
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NEA
NUCLEAR ENERGY AGENCY

Unclassified **NEA/CSNI/R(2015)5**

Organisation de Coopération et de Développement Économiques
Organisation for Economic Co-operation and Development **01-Jun-2015**

English text only

NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

NEA/CSNI/R(2015)5
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JT03377502

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ORGANISATION FOR ECONOMIC CO-OPERATION AND DEVELOPMENT

The OECD is a unique forum where the governments of 34 democracies work together to address the economic, social and environmental challenges of globalisation. The OECD is also at the forefront of efforts to understand and to help governments respond to new developments and concerns, such as corporate governance, the information economy and the challenges of an ageing population. The Organisation provides a setting where governments can compare policy experiences, seek answers to common problems, identify good practice and work to co-ordinate domestic and international policies.

The OECD member countries are: Australia, Austria, Belgium, Canada, Chile, the Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Israel, Italy, Japan, Luxembourg, Mexico, the Netherlands, New Zealand, Norway, Poland, Portugal, the Republic of Korea, the Slovak Republic, Slovenia, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The European Commission takes part in the work of the OECD.

OECD Publishing disseminates widely the results of the Organisation's statistics gathering and research on economic, social and environmental issues, as well as the conventions, guidelines and standards agreed by its members.

NUCLEAR ENERGY AGENCY

The OECD Nuclear Energy Agency (NEA) was established on 1 February 1958. Current NEA membership consists of 31 countries: Australia, Austria, Belgium, Canada, the Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Japan, Luxembourg, Mexico, the Netherlands, Norway, Poland, Portugal, the Republic of Korea, the Russian Federation, the Slovak Republic, Slovenia, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The European Commission also takes part in the work of the Agency.

The mission of the NEA is:

- to assist its member countries in maintaining and further developing, through international co-operation, the scientific, technological and legal bases required for a safe, environmentally friendly and economical use of nuclear energy for peaceful purposes;
- to provide authoritative assessments and to forge common understandings on key issues, as input to government decisions on nuclear energy policy and to broader OECD policy analyses in areas such as energy and sustainable development.

Specific areas of competence of the NEA include the safety and regulation of nuclear activities, radioactive waste management, radiological protection, nuclear science, economic and technical analyses of the nuclear fuel cycle, nuclear law and liability, and public information.

The NEA Data Bank provides nuclear data and computer program services for participating countries. In these and related tasks, the NEA works in close collaboration with the International Atomic Energy Agency in Vienna, with which it has a Co-operation Agreement, as well as with other international organisations in the nuclear field.

THE COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made up of senior scientists and engineers with broad responsibilities for safety technology and research programmes, as well as representatives from regulatory authorities. It was created in 1973 to develop and co-ordinate the activities of the NEA concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations.

The committee's purpose is to foster international co-operation in nuclear safety among NEA member countries. The main tasks of the CSNI are to exchange technical information and to promote collaboration between research, development, engineering and regulatory organisations; to review operating experience and the state of knowledge on selected topics of nuclear safety technology and safety assessment; to initiate and conduct programmes to overcome discrepancies, develop improvements and reach consensus on technical issues; and to promote the co-ordination of work that serves to maintain competence in nuclear safety matters, including the establishment of joint undertakings.

The priority of the committee is on the safety of nuclear installations and the design and construction of new reactors and installations. For advanced reactor designs, the committee provides a forum for improving safety-related knowledge and a vehicle for joint research.

In implementing its programme, the CSNI establishes co-operative mechanisms with the NEA Committee on Nuclear Regulatory Activities (CNRA), which is responsible for issues concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with other NEA Standing Technical Committees, as well as with key international organisations such as the International Atomic Energy Agency (IAEA), on matters of common interest.

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FOREWORDS

This report has been written in the framework of the Working Group on Integrity and Ageing of Components and Structures (IAGE WG) of NEA, and more precisely with an active participation of members of the Concrete sub-group. This activity is fully consistent with the OECD/NEA-medium-term strategies for IAGE concrete working group (2006) and specifically with the proposed priorities (High priorities: Containment & Tightness and Prestressing Losses). This report is the result of three workshops:

- The Lyon April 20th and 21th 2011 workshop dedicated to technical presentation in order the share technical experience and feedback, and to agree the organization of the document and main work load distribution

- The Lyon November 26th and 27th 2011 workshop dedicated to the detailed definition of the document content and additional technical presentations focused on critical issues.

- The Helsinki December 9th and 10th 2013 workshop dedicated to the detailed review of an official draft given to everybody before the meeting

But this activity has to be considered not only on the basis of this report but also as an actual international platform for technician exchanges. These exchanges have been particularly intense not only during meeting, but also between meeting, and we hope also after.

OECD/NEA thanks every contributors whose list is just here after and specifically the two inseparable and complementary pilots of this CAPS activity who was Etienne Gallitre and Pentti Varpasuo.

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LIST OF ABBREVIATIONS

ACI:	American Concrete Institute
AFCEN:	French Association for Nuclear Reactor Codification
ASME:	American Society of Mechanical Engineers
CCB:	Concrete Containment Building
CEN:	European Committee for Standards
CSA:	Canadian Standards Association
COFRAC:	Le Comité français d'accréditation
EN:	European standards maintained by CEN (European Committee for Standardization)
EPR:	European Pressurized Reactor
ETA:	European Technical Approval
ETAG:	European Technology Assessment Group
ETC-C:	EPR Technical Code for Civil works
FIB:	International Organization for Concrete
FE:	Finite Element
FEM:	Finite Element Method
GSS:	Greased Sheathed Strands
HPDE:	High Density Polyethylene
ILRT:	Integrated Leak Rate Test
ISIT:	Initial Structural Integrity Test
JSME:	The Japan Society of Mechanical Engineers
LOCA:	Loss of Cooling Accident
LCPC:	(French) National Laboratory for Bridges and Roads (today: IFSTTAR)
LVDT:	Linear Variable-Differential Transformer
LRT:	Leak Rate Test
NPP:	Nuclear Power Plant
OCN:	Oconee Nuclear station
OSS:	Optimal Surveillance System (French practice)
PCC:	Prestressed Concrete Containment
PP:	Polypropylene
PPT:	Preoperational Proof Test
PE:	Polyethylene
PVC:	Polyvinylchloride
RC:	Reinforced concrete
RG:	(US) Regulatory Guide
SCOSS:	(UK) Standing Committee on Structural Safety

SETRA:	(French) State Division for Transportation Studies
SIA:	Swiss Society of Engineers and Architects
SL-2:	Seismic Level 2 (usually design earthquake)
THTR:	Thorium High Temperature Reactor
USNRC:	Nuclear Regulatory Commission for the United States of America
VIPP:	Bridge with simply supported spans using prestressed beams
VWSG:	Vibrating Wire Strain Gauge

VOCABULARY

Anchor head:	End of tendon where prestressed forces are transferred to the concrete through a dedicated metallic device
Bonded:	With mechanical connection
Containment:	Structure surrounding the reactor which prevents the release of radioactive products and can withstand to pressure in case of accident
Creep:	Property of material which volume decreases when submitted to load
Duct:	Pipe put in place in the formwork which contain the tendons
Design Basis Domain:	Set of load cases covered by national codes
Design Extension Domain:	Set of load cases not covered by national codes, but which have to be taken into account for design or for safety assessment
Discretization:	Modelling technique consisting on dividing structure element into small parts called finite elements
Eurocode:	European Standard for Civil Works Design
Grease:	Organic semi-liquid material dedicated to tendon anti-corrosion protection
Grout:	Product made of cement and water which protects the tendons from corrosion
Lift-off:	Short uplift of prestressed tendon end in order to measure the effective tension
Relaxation:	Diminution of stress with time
Sheathed:	Located inside a protection sheath
Shrinkage:	Property of material which volume decreases with time
Strand:	Set of wires for prestressing system (a set of strands is a tendon)
Tendon:	Set of strands or set of wires if these wires are not gathered in strands
Unbonded:	Without mechanical connection
Wax:	Solid greased dedicated to tendon anti-corrosion protection
Wire:	Unique filament of metal

EXECUTIVE SUMMARY

OECD/NEA/CSNI Working Group on Integrity and Ageing of Components and Structures (WGIAGE) has the main mission to advance the current understanding of those aspects relevant to ensuring the integrity of structures, systems and components under design and beyond design loads, to provide guidance in choosing the optimal ways of dealing with challenges to the integrity of operating as well as new nuclear power plants, and to make use of an integrated approach to design, safety and plant life management. The work related to the risks of the loss of pre-stressing force in concrete structures has been in high priority during the activities of the concrete sub-group of WGIAGE. Therefore, the CAPS of WGIAGE: Study on post-tensioning methodologies in containments, was approved by CSNI in June 2009.

In this study the two post-tensioning methodologies: bonded and unbonded methods and their technological features are analysed. In the bonded technology, the tendon cannot slide in its duct due to the cement grouting which is injected after tensioning. In the unbonded technology, the tendon can slide inside its duct, the corrosion protection is given by grease, wax or dry air. A key point concerning the assessment of durability and safety of prestressed concrete containments is the technology chosen for tendon protection: bonded with cement grout or unbonded and protected by grease or soft products. The mechanical behaviour of the containment is directly influenced by the adherence of the tendons to the concrete, locally and under high stresses in case of severe accident. The bonded or unbonded tendons of post-tensioned concrete containment of the Nuclear Power Plants have the major role of containment (balance of the pressure effect during design basis and beyond design accident). Many difficulties around the design, the construction and the in service inspection are related to the tendons.

The main goal of the CAPS work was to clarify the consequences and necessary procedures when choosing the post-tensioning technologies in terms of design basis, in terms of behaviour during severe accident, in term of construction requirements as well as in term of monitoring and in-service inspection of the containment. The choice of the post-tensioning technology is related to the life time extension procedures of old plants as wells as to the construction methods of new NPP's. Today, the performance of the containment in severe accident conditions is a part of the design safety assessment and this report presents how the different technologies impact these methods.

The work was done by collecting the information on methods and different methodologies including their application and operating experiences in the existing plants and ongoing projects in member countries during three workshops in years 2011-2013. The document has three parts: design, construction and in service inspection in order to cover all aspects, from the beginning to the end: every part will be summarised in the full paper. This work led to a very productive exchange platform, where the main experts of this subject have contributed with their experiences regarding these problems and methods used to solve them.

The results of the work are summarised in chapter 10. The most important results are as follows:

- For new structures, data collection should start during construction (at least at the beginning of the tensioning of the cables) so that the initial state, i.e. baseline parameters could be established at the end of the construction period.

- Differences of the unbonded and for bonded tendons to the maintenance aspects are taken into account in the early design of new construction to be sure that all aspects to the accessibility, inspections and preventive maintenance are managed in a relevant way.

- Procedures of monitoring and In-Service Inspection methods of containment should carefully be defined and approved with special features of chosen technology for the new and existing plants to make sure that all safety goals are fulfilled

- Direct monitoring of the bonded post-tensioning system is currently not possible, proven reliable indirect methods should be used for the new and existing plants to ensure continuous integrity of the post-tensioning system over the service life of the plant.

One important recommendation of this study is that the correct modelling of the tendon bonding condition with concrete around the tendons is important, especially when the Design Extension Conditions are taken into account in the design of new constructions. For the new and existing plants the lifetime monitoring, maintenance and testing procedures must be designed and reviewed according to the choice of the protective system for tendons, namely, bonded or unbonded protective system.

1. CONTEXT AND OBJECTIVE

1.1 Context

This OECD activity has been launched in the frame of a NEA/CSNI CAPS as it is usually done for such a technical topic. The context is related to two main challenges for nuclear engineering, life time extension of existing plants and construction of new ones. The post tensioning methodology has to be carefully addressed in both situations due to the high importance of the tendons for the containment capacity. To perform this analyse two aspects have to be tacked: The performance of the containment in severe accident conditions is today a part of the plant safety re-assessment: such re-assessment needs numerical simulation. The way to represent the tendons and their link to the concrete may have an impact on the results. This question concerns both new and existing plants

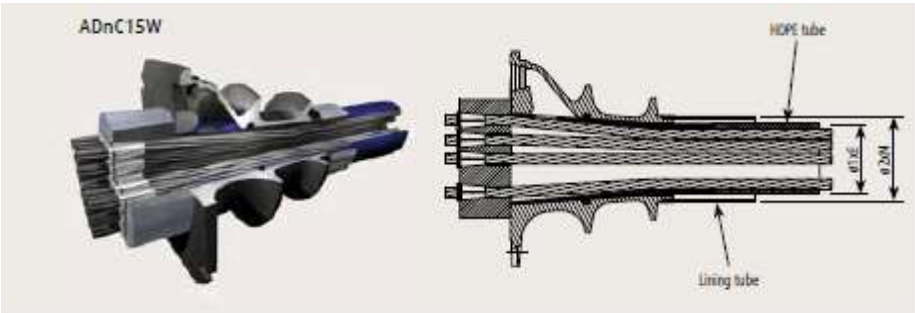
- For existing plants the questions are mainly related to In-Service Inspection and Monitoring which obviously depend on the technology. Monitoring has to be linked to parameter values intended for safety assessment and ISI scoping has to be consistent with variability of the materials and the ageing effects.

These two issues are not so easily solved, and an international cooperation is necessary in order to share good and bad experiences.

1.1.1Definitions

- Unbonded technology: in this technology, the tendon can slide inside its duct, the corrosion protection is given by grease, wax or dry air (GSS belong to that family)

Figure 0.1 Unbonded tendons



- Bonded technology: in this technology, the tendon cannot slide in its duct due to the cement grouting which is injected after tensioning.

Figure 0.2 Bonded tendons (grouted with cementitious grout)

A key point concerning the assessment of durability and safety of prestressed concrete containments is the technology chosen for tendon protection: bonded with cement grout or unbonded and protected by grease or soft products. The mechanical behaviour of the containment is directly influenced by the adherence of the tendons to the concrete, locally and under high stresses in case of severe accident.

1.2 Objective

One objective is to clearly identify the technical considerations of both technologies (bonded versus unbonded), not in order to rank one before the other, which would be useless due to the reliability of both, but in order to properly address the drawbacks.

In this document, we will use the term bonded instead of grouted to emphasize on mechanical contact between tendons and concrete by bond. Instead of “greased”, we will use the term “unbonded” which emphasizes the inspectability of the tendons during the lifetime of the plant.

To do this it is necessary to understand the mechanisms that affect the materials, our ability to catch them in reality, our ability to represent the behaviour of structure, in order to summarize our ability to predict the containment performance under accidental conditions, taking into account ageing effects.

Every participant in this OECD activity has actual experience of areas of these issues but needs more information and more cross check validation due to the complexity of the situation.

The main objective of this CAPS is, obviously, to produce a guideline to help every country in the choice of technology but mainly to clarify the consequences of their choice in terms of design basis, in terms of behaviour during severe accident, in term of construction requirements, in term of monitoring and in-service inspection of the containment.

2. SAFETY OBJECTIVES OF THE CONCRETE PRESTRESSED CONTAINMENT

2.1 Safety objectives

The containment is the final barrier to protect the environment and public against uncontrolled release of radionuclides, so the PCC has to fulfil the following requirements:

- Confinement (tightness): improving the leak tightness of the wall (in the absence of steel liner or complementary coating),
- Resistance against pressure loads by limiting its membrane tension forces and flexural moments when submitted to inner pressure (by pressure test or internal LOCA or Severe accident),
- Limitation of concrete cracking in all situations, namely along the inner face (in contact with the steel liner or without liner) improving the reversibility of the mechanical behaviour (elastic behaviour) of the wall,
- Protection against internal and external hazards,
- Stability of the containment wall, during the lifetime of the installation.

2.1.1 Defence in depth

In addition to the design provisions, the prestressing shall exhibit sufficient ruggedness/robustness, in order to avoid any cliff-edge effect, when considering any variation of parameters of interest on the actions side (pressure, temperature, concrete creep and shrinkage) and on the resistance side (yield limits, ultimate deformation, losses due to tendon/sheath friction, steel relaxation, anchorage draw-in, tensioning force). In particular, the tendons are generally the last component of the containment that remain in its elastic domain when the inner pressure level increases and they can be considered as the ultimate structural reinforcement responsible for the leak tightness (by limitation of the steel liner deformations) and later of the overall stability. So the following defence in depth principles have to be considered.

2.1.1.1 Proven engineering design

Robustness in conception and design itself shall be found by using proven engineering design, for instance by using hand calculation for the spacing of the tendons in parts of the containment wall. The FE models are used for checking the stress state in concrete and for the general overview of the losses of prestressing during the construction phases and at the end of lifetime of the installation.

2.1.1.2 Design extension and margins

The bonded tendons adhere to their ducts and therefore to the rest of the structure; the unbonded tendons are not continuosly fixed onto the surrounding structure and, in case of overtension, they tend to slip along the duct when the imposed deformation by the wall is not homogeneous: this situation is difficult to simulate by calculation and may initiate wire/strand/tendon rupture. In case of wire rupture, the missing force is added to the adjacent wires that may in turn induce successive wire ruptures, up to strand failure then up to the tendon failure itself: a local rupture of any bonded tendon induces the loss of all the

prestressing from the tendon. The margins in prestressing are to be found in the tensioning level (the lower the tensioning force, the higher the margin) and in the losses of prestressing in the tendon (due to anchor draw-in, friction coefficient, deferred permanent deformation). New concepts for tendons (like sheathed greased strands) exhibit a better use of the prestressing steel, with less losses of prestressing (namely by reducing the friction coefficients), but, conversely, they exhibit less margins regarding beyond design considerations.

2.2 General provisions for the containment prestressing from a safety point of view (for each phases of the project)

2.2.1 General provisions

Design the prestressing force distribution shall counteract, for the best, the external applied forces by adapting the tendon locations (through their position in the section and their spacing inside the grid) and a modified level of tensioning force; the design shall comply with a design code for prestressing that complies with the approved reference described in the Safety Report.

2.2.2 Modification of containment

Bonded tendons do not allow any major modification of the containment (like widening the equipment hatch or creating a new penetration), as opposed to the unbonded tendons that may be (partially) detensioned, replaced (if necessary) and retensioned. However, for this type of tendon, great attention has to be paid to the detensioning and retensioning, in terms of quantity of tendons and of operations phasing, in order to limit any detrimental effects on the structure.

2.2.3 Construction

Competence: the civil Contractor in charge of the implementation of the prestressing shall be competent in this highly specialized domain; it is recalled that the prestressing is a system (not only a collection of items and a succession of acts), generally patented, for which the final responsibility cannot be simply shared.

Procurement: the procurement and storage conditions shall comply with the standard of the provider(s) for all necessary items involved in the prestressing, these standards being mentioned in an approved reference that shall be described in the Safety Report.

Erection: all the activities dealing with the prestressing erection before tensioning shall comply with the standards of the Contractor in charge, these standards being mentioned in an approved reference that shall be described in the Safety Report; special attention must be paid to the location of the ducts within the formwork according to tolerances defined by the Designer.

Tensioning: all the activities dealing with the tensioning shall comply with the standards of the Contractor in charge, these standards being mentioned in an approved reference that shall be described in the Safety Report; the tensioning sequences have to be defined by the Designer in order to minimize potential harmful effects and any field modifications shall be approved by the Designer; the mechanical parameters responsible of the losses of prestressing are to be checked in situ and interpreted by the Designer, before the tensioning; the effectiveness of the tendon forces shall be confirmed by monitoring during the tensioning; the tendon extension has to be recorded by the Contractor, transmitted to the Designer and compared to its theoretical value.

Protection: the materials for protection (cementitious grout mix, wax or grease) and the procedure for injection shall be defined, approved and tested on site by mock-ups.

2.2.4 Operating

Monitoring: the maintenance of the tension in the tendon and/or of the compression in concrete wall shall be checked by adequate monitoring and recording throughout the life of the containment and during periodic tests (mechanical and air leakage); the results should be reported to the responsible parties and analysed properly Maintenance: all visible parts of the containment shall be periodically inspected, and if necessary repaired; failed monitoring sensors shall be replaced.

2.2.5 Decommissioning

Decommissioning and dismantling: dismantling shall be forecast, at conception and at design stage, in order to minimize the risks for the operators during tendon detensioning.

3. OVERVIEW OF DIFFERENT PRESTRESSING TECHNOLOGIES COMPONENTS

3.1 General

The safety of all nuclear facilities depends on the integrity of civil structures, among them the prestressed concrete pressure vessel of advanced gas cooled reactors and the PCC of PWRs are key structures. The containment building is a large volume reinforced concrete structure, which houses the reactor, reactor cooling or recirculation system, pressurizer and pumps.

The PCC is highly reliable under maximum design conditions and accident loads and actions from outside. A typical PCC is a large vertical cylinder closed at the bottom with a flat slab and at the top with a convex hemi-spherical or sloping ellipsoid dome.

The high in-service reliability of the PCC is determined primarily by the fact that operating as well as the emergency loads are supported by the post-tensioning system.

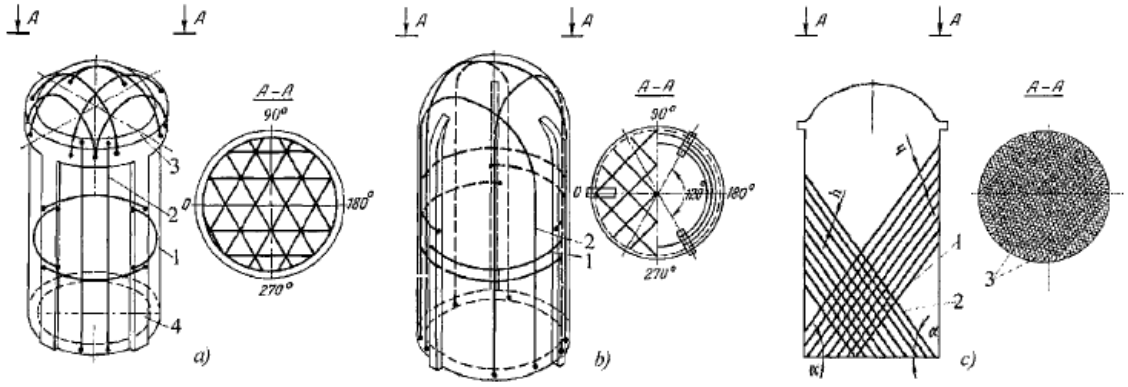
This system consists of a large number of prestressed tendons made of high-strength wire, strand or bar. The tendons are inserted in tendon sheaths, which form ducts through the concrete shell between anchorage points. The duct through which the tendon passes can be filled with a corrosion inhibiting grease. Tendons are tensioned and then anchored to the hardened concrete forming the PCC.

Tendons operate independently with multiple overlapping of the action zones of each. An unlikely failure of one or several tendons does not result in any appreciable changes in the stress and strain state of the PCC and it is able to sustain high internal pressures (in incidents) and protects the reactor against extreme climatic and external effects in-service.

Three major categories of prestressing reinforcement exist depending on the type of tendon utilized: wire, bar or strand tendons. The different schemes of tendons layout in PCC adopted in various countries are presented in

Figure 0.1 below.

Figure 0.1 Different schemes of tendon layout



3.2 Prestressing reinforcements example (EPR)

The prestressing tendons are made of high-strength wire. In the European PWR project, they are composed of steel class 1860 MPa strands which comply with the following requirements:

- The nominal diameter of strands shall be 15.7 mm. Each strand are composed of 7 wires and have a nominal cross section area of 150 mm².

- Tendons are composed of a maximum of 55 strands. The number of strands per tendon and the number of tendons is defined by the detailed designer specifications.

Other class of strands (higher class only) may be used, but are to be justified in accordance with the design and the whole objective of prestressing.

Strands are certified, according to the regulation, by the Approved Body (or Notified Body) as defined by ETAG 013. The following requirements are included in the certification rules:

- Strands are constituted of special non-alloy special steels according to EN 10020

- Standard Pr EN 10138 applies, modified by the following requirements:

  - The diameter of the central wires of the strands are at least 1.02 times greater than the diameter of the outer peripheral wires of the strands,
  - The relaxation of the strands are evaluated with loads equal to 0.7 Fm,a and 0.8 Fm,a and the results are respectively lower than or equal to 2.5% and 4.5% at 1000 h. The relaxation test are performed over at least 240 h, and results may be extrapolated to 1000 h,
  - Corrosion strength under stress is evaluated with solution A (as defined in EN 15630-3). Durations are greater or equal to 1.5 h and the average duration are greater or equal to 4 h.

3.2.1 Anchorage components

All anchorage components comply with the ETA of post-tensioning system (including the permanent or temporary anchorage caps).

The bearing device consists of a cast-iron plate with a non-separate guide. The guide shall possess two load transfer flanges.

3.2.2 Ducts

Ducts can be either of the 2 following types:

- Rigid hand-bendable steel strip sheaths: used for non-deviating horizontal tendons (hoops with radius of curvature ≥ 80 times the inner diameter of the sheath),

- Machine-bendable steel tubes: used for all other ducts or parts of ducts.

3.2.3 Steel tubes

Tubes are round steel section, fabricated from hot-rolled products and longitudinally welded. Welds have to be smoothed. The tubes have to be capable of withstanding the following shaping operations:

- Cold forming at ends so as the minimum internal diameter at bell-mouth ends is equal to the external diameter of the tube in a continuous section plus a value justified in the dedicated procedure of the post-tensioning specialist company;

- Bending by machine with a regular minimum radius. If the section becomes ovalised, the minimum internal diameter has to be greater than the specified minimum value given in the dedicated procedure of the post-tensioning specialist company.

3.2.4 Steel strip sheaths

Steel strip sheaths are fabricated from rolled steel strips. Sheaths are, in general, phosphated and soaped: the friction coefficient is checked by tests.

3.2.5 Heat-shrinkable sleeves

Heat-shrinkable sleeves can be made with reticulated modified polyolefin, with a heat-fusible internal coating and dimensions such that the external nominal diameter of the duct and the sleeve diameter before shrinkage are adapted to the ducts and tubes.

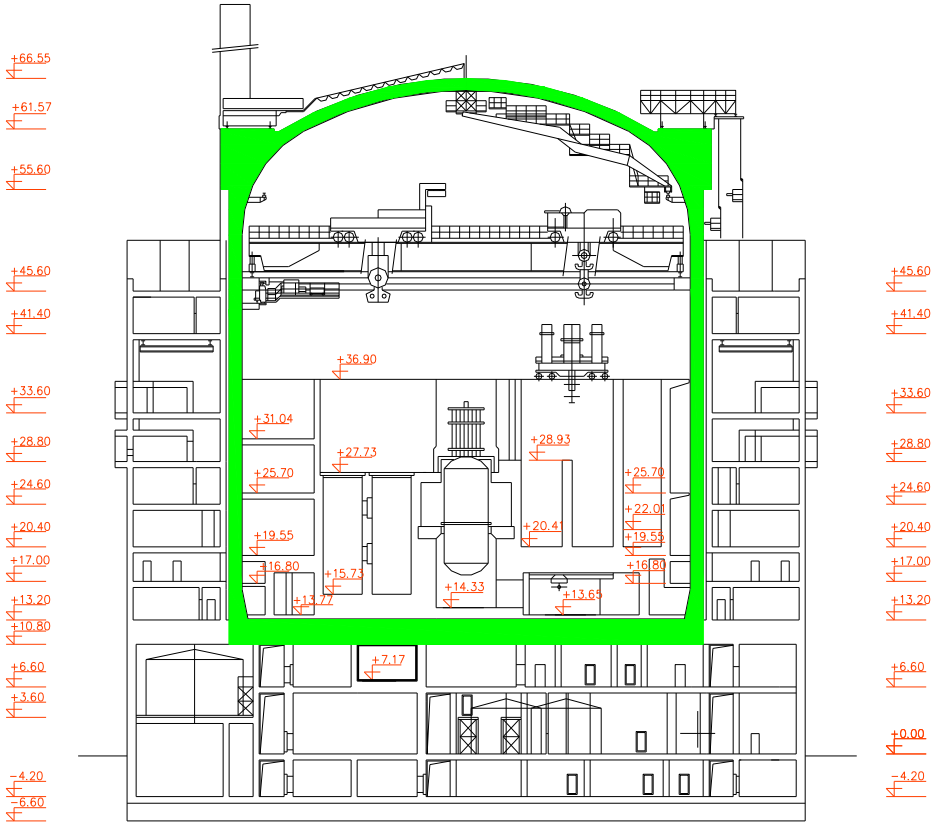
3.3 Overview of different prestressing technologies for nuclear containments VVER 1000 V-320 type containment

The VVER1000 V-320 type unit has a PWR reactor and a containment of prestressed reinforced concrete. The containment structure is a part of the reactor building and is placed on a reinforced concrete slab of thickness 2.4 m at a level of +13.20 m – see

Figure 0.2. The containment consists of a cylindrical and a dome part. The wall thickness of the cylindrical shell is 1.2 m, the dome wall thickness is 1.1m. Connection between the cylindrical and dome parts is made with the help of a rigid ring beam in which the anchoring blocks of the prestressing tendons are placed. There is a gap between the containment structure and internal structures as well as between the containment structure and external surrounding structures. The containment is made of concrete, grade B40 according to the Czech standards (CSN) – approximately corresponds to 30/37 according EN 1992-1-1 [12]. The tightness of the containment is ensured by the steel liner of a thickness of 8 mm made of carbon steel.

The prestressing unbonded tendons are conducted in polyethylene tubes. The cylindrical part of the containment is prestressed by 96 tendons running in a helical direction – see Figure 0.3. The tendon anchors are installed in the upper part of the ring beam, the bending of the tendons takes place in the slab at a level of +13.20 m. The dome part of the containment is prestressed by an orthogonal grid plan of 36 prestressing tendons. Two tendons are always conducted against each other, anchors of one tendon and bending of the other one are situated on one side. The tendons of the cylinder and dome parts are of the same structure and cross section. Every tendon is formed by 450 wires featuring a diameter of 5 mm, the scheme of the tendon is in **Figure 0.4**. Low-relaxation wire is used for production, its yield point being 1620 MPa. The initial nominal prestressing force according to the design is 10 MN. Tendon preservation is made with grease during production (there is no filling of ducts by grease), preservation of anchors is made by grease after prestressing.

Figure 0.2 Section view of VVER1000 V-320 type Reactor building, concrete containment is marked by green filling



.Figure 0.3Scheme of prestressing of VVER1000 V-320 type containment.

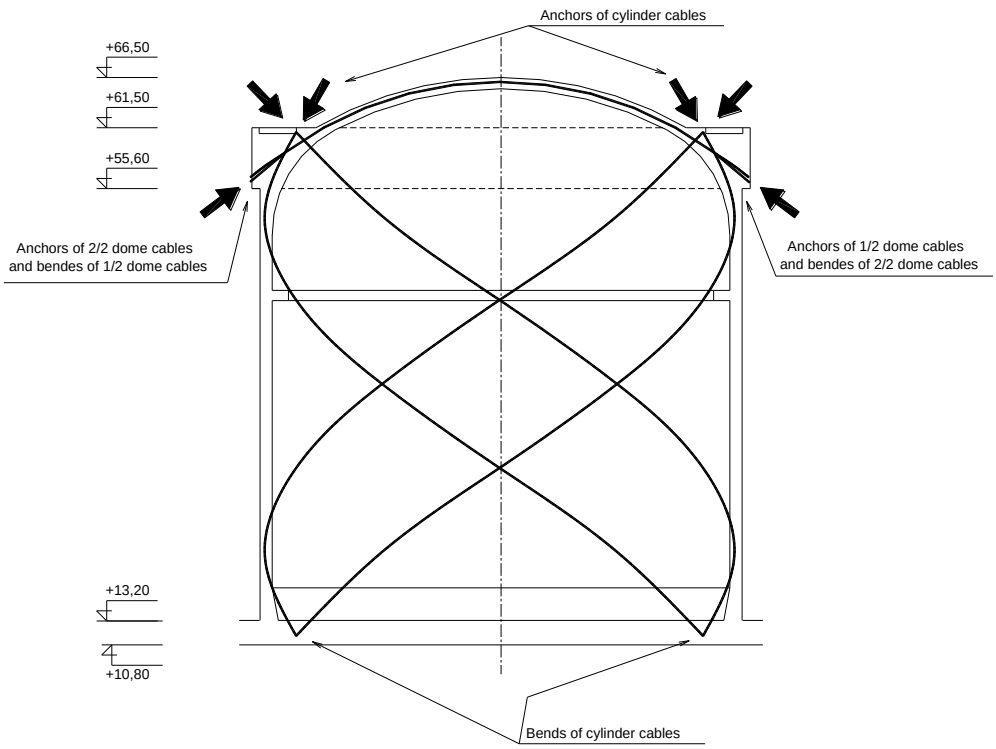
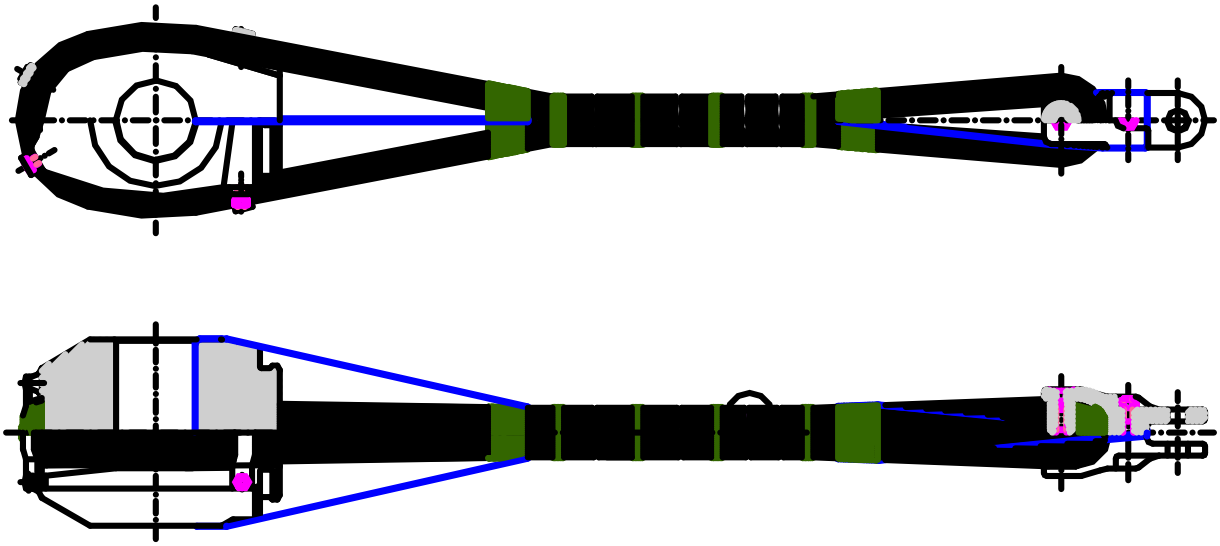


Figure 0.4 Scheme of tendon – final anchor head in on the left side and inserter used for installation of the tendon into the structure is on the right side.



3.4 Greased sheathed strands (GSS) technology

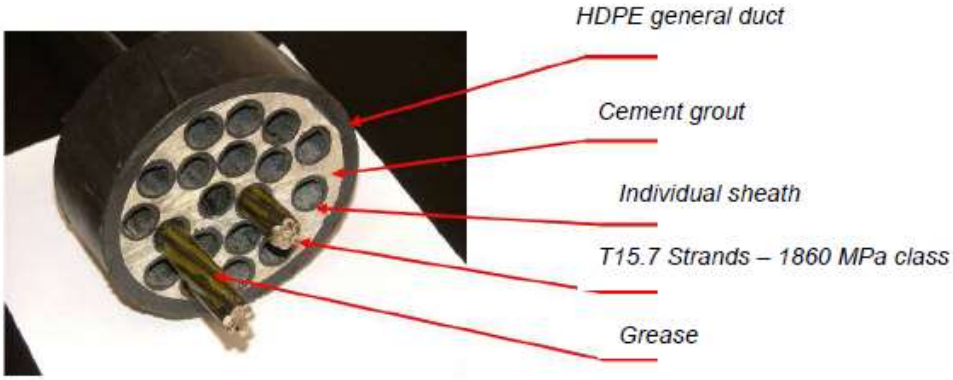
3.4.1 Technology overview

In the field of Nuclear Civil Engineering, prestressing technologies by post-tension are generally separated into two broad categories: prestressing technologies injected with cement grout (bonded) and

prestressing technologies injected with grease or wax (unbonded). In each of these cases, the injection process aims to ensure effective protection of the tendon against corrosion during the operating life of the structure.

An alternative technology was developed: prestressing by Greased Sheathed Strands (GSS), see **Figure 0.5**. These tendons have the particular feature of being made up of greased strands, individually sheathed at the manufacturing stage by a layer of hot-extruded high density polyethylene (HDPE). All strands are then gathered into a general duct (thick HDPE or metallic). This approach lies in the fact that the duct is injected with cement grout before tensioning. This allows setting the geometry of the tendon to limit interactions between the strands at the tensioning phase. Indeed, it prevents damage to their individual protection (consisting of grease and the individual sheath)

Figure 0.5 Section view of a GSS tendon - Example of a tendon 19T15, 7



Thus, this technology allowing each strand to slide freely in its sheath gives an optimal anti corrosion protection formed by the individual HDPE sheath, grease and by injecting cement grout in the general duct. This protection is provided from strand manufacturing, at the stage of implementation on site and throughout the operational phase of the plant. In addition, the friction coefficient between the tendon and the sheath, which is very low, allows optimisation of the prestressing design. Finally, as this prestressing technology is unbonded, tendons restress or replacement during the operating life of the structure is possible. Nevertheless, the design of a containment building with prestressing (partial or total) through a Greased Sheathed Strand (GSS) solution results in a certain number of notable impacts on the design and requires to take into account specific structural arrangements.

Figure 0.6 Bright strand and sheathed greased strand T15, 7

3.4.2 Geometric characteristics

3.4.2.1 Strands

Sheathed greased strands have geometric characteristics identical to bright strands used in a prestressing injected with cement grout, grease or wax. Their diameter is slightly increased due to the presence of the individual HDPE sheath.

3.4.2.2 Ducts

For GSS tendons, the ducts used can be thick HDPE duct, steel strip sheaths or metallic tubes. However, whatever the type of duct selected, they are systematically larger in diameter than the diameter used for tendon injected with cement grout or grease of the same unit. Thus, this parameter must be taken into account when defining the location of the ducts in the wall and can affect the thickness of the different parts of the containment (cylinder wall, dome).

3.4.2.3 Anchorages

GSS tendon anchorages (anchorage blocks, wedges) may be identical to anchorages used for tendons injected with cement grout or grease of the same unit as the individual sheaths of the strands are removed near the anchorage area. The entire anchoring system must be covered by technical approval.

4. GENERAL PHILOSOPHY OF BONDED AND UNBONDED, AIRCRAFT ASPECTS AND TECHNICAL CONSIDERATIONS

4.1 General philosophy

The containment is a prestressed concrete structure and it goes back to the beginning of the decade of the 60's. The evolution followed the growing of nuclear power reactors with the consequences of a reference accident more severe in terms of pressure and temperature (from around 0.1 MPa for the graphite-gas, the pressure rose to 0.6 MPa for the last generation of PWR and even higher in case of beyond design accidents).

The prestressing is "THE" solution to sustain such pressures despite all the drawbacks in terms of difficulties in design, construction work and monitoring. The prestressing in a nuclear power plant, has an essential function in case of internal accident. A containment is a reinforced "concrete + prestressed" structure which can suffer of durability problems under certain conditions. In many cases, such problems were accompanied with corrosion of the non-prestressed and prestressed reinforcement in the structure. However, the corrosion of the reinforcement is usually not the root cause of the durability problem but rather a consequence of inadequate consideration for durability in the overall design of the structure.

The problem is that the design life of existing NPPs was often chosen to be 30-40 years. However, the economical constraints push the utilities to extend plant service life (60 years total being a quoted target) and decommissioning strategies that involve use of the containment as a "safestore" for periods of up to 100 years, mean that the containment buildings may have to perform safety functions for a time period significantly greater than their initial design life.

The containment is, with the vessel, the 2 main components impossible to replace. So the requirements in terms of durability of the prestressing system are very stringent.

The main concern is the loss of prestressing forces due to ageing phenomena of materials (creep and shrinkage of concrete, relaxation or corrosion of steel). Every phenomenon has been studied for many years and can be handled in a proper manner (high performance concrete, new material for tendons and rebars). The protection against corrosion remains a pending option, mainly the question of grouting or not grouting the tendons into their ducts.

Almost half of the nuclear containments in the world are unbonded and half are bonded.

The **philosophy** is the following:

- ***Bonded technology:*** these countries favour the mechanical aspects of the structure behaviour and prefer to make an important effort at the construction phase (injection has to be "qualified" by a mock-up) and perform an visual inspection coupled with mechanical structural analysis with the periodical pressurization test thanks to instrumentation which is a good tool to assess the global mechanical behaviour of the prestressing system.

- ***Unbonded technology***: these countries have the "St Thomas" philosophy; they believe only on what that can be seen during the NPP life time and this technology is the only one that can directly give direct information. Retensioning of tendons is available in case of prestressing loss.

The mechanical behaviour, using the unbonded technology, is slightly different from the bonded technology because locally the tendons can slide inside the ducts, particularly in case of severe accident (see section 0 regarding beyond design).

The lessons learned from the past showed that for the plants in operation the ageing of concrete and consequently the loss of prestressing force was often encountered (see the OECD WS in Civaux in 1997) but also some evidences of steel corrosion in unbonded tendons.

Safety considerations have to be recalled and kept in mind when comparing advantages and drawbacks of the tendon protection either by cementitious grouting or by grease/wax injection:

4.2 Aircraft impact aspects

4.2.1 Introduction

In the event of an aircraft impact, a concrete containment can be subjected to loads and damage, possibly reaching the ultimate capacity of the structure. Thus, each part of the structure (the concrete itself, reinforcing steel, prestressing steel, liner, etc.) plays its role in the resistance and in the leak tightness of the wall. As investigated below, the behaviour and the confining performance of a prestressed wall submitted to an impact may depend on whether the tendons are bonded to the surrounding concrete or not.

4.2.2 Structural behaviour of the containment wall

An aircraft crash on a civil structure has, in broad outline, four types of consequences on that structure: axial and bending deformations of the wall, shear deformations of the wall, local damage (for instance spalling and penetration) of the wall and induced vibrations.

Figure 0.1 Missile impact effects on concrete target (from Q. M. Li et al. International Journal of Impact Engineering 32 (2005) 224-284)

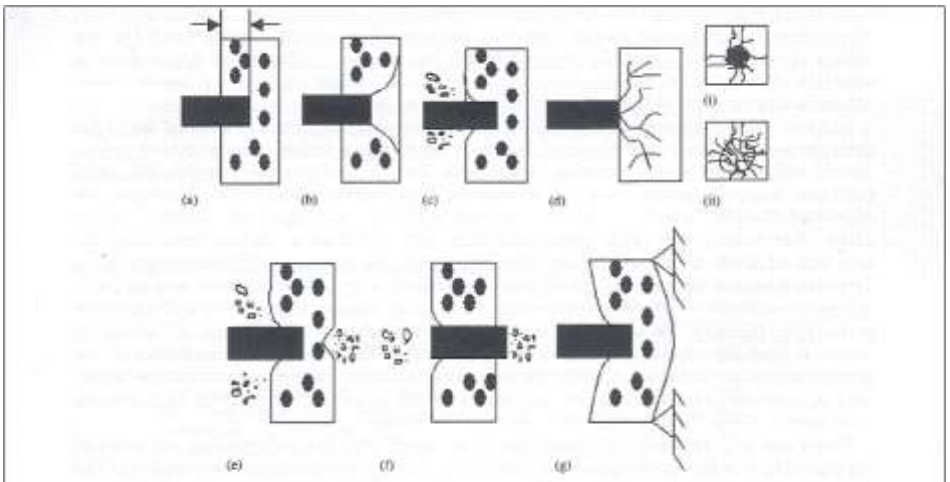


Fig. 1. Missile impact effects on concrete target, (a) Penetration, (b) Cone cracking, (c) Spalling, (d) Cracks on (i) proximal face and (ii) distal face, (e) Scabbing, (f) Perforation, and (g) Overall target response.

For the present thoughts, the depth of the local damages being a key parameter, two cases are considered, whether the tendons are damaged or not.

a) The tendons are not damaged

In the case the tendons are not directly harmed during the crash, the projectile might have caused some spalling of the front face of the wall, possibly has penetrated in the wall, but leaving the ducts and the tendons undamaged.

For a given impact, the dynamic response of the impacted structure depends on its dimensions, on its reinforcement and on the prestressing forces applied to the wall. During the impact, the curvature of the wall does not change much and the prestressing forces remain more or less identical, whatever the protection is (bonded or unbonded tendons). In the case of unbonded tendons, some sliding of the tendons in their ducts may occur, but no significant modification of the prestressing forces is expected.

Finally, the main difference between the two technologies lies in the fact that, in the case of bonded tendons, the tendons act, in addition to the structure prestressing effect, as passive steel (like rebars) and contribute to the resistance of the reinforced concrete sections in the impacted zone, increasing the mechanical resistance of the wall. That advantage has to be quantified on a case by case basis.

b) One or several tendons are damaged

In this case some parts of the projectile have caused significant damage to the wall; their penetrations into the concrete have reached the tendons, so that strands or complete tendons are ruptured. Due to the metallurgical nature and to the tension state of the prestressing steel, any notch might lead to the brittle failure of a wire, strand or tendon. Those failures of impacted tendons lead to a thorough loss of prestressing forces if the tendons are unbonded. If the tendons are bonded, the loss of prestressing is only localised, because of the link between tendons, bonded ducts and concrete.

Then, the capacity of the containment following an impact can differ: if a few tendons are broken and completely untensioned (unbonded tendons), the ability of the containment wall to withstand an internal pressure will be affected, possibly leading to early releases in the environment if ever a LOCA is the consequence of the airplane crash. Conversely, if the broken tendons are bonded, the loss of prestressing forces is limited to the vicinity of the damaged zone, and a possible internal pressure may be balanced by the wall. Of course, that conclusion is valid only if no perforation or important through-wall cracking of the wall occurs.

Besides, in the case of unbonded tendons, the ducts being not completely filled with resistant material (steel or grout), the concrete structure is less monolithic than in the case of bonded tendons. Leak tightness of the containment wall

Following an aircraft impact, the leak tightness requirement assigned to the containment depends on the accident scenario, which can be with or without inner pressure.

In the case of heavy damage to the containment wall induced by an aircraft crash, the leak tightness of that wall depends on its crack network. Important and numerous cracks mean not only a poor leak tightness of the concrete itself, but also significant strains imposed to the steel liner, increasing the probability of its tearing. In that situation, the possibly pressurized air and steam fluids present in the reactor building will seek their way out through the wall, even more easily if they find some voids or possible paths that interconnect the cracks. If not completely sealed, the prestressing ducts could offer such paths. In that respect, the bonded ducts technology appears as safer.

4.2.3 Conclusion

As a conclusion, a containment wall whose tendons are protected by grouting, thus bonded to the structure, appears as more robust to impact loads than a containment whose tendons are greased or waxed, thus unbonded to the structure. The main reasons are the ability of bonded tendons to remain anchored to the concrete even if they are locally ruptured, and the contribution of bonded tendons (as passive steel) to the capacity of the reinforced concrete sections in the zone of the impact.

4.3 Technical considerations for both technologies

4.3.1 Durability & ageing management

The ageing management has to be a concern from the very beginning of a project, all along the life of the prestressing items. The main difference between bonded and unbonded tendons relies on the fact that bonded tendons cannot be directly tested and cannot be replaced. Bonded tendon durability relies on the control and surveillance of all related activities under quality procedures and, once bonded, on the monitoring of the deformation of the containment wall. Unbonded tendons can be tested (by lift-off) and replaced if necessary. The program of inspection shall be consistent, in terms of quantity of tendons to be tested, of periodicity and of adequacy of protection by wax or grease.

4.3.1.1 Bonded tendons

The cementitious grout surrounds the tendon in an alkaline environment that will inhibit corrosion of the steel, and prevents the ingress and circulation of corrosive fluids. In case of break of a tendon, due to the bond with the grout, part of the prestress remains transmitted to the concrete. Therefore bonded tendons are less vulnerable than unbonded tendons to local damage. They reduce the risk of the containment being by-passed via tendon ducts, particularly important where the containment is unlined. However, bonded tendons cannot be visually inspected, mechanically tested or re-tensioned in the event of greater than expected loss of prestress.

4.3.1.2 Unbonded tendons

Prestressing force is transmitted to the concrete, primarily, at the location of the anchorages. Corrosion is prevented by organic petroleum based greases or corrosion inhibiting compounds. These are either applied to the surface of the tendon prior to installation or injected into the tendon duct following completion of the stressing sequence. Some countries use a combination of both coating and injection. Tendons can be removed for visual inspection/replacement; mechanically tested in-situ; and retensioned to maintain prestress. Ducts may provide a route for containment by-pass in unlined containments, although the practice of keeping ducts filled with corrosion protection medium reduces the likelihood of by-pass.

4.3.2 Synthesis of technical considerations

A list of technical considerations is given below:

- Unbonded tendons have a lower construction cost: installation is quicker than for bonded tendons,
- The possibility of monitoring the state of the unbonded tendons during the entire lifespan of the plant and in particular for identifying the breakings of strands is a good point,
- Ease of replacement of defective elements of unbonded tendons is a good point,
- The possibility of retensioning the unbonded tendons if necessary is a good point,
- Unbonded tendons need more effort during lifetime for corrosion protection,
- The sensitivity to corrosion of unbonded tendons requires increased strict maintenance procedures,

- In the case of double containment, unbonded tendons are more vulnerable to fires: in particular there is a risk of losing horizontal cables because the anchoring areas are located in the inter space between reactor containments, which is classified as safety fire zone (SFZ). This sector type is created to protect the safety lines from common cause failure. The walls boundaries of these safety fire sectors must have an (R) El 120 level of fire resistance and active or passive fire protection devices should be installed, where necessary, to ensure their integrity, in the event that the 120 minutes resistance is exceeded. That is to say that the cap, the anchorage heads and the cables must have to be protected against the fire effects. It should be noted that the problem deals with the soft protecting product which is sensitive to high temperatures (the temperature in such a closed space can reach at least 500°C after 5 minutes),

- The availability of retensioning the unbonded tendons is limited by the capacity of resistance: in some cases, the tension of the unbonded tendons may be limited by the capacities of resistance in compression of the metallic liner, which is already reached only under operational conditions, including creep and shrinkage of concrete. Moreover, some feedback shows that the rare ruptures of strands observed occurred during tensioning phases at an early stage,

- Any detensioning or retensioning operations of unbonded tendons must be carefully planned and executed taking into account,

- The practicability of retensioning unbonded tendons depends on the prestressing system employed. The number of tensioning and detensioning operation completed may limit the life on the cables,

- Other issues about restressing possibilities of unbonded tendons may be limited for technological reasons (number of detensioning/retensioning, values. relaxation, etc.),

- The rupture of a bonded cable is irremediable, whereas in the case of the rupture of a bonded cable the effect of adherence between cable and concrete enables the cable to continue to provide passive reinforcement, albeit with a reduced capacity.

4.3.3 Technical considerations about GSS technology

The GSS tendons allow:

- Strength of all the tendons can be checked, tendons can be restressed, removed and replaced (unbonded technology),

- Very low friction coefficient: less tendons are necessary and the prestressing strength is better spread,

- Very good protection against corrosion due to many fences: grease and HPDE individual sheath (from strand manufacturing), cement grout and general duct,

- Due to the individual protection, strands can be threaded inside the ducts and cement grouted during the containment construction: the prestressing schedule can be reduced.

On the other hand, GSS technology requires:

- Greater diameter of the ducts used for GSS tendons than those of injected tendons,

- Participation of tendons cannot be taken into account in the reinforcement calculation (except for membrane strains),

- If a tendon failure occurs, the prestressing strength is lost all along the tendon (unbonded tendons) contrary to bonded tendons,

- Significant impacts on layout: sufficient excess length of the tendons to consider, maintenance area near the ribs, the upper ring beam and prestressing gallery to take into account.

5. FEEDBACK EXPERIENCE OF THE PERFORMANCE OF PRESTRESSING TENDONS

5.1 Introduction

In fact, due to the strong connections between nuclear and non-nuclear activities, civil engineers were necessarily influenced by the problems encountered on bridges or other structures like tanks or silos. Concerning the problem of tendon corrosion, unbonded tendons have been preferred by Dischinger in early posttensioned structures. However, under the influence of Freyssinet and other prominent engineers, the advantages of structures with bonded tendons were emphasized and this type of tendon became the common practice.

5.2 Non-nuclear domain

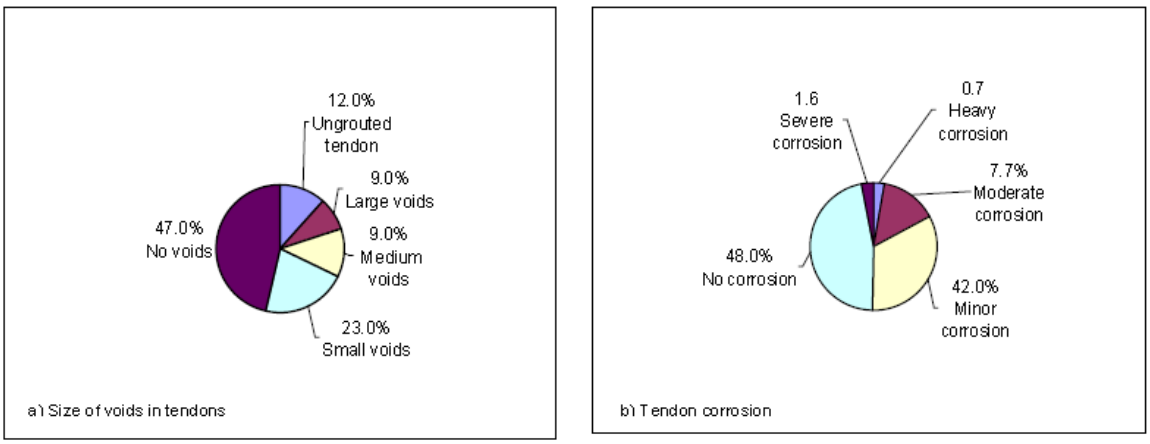
Tendons that are left permanently unbonded can also be used in bridges. The most common such application in new construction is the use of external tendons in precast segmental structures. One of the advantages of unbonded tendons may be the speed of the construction.

The prestressing steel, enclosed in plastic ducts and injected with grease or wax, is protected from severe environment. Unbonded tendons can also be used for rehabilitation of existing bridges. For deck slabs with high concentration of chlorides but otherwise sound concrete and reinforcement, for example, providing an additional layer of concrete reinforced with transversely with unbonded tendons may be less costly alternative to deck replacement.

Between 1979 and 1992 the UK Standing Committee on Structural Safety (SCOSS) published annual reports (SCOSS, 1979-1992) which included reports on suspected deficiencies of grouting of post-tensioning tendons and in 1992 the UK Department of Transport banned ducted bonded post-tensioning in bridges. This marked the beginning of a process of investigation and review of standards and procedures which have lead over 20 years to national and international bodies introducing improvements to help ensure the safety and reliability of post-tensioned structures.

There are significant lessons to learn from the feedback experience for structural designers, material scientists, and owners of structures both for new designs and for asset maintenance.

Figure 0.1 Results of post-tensioning tendon inspection of 447 bridges in the UK, [62]



External unbonded tendons were banned in the UK in the 1970's after some problems have been found. External tendons have later been strongly promoted by Jean Muller and other French engineers in

conjunction with precast segmental bridge construction in France and in particular in Florida. Under the auspices of SETRA (State design office of highway authority in France) many bridges have been built in France using either external tendons only or a combination of internal bonded, and external unbonded tendons.

While this construction practice was not accepted previously, the German highway administration recently declared unbonded tendons as the preferred type of bridge tendons.

The feedback experience documents that there is not one superior type of technologies but it may be appropriate to repeat the strengths and weaknesses of bonded and unbonded tendons.

5.2.1 Experience feedback from French sector

Concerning the problems encountered on prestressed structures, the area for which the most complete experience feedback is available is that of bridges. The birth of French prestressing engineering was in 1939 when E. Freyssinet developed the conical friction anchorage and the double-acting jack. However, the origins of prestressed concrete are much older, when Freyssinet built in 1908 an experimental arch at Moulins with 50 m span and 2m rise. The 'injection' was very simple – only sand-sealed by a hydraulic mortar – and when excavations were carried out in 1993, the steels rebars were still in good condition. Since the Second World War, much attention has been given to the problem of durability of tendon protection.

5.2.1.1 Problems arising from the cement grouting

At the time of construction of the first generation of VIPP (bridges with simply supported spans using prestressed concrete beams) cement grouting methods were far from being mastered. However, with experience, manufacturers improved their products and contractors improved grouting methods in order to solve the many existing problems. From 1970 onwards, the gammagraphy examination of tendons, by ensuring a control of the results of grouting, made it possible to detect filling defects and to improve the technique.

The Provisional Directive for grouting the ducts of prestressed concrete bridges was issued on 28 March 1973 by LCPC and SETRA (SETRA is the technical department of the French Ministry responsible for public works and transport in charge of design and construction of motorways). It deals with the specifications which the grouting should meet (traditional and special grouting), the consistency of the specimens for design and conformity, the manufacture of the grout, the carrying out and checking of the injection, the reporting of any incidents, etc.. This Directive made it possible to solve most grouting problems discovered between 1970 and 1973; it was accompanied by the development of regional formulations of grouts and the implementation of a policy of controlling duct filling using gammagraphy.

From 1978, the change introduced to the standardization of cements led to a progressive neglect of regional grout formulas, and from the beginning of the1980s the use of gammagraphy to control duct filling decreased rapidly to the point of not being implemented at all except for in occasional cases, due to the opinion that grouting operations had been sufficiently mastered.

During the last 20 years, progress has been made with the use of increasingly fluid grouts to facilitate the injection of the cable ducts. From the 1980s onwards, the search for a greater fluidity was accompanied by a generalised use of super-plasticizers. These developments in grout formulation and grouting methods were imposed by the contractors themselves, without, however, regularly ensuring the effective filling of the ducts.

However, other anomalies were detected during a gammagraphy survey on a box girder bridge under construction in 1994, consisting of a lack of filling and a presence of soft and wet paste. As it was not the first anomaly of this type, LCPC reproduced the defects during tests of injection of cement grout with admixture carried out in tilted transparent tubes. When certain conditions are met, there is a separation by density between the cement suspension in the course of hydration and these lighter mineral materials. A whitish product is thus created and transported by air bubbles to form a layer of whitish paste at the end of the test. This paste was analysed and all the components identified. This phenomenon is amplified by the super-plasticizer which can present an incompatibility with the cement.

Thus, in absence of a sweating of the grouting, and even using a grout without exudation problems, the injection of very long tendons (200 to 300 m) can suffer problems such as air pockets. For this reason, the LCPC has developed a test on a tilted 5 m length, in order to validate a normalized test and the different parameters important for avoiding these anomalies.

5.2.1.2 Problems with sheaths

Problems with sheaths are very rare on bridges due to the properties of the material used: High Density Polyethylene (HDPE). Only 2 cases are reported for sheaths cracking, due probably to an inadequate formulation of the HDPE.

The only concern of the sheaths is his leak tightness and on this fact, his influence is more sensitive is the nuclear domain.

5.2.1.3 Problems with grouting using soft products

Injection by soft products covers the use of grease and oil wax. The first injections of external tendons with grease did not give completely satisfactory results, mainly because grease proved to be unstable in the short-term and because of leakage through ducts, often constituted of the oily part of the grease (example of the strengthening of the Bayonne Bridge by additional prestressing). It is for this reason that soft injections are currently carried out with wax which is a more stable product in the long-term, and which until now has not posed any problems with regards to durability, according to the few sheath or anchoring cap openings which have been made.

It remains also progresses and R&D to achieve in the domain of soft products.

5.2.2 Historical background of post-tensioning in Great-Britain

Post-tensioning is a relatively young technology compared with reinforced concrete, and dates from the mid-20th century. The theory and history of post-tensioning is well documented and mainly falls into two types – external and internal post-tensioning, with both being pioneered and leading to distinct structural forms.

High carbon high strength steels are used to provide this tensile strength to concrete structures in the parts subjected to tension, with the concrete itself providing the compressive resistance in compression zones. However these high strength steels are more sensitive to corrosion in the presence of chlorides than normal reinforcement, and the common use of de-icing salts on roads has caused problems.

Common to most systems was the need to inject cement grout into the ducts to protect the steel from corrosion and, for internal tendons, to provide bond to the structure. Ducts for internal tendons were seen as a means to provide a conduit to place the steel where it was needed, and were sometimes initially simply cardboard tubes, or formed by inflated rubber tubes, but later became more commonly of steel.

Other systems rely on grease and plastic sheathing to provide corrosion protection. Early use included polyvinylchloride (PVC) sheathing which was subsequently found to be susceptible to release of hydrogen ions and damaging to high carbon steels. Modern plastic sheathing is normally polyethylene (PE) or polypropylene (PP). External post-tensioning systems usually had purpose cast or forged steel deflectors connected or cast into the concrete structure to deviate the tendons.

One could describe such post-tensioned structures as “highly strung” in that they are finely tuned and there does tend to have less redundancy or reserve of strength, although this depends on the disposition of steel reinforcing bars and the structural form. For example precast concrete segments held together simply by post-tensioning are totally reliant on the prestressing, whereas an in-situ concrete structure with continuous reinforcement has some degree of additional capacity.

5.2.2.1 Review of problems

From quite early on there were problems discovered in some UK post-tensioned structures, primarily bridges, for various reasons. Where inspection was possible, often this was attributable to leaking of water reaching the tendons of external post-tensioned bridges.

Other problems related to materials or were unaccounted for. Two significant externally post-tensioned bridges, Braidley Road Bridge in Bournemouth and the A3/A31 Bridge over the A3 in Guildford, both had their external tendons completely replaced in the 1980’s. The fact that such replacement could be achieved is a pointer to one of the major advantages of external post-tensioning.

As most of the early evidence of problems had been concentrated on externally post-tensioned structures, simply because their cables were visible for inspection, there was a ban introduced by the UK Department of Transport in 1977 on use of external post-tensioning in bridges. With the benefit of hindsight, one can challenge whether this was a sensible decision. There is little doubt that external post-tensioning has a significant advantage in that it is relatively easily inspectable, whereas internal post-tensioning is buried inside the body of the concrete and despite significant investment in recent years has proved extremely difficult to inspect.

Nevertheless fortunately development of external post-tensioning technology continued outside of the UK and now forms an important part of many significant concrete bridges. In the UK the Camel Viaduct in Cornwall, completed in 1993 was the first to utilise new design rules developed by the Highways Agency (1994) and included innovative features to allow replacement of the post-tensioning cables in the future.

In 1985 the sudden collapse of the Ynys-y-Gwas Bridge in Wales had sparked investigations which raised serious concerns over the underlying durability of internal post-tensioning. In that instance it was a single span precast segmental bridge with internal tendons which passed through thin concrete joints between the units in ducts formed with cardboard. Over the years de-icing salts had penetrated and corroded the prestressing steel at the joints resulting in collapse.

In 1986 the bridge over the Mandovi River in Goa, India collapsed after less than 20 years in service due to corrosion of the post-tensioning cables. Cracks were unattended for six years and corrosion of prestressed wires, which was noticed in 1983, was neglected until the bridge collapsed. It is reported by Arullappan (2010) that the available workforce was probably not able to provide the required standard of workmanship to construct the bridge.

In 1992 a further bridge collapse occurred in Belgium. The Malle Bridge over the river Schelde relied entirely on buried tie-down prestressing tendons for its stability which failed due to corrosion (ASBI 1998). The tie down cables were completely uninspectable and this is a prime example of a high-risk form of structure. Such details should be avoided.

It was not only bridges which experienced durability problems with prestressing. In Berlin the roof of the West Berlin Congress Hall collapsed in 1980. The thin shell roof had post-tensioning bars in ducts which had corroded partly due to poor quality of grouting (Buchhardt et al, 1984).

Doubts were shed on the adequacy of grouting materials and techniques and some of the construction details, and following the SCOSS concerns a ban on bonded duct post-tensioning was introduced in 1992 by the Department of Transport.

There already existed the ban on externally post-tensioned bridges due to previous failures. It was generally accepted that this did not mean that all post-tensioned structures were unsafe, but the immediate response was to carry out a study of different types of post-tensioned structures and systems and broadly categorise them by risk assessment of the potential impact of deficient grouting and possible corrosion of the tendons. At the same time a programme of physical intrusive investigations was embarked upon to assist in understanding the degree of deficiencies and current extent of damage with a view to establishing a programme of repair options. Early studies published by Woodward (1981) had shown a mixed picture of different conditions in existing structures. This manifested itself in some cases as fully bonded but in others as complete lack of grouting, or just a thin coating of the steel. However, generally little corrosion was found.

The Highways Agency in 1994 started a significant structured series of inspections of their bridges and this gradually provided evidence that there was a growing problem with some post-tensioned structures.

5.2.2.2 Lessons learned and areas for improvement

In 1992 the Concrete Society and the Concrete Bridge Development Group plan and instigate a program of studies with a view to making recommendations for improvements for future structures, with the intent to allow the ban to be lifted. This involved reviewing existing standards, carrying out development and testing of new materials, consultations with consultants, suppliers and contractors.

Concrete Society issued different reports in 1995, the Technical Report No 47 (1996) which resulted in the ban being lifted. It has been revised over the years and is now Technical Report No 72 (Concrete Society 2010). This has a comprehensive record of the work carried out and contains many recommendations.

In the rest of the world there was a growing awareness of the problems in the UK disseminated through bodies such as the International Federation for Structural Concrete (*fib*) and gradually it became apparent that the problem was not just limited to the UK. In the USA important bridges in Florida revealed problems at Sunshine Skyway, Niles Channel and some other major bridges and other states. Swiss, German, Japanese and French experience was shared. *Fib* established a working group to prepare guidance on grouting published as Bulletin 20 (*fib* 2002) and another to give guidance on use of plastic ducts. A workshop on durability of post-tensioning tendons was held in Belgium (Ghent) 15-16 November 2001 (Bulletin 15).

5.2.2.2.1 Grout

For many post-tensioned structures grout is used to protect the steel from corrosion. This has proved to be a fundamental weakness in many cases and something to which major attention has been paid over recent years to improve. Long used test methods were found to be inadequate to distinguish between good and bad grout formulations, which resulted in some tendons having voids in their protection or poor quality of grout leaving the steel susceptible to attack by penetrating chlorides. An understanding has been gained through research of the key qualities of grout needed to assist ability to fully grout the ducts and to provide

long-lasting protection, with a particular focus on control of bleeding. Use of factory formulated pre-bagged grouts has now become almost the norm in the UK and a certification scheme is in operation.

5.2.2.2.2 Ducts

The role of long term protection provided by the ducts was identified at an early stage as an area for possible improvement. In aggressive environments such as bridges steel ducts are not considered adequate where de-icing salts may penetrate to corrode them, especially at joints in concrete members. The requirements for the development of new product were essentially simple – what was needed was a sufficiently robust non-corrodible watertight duct to be able to be set to the required profile, withstand the duress of concreting and strand installation and stressing operations and be fully fillable with grout. It also had to be able to transmit cable force by bond to the concrete body.

The challenge of coordinating introduction of requirements for new non-corrodible plastic ducts was taken up by *fib* who published an authoritative guideline Bulletin 7 (*fib* 2000) which has become referenced internationally. Plastic ducts had been previously used in Switzerland with good experience but a new generation of ducts needed to be brought to market.

5.2.2.2.3 Quality and skills

The UK also proposed an industry certification scheme which ensued from the process in direct response to requirements of the new recommendations, covering requirements for post-tensioning companies to satisfy certain levels of training and experience of personnel, as well as having documented QA procedures for carrying out post-tensioning work. This also raised a classic dilemma; how to require companies to demonstrate skills and experience with a new set of standards before the standards were in operation.

The solution was to introduce, with the support of the UK Highways Agency, an interim pre-certification scheme to allow close scrutiny, monitoring and recording, which would lead to full certification. Similar schemes are not universally used around the world although the Post Tensioning Institute in USA has comprehensive skills and training qualification for firms and their operatives, and many global post-tensioning companies now have their own documented requirements in this area. In Europe similar requirements are now covered by EN13670 (BSI 2009) and the need for companies to hold a European Technical Approval for their system (EOTA 2002) and to comply with a CEN Workshop Agreement for installation (CEN 2003).

5.2.2.2.4 Detailing

In addition to the practical aspects of improving the materials it was considered vitally important to learn from design details which had failed or proved to be troublesome and provide guidance for structural designers.

Experience of corrosion of tendons in or near their anchorages, informed the need for added protection to anchors placed in vulnerable locations under expansion joints or construction joints. This was developed to provide the concept of multi-layer protection whereby a second or third line of defence was provided to the tendons rather than simply relying on one single protection method. It was recommended to avoid anchorages in pockets in the top slab of bridges as these appeared particularly vulnerable to leakage. Additional double layers of deck waterproofing was recommended in certain locations and guidance given on placement and sealing of duct vents which had been shown from trials to help complete grouting at vulnerable high-points in duct profiles.

Full scale prototype trials were advocated on projects to verify the suitability of proposed grouting materials and techniques prior to full scale use.

Experience on these was invaluable to reassure clients and designers that the ducts could be fully bonded and also inform simple but crucial detailed changes to, for example, grout vent locations and grouting procedures.

The concept of multi-layer protection and suggested design details to avoid such problems are described in Concrete Society Report TR72 [33] and in *fib* bulletin No. 33 [25].

5.2.2.2.5 Inspection

Owners of post-tensioned structures, especially bridges, have had a wake-up call and special inspection procedures and risk assessments for these structures were introduced by many authorities. Even so it is still the case that defects are in some cases a surprise when discovered such as recently on the Hammersmith Flyover in London which is currently the subject of a major emergency strengthening program (New Civil Engineer 2012). This precast segmental bridge, post-tensioned both internally and externally was built in 1962 and has classic features of high risk with in situ concrete joints. It does not seem surprising to the author that the post-tensioning cables in ducts at the top slab over the piers could be suffering from chloride induced corrosion, the extent of which has only now been discovered 50 years on.

Methods of inspection are well documented (Concrete Society 1996) and intrusive investigation is generally recognized as the only reliable way to check condition of internal post-tensioning. In many cases this is difficult but initial examination of design details can and must be used to inform targeted investigations.

It is clear that in most cases a post-tensioned structure will be unlikely to show visual signs of distress when close to failure. This makes it all the more important to carry out full investigations where there may be vulnerable high risk details.

Owners of post-tensioned structures must be helped to understand that these structures are unique and very careful consideration needs to be given to their inspection.

5.2.2.2.6 Maintenance

Experience of repairs of post-tensioned structures is relatively limited. Several bridges have been so complex to evaluate the overall state with confidence that they have been demolished and replaced. However where there has been significant confidence to design strengthening or replacement, various techniques have been used, including supplementary tendons, as in the case of Hammersmith Flyover, passive FRP strengthening, or tendon replacement.

The bridge at Palau in Indonesia collapsed suddenly in 1992 some weeks after completion of a strengthening program and a protracted legal case resulted in details not being revealed until several years later. This was an extremely long span internally post-tensioned box girder cantilever bridge with a pin at midspan. It was initially hypothesized by some that the additional prestressing which was applied to strengthen the structure and help correct significant sag of the main span may have contributed to the failure. However a later published paper (Burgoyne et al, 2008) postulates (with a cautionary note) that there were weaknesses in the original design and workmanship which could have been the main reason. Tendon corrosion was not apparently a contributor.

It is clear that many poor details on existing post-tensioned bridges should be corrected to ensure longevity and arrest possible deterioration. For many, however, without any obvious poor details, experience is that they should remain trouble-free. Post-tensioned structures when designed and detail properly certainly have the potential to be one of the most durable and long-lasting forms of construction.

5.2.3 New standards: UK, European and international developments

Publication of substantial new recommendations in the first edition of Concrete Society Report TR47 (1996) had led to the UK Highways Agency announcing a lifting of the ban on internal post-tensioned bridges. This was welcomed by industry but it was recognized that further development was necessary, assisted and informed by the International community.

In parallel CEN had instigated a review of existing standards for grouting, ENs 445, 446 and 447 (BSI 2007a, 2007b, 2007c) and publication of these was the product of a consensus view across Europe for improved grouting materials and injection techniques.

A second edition of TR47 followed in the UK in 2002 to reflect the European experiences. In the meantime international developments were being followed and *fib* had published bulletin 33 (*fib* 2005) which further disseminated state-of-the-art knowledge.

By the mid 2000's Engineers were gradually coming to a consensus view of the ways to improve the durability of grouting for post-tensioning, so an ISO Committee was convened in 2008 to develop International Standards for grouting informed by the ENs, new National standards in USA, Japan and many European countries.

ISO14824 Parts 1, 2 and 3 published in 2012 covers basic requirements, test methods and grouting procedures. However there is still remaining much to learn.

Experience from developing these standards has revealed that the current European requirement for grout to contain less than 0.01% of sulfide ion is apparently impossible to ascertain from any current ISO test (it is below measureable tolerances). It has emerged that there has been very little research into threshold limits for sulfide ions in contact with prestressing steel and even more concerning, that it appears that although this has been a required limit in the current EN for some 10 years suppliers appear not to have tested their compliance. This is a note of caution for standard developers to always seek firm evidence as back-up before including requirements proposed by one country.

5.2.4 Recent experience and future developments

5.2.4.1 Recent problems

In the USA it was revealed in 2011 that SIKA, a pre-bagged grout supplier, had been supplying grout which had chloride levels significantly higher than the USA specification limit of 0.08%. This apparently had been supplied undiscovered and untested for several years, with the supplier relying on the supplier of the bought cement to comply with its certification regarding chloride levels and no further testing being carried out. It is reported (US FHWA 2012) that this grout has been used on over 30 bridge projects in the USA and may have serious implications which are currently under investigation.

This is potentially seriously damaging for the credibility of self-certification as well as for post-tensioning in general. We must learn from this that Audit testing MUST be a routine part of every supplier's procedure.

The 2011 findings of major corrosion to tendons in the 50 year old Hammersmith Flyover in London have luckily come in time to potentially save that vitally important structure. We must learn from this to be more questioning and reinforce to owners of structures the need to make deeper and more searching investigations where it appears fairly likely that our older structures have vulnerable details, even though it may be more costly.

5.2.4.2 Successes

We should also champion successes. Relatively few post-tensioned bridges have been constructed in the UK in the last 20 years but those which have will have benefited from much improved technology. Better still is the fact that many post-tensioned elements of UK buildings are also now using much improved standards having learned from the experience gained in bridges. This is an important example. The latest edition of the National Structural Concrete Specification (Concrete Centre 2010) embodies many of the key features of good practice for grouting learnt by industry as well as embracing Eurocodes.

Techniques have developed considerably in recent years for investigation and monitoring of post-tensioned structures, and in particular the health of their prestressing cables. However there is certainly further to go and scope for further development. These include acoustic and ultrasound technology as well as use of sensors embodied within the cables at the time of construction. Time has yet to tell whether the latter will prove valuable for long term health monitoring.

New materials are always being developed. Currently there are higher strength steels, carbon based composites and new epoxy fillings for coated strands. Each of these will need to be proven but what we have learnt from the past should enable us to take care about their use.

5.3 Nuclear domain

5.3.1 Feedback from the United States

In 1999, the U.S. Nuclear Regulatory Commission (NRC) has issued an information notice to alert addressees to the degradation of prestressing systems components of prestressed concrete containments (PCCs). Inspections of PCCs and PCC tendons have identified a number of concerns related to the degradation of prestressing tendon systems in PCCs. Findings relevant to these concerns are the following:

Breakage of prestressing tendon wires

Recent observations related to containment prestressing systems have revealed conditions that may precipitate breakage of tendon wires. Conditions such as uneven shim stack heights on the anchor-heads, spalling and cracking of concrete beneath the anchor-head base plates, free water in the bottom grease caps, poorly drained top anchorage ledges, and the absence of filler grease in various areas can lead to corrosion of tendons and eventually to wire breakage. Specific plant observations and instances of the failure of tendons and associated anchorages are detailed in Attachment 1.

Effects of high temperature on the prestressing forces in tendons

Licensees at a number of plants have reported lower-than-predicted prestressing forces for vertical, hoop, and dome tendons. Investigations and analyses have indicated that *the relaxation losses in prestressing tendon range from 15.5 to 20 percent over 40 years at an average sustained temperature of 32°C around the tendons.* However, the tendon relaxation loss values assumed in the original design of PCCs vary between 4 and 12 percent. These values were determined at the presumed ambient temperature of 20°C. The relevant plant observations and discussions are reported in Attachment 2.

Comparison and trending of prestressing forces

It is important to adhere to the guidance in Regulatory Guide 1.35.1 ("Determining Prestressing Forces for Inspection of Prestressed Concrete Containments") or equivalent methods to maintain the safety function of the prestressing tendon system and the concrete containment. Moreover, proper comparison and trending analysis are critical in determining the future trends in prestressing force in PCCs. Licensees have reported *losses using the average forces determined from the lift-off testing*, thereby masking the true variation in the loss of prestressing forces. An analysis using an average of the lift-off forces for regression

analysis does not give results that are statistically valid. Attachment 3 contains the staff's discussion of the variation in trend analysis of tendon prestressing forces.

In the United States, reviews of the performance of the prestressing tendons in these structures have revealed that corrosion related incidents are extremely limited. The evolution of corrosion inhibitors and the use of organic-petrolatum-based compounds designed especially for corrosion protection of prestressing materials have virtually eliminated corrosion of prestressing materials.

The **few** incidences of **corrosion** that were identified, occurred early in the use of prestressed concrete for containment structures. Where these failures involved tendons coated by petroleum-based materials, the failures generally resulted from the use of off-the-shelf corrosion inhibitors that had not been specially formulated for prestressing materials.

5.3.1.1 Problems and experiences during construction of PCCs

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

- At Calvert Cliff nuclear plant (Units 1 and 2) some of the bearing plates under anchor heads of vertical tendons became depressed into the concrete. These depressions ranged in size from 0.8 mm (0.03 in.) to 4.8 mm (0.19 in.) and were generally on the inside edges of the plates. The problem was corrected by detensioning the tendons at affected plates, reinstalling the plates, pressure grouting and retensioning.

- At Bellefonte nuclear plant (Units 1 and 2), failures occurred in the top anchor heads of 170-wire rock anchor tendons during installation of the rock anchorages. Anchorage of the 12.2 m (40 ft.) long tendons to the rock was to be performed using a two stage grouting operation. Failures of the top anchor heads were observed just prior to the second stage of grouting that during the period between the first and second stage grouting despite the fact that the top anchor heads were covered with grease; metallographic and fractographic examinations in conjunction with the study of the environment indicated that the failures were the result of stress corrosion cracking.

- In November 1979 four anchor heads of 179-wire tendons failed between 1 and 64 days after post-tensioning the Unit 1 containment at the Byron nuclear plant. Failures were timed delayed and occurred in a decreasing stress field.

- Concrete cracking and grease leakage were noted at various locations on the dome surface, predominately in the southern portion, after tensioning of approximately two-thirds of the dome tendons at Turkey Point Nuclear Power Plant (Unit 3). It was concluded that the dome delaminations were caused by the combined action of inadequate concrete consolidation and weakness at construction joints.

Some engineers at NRC, however, believe that well designed radial reinforcing would help prevent the situation from repeating in the domes of similar containments.

- In April 1976, surface cracking and voids in the dome concrete at Unit 3 of Crystal River Nuclear Power Plant were discovered (by accident) after the dome had been constructed and fully post-tensioned. Primary causes of the delaminations were thought to be the use of low quality coarse

aggregate materials accompanied by high radial tension forces above the top tendons, and compression-tension interaction. Other potential contributing factors were tendon misalignment and construction methods. Corrective measures included detensioning of some of the tendons, removal of the delaminated cap, installation of top orthogonal and radial reinforcing, and installation of a new cap concrete.

- In April 1998, the NRC staff visited the Oconee Nuclear Station (OCN) to discuss issues related to the licensee's license renewal technical report. During the visit, the staff performed a walkdown inspection of the OCN containments and other structures.

The following observations are related to the prestressing system degradations reported by the staff:

- At Tendon 12V6, the concrete beneath the 5.1-centimeter (2-inch) thick anchor-bearing plate had spalled along the outer edge; a cavity existed below the plate. Cracks in the concrete beneath the outer edge of the bearing plates were observed for a number of tendons.

- Tendon grease had leaked from a significant number of hoop tendons in the containments of all three units at OCN.

- The Unit 1 tendon access gallery showed water infiltration and standing water at several locations. The licensee indicated that the Unit 2 tendon access gallery at one time held as much as 51 centimetres (20 inches) of water. The licensee is periodically purging the tendon galleries of all three units to remove water.

5.3.1.2 Feedback on Crystal River 3 (CR3) containment delamination

In October 2009, a delamination was discovered in the cylindrical concrete wall of the Crystal River Unit 3 post-tensioned 6-buttress containment during construction activities for creating a construction opening for steam generator replacement. The licensee’s condition assessment determined that the extent of the delamination was limited to bay 3-4, and corresponded to an hourglass shaped area including the steam generator replacement construction opening. Following the incident, the licensee performed an extensive investigation and developed a repair program.

The delaminated condition was not an immediate safety concern, since the plant was shut down when the condition was discovered and remained in a safe shutdown condition since discovery. Along with other contributing causal factors, the technical root cause was determined to be inadequate scope and sequence of detensioning of tendons associated with the steam generator replacement construction opening activities, which resulted in redistributed stresses that exceeded the tensile capacity of the concrete. Other contributing factors included containment concrete with lower than normal tensile strength and limited crack-arresting capability because of the fragility of the soft coarse aggregate used, relatively higher level of containment prestress, and resulting stress concentrations around the tendon sleeves. Through state-of-the-art computer models, the licensee determined that the complex interplay between all the contributing causes resulted in the delamination, with the driver being tendon detensioning.

In March 2011, during the final phase of tendon retensioning to complete the repair of Bay 3-4, the CR3 containment experienced a second delamination in Bay 5-6. Subsequently, a third delamination was observed in Bay 1-2. In June 2011, the licensee announced a tentative plan to repair the delamination(s) in the Unit 3 containment building. The plan posed significant construction challenges since the containment bays that needed to be repaired were surrounded by rooflines, and in some areas inside, obstructed by adjacent buildings. The proposed repair was estimated to take up to 5 years to complete and would involve significant construction and replacement power costs. In February 2013, the licensee decided to terminate

the proposed repair of the containment and announced that it would permanently cease operations at Crystal River Unit 3 and eventually decommission the unit.

5.3.2 Feedback from the Czech Republic

5.3.2.1 Problems with prestressing system of VVER1000 V-320 type containments

Breakage of prestressing tendon wires

Results of on-site inspections and inspections and laboratory tests done on removed tendons indicate that the most sensitive part of tendon to breakage of wires is at the anchor head. There is a concentration of mechanical damage of wires that occur during the process of fabrication and installation of a tendon increased in some cases by man indiscipline (e.g. not removing of fixation tapes during installation of tendon). Another possible source of wires breakage is crossing of the individual layers of wires on anchor head arising during tendon installation which could cause local damage of crossed wires due to local pressure in crossing after prestressing of the tendon. Above mentioned potential sources of wire breakage could be eliminated by rigorous manufacturing processes. Furthermore, in Czech design the changes of anchor head have been made in comparison to original design with the aim to minimize the risk of wire breakage.

Degradation of grease

Inspections that have been done on removed tendons yet show that degradation of grease occurs in the area of anchors under anchor covers. The reasons are hard climatic conditions under anchor covers due to the enclosure of this space and the direct exposure to external environmental conditions. Depending on the site climatic conditions, there is a great difference in protective properties of the grease influenced mainly by air and condensed humidity and also thinning of the layer due to higher temperatures. Impact of grease degradation on tendon corrosion is eliminated by periodical inspections of anchors along with eventual replacement of grease. The effects of cover aeration have been tested also but without any final decision for the present. Considerable occurrence of grease degradation in areas out of anchors have not been noted yet.

5.3.3 Feedback from Canada

The CANDU reactor building at Gentilly-1 (G-1), Quebec, Canada (250 MWe) was built in the early 1970s and is currently in a decommissioned state. The structure at present is under surveillance and monitoring. In year 2000, a field investigation was conducted as part of a condition assessment and corrosion was detected in some of the unbonded post-tension cable strands. However, no further work was done at that time to determine the cause, nature, impact and extent of the corrosion.

An investigation of Gentilly-1 containment building is currently underway to assess the condition of unbonded post-tensioning cables and reinforced concrete. At two selected locations, concrete and steel reinforcements were removed from the containment building wall to expose horizontal cables. Individual cable strands and reinforcement bars were instrumented and measurements were taken in-situ before removing them for forensic examination and destructive testing to determine the impact of ageing and corrosion. Concrete samples were also removed and tested in a laboratory.

The purpose of the field investigation and laboratory testing, using this structure as a test bed, was also to collect material ageing data and to develop potential Non-destructive Examination (NDE) methods to monitor Containment Building Integrity.

What where the results? The first phase of the field investigation yielded a wealth of information on the ageing of the containment building wall of the CCB nearing 40 years of age. The second phase of the investigation will include the dome of the CCB.

The concrete and reinforcement bars were found to be in good condition. The concrete ultimate compressive strength obtained by laboratory testing was 40% higher than the design specification of 35 MPa. Concrete suffered micro cracking probably due to alkali aggregate reaction. However, this has not affected the performance of the concrete in the orientation in which the testing was conducted. This could be due to the confining effect of post-tensioning system. Ultimate strength of reinforcement bars ranged between 865 MPa to 900 MPa indicating that they have suffered no apparent adverse ageing effects.

The horizontal post tensioning cable strands in both test areas exhibited a varied range of surface corrosion. In spite of that, the strands appeared to be performing well, still meeting the design specifications in ultimate strength. The corrosion of the strands likely occurred during construction when the horizontal wall cables remained in the ducts for an extended period of time before being tensioned and unbonded. It seems that the cement grout has well protected steel cable strands from further degradation.

The in-situ residual strain in most of the cable strands measured was in the range of 4000 to 5000 micro strain. This compared well with the original design value of 4600 micro strain corresponding to the specified design working stress of 141,000 psi at 0.6 of the design ultimate stress of 235,000 psi. The following topics will be further investigated in the second phase of the project:

- Investigation of the ageing of the CCB dome,
- The cause and impact of pattern micro-cracks in concrete,
- Reasons for the high in-situ tensile stress in horizontal reinforcement bars,
- Development of NDT technologies to monitor the condition of PT cables.

5.3.4 Feedback from France

5.3.4.1 Operational experience on instrumentation

If relevant instrumentation is required for In-Service Inspection, this raises the question of sensor life expectancy. A classical maintenance program can be considered for monitoring devices that are mounted on the containment surface and then remain accessible during operation. However, it is no longer possible to achieve direct maintenance for embedded sensors (extensometers or thermometers). If the functional and metrological test should be performed regularly, repairs or replacements are hardly possible. Then, an adequate redundancy of sensors should be included in the design of the monitoring system, to cope with the possible failures during operation. Among more than 3.000 embedded sensors, EDF's experience shows that the mean failure rate is less than 1% per year in operation, leading to about 25% of failed sensors after 40 years operation and 33% after 60 years. It was founded that the concreting is a critical phase for the monitoring systems. According to EDF's experience, the failure rate can increase to 10%. During concrete pouring special care should be taken to mounting specifications and protection systems. Other data are provided by the literature, for example in [56] that reports a 50% failure rate after 25 years operation.

5.3.4.2 Operational experience on Bugey "specific monitoring"

In France, EDF installed a rather large number (more than 400 per unit) of specific VWSGs within 4 NPP (Bugey 2, 3, 4 and 5). It was a kind of experiment, dedicated to an enhancement of the knowledge on containment behaviour at time where construction of the PWR fleet had just started in France. Some of the sensors were attached to the ducts and monitored the duct strain to follow up the tendon integrity over time. The intent was to analyse the strain histories to detect any possible tendon breakage by a change in the trend of the strain curve. The amount of data provided by these sensors was so large that EDF decided to stop the experiment in the early 1990s, after about 12 years of "intensive" monitoring. No sign of tendon degradation has been found out during this period.

5.3.5 Feedback from Germany

5.3.5.1 Experience with the prestressed concrete reactor vessel of the THTR NPP

The large cylindrical prestressed concrete pressure vessel (PCPV) of the 300 MWe THTR (Thorium High Temperature Reactor) prototype NPP in Hamm-Uentrop, Germany (see

Figure 0.2 and **Figure 0.3**) is - mainly because of its prototype character - equipped with extensive measuring instrumentation (see

Figure 0.4), which has been monitored over the whole period of the vessel life from erection until decommissioning of the NPP.

Figure 0.2 Model of the THTR PCPV

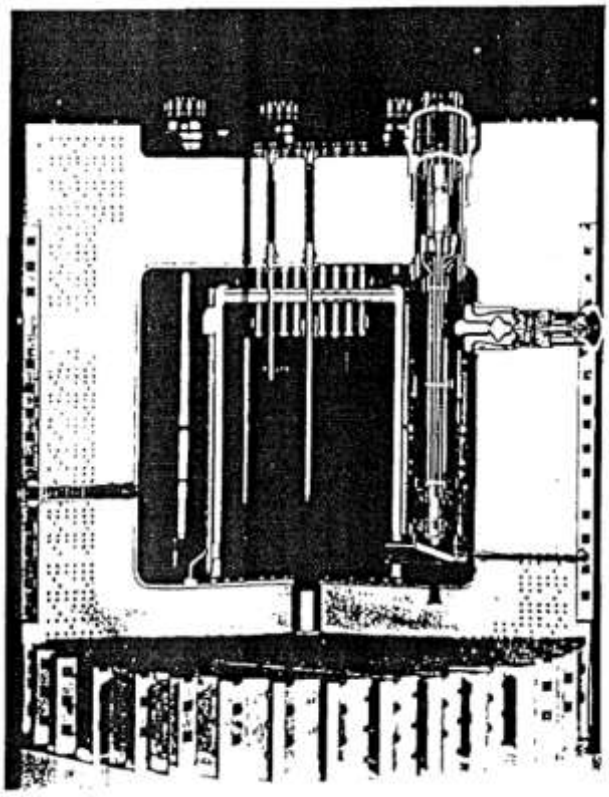


Figure 0.3 Section through THTR PCPV

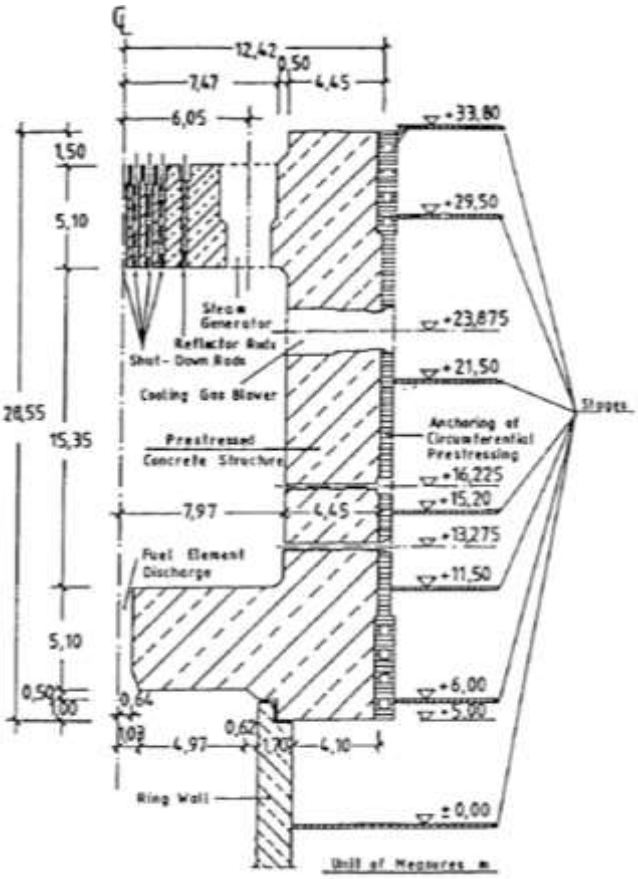
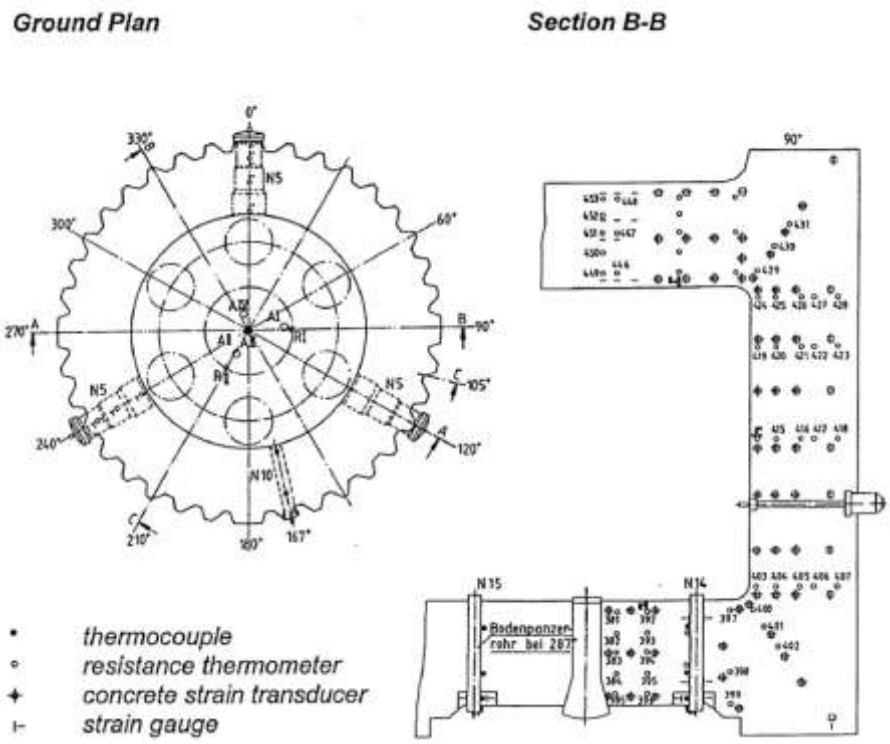


Figure 0.4 Measuring instrumentation



The prestressing system consists of 292 vertical and 560 circumferential, buttress anchored tendons of the BBRV System, each with a bearing capacity of about 9000 kN. A total of 30 prestressing force transducers are installed at 10 vertical and 10 horizontal circumferential tendons, which are not grouted like the others but filled with the grease "Astrolan". Each monitored vertical tendon is equipped with 1 transducer because the vertical tendons are prestressed only from one end. The horizontal tendons have transducers at both anchors.

Measurements have been recorded periodically from 1976 to 1995 during erection, commissioning, test operation and decommissioning. The evaluation of these measurements is based on a comparison of predicted and measured values. The mechanical calculations of the vessel history have been carried out mainly by use of the axisymmetric model of its prestressed concrete structure shown in **Figure 0.5**. A realistic calculation of stresses and deformations of the PCPV at particular points of time based on the time dependent material behaviour required the load history until that time to be taken into account. The idealized load history of the THTR PCPV is depicted in **Figure 0.5**.

The measured and calculated prestressing forces of one selected hoop tendon at both anchor heads are depicted vs. time in Figure 0.6. The measured values are plotted as crosses. The calculated values are linked by straight lines. The dotted curves characterize the tolerances due to the effects of measuring inaccuracies quantified as follows:

- 1% of the measured value due to the influence of the measuring facility,
- 2.5% of the measured value due to deviation from linearity,
- 300 kN due to the installation of the transducers under site conditions.

The abscissa unit is the time in days since the midst of the prestressing period.

Figure 0.5 Left: Axisymmetric computation model. Right: Idealised load history

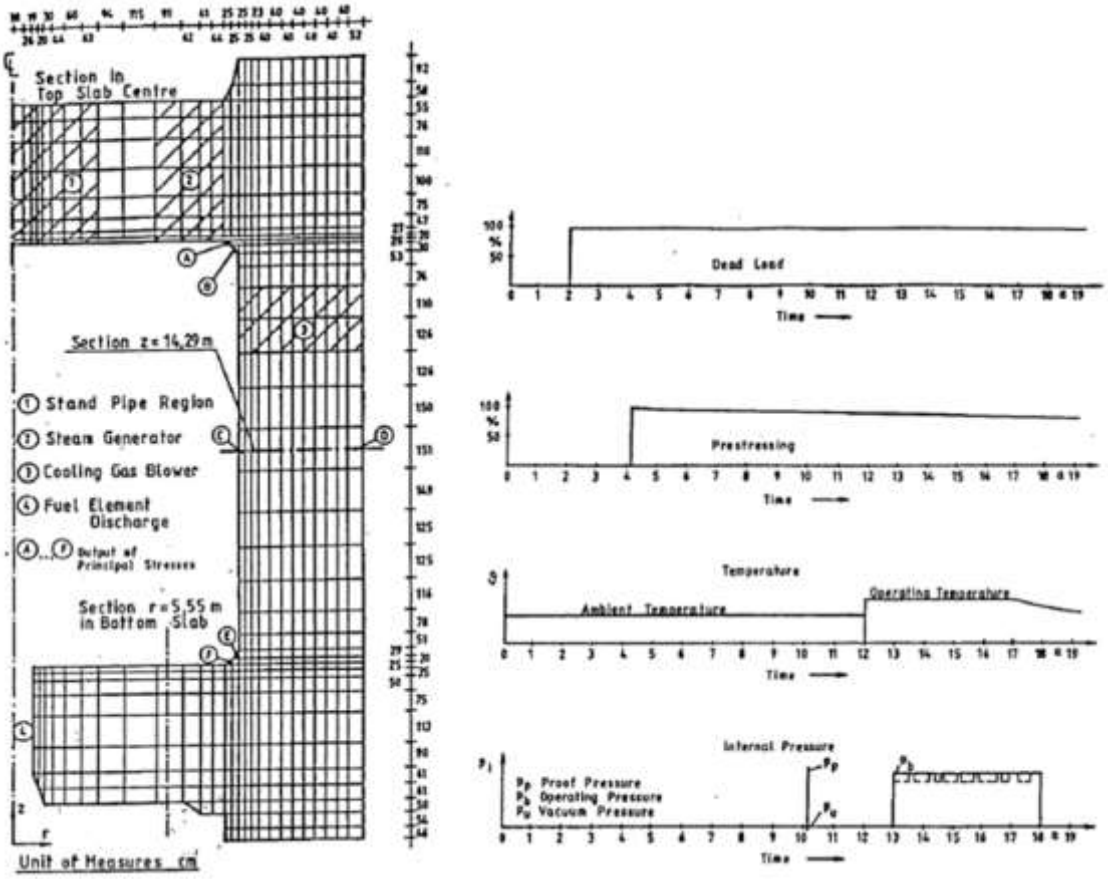
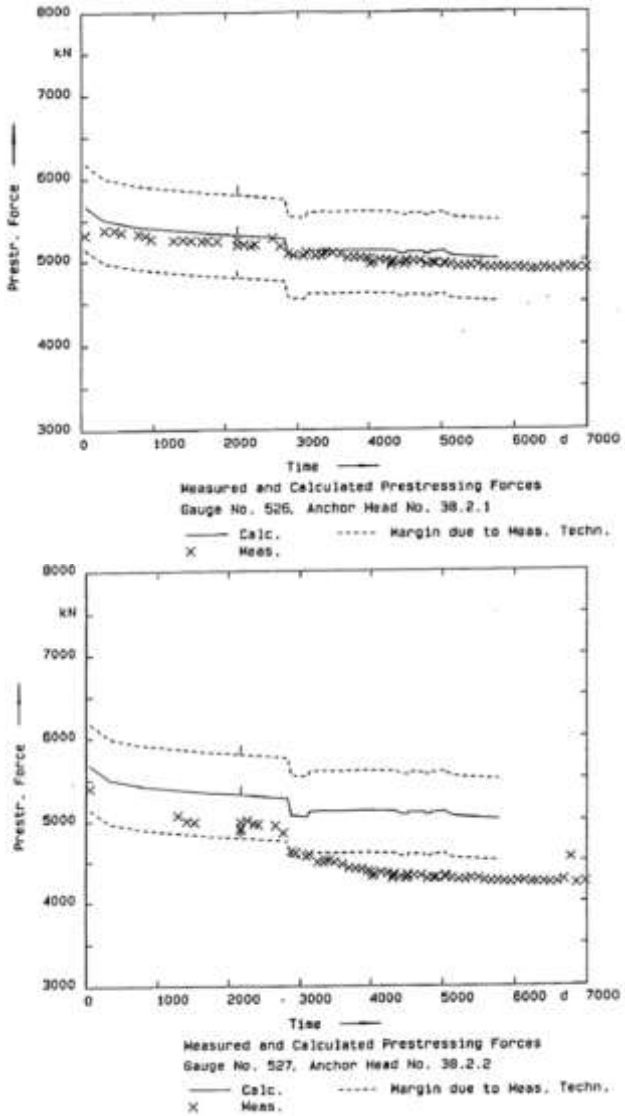


Figure 0.6 Measured and calculated prestressing forces of one hoop tendon



The conclusions from the long-term measurements are as follows:

- In the first time phase, the measured prestressing forces decreased less than the calculated values due to substantially lower initial creep deformations.

- After this initial period, the measured and calculated prestressing forces mainly proceed in parallel with mostly small deviations.

- 43% of all force transducers have transmitted measurements with similarly good agreement of calculated and measured prestressing forces (as shown in the left diagram of Figure 0.6 for one anchor head of the examined tendon). At 13% of the transducers, prestressing forces have been measured which run out of the lower margin (as shown in the right diagram). Fortunately, in the case of this tendon it can be concluded from the measurements at the opposite anchor head that the tendon is all right. Hence, the assessed deviations must be connected with defaults of measuring. The left over transducers have transmitted measuring results, which remain within the tolerable limits.

5.4 Conclusions from non-nuclear and nuclear feedback

There is little doubt that much has been learned over the past 20 years and more will be learned in the future to enhance the durability and longevity of post-tensioned structures.

SCOSS were absolutely correct to start raising concerns as early as 1979 about the sufficiency of grouting, which has been a key influencing factor in starting the drive for improvements.

Poor detailing has been as much to blame as poor grouting.

Although properly designed, well detailed and constructed post-tensioned structures have potentially long life if well maintained, we should and must not be complacent about post-tensioned structures – especially the older ones. Owners and Engineers should be pro-active to ensure these have full and proper investigations, especially where tendering requirements may act against thoroughness of approach.

The importance of suitably frequent audit testing for materials must be emphasized, however comprehensive other requirements for compliance with standards may appear to be.

IAEA TECDOC 1025 [22] provides guidance on ageing management of concrete containment buildings. The IAEA document was issued in 1998 and is currently being revised to incorporate lessons learned and to include other concrete structures (e.g. spent fuel bays). The revised document contains guidelines for assessment, monitoring and inspection of concrete containment structures using bonded or unbonded post-tensioning system including methods to monitor prestressing force losses. Furthermore, corrosion evaluation techniques are discussed.

6. DESIGN

6.1 Introduction

Today attention of the containment designers should be put on the evolution of the design domain. In order to clarify this point, now we use to break down the design domain into two sub-domains:

- Design Basis Domain
- Design Extension Domain

6.1.1 Design Basis Domain

This domain comprises itself two sub-domains the Design Basis Conditions (depending on reactor conditions: with basic accidents such as Loss of Cooling Accident or Steam Pipe Break) and Design Basis Hazards (hazards such as SL-2 earthquake). In the Design Basis Domain the justification principles are based on:

- Codes and standards adapted to size such structures in accidental conditions, with tightness function depending on national requirements.

6.1.2 Design Extension Domain

This domain comprises it-self two sub-domains, Design Extension Conditions and Design Extension Hazards for which the justification principles are based on:

- Realistic assumptions and best-estimate rules and methodologies with the purpose of further improving the safety of the NPP by enhancing the plant's capabilities to withstand, without unacceptable radiological consequences, accidents that are either more severe than design basis accidents

Note: as required in the IAEA document NS-G-1.10 [23], for new plants, severe accident which scenarios are inside the design extension domain should lead to limited permanent deformation.

6.1.3 Accidental scenarios that did not belong to Design Basis nor to Design Extension Domains

The other scenarios are managed by Ultimate Capacity Assessment of the containment that can be used for Probabilistic Safety Assessment.

6.2 Design Basis Level

6.2.1 Introduction

Prestressing system plays a vital role in ensuring the structural integrity of the prestressed containment structure through the design life of the reactor building. The prestressing system of a prestressed concrete containment is a principal strength element of the structure in the case of any accidental loads. The ability of the containment to withstand the extreme loads and severe accidents during the service life (i.e. short-term and long-term) strongly depends of the functional reliability of the prestressing tendons and anchorages system. Therefore, in addition to design for structural loads, the prestressing system should be designed for durability in order to maintain the functional and performance requirements through the design life of the containment structure. Various proprietary post-tensioning

systems are available. These systems differ in the type of tendon that they employ, in the manner in which the tendons are tensioned, and in the anchorage devices which are used.

The designer of prestressed concrete structures must be knowledgeable and experienced about the technology associated with prestressing and must be familiar with the terminology. Prestressing forces on the containment structure are strongly affected by the material characteristics of the concrete (i.e. mechanical properties, shrinkage, creep, thermal properties and ageing) and tendons (i.e. mechanical properties, relaxation, thermal properties and stress-corrosion) as well as the prestressing system (i.e. tendon layout, anchorage devices, bonded vs. unbonded and friction losses). In addition, the prestressing forces are time dependent and may vary during the service life of the structure. Any unrealistic design assumption for predicting prestressing forces can result catastrophic failure and may present potential risk to the public safety.

Due to the complex nature of nuclear structures, in addition to design consideration during operating life, designer of the nuclear power plant should take into account any impact from construction phase and decommissioning phase during his design process.

6.2.2 Design objectives

The main objective of this section is to provide technical information for the designer of prestressed containment structure with respect to the prestressing system of both bonded and unbonded tendons.

6.2.3 Prestressing system components in containment structure

The basic components of the post tensioning system are as follows:

- Ducts (sheaths or tubes), the conduits through which the post-tensioning tendons run;
- Tendons, a number prestressing strands made from steel wire wound together;
- Anchorages, consist of a thick steel bearing plate to transfer the prestressing tendon force to concrete, usually with an integral transition guide or trumpet that mates with the duct;
- Anchor heads and wedges, holds the ends of prestressing tendons against the post-tensioning force.
- In-fill material such as grout or greased materials

6.2.4 Background

As indicated earlier, an expert meeting on prestressing system was held in Lyon, France in April 2011. A summary of issues, concerns and findings with respect to the design part is given below:

- Comparison of design criteria and material specifications of regulatory requirements, international standards and codes of practice and specify similarities and differences
- Realistic modelling of the containment geometry and tendon paths around singularities (disturbed regions)
- Effect of concrete shrinkage and creep on containment design and prestressing

- Tendon rupture mechanism for bonded vs. unbonded tendons
- Prestressing losses
- Development of finite element modelling to represent the realistic behaviour of prestressing system and containment structure
- Modelling of bonded tendons vs. unbonded tendons
- Non-linear vs. linear analysis
- Sensitivity analysis
- Simplified method vs. sophisticated methods of analysis

It should be noted that a status report on ageing of concrete NPP structures was prepared during 1995 by a task group to initiate activities in this field under PWG3/OECD. The topic of prestress loss was identified as one of the highest priority. This topic was also notified in 2011 expert meeting on prestressing system.

6.2.5 Prestressing losses

The initial prestressing force applied to the containment structure undergoes a progressive process of reduction immediately after prestressing and over a period of time. Consequently, it is important to determine the level of the prestressing force at any loading stage, from the stage of transfer of the prestressing to the concrete, to the various stages of prestressing available at service load, up to the end of life of the NPP. Essentially, the reduction in the prestressing force can be grouped into two categories:

- Immediate prestress losses due to:
    - Friction loss
    - Anchorage seating loss
    - Elastic-shortening of concrete
- Long-term prestress losses due to:
    - Shrinkage of concrete
    - Creep of concrete
    - Relaxation of tendons

An exact determination of the magnitude of prestress losses, particularly the time-dependent ones, is not feasible since they depend on a multiplicity of interrelated factors (i.e. material characteristics of concrete and tendons).

To predict the prestress losses due to friction, anchorage seating loss, tendon relaxation and elastic shortening of concrete the technical information on curvature friction, wobble friction and anchorage devices shall be provided from the manufactures of prestressing system and to be used in the analysis. Factors affecting losses due to elastic shortening of concrete, shrinkage and creep can be obtained by concrete stress-strain, shrinkage and creep values of concrete. More realistic prediction of prestress losses results in more accurate determination of effective prestressing forces for the design purpose.

The immediate prestress losses can be assessed by measurements before and during the tensioning. The friction factor can be checked by friction tests and by elongation measurements of each tendon and by comparison between measurements and calculation. In the same way, the anchorage seating is measured for each tendon. The elastic-shortening of concrete can be assessed by monitoring device during the tensioning. So the initial tension in tendon can be estimated after tensioning, but in order to obtain margins it is necessary during the design process to introduce conservative design assumptions.

The long term losses can be assessed by monitoring device and extrapolation until the NPP end of life can give the final tension in the tendons. But it is more difficult to predict at the design stage the long term losses and higher design safety margins shall be taken into account, for example with small concrete Young's modulus and small ambient humidity.6.2.6 Comparison of bonded and unbonded tendons

Comparison of bonded and unbonded tendons showed that:

- Bonded tendons provide good corrosion protection considering good quality of the grouting (materials and construction process). However, properly placed grease will provide adequate corrosion protection for unbonded tendons,

- Bonded tendon provide a degree of bond between tendons and concrete,

- Bonded tendons are not available for direct inspection and force measurements. Therefore, Prestressing force should be measured by supplementary methods such as lift off beam tests and monitoring analysis; Unbonded tendons can be physically inspected if required,

- Unbonded tendons can be re tensioned and replaced if required, but it is necessary to adapt the layout in order to permit the access close to the anchorages during the NPP life. The re-tension or replacement is feasible but particular care should be watch out for damage to the strand by the wedges.

6.2.7 Design approach - Design process

As a general design guide the first step that shall be considered is to assure that all the design requirements including safety requirements, regulatory requirements, applicable standards and codes of practice are well established. Performance based design may be implemented particularly for durability. In order to perform analysis, the material characteristics of concrete, reinforcement and prestressing system shall be specified. Analysis models shall represent the realistic behaviour of the geometry of the containment structure, time dependent material characteristics and boundary conditions.

The main phases of the design process are the following in order:

a) Choice of the prestress unit which could govern the concrete containment thickness,
b) Choice of the global containment geometry (dome geometry and ribs number) which governs the tendons layout,
c) Determination of the tendons number or the spacing between two tendons,
d) Layout of the tendons around the small or big opening and optimisation of this layout,
e) Linear calculations for the several design load combinations "normal loads", "accident loads at the design pressure" and "some severe accident loads at higher pressure", in order to check the concrete compressive stresses, to determine the reinforcement and to obtain the liner strains imposed by the concrete.
f) Specific liner studies to justify the no-tearing of the liner,

g) Suitable behaviour of anchorages,
h) First estimation of the ultimate pressure,
i) Verification of some severe accident scenario with beyond design pressure.

The choice of the prestress unit which correspond to a choice of ducts diameter imposes a wall thickness in order to install all the type of tendons inside the wall thickness.

The choice of the global containment geometry imposes the global tendons geometry, with gammas tendons, inverted U tendons, horizontal tendons in the dome or not.

The spacing between two tendons is determined in order to balance by the prestress, the force due to the pressure and due to the liner thrust. This calculation is depending to the design criteria. In the ASME III division 2, formally no criteria are given concerning the equilibrium by the prestress and by the rebar. In the ASME III division 2, the governing load case in case of accident is a factored load cases "D + F + 1.5 Pa + Ta". In the ETC-C-2010 as in the old RCC-G, criteria are given which imposes a minimal prestress force in order to avoid tensile stresses in the concrete in case of tests or in case of accident at the design level. In this case the load combination is an un-factored load cases "D + F + Pa + Ta". These last criteria are given in current part of the containment and in areas where these criteria is not complying, additional reinforcement is required in order to avoid high cracks opening through the wall thickness.

After the choice of the spacing between two tendons, a major design phase is the layout of the tendons. It is important to optimise the tendons layout in order to avoid an important unbalanced force between pressure and prestress. Higher is the tendon layout optimisation, smaller is the reinforcement required for the unbalanced forces.

The tendons modelling shall take into account the precise tendons position, because, as the tendons force is high, a layout imprecision can generate higher friction losses or high unintentional bending moment.

The scope of the preliminary linear calculation is to check the prestress layout by the checking of the concrete compressive stresses, the checking of the liner strains imposed by the concrete. Another criterion is the reinforcement quantities which are needed for the equilibrium (membrane force and bending moment). In some case an iterative process is necessary with a better optimisation of the tendon layout.

It is to notice that specific liner studies are necessary in order to obtain a design of the liner and of the anchorage which assure a leak-tightness behaviour. The main objective of these specific studies is to avoid tearing of the liner in particular close to the anchorage.

When the containment is defined by the wall thickness, the prestress, the liner and the reinforcement quantities, a first estimation of the ultimate pressure can be implemented in order to compare several failure modes and to detect eventually some weaknesses. This study is the first step of the Probability Safety Assessment (level 2) of the containment. The PSA analysis can be more precisely achieved only when the all the construction material characteristics are knows.

After these Design level studies, other studies can be applied with non-linear models in order to define the containment behaviour under some beyond design accident scenario.

6.2.8 Summary of standards and codes of practice on prestressing steel

This section provides a brief description for prestressing systems in nuclear codes and standards. Discussion of findings is given below for the specified codes.

The codes consist of General Rules, Materials and Designs. General Rules part contain scope and terminology. Materials part contain cement, aggregate, mixture, rebar, tendon, anchorage, corrosion protection medium, liner plate, etc. Designs contain design of CCV, concrete part, liner plate and anchor, allowable stress, etc.

6.2.8.1 Canada and internationally

- CSA N287.2-08, "Material Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants", Canadian Standards Association, Mississauga, ON, 2008.

- CSA N287.3-93, "Design Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants", Canadian Standards Association, Rexdale,, ON, 1993.

- ASME III Division 2, "Code for Concrete Containments - Rules for Construction of Nuclear Facility Components", American Society of Mechanical Engineers", New York, NY, 2011. Note: Also ACI 359-10.

- ETC-C-2012, "EPR Technical Code for Civil Works", French Society for Design, Construction and In-Service Inspection Rules for Nuclear Island Components (afcen), Paris, 2010.

- EJ/T 926-95, "The Design Code for the Prestressed Concrete Containment of the PWR NPP" Nuclear Industry Standards of the People's Republic of China.

It should be noticed that the ETC-C 2010 was written in order to take into account some severe accident at the design stage.

6.2.8.2 Japanese code for materials and design of the concrete containment vessels

- JSME S NE1-2011, "Rules on Concrete Containment Vessels for Nuclear Power Plants"

This standard applies to materials and design of concrete, liner plate, liner anchor, penetration sleeve, penetration anchor, attachment, knuckle and body anchor for the concrete containment vessels. The concrete containment vessels are including prestressed concrete containment vessel and reinforced concrete containment vessel.

6.2.8.3 Russian standard used for design of NPP containment buildings

- NTD, tema 08.05.50. Нормы проектирования сооружений локализующич систем безопасности атомных станций (Normy projektirovanija sooruzhenij lokalozujushhih sistem bezopasnosti atomnych stancij – Standard on Design of Safety Important NPP Buildings). Interatomenergo, Moscow 1989.

- PiNAE G-10-007-89 Нормы проектирования железобетонных сооружений локализующич систем безопасности атомных станций (Normy projektirovanija zelezobetonnych sooruzenij lokalozujushhih sistem bezopasnosti atomnych stancij – Standard on Design of Reinforced Concrete Structures of Safety Important NPP Buildings). Gosatomenergonadzor USSR, Moscow 1989.

- SNiP 2.03.01-84 Бетонные и железобетонные конструкции (Betonnyje i železobetonnyje konstrukcii – Concrete and Reinforced Concrete Structures). Gosstroj, Moscow 1985.

6.2.8.4 Codes for the design of the NPP prestressing systems in Sweden

The codes (regarding prestressing) used for the Swedish nuclear power plants has varied over time. The oldest power plant in Sweden still in operation is Oskarshamn 1 (built: 1966-1968, start of operation: 1972). The youngest power plant in Sweden still in operation is Oskarshamn 3 (built: 1979-1982, start of operation: 1985). All power plants in Sweden are prestressed with either BBRV or VSL systems. Both bonded and unbonded tendon systems are used in Sweden.

In addition to the general containment design principles, the applied codes for the prestressing system used for the prestressed concrete containments were:

- Kungliga Väg- och vattenbyggnadsstyrelsen, Statliga Betongbestämmelser 1949. (Oskarshamn 1)
- Kungliga Väg- och vattenbyggnadsstyrelsen, Brobyggnadsanvisningar 1965.

For later designs regarding the prestressing system:

- Statens Planverks spännbetongnormer för husbyggnader SBN-S25:21, supplemented with AB Strängbetongs arbetsbeskrivning av den 10.4.1970.

The concrete containment building at Oskarshamn 3 were partly designed using ASME Boiler and Pressure Vessel Code, Section III, Div 2, Article CC3100-CC3522 and CC3900-CC3932.

6.2.8.5 Prestressing steel

The CSA N287.2-08 provisions for prestressing steel allows a number of different types of tendons (strand, wires and bars). The 2010 ASME III Division 2 requirements have a more limited choice of prestressing steel. The ETC-C-2010 code requires the use of 15 mm low-relaxation strand (relaxation equal to 2.5% at 1000 h). The CSA N287.2-08 code should consider providing a list of possible prestressing steel that is limited to more practical cases as is done in the 2010 ASME III Division 2. For example, the revision 2007 of the ASME II division 2, permits the use of the strand with a section equal to 150 mm² and grade 1860. It is the strand used for the EPR project.

The advantage of the strand system is the possibility to obtain large unit of prestress forces.

6.2.8.6 Bonded versus unbonded

CSA N287.2-08 and the 2010 ASME III Division 2 requirements permit the use of unbonded and bonded tendons. The ETC-C-2010 code requires that bonded post-tensioning systems be used with a cement grout.

6.2.8.7 Ducts

The ETC-C-2010 code requirements for ducts are more detailed than the requirements in the CSA N287.2 and the 2010 ASME III Division 2 codes. In particular the frictional properties of the ducts are to meet the values used in the friction loss calculations and there are more detailed requirements for the bending of ducts.

The ETC-C-2010 specifies also the ducts type, either steel tubes for vertical or dome tendons and on specific part of the horizontal tendons, either steel corrugated sheaths for the main part of the horizontal tendons.

6.2.8.8 Grout

The CSA N287.2-08 code for grout requires minimum air content as a function of temperature. This differs from the 2010 ASME III Division 2 and the ETC-C-2010 codes which have no specified air content requirement.

Generally, the grout quality and the grouting methodology is checked by tests on mock-up.

6.2.8.9 Prestressing steel jacking and anchorage stress limits

The N287.3-93, the 2010 ASME III Division 2 and the ETC-C-2011 codes give very similar limits on the stress in the prestressing steel as shown in the Table 0.1.

Table 0.1 Comparison of jacking and anchorage stress limits for prestressing steel.

Code	Stress limit
N287.3-93 - Jacking - Transfer - At anchorages or couplers	 $0.94f_{py}$ but not greater than $0.85f_{pu}$ $0.82f_{py}$ but not greater than $0.74f_{pu}$ $0.70f_{pu}$
2010 ASME III Division 2 - Jacking - Transfer - At anchorages or couplers	 $0.94f_{py}$ but not greater than $0.80f_{pu}$ $0.81f_{py}$ but not greater than $0.73f_{pu}$ $0.70f_{pu}$
ETC-C-2011 - Jacking - Transfer - At anchorages or couplers	 $0.90f_{p0.1k}$ but not greater than $0.8f_{pk}$ $0.85f_{p0.1k}$ but not greater than $0.75f_{pk}$

6.2.9 Analysis techniques and design guide

6.2.9.1 Introduction

Both linear and non-linear analysis may be performed during design process. Obviously use of reliable software and benchmarks are needed prior to perform any finite element analysis. Software shall be verified and validated before use and their limitation shall not impact on the design process. Linear analysis can be used during normal operations while non-linear analysis may be more adapted in certain cases for accidental loads. In general prestressing tendons can be modelled in two ways. Prestressing tendons can be replaced by “equivalent prestressing forces” acting to the containment structure or implementing finite elements such as “truss elements” in the analysis. It was found that modelling prestressing tendons as equivalent prestressing force is only appropriate for elastic analysis (for bonded and unbonded tendons) and it can be inappropriate for severe accident analysis when the concrete cracking is important.

It is always argument of simplified method vs. sophisticated methods. Simplicity is an advantage at the conceptual design phase however, oversimplification may result unrealistic output in the analysis.

Simplified methods usually need more assumption in the pre-processing stage of the analysis and also necessity of the interpretation of results in the post-processing. An example of using simplification in the analysis is to model creep and shrinkage of concrete with thermal strains and prestressing tendons by equivalent force. More sophisticated modelling requires more preparation time, computation time, numerical difficulties and extra input data. However, in return, the analysis output provides more realistic information on the structural behaviour particularly at disturbed regions and local areas. The rules of interpretation depend on thresholds and guidelines adapted to the modelling techniques and parameters adopted (initial states, material properties evolution, loading conditions and boundary and environmental conditions). Sensitivity analysis particularly mesh size sensitivity analysis is necessary to be carried out to verify and validate the accuracy of the results.

In the design level, the main non-linear effect is the liner plasticity and equivalent force which represents the liner plasticity can be used with linear behaviour of the concrete wall.

Another non-linear effect is the concrete cracking in case of severe accident. In order to calculate the rebar stresses it is necessary to take into account the concrete cracking, but the wall stiffness shall be estimated with the concrete tensioning effect.

6.2.9.2 Finite element discretization

In general, there are multitudes of ways in which a containment structure can be discretized into a finite element model. The discretization may depend on the loads that are to be analysed, the FE-software that is used, the level of detail that is required, the geometry of the containment, whether non-linear behaviour is expected etc. It is also important to note that bonded tendons are modelled differently from unbonded tendons.

E.g., the containment model shall capture the non-linear temperature distribution across the section thickness. Both solid elements and shell elements are suitable for modelling the non-linear temperature distribution through the containment wall. For the reinforcement calculation, the reinforcement model has to calculate the stress integration in order to obtain membrane forces, bending moments and shear forces. It is important that the model captures the behaviour of the chosen tendon system. E.g., in case off unbonded tendons, special attention to the interaction between tendon ducts and the tendons (friction, forces, temperature, etc.) has to be taken.

A suggested and common method to discretise a concrete containment structure into a three-dimensional finite element model is to model the containment wall concrete with solid elements, while the steel liner can be modelled with shell elements rigidly attached to the inside of the concrete wall and the steel reinforcement bars can be modelled with membrane elements embedded in the solid elements. The tendons are commonly represented with two-node truss elements. See Figure 0.1 for an example model.

The concrete can also be modelled with shell elements. In that case, considerably less elements are needed, but some geometrical details cannot be represented. Furthermore, non-linear shear deformation cannot be calculated with most shell element types available. The bending reinforcement is usually represented as rebar layers, i.e. equivalent stiffness, within the shell elements. See Figure 0.2 for an example model.

Note that a check-up of the validity of the model always has to be carried out regardless of the chosen method of discretization (benchmarking, hand-calculation using analytical solution formulas etc.). Furthermore, a mesh convergence study shall be performed in order to satisfactorily balance accuracy and computing resources.

Figure 0.1 An example of an FE-model where the containment concrete wall is modelled with solid elements. The tendons are modelled with truss elements

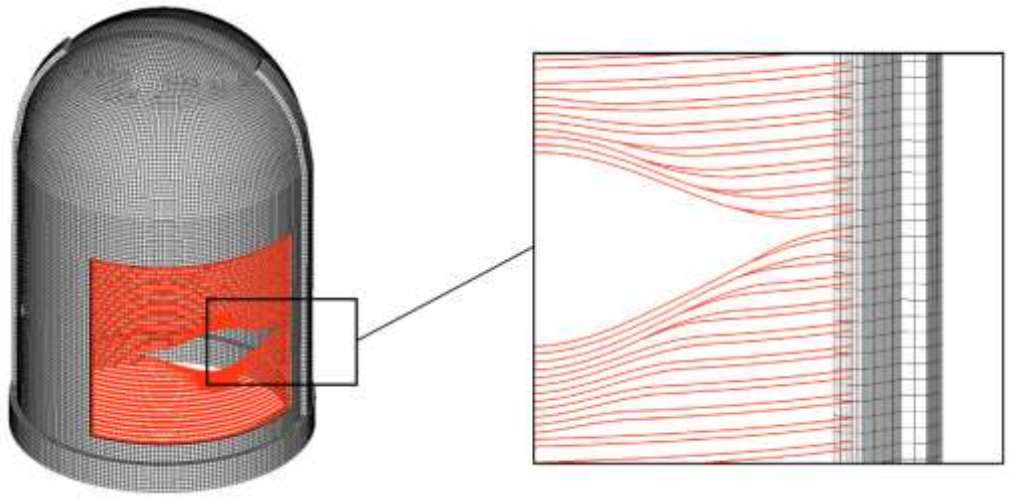
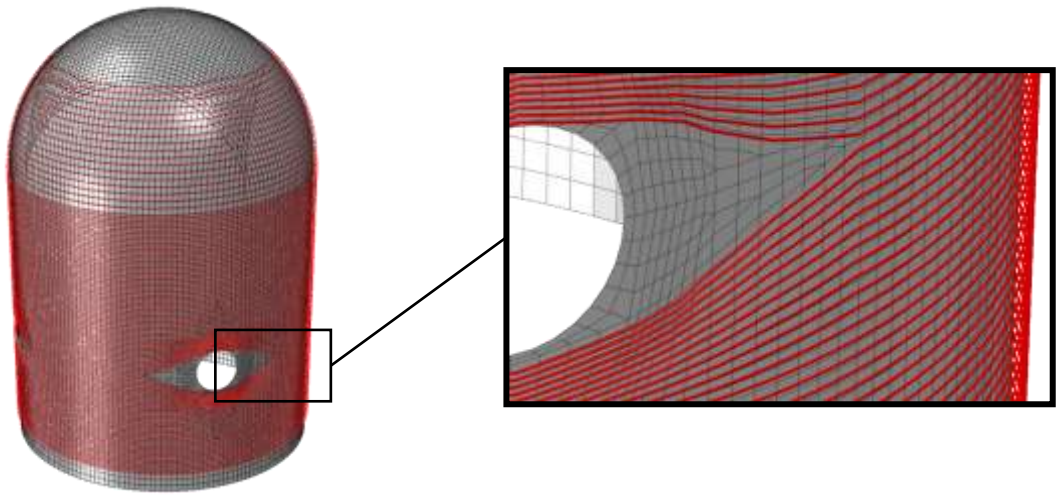


Figure 0.2 An example of an FE-model where the containment concrete wall is modelled with shell elements. The tendons are modelled with truss elements



6.2.9.3 Material modelling

Modelling of materials should represent the realistic behaviour of the materials for both short-term and long-term responses.

If concrete structures are to be modelled and concrete material models are to be used, special attention should be paid to material characteristics. In addition to mechanical properties (stress-strain curve), shrinkage, creep and thermal properties shall be considered. Tension stiffening shall be taken into account when non-linear analysis is carried out.

A damage model for tensile cracking and compression crushing should also be included in non-linear cases with high accidental loads. It is important, in the FE model, that the concrete is able to lose its

stiffness, i.e. the capacity to carry the loads from a high inner overpressure. Thus, the loads shall be taken by the remaining load-carrying components, mainly the tendons and the reinforcement. The same applies to other components as well, especially if an assessment of the containment ultimate capacity is to be carried out.

The material model used for the prestressing tendon steel shall include thermal properties in addition to the mechanical properties. Since the behaviour of concrete and prestressing tendons vary with time, the analysis should consider both short-term and long-term responses

6.2.9.4 Tendon modelling

In general, prestressing tendons can be modelled in two ways. Prestressing tendons can be replaced with "equivalent prestressing forces" acting on the containment structure or they can be explicitly modelled using finite elements such as truss or beam elements. Truss elements do not have bending stiffness themselves, but they contribute to the bending stiffness of the containment wall section. It was found that modelling prestressing tendons as equivalent prestressing force (i.e. no bending stiffness contributed to the section) is only appropriate for elastic analysis (for both bonded and unbonded tendons) and it can be inappropriate for severe accident analysis when the concrete cracking is important[64].

6.2.9.5 Bonded vs. unbonded

It should be noted that the bonded tendons have different behaviour than unbonded tendons, either at design pressure, or at ultimate pressure.

6.2.9.6 Behaviour at design pressure

In the event of concrete cracking and if the tendons are bonded, the cables contribute as passive reinforcement to limit the cracks opening. The over-tensions along the tendon are not uniform with higher over-tension on the crack than over-tension between two cracks. Even if the rebar are yielded, the bonded tendons can contribute to limit the cracks opening. For unbonded tendons the over-tension is about uniform along the tendons and is limited by the average concrete strain.

If bonded tendons are not explicitly represented in the model as bar elements (e.g. beam or truss elements) but only by their forces effects, they can be taken into account as additional reinforcement section.

If tendons are explicitly represented in the model as bar elements, it is preferable to use two types of mesh, a concrete mesh and a tendons mesh, with kinematic connections between the nodes of the two meshes. The tendons mesh shall be more refined than the concrete mesh. This methodology is more consistent for bonded tendons than unbonded tendons because the connections are not perfectly rigid in case of unbonded tendons[63].

6.2.9.7 Behaviour at ultimate pressure

In the event of rupture of a bonded tendon, the cables contribute to re-anchor themselves and continue to act as passive reinforcement and resist some of the force previously resisted by the ruptured tendon. For unbonded tendons, when a rupture occurs the force that was resisted by the ruptured tendon must be taken up by the surrounding tendons which can cause overstress in the remaining tendons.

The localisation of the containment failure under over-pressure is different for unbonded tendons and for bonded tendons. The localisation of the main failure mode of the containment is close to the ribs in case of bonded tendons, because it is the point where the initial tendon tension is higher. The localisation of the

main failure mode is more uncertain in case of unbonded tendons, because a redistribution of the over-tension along the tendons.

Modelling unbonded tendons as (perfectly) bonded may produce approximate and acceptable results but can lead to premature prediction of tendon rupture, as tendon strain increase is equal to containment wall strains. Modelling bonded tendons as rigidly embedded into the elements modelling the concrete wall will produce realistic results for most studies. However, great care must be taken in implementing the proper stress distribution in the tendons (the stress distribution prior to grouting, which is given automatically in case the tendons are modelled unbonded and the prestressing is simulated). As expected, the displacements in the bonded model are more variable (not as uniform) as those in the unbonded model. Due to the force transfer mechanisms creep of concrete in bonded and unbonded tendons are different and may impact on the analysis results.

If a combination of bonded and unbonded tendons is used in a containment structure this effect shall be considered in the modelling.

6.2.9.8 Disturbed regions - local effects

Disturbed regions (i.e. geometry discontinuities, load concentrations) should be considered in the modelling. Tendon profile and any tendon deviation particularly around opening shall be considered in the modelling. Modelling of anchorage zones as close as possible to the anchorage behaviour is necessary to predict the true behaviour in disturbed regions. There is an advantage to use a complementary set of local models. It is generally the case for the local analysis of steam pipes under local rupture. The opening may be ignored in the modelling if the size of the opening is less than the containment wall thickness.

6.2.9.9 Impact of construction phase on design

Since the construction of the containment structure and prestressing the tendons take a number of years, the impact of the construction phase shall be considered in analysis and design. In a normal practice it is assumed that the analysis is carried out using the completed containment structure with all prestressing tendons stressed at the same time. Sequencing of prestressing has impact on concrete cracking and on prestress losses and effective prestressing forces: simplified methods can be used to address these phenomena. Concrete pouring at different stages (i.e. fresh concrete vs. relatively hardened concrete) may result non-uniform restrained shrinkage and creep along the containment wall and dome. The effects of the pouring stages and of the tensioning sequences can be checked by an analysis of the measurement given by the monitoring devices.

This non-uniform shrinkage and creep should be addressed by material laws that consider the thickness of the concrete element to assess the delayed deformations. This phenomenon leads also to a non-uniform stress distribution in the concrete section which has an impact on cracking pattern and some codes as ETC-C rev 2010 (principles coming from Eurocode 2) propose simplified procedures.

In the design phase, the containment geometry shall avoid high concrete thickness variation, in order to avoid differential deformations between to area of the containment.

The scope of these construction phases studies (mainly the tensioning phases) is to check that the phases does not create damage in the containment. Generally the tensioning sequences are defined in order to no add specific reinforcements, except for the delamination reinforcement. Another exception concerning the reinforcement under construction phases is the polar crane which is generally tested before the tensioning and for which reinforcement is needed.

6.2.9.10 An example of French analysis practice

The French practice for the design level (as specific criterion are given for minimal prestress force) is to use a linear equivalent model, which takes into account the nonlinear effect due to the liner plasticity and concrete cracking.

For all the design load combinations which represent situations without liner yielding (construction, normal operating, testing, etc.), the liner and the concrete are modelled with linear properties. The concrete basic creep is taken into account by modifying the Young's modulus. The shrinkage and the desiccation creep are modelled by additionally imposed strains.

For all the design load combinations which represent situations with liner yielding (mainly due to the temperature increase in case of accident), the concrete is modelled with finite elements and the liner is modelled by equivalent load forces. In this case, the concrete is linear, but if the concrete is cracked due to the thermal gradient, reduction factors are taken into account for the estimation of the thermal solicitations which are used for the reinforcement calculations.

6.2.9.11 Analysis steps

An example of a general analysis set-up is presented below, regarding load application for analysing containment behaviour when subjected to accidental loads. Note that including long-term effects might be un-conservative for some load combinations.

Step 1) Dead load, service loads

Step 2) Tensioning of the tendons and seating (locking)

Step 3) Application of long-term effects (creep, shrinkage, relaxation)

Step 4) Accidental loads (internal overpressure, point loads, temperature etc.)

6.2.10 Durability design considerations for prestressing system

Prestressing system and prestressing forces shall perform with acceptable level of confidence during the design life of the containment structure. Therefore, in addition to structural design (i.e. strength and serviceability), durability design is a major requirement during the design life of the containment structure. Modern structure codes are increasingly based on the performance of the structures (performance based Design). Major issues on durability design with respect to prestressing system are long-term prestressing losses and stress corrosion. Protections during the construction phases are necessary in order to limit friction losses and to avoid corrosion before tensioning (use of sufficient concrete cover, use of stoppers at the end of ducts).

Low friction coefficients (curvature and wobble friction coefficients) can be obtained with a high quality of construction (detailed specification, tolerances and site checking). Prestressing losses may be reduced by using prestressing tendon with low relaxation and prestressing losses due to shrinkage and creep can be reduced by using high-performance concrete with lower shrinkage and creep coefficients. In order to protect tendons against stress corrosion appropriate permanent corrosion-preventive coatings, sufficient concrete cover and high-performance concrete with special characteristics such as lower water cement ratio may be used.

6.2.11 Design assessment

Design assessment shall be carried out during the plant life of the structure including operating period and decommissioning phase. Existing material characteristics of concrete, reinforcement and post-tensioning system such as strength, stress-strain behaviour, shrinkage, creep, relaxation and thermal

properties should be implemented for the purpose of design assessment. Standards and codes of practice used for original design should be reviewed and any change on the design requirements during the service life of the structure that has any impact on the structural performance should be considered.

6.2.12 Innovations in material performance

Implementing latest innovations in concrete and prestressing characteristics have major impact on design performance. In the design of prestressing system extensive knowledge on the latest innovations developed in concrete and prestressing industries are inevitable. Performance of the prestressing is strongly relies on the performance of both concrete and prestressing system. Many recent innovations in advanced concrete materials technology have made it possible to produce modern concrete such as high-performance concrete with exceptional performance characteristics. Prestressing system also has gone to many developments since the last decades. The designer of the prestressing system is expected to be aware of any innovations and manufacturer information in both concrete and prestressing system that can be used in the analysis input and design process.

For example, the evolution of the prestress technology is to use prestress system which was developed mainly for the nuclear structures, with high prestress force with more than 50 strands, with specific devices in order to avoid high tension dispersion between each strand or with high jack displacement.

Another example is the possible use of “individually sheath greased tendons”. This technology bring many answers in term of durability, but leads to additional questions such as monitoring, possibility to re-tension or to replace, behaviour under the temperature effects and test pressure level: these questions are addressed in the following sections below.

6.2.13 Design for greased sheathed strands (GSS)

6.2.13.1 Impact on the calculation phase

6.2.13.1.1 Tension losses

One main special feature of GSS tendons is a very low friction coefficient between the strands and the sheaths linked to a high transmission coefficient value for the tendon ensuring high prestressing efficiency.

ETC-C (for tendons injected with cement grout) and ETA-06/0226 (for GSS tendons) gives the following values presented in Table 0.2, fitted with the specific features of containments.

Table 0.2 Comparison of friction coefficients.

	Tendons injected with cement grout	**GSS Tendons**
Friction coefficient μ [rad-1]	0.18 (in steel corrugated sheath) 0.16 (in metallic tube)	0.05
Wobble coefficient k [rad/ml]	0.009 (horizontal and dome tendons) 0.005 (vertical tendons)	0.012

Thus, the initial tension of a GSS tendon will exceed that of a tendon injected with cement grout or grease to optimise prestressing. So, it appears that the initial tension of the GSS tendons is about:

- 25% higher for non-deflected horizontal tendons than for tendons with cement grout (or grease),
- 1% higher for vertical non-deflected tendons than for tendons with cement grout (or grease),

- 10% higher for non-deflected dome tendons than for tendons injected with grout (or grease).

Gains are higher for deflected tendons because losses due to deflection near penetrations are also lower for GSS.

Gains in terms of initial tension linked to the use of GSS tendons can be assessed in several ways:

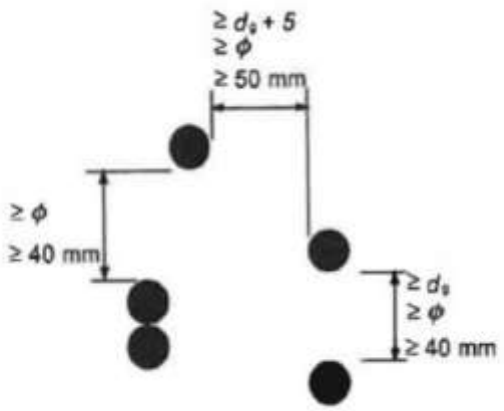
- By reducing the number of tendons and thus their spacing (leading to gains in prestressing planning, reduced cost and improved constructability),
- Considering weaker tendons units to optimise design (reduced wall thicknesses, for example).

For GSS tendons, relaxation of the tendons is a little greater because of the increased initial tension (the steel grade is the same), particularly for horizontal tendons and dome tendons (to a lesser extent).

6.2.13.1.2 Diameter of the ducts - Positioning of the elements in the wall

The ducts used for GSS tendons have a diameter greater than those of injected tendons (grease or cement grout). At the design stage, this may have an impact on the layout of all the elements in the wall (steel, prestressing ducts, liner) and their relative positions. The greater overall dimension of the ducts used for GSS tendons must be considered when determining the radii for tendon layout (vertical, various horizontal beds) in the cylinder wall and dome.

Figure 0.3 Recommendations for the implementation of prestressing ducting [12]



With ϕ the diameter of the prestressing duct and d_g the dimension of the biggest aggregate

For GSS tendons, it is possible to optimise the spacing usually provided when using tendons injected with cement grout or grease because the GSS tendons injection is performed before tensioning. The risk of collapsing a duct when prestressing is reduced significantly. However, ensure that the ducts are not too compressed, which could pose difficulties during tensioning or while changing a strand. Thus, two ducts may be in contact if they are stacked vertically. In this case, it is recommended to use metallic tubes.

Horizontally, it could be planned to reduce the distance between two ducts to $\phi/2$.

The application of these structural arrangements associated with larger diameter ducts may require increasing the thickness in some areas (cylinder wall, dome) to:

- Avoid bending moment caused due to the eccentricity of vertical tendons relative to the mean fibre of the wall,

- Ensure adequate coating,
- Limit vertical tendons closeness to intrados facings, strongly heated by accident.

The choice of a weaker prestressing unit can eventually limit the impact on the thickness of the cylinder wall or dome.

6.2.13.1.3Participation of tendons in the reinforcement calculations

In the common area, for tendons injected with cement grout, a part of the horizontal tendons section participates in the limitation of crack openings. This is no longer the case with GSS tendons, which are considered in the construction codes as unbonded prestressing technology.

Since the strands can slide in their sheath, there is no compatibility of the strains between the GSS tendon steel and the surrounding concrete. Hence, unlike passive steel and bonded prestressing steel, the GSS tendon cannot contribute to resist traction locally when concrete is cracking under bending. Therefore greater quantities of passive steel are required when GSS tendons are used instead of bonded tendons.

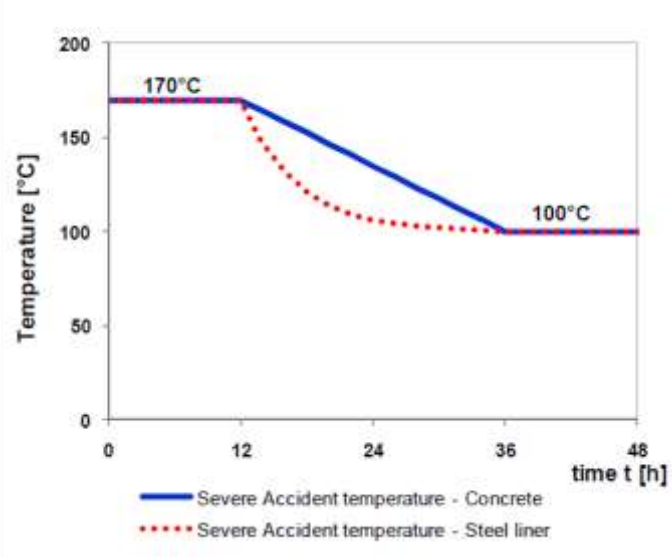
Their participation is limited to a uniform increase of the stress in the tendon inducing excess tension, limited by design codes. The consequences result in increased quantities of passive steel to implement in the cylinder wall and dome common area, mainly for thermal loads.

6.2.13.1.4Tendons behaviour under thermal load case

In the event of a severe accident, the containment is subjected to high temperatures maintained for a long time.

According to the hypothesis of ETC-C (Part 1), taking into account a very conservative estimate of the evolution of the temperature within the containment, up to 170°C for 12 hours (cf. graph below), vertical tendons reach a temperature between 55°C and 70°C in the long term (thickness between 1,30 and 1,80m), between 40°C and 55°C, for the warmest horizontal layers, around 70°C for the warmest tendons in the dome (thickness of 1,00m). These temperatures would be much lower for a thicker containment.

Figure 0.4 Severe Accident Scenario - Evolution of the temperature in the containment [4]



The problem is to ensure that the mechanical behaviour of materials (sheaths strands and grease) is not affected when the temperature reaches 70°C for a relatively long time after the beginning of the accident to avoid risk of significant loss of tension in the tendons.

Polyethylene ducts can withstand over 120°C (123°C corresponds to the softening point of polyethylene) and grease melts at 130°C. Maintaining a temperature of 70°C should not damage the tendon. It should be noted that when using GSS tendons in the field of cable-stayed bridges, they may bear such temperatures under the effects of sunlight.

To confirm these points, tests could be performed aimed at characterizing the mechanical behaviour of the tendons at those temperatures, and the effects on the strands.

6.2.13.2 Impact on the layout

Some arrangements must be taken into account at the design stage, to enable the measurement, the initial tensioning, retensioning or detensioning of the GSS tendons:

- A sufficient tendon excess length,
- A maintenance area near the ribs, the upper ring beam and prestressing gallery,
- The impact on the geometry of the upper ring beam for a double containment.

6.2.13.2.1 Tendon excess lengths

When the tendon is inserted, it must have sufficient excess length (fastening excess length, marked G) to be hung by the jack. This value depends on the type of jack considered.

After tensioning, the excess length of the tendon corresponds to the previous value (G) to which it is appropriate to add the elongation of the tendon.

To remove the jack, at least a distance equal to the latter to which the length of the jack must be added (marked E) is required (assuming the tendon is not cut before removing the jack).

Thus, the distance needed is E + G + elongation.

The diameter of the jack depends on the type of jack considered.

For tendons injected with cement grout, after receiving the tendons, the strands can be cut to less than 200 mm and injected. For GSS tendons, in order to retain the ability to measure or restress the tendon, it is necessary to maintain an excess length at least equal to the sum of the fixing excess length (G) and the elongation of the tendon, except if a single-strand jack is used, which allows to reduce slightly excess lengths. However the clearance must remain at least equal to that provided for measuring tendons (approximately 1.00 m excess length necessary for detensioning and retensioning with a single-strand jack in comparison with approximately 3.50 m for a multi- strand jack).

Thus, it can be stated:

- When using injected tendons, a minimum clearance during work of about E + G + elongation is required to achieve the tensioning. After tensioning, the tendons do not exceed the dimension of the protective cap (about 250 mm),

- When using GSS tendon intended to be measured and retensioned, but not changed, an extra length of tendon is required for all tendons (at least equal to G). A clearance has to be available to perform measuring or retensioning operations. The value depends on the type of the jack, at least upper than E + G + additional elongation during the retensioning operation,

- When using GSS tendons intended to be detensioned if needed with a multi-strand jack, an extra length of tendon is required for all tendons (at least equal to G + elongation). The clearance to perform detensioning and retensioning operations. The value depends on the type of the jack, at least upper than E + G + total elongation during the tensioning operation,

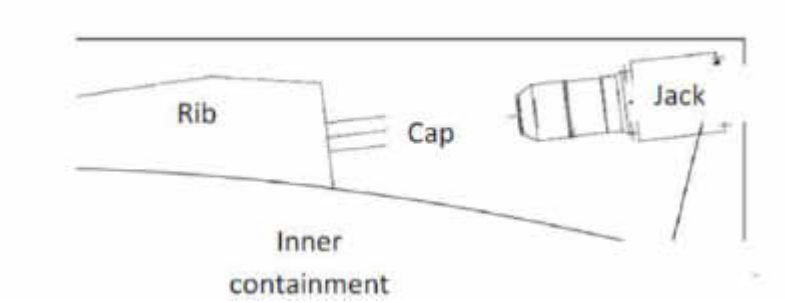
- When using GSS tendons intended to be detensioned if needed with single-strand jack, an extra length of tendon is required (at least equal to G + elongation). The value of G is much lower than with a multi-strand jack.

6.2.13.2.2 Maintenance areas to be kept available

The size of the ribs is not sensitive to the prestressing technology used. However, maintenance areas around the ribs are to be provided and maintained during the entire operating life of the containment. The dimensions of these areas must be compatible with excess lengths defined above.

Thus, no penetration, no floor (except for removable floors) may be installed in this area to allow for a pod of the type used for initial prestressing of the containment in the construction phase, to circulate over the entire height of the rib during measuring or retensioning operations. The space to keep around the ribs must be similar to that provided in the initial prestressing phase. Access for tendons maintenance at the rib must be provided to carry the necessary equipment.

Figure 0.5 Space to provide for the maintenance area around the rib



6.2.13.2.3 Geometry of the upper ring beam - Double wall containment case

For a containment having an upper ring beam, the geometry must include:

- Tendon tensioning phasing (impact on the section),
- Limiting tension losses in the tendon in the dome according on the inclination angle of the tendons.

For GSS tendons, it is necessary to consider prestressing work after completing the construction. Thus, for a double containment, the minimum distance between the inner and outer containment must be compatible with the operations to be performed on the prestressing tendons.

In this case, the use of GSS tendons may require changing the output angle of the tendons to minimise impact on the size of the spacing between the containment and the upper ring beam. The upper ring beam has a larger section, with more slanted tendons to allow them to be measured or retensioned. Tension in the tendons will be smaller due to the additional deflection to be provided (about 3% additional).

This new dome geometry is also reflected by a decrease in the height of the ribs. As the GSS gamma tendons are very slanted at the upper ring beam outlet, the horizontal stress no longer allow to apply horizontal prestressing to the bottom portion of the upper ring beam. This decrease of horizontal stress at the upper ring beam will result in an increase in the passive steel section to support the additional stress created in the event of internal overpressure.

Figure 0.6 Development of the upper ring beam geometry when using GSS tendons – Double containment design

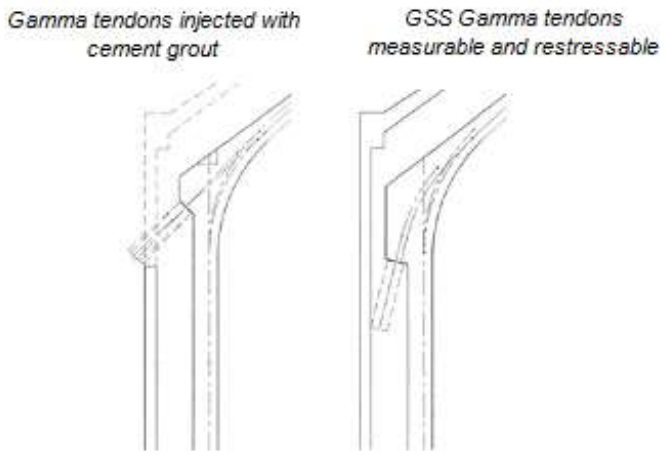


Figure 0.7 Evolution of the temperature in the wall during a Severe Accident (thickness: 1.30 m)

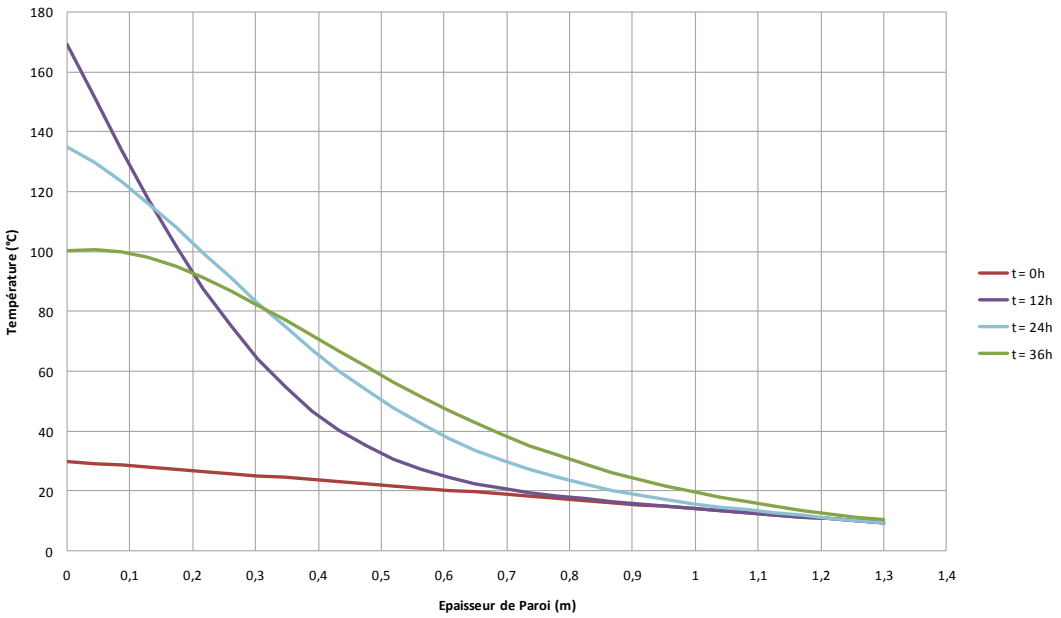


Figure 0.8 Evolution of the temperature in the wall during a Severe Accident (thickness: 1.80m)

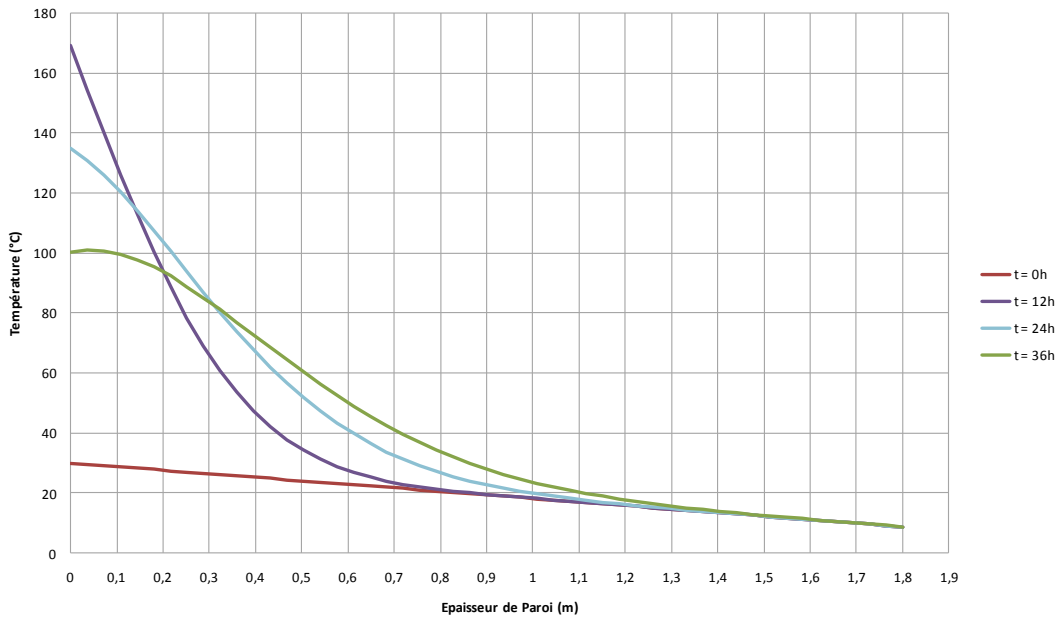
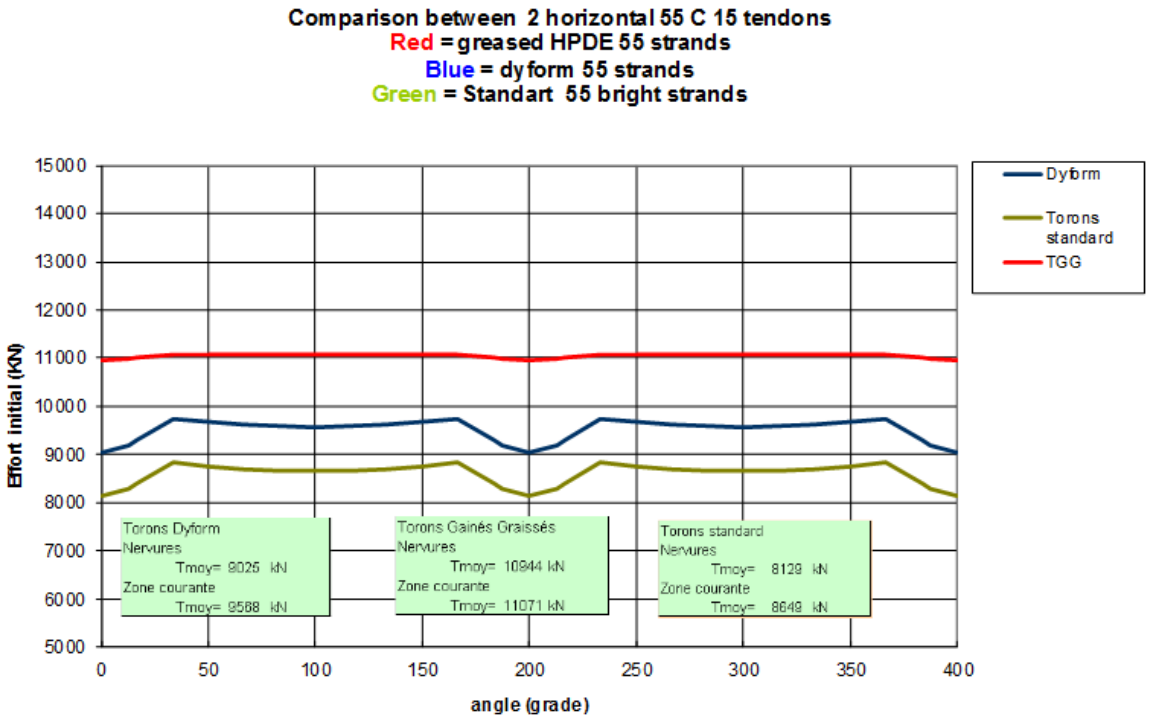


Figure 0.9 Evolution of the initial stress in a prestressing tendon for different technologies



6.3 Beyond design

Two main events have to be considered for the assessment of strengthening of containment beyond design:

- Internal event: severe accident,
- External event: large aircraft impact (see section 0).

6.3.1 Scope

This document addresses deterministic methods for structural integrity evaluations of containment structures of prestressed concrete containments with bonded or unbonded tendons.

Prestressed concrete containments include the structural concrete pressure-resisting shells and shell components, steel metallic liners, penetration liners extending the containment liner through the surrounding shell concrete, and tendons and anchorage system.

6.3.2 Severe accident

This document describes methods for predicting by analytical means the internal pressure capacity for containment structures above the design-basis accident pressure.

The internal pressure capacity in this estimation is an internal pressure capacity at which the structural integrity is retained and a failure leading to a significant release of fission products does not occur.

This document provides methods for an analysis that specifically address the issues for containment structures in nuclear power plants under severe accident conditions.

6.3.3 Design objectives and methods for Beyond Design condition [40]

The objective is to assess the pressure capacity of the containment at which the structural integrity is retained, and a failure leading to a significant release of fission products does not occur. For estimating the pressure capacity of the containment structure by deterministic structural simulation with the aid of the static nonlinear 3D finite element analysis will be needed for predicting the global response. Large penetrations are usually included in the finite element model; smaller penetrations and penetration closure components are analysed using a local finite element models either separate or as parts of modified global finite element model.

For cylindrical containment structures the use of analytical solution formulas to estimate the pressure capacity are possible as first link in the analysis sequence.

The initial condition for the nonlinear analysis of the containment structure should be the linear elastic response caused by dead load and design pressure, at the design temperature. The internal pressure is incrementally increased until a failure is reached.

The nonlinear stress-strain curve for steel materials should be based on the code-specified minimum yield strength for the specific grade of steel and a stress-strain relationship beyond yield that is representative of the specific grade of steel in the relevant temperature conditions. The nonlinear stress-strain curve for concrete should be based on experimentally verified test results corresponding to relevant temperature conditions.

The pressure capacity for cylindrical prestressed concrete containments is to be based on the contribution from each structural element considered in the analysis, using the stress-strain curve for each material and the strain level in each material, as determined based on overall strain compatibility between all of the credited structural elements such as concrete, liner, tendons and reinforcement bars.

In addition for estimating the cylindrical shell capacity in ideal membrane state, the analysis should consider additional failure modes, such as concrete shear and concrete crushing which may occur near discontinuities.

The analysis methods described above apply to the global containment capacity. A complete evaluation of the internal pressure capacity should also address major local components such as equipment hatches, personnel airlocks, and major piping penetrations.

6.3.4 Modelling considerations for beyond design analyses for bonded and unbonded tendon systems and for concrete cracking in PCCV's

The possibility of slip between tendon and tendon duct should be enabled by some suitable modelling arrangement like with the use of slot connectors modelling the contact between tendon and grout or grease in the tendon duct. This means that the tendons as general rule should be modelled explicitly using bar elements in the containment model. The preferable arrangement of the meshing of the composite model consisting of tendons and containment wall concrete is the coincident nodes of tendon mesh consisting of rod elements and concrete mesh consisting of shell or solid elements. Regulating the connector stiffness of slot connector either perfect or total lack of bond between tendon and concrete can be simulated.

Cracking of concrete can be simulated using various model approaches depending on the precision and size of the model. So called smeared crack concept is one modelling possibility assuming that

decreased stiffness caused by concrete cracking is distributed evenly on the tributary area of material connected to one integration point inside one particular finite element. Other possible modelling approaches could be the brittle cracking modelling approach or damaged plasticity modelling approach or individual micro-plane modelling approach specifying the one dimensional stress-strain relationship of the concrete material for a multitude of directions called micro-planes.

6.3.5 Estimation of the leak - rate through the containment wall based on liner strains simulated on a specific value of the internal pressure load [60]

For pure membrane loads, the width of the crack could be assumed to be uniform through the containment wall thickness. By using this assumption the flow through a crack in the containment wall can be approximately modelled as a flow through a gap between two parallel plates. The mass flow rate and the friction are assumed to be constant along the flow path. The expression for momentum balance, which states that the summation of forces should be equal to the change in the momentum, is given with the aid of the following equation:

$$PA - (P-dP)A - 2\tau_0 Bdx = \rho AV(V+dV) - \rho AV(V) \qquad \text{Eq. 0.1}$$

Equation 0.1 can be also written in the following form

$$dP + \rho VdV + 2\tau_0(B/A)dx = 0 \qquad \text{Eq. 0.2}$$

In Equation 0.2, P is the absolute gas pressure at any section, V is the velocity of gas at any section, ρ is the gas density, B is the length of the crack, A = B*W and W is the crack width and τ_0 is the shear stress due the friction in containment wall and dx is differential distance in the direction of gas flow. The mass flow rate and the friction coefficient are assumed to be constant along the flow path. Thus in a steady and uniform flow in a conduit with constant cross-section, shear stress can be expressed in terms of gas velocity V, gas density ρ and dimensionless friction coefficient f as follows: $\tau_0 = (f/4)*(\rho V^2/2)$. Using this expression for shear stress and integrating Equation 0.2 along the depth of the crack in the direction of flow L and neglecting insignificant terms the Equation 0.3 or so called Rizkalla [53] equation is obtained:

$$((P_1^2 - P_2^2) / L) = (k^n / 2)(\mu / 2)^n (RT)^{n-1} |(P_2 q_2) / B|^{2-n} (1/W^3) \qquad \text{Eq. 0.3}$$

In Equation 0.3 L is the wall thickness, μ is the gas viscosity, R is the molar gas constant, T is the temperature, B is the crack length, W is the crack width, P_1 is the containment internal pressure, P_2 is the pressure outside the containment (1atm) and q_2 is the leak volume rate outside the containment (atmospheric conditions). Entities k and n are dimensionless parameters. The q_2 in Equation 0.3 can be solved in the following way:

$$q_2 = [((P_1^2-P_2^2)W^3) / L / (k^n / 2) / (\mu / 2)^n / (RT)^{n-1}]^{(1/(2-n))} B / P_2 \qquad \text{Eq. 0.4}$$

The parameters *k* and *n* are defined to be as follows:

$$k = 2.907*10\text{-}7*(W^3)^{0.428}*(1 / (0.3048^3)^{0.428})) \qquad \text{Eq. 0.5}$$

$$n = (0.133 / (W^3)^{0.081})*(0.3048^3)^{0.081} \qquad \text{Eq. 0.6}$$

For other parameters in the Equations 0.3 - 0.6 the following values were used:

R = 8.3145(J/(mol*K)) / 0.02802(kg/mol) = 296.73(J/kg/K)

m = 1.76E-5(Ns/m^2)

L = 0.325 m

B = 0.34 m

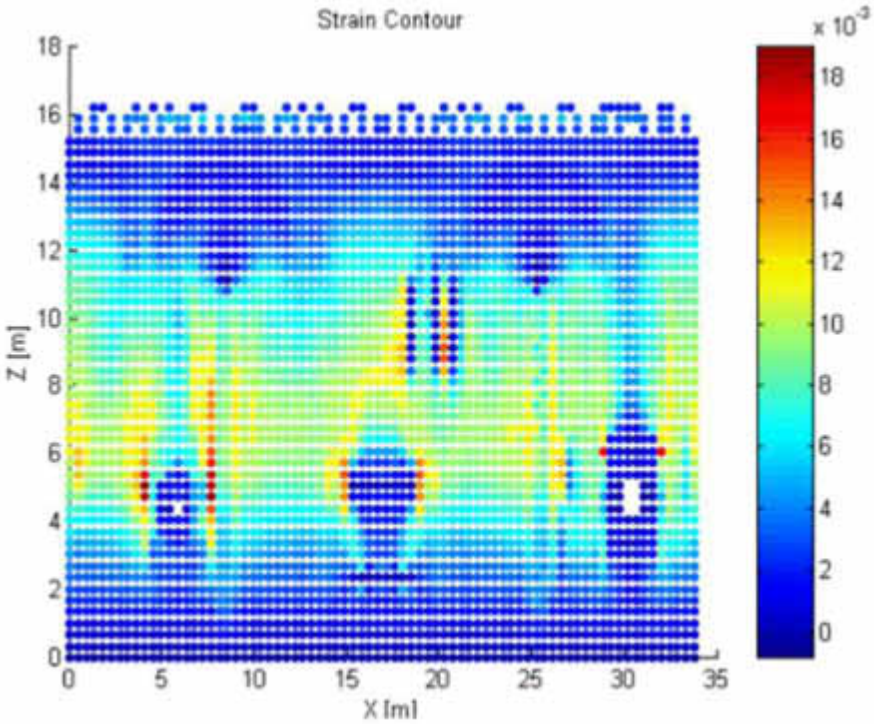
T = 300 K

The crack width W was defined by the Equation 0.6 as follows:

$W = W(L_e, \varepsilon_\theta) = L_e(c_1\varepsilon_\theta + c_2\varepsilon_\theta^2)$ Eq. 0.7

The hoop strain field ε_θ in Sandia test at the internal pressure of $3.5*P_d$ = 1.365 MPa is given in **Figure 0.10**.

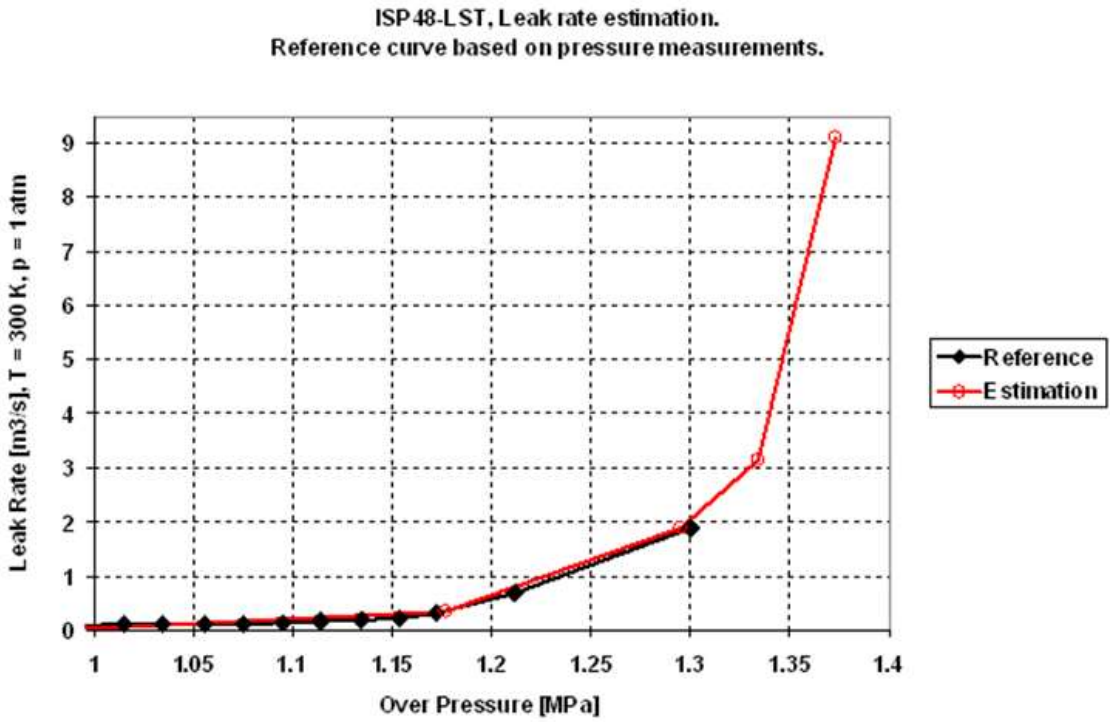
Figure 0.10 The hoop strain field εθ in Sandia test specimen at the internal pressure of 3.5*Pd = 1.365 MPa



In Equation 0.7, L_e = 0.46 m, which is the representative width of the liner element used in hoop strain field analysis. ε_θ is the hoop strain at particular point on the liner. The parameters c_1 and c_2 are tuning parameters to be determined so that Equation 0.4 yields good match to the reference leak data from Sandia test. The total leak rate is determined as the sum of all the leak rates calculated for the strain value locations. The strain points at the buttress locations were omitted from the procedure. It should be noted that we use unit system [m, kg, K° and s] in our equations. That is why k and n have additional multipliers. After optimization it was found that the tuning parameter values c_1 = 7.5E-03 and c_2 = 2.25 yield good estimation of the reference curve from the Sandia test. The curve fit can be seen in

Figure 0.11 [65].

Figure 0.11 Estimation based on Rizkalla equation in the Sandia ISP-48 1:4 scale test against the reference data, which is provided with a separate 1:4 scale test (NUREG 6810)



6.3.6 Illustrative examples for estimating the ultimate pressure capacity and beyond design load behaviour of containments

6.3.6.1 Use of analytical solution formulas

6.3.6.1.1 Sandia scale model

The following definitions refer to the formulae in this subsection to use analytical formulas to develop the pressure capacity of 1:4-Scale Prestressed Concrete Containment Vessel Model tested to ultimate failure in Sandia laboratories [36].

The tendons in Sandia containment model specimen were unbonded.

Prestress load and corresponding radial displacement

ρ= reinforcement ratio = 0.0228

$\rho_{hoop\ rebar} = \rho_{hr}$ = Area of hoop reinforcement/gross concrete area = 0.00865

ρ_{liner} = Area of liner/gross concrete area = 0.00492

$\rho_{hoop\ tendons} = \rho_{ht}$ = 0.00927

t_{liner} = thickness of liner = 0.16 cm

t_{eq} = equivalent concrete thickness or transformed section thickness (concrete section area

With steel portion transformed by ratio of Young's Moduli) = 35.2 cm

$t'_{eq} = t_{eq}$ including rebar and tendons 37.0 = cm

t_c = thickness of concrete wall = 32.5cm

σ_o(concrete) = compressive concrete stress after prestressing = -8.83 MPa

R = Inside radius of cylinder = 538 cm

E_{rebar}, E_{tendon}, E_{liner} = Young's Moduli of rebar, tendon, and liner = 200 000 MPa

Ec = Young's Modulus of concrete = 33 000 MPa

ε_{cr} = Concrete cracking strain = 80 x 10^{-6}

ε_{ly} = liner yield strain = 270/200 000 = 0.00135

ε_{ry} = rebar yield strain = 470/200 000 = 0.00235

ε_{ry} = tendon yield strain = 1300/200 000 = 0.006

$\sigma_{linerult}$ = liner ultimate strength = 498 MPa

σ_{barult} = rebar ultimate strength = 658 MPa

$\sigma_{tendonult}$ = tendon ultimate strength = 1876 MPa

Stress at 4.77% strain was used in this analysis as an upper bound; the Prestressed Concrete Vessel test at Sandia [36] suggests ultimate tendon strains of 1 to 2% might be more realistic.

Prestress load and corresponding radial displacement

There are four hoop tendons of area 3.39cm^2 in every 45cm wall segment.

$\rho_{hoop,tendons}$ = (4 * 3.39 cm^2) / (32.5 * 45) = 0.00927

In compression under tendon action,

σ_0(concrete) = $-\rho_{tendon}\ \sigma_{itendon}$ = -0.00927 x 953 MPa

(Average prestress in hoop tendons including assumed losses).

σ_0 = -8.83 MPa

Inward pressure $P_{inward,\ prestress}$ to correspond prestress, $F_{prestress}$ is

$P_{inward,\ prestress} = \sigma_0 * t_{eq}/R$ = -8.83 * 35.3 / 538 = -0.580 MPa

Corresponding inward radial displacement = (-8.83) / (33000) * 538 * 10 mm ≈ - 1.44 mm

Pressure at which cylinder stress overcomes prestress, P_o

In compression under tendon action,

σ_0(concrete) = $-\rho_{tendon}\ \sigma_{tendon}$ = -0.00927 x 953 MPa

(Average prestress in hoop tendons including assumed losses)

σ_0 = -8.83 MPa

Pressure to overcome prestress, P_o is

$P_0 = \sigma_0 * t_{eq} / R$ = 8.83 * 35.3 / 538 = 0.580 MPa

Corresponding radial displacement = 0 cm

Cylinder hoop cracking pressure, P_{hc}

$P_{hc} = t_c * E_c * \varepsilon_{cr} / R + P_0$ ≈ 32.5 * 33000 * 80 * 10^{-6} / 538 + 8.83 * 35.3 / 538 = 0.74 MPa

Radial displacement at concrete cracking ≈ 0.00008 * 538 * 10 mm ≈ 0.43 mm

Pressure at Liner Yield, Ply

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

$\varepsilon_{ly} = \sigma_{ly} / E_{steel}$ = 270 / 200000 = 0.00135

Solving,

P_{ly} = (0.00135 * 0.0228 * 32.5 * 200000) / 538 + 8.83 * 35.3 / 538 = 0.95 MPa

Liner yield displacement ≈ 0.001 * 538 * 10 mm ≈ 5.38 mm

(To calculate the liner yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

Pressure at rebar yield, P_{ry}

Assuming the tendons have not yielded, the hoop stiffness after cracking is the stiffness of elastic rebar, liner and tendons acting alone. Therefore,

$\varepsilon_{ry} = \sigma_{ry} / E_{steel} = 470 / 200000 = 0.00235$

Solving,

P_{ry} = (0.00235 * (0.0228 - 0.00492) * 32.5 * 200000) / 538 + (0.00135 * 0.00492 * 32.5 * 200000) / 538 + 8.83 * 35.3 / 538 = 1.17 MPa

Rebar yield displacement $d_{rebar,yield}$ ≈ 0.002 * 538 * 10 mm ≈ 10.76 mm

(To calculate the rebar yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

Ultimate cylinder membrane failure based on ultimate strengths of steel components, P_{ult}

P_{ult} = (0.006 * (0.0228 - 0.00492 - 0.00865) * 32.5 * 200000) / 538 + (0.00235 * 0.00865 * 32.5 * 200000) / 538 + (0.00135 * 0.00492 * 32.5 * 200000) / 538+8.83 * 35.3 / 538 = 1.57 MPa ≈ 4 * P_d

Ultimate radial displacement, $d_{ultimate}$ ≈ 0.006 * 538 *10 ≈ 32.3 mm

The developments calculated above are presented in Table 0.3 and plot format in

Figure 0.12 and **Figure 0.13** below.

Table 0.3 Pressure – displacement relationship showing the conceptual sequence developed by using analytical formulas for the cylinder of the infinite length of Sandia 1:4 model test up to the pressure capacity.

1:4 Sandia containment model test	**Pressure [MPa]**	**Rad disp. [mm]**	**Corresponding pressure at test (** Figure **0.11)**	**Corresponding displacement at test (** Figure **0.11)**
prestress	0	-1.44	0 (green curve)	-1.78
pressure to overcome prestress	0.58	0	0.582 (green curve)	0
concrete cracking	0.74	0.43	0.595 (green curve)	0.662
liner yield	0.95	5.38	0.972 (green curve)	8.13
rebar yield	1.17	10.76	1.111 (green curve)	11.7
tendon yield	1.57	32.3	1.400 (red curve)	79.2

Figure 0.12 Conceptual corner points of Sandia 1:4 containment model test, the design pressure for the Sandia containment model was 0.39 MPa

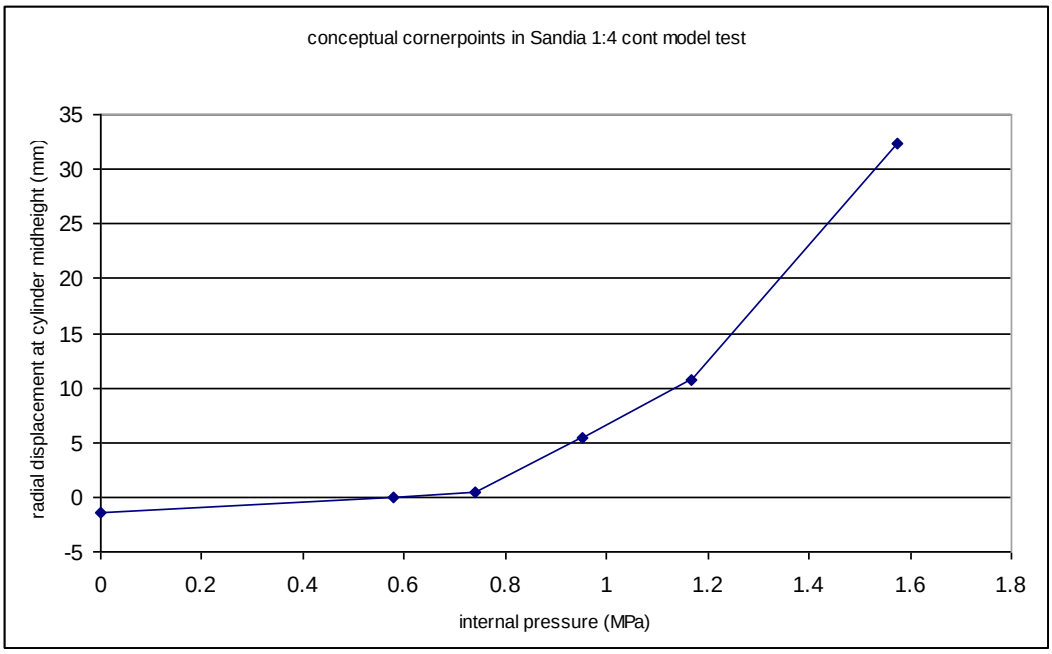
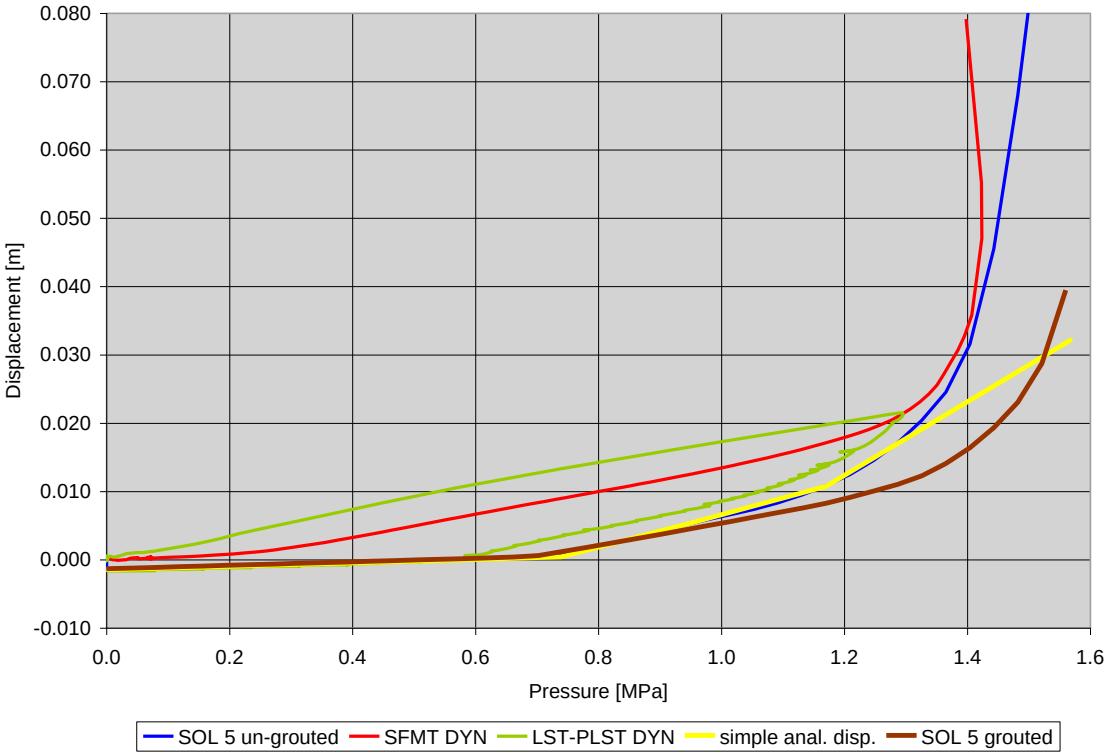


Figure 0.13 The actual pressure – radial displacement relationship in the Sandia 1:4 containment model test



The green curve in **Figure 0.13** shows pressure – radial displacement relationship in cylinder mid-height for leak-rate (LST) test performed for un-cracked containment and the red curve shows radial displacement

relationship in cylinder mid-height for ultimate pressure capacity test (SFMT) performed for already cracked containment. The yellow curve (simple anal. disp.) repeats the conceptual corner points of the curve in

Figure 0.11. The blue curve (SOL 5 unbonded) shows the FEM simulation results of the Sandia test specimen with unbonded tendons given in [27]. The brown curve (SOL 5 bonded) shows the FEM simulation results of the Sandia test specimen with unbonded tendons given in [27].

It can be seen from **Figure 0.13** that estimated ultimate capacity in tendon modelling with bond is 1.56 MPa and that estimated ultimate capacity in tendon modelling without bond is 1.44 MPa meaning 8% difference in estimated ultimate capacity.

6.3.6.1.2 Bechtel-IVO design on LO3 VVER440 containment, 1985 [61]

The tendons in LO3 containment design are unbonded.

Prestress load and corresponding radial displacement

There is one hoop tendon of area 51.15 cm^2 in every 44 cm wall segment with wall thickness of 1.2 m.

$\rho_{hoop\ tendons}$ = (51.15cm^2)/(120*44 cm^2) = 0.00968

In compression under tendon action,

σ_0(concrete) = $-\rho_{tendon}\ \sigma_{itendon}$ = -0.00968 x 976 MPa

(Average prestress in hoop tendons including assumed losses).

σ_0 = -9.45 MPa

Inward pressure $P_{inward,\ prestress}$ to correspond prestress, $F_{prestress}$ is

$P_{inward,\ prestress} = \sigma_0 * t / R$ = -9.45 * 1.2 / 21.60 = -0.525 MPa

Corresponding inward radial displacement = (-9.45)/(33500)*21600 mm ≈ - 6.09 mm

Pressure at which cylinder stress overcomes prestress, Po

In compression under tendon action,

σ_0(concrete) = $-\rho_{tendon}\ \sigma_{itendon}$ = -0.0068 x 996 MPa

(Average prestress in hoop tendons including assumed losses).

σ_0 = -9.45 MPa

Pressure to overcome prestress, P_o is

$P_0 = \sigma_0 * t/R$ = 9.45*1200/21600 = 0.525 MPa

Corresponding radial displacement = 0 cm

Cylinder hoop cracking pressure, P_{hc}

$P_{hc} = t_c * E_c * \varepsilon_{cr} / R + P_0 \approx 1200 * 33500 * 100 * 10^{-6} / 21600 + 9.45 * 1200 / 21600 = 0.71$ MPa

Radial displacement at concrete cracking ≈ 0.0001 * 21600 ≈ 2.16 mm

Pressure at liner yield, P_{ly}

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

$\varepsilon_{ly} = \sigma_{ly} / E_{steel}$ = 270 / 200000 = 0.00135

ρ = reinforcement ratio = 0.0167

$\rho_{hoop\ rebar} = \rho_{hr}$ = Area of hoop reinforcement/gross concrete area = 0.002

ρ_{liner} = Area of liner/gross concrete area = 6 / 1200 = 0.005

$\rho_{hoop\ tendons}$ = (51.15 cm^2) / (120 * 44 cm^2) = 0.00968

Solving,

P_{ly} = (0.00135 * 0.0167 * 1200 * 200000) / 21600 + 9.45 * 1200 / 21600 = 0.775 MPa

Liner yield displacement ≈ 0.00075 * 21600 ≈ 16.2 mm

(To calculate the liner yield displacement the steel strain is assumed to be 40% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

Pressure at rebar yield, P_{ry}

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

$\varepsilon_{ry} = \sigma_{ry} / E_{steel} = 470 / 200000 = 0.00235$

P_{ry} = (0.00235 * (0.0167 - 0.005) * 1200 * 200000) / 21600 + (0.00135 * 0.005 * 1200 * 200000) / 21600 + 9.45 * 1200 / 21600 = 1.17 MPa

Rebar yield displacement $d_{rebar,\ yield}$ ≈ 0.002 * 21600 ≈ 43.2 mm

(To calculate the rebar yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

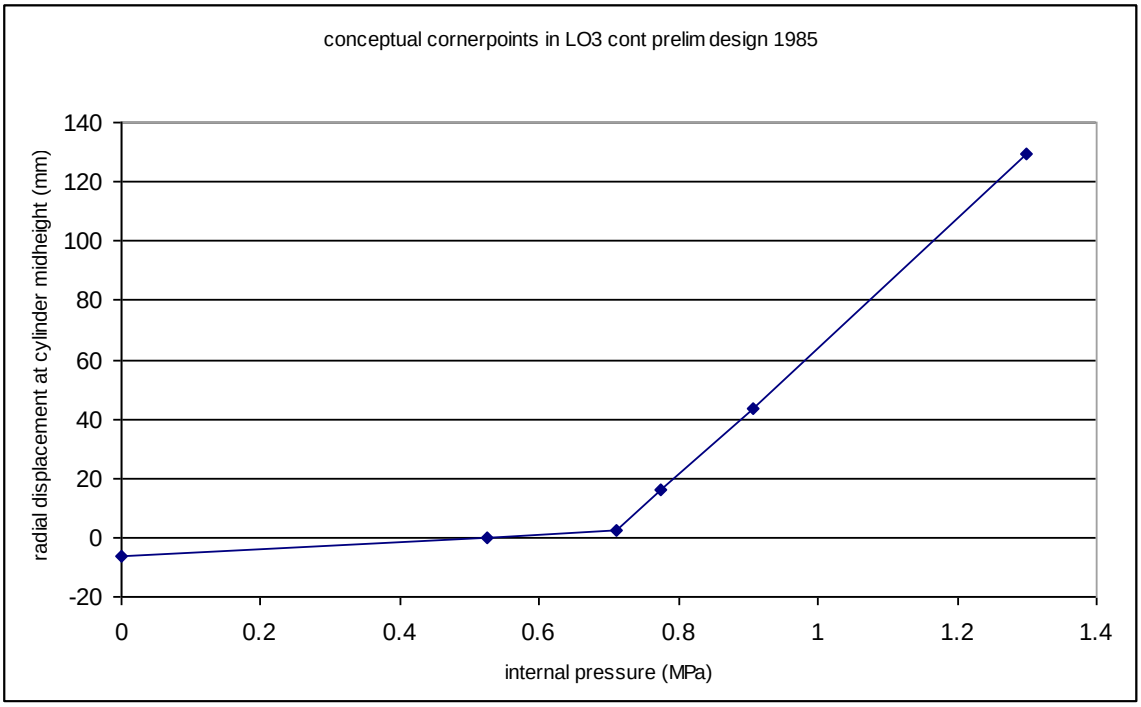
Ultimate cylinder membrane failure based on ultimate strengths of steel components, P_{ult}

P_{ult} = (0.006 * (0.0167 - 0.005 - 0.002) * 1200 * 200000) / 21600 + (0.00235 * 0.002 * 1200 * 200000) / 21600 + (0.00135 * 0.005 * 1200 * 200000) / 21600 + 9.45 * 1200 / 21600 = 1.3 MPa ≈ 3.7 * P_d

Ultimate radial displacement $d_{ultimate}$ ≈ 0.006 * 21600 ≈ 129.6 mm

The ultimate stress estimate by BECHTEL/FINEL nonlinear axisymmetric finite element code was 2.2*P_d.

Figure 0.14 Conceptual corner points of LO3 containment preliminary design, the design pressure for LO3 containment was Pd = 0.35 MPa



6.3.6.1.3 OL3 EPR containment, 2005 [43]

The tendons in LO3 containment design are bonded.

Prestress load and corresponding radial displacement

There is one hoop tendon of area 154 cm^2 in every meter wall segment with wall thickness of 1.3 meters.

$\rho_{hoop\ tendons}$ = (187 cm^2) / (130 * 100 cm^2) = 0.01438

In compression under tendon action,

σ_0(concrete) = $-\rho_{tendon}\ \sigma_{itendon}$ = -0.01438x 960 MPa, (Avg. prestress in hoop tendons including assumed losses)

σ_0 = -13.80 MPa

Inward pressure $P_{inward\_prestress}$ to correspond prestress, $F_{prestress}$ is

$P_{inward\_prestress} = \sigma_0 * t / R$ = -13.80 * 1300 / 24050 = -0.746 MPa

Corresponding inward radial displacement = (-13.80) / (33500) * 24050 ≈ - 9.91 mm

Pressure at which cylinder stress overcomes prestress, P_o

In compression under tendon action,

σ_0(concrete) = $-\rho_{tendon}\ \sigma_{itendon}$ = -0.01438 x 960 MPa

(Average prestress in hoop tendons including assumed losses).

σ_0 = -13.8 MPa

Pressure to overcome prestress, P_o is

$P_0 = \sigma_0 * t / R$ = 13.8 * 1300 / 24050 = 0.746 MPa

Corresponding radial displacement = 0 cm

Cylinder hoop cracking pressure, P_{hc}

$P_{hc} = t_c * E_c * \varepsilon_{cr} / R + P_0 \approx 1300 * 33500 * 100 * 10^{-6} / 24050 + 13.8 * 1300 / 24050 = 0.93$ MPa

Radial displacement at concrete cracking ≈ 0.0001 * 24050 ≈ 2.4 mm

Pressure at liner yield, P_{ly}

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

$\varepsilon_{ly} = \sigma_{ly} / E_{steel} = 270 / 200000 = 0.00135$

ρ= reinforcement ratio = 0.004615 + 0.004615 + 0.01438 = 0.02361

$\rho_{hoop\ rebar} = \rho_{hr}$ = Area of hoop reinforcement/gross concrete area = 60 / 130 / 100 = 0.04615

ρ_{liner} = Area of liner/gross concrete area = 6 / 1300 = 0.004615

$\rho_{hoop\ tendons} = (187\ cm^2) / (130 * 100\ cm^2) = 0.01438$

Solving,

$P_{ly} = (0.00135 * 0.02361 * 1300 * 200000) / 24050 + 13.8 * 1300 / 24050 = 0.775$ MPa

Liner yield displacement ≈ 0.00075 * 24050 ≈ 18.04 mm

(To calculate the liner yield displacement the steel strain is assumed to be 40% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

Pressure at rebar yield, P_{ry}

Assuming the tendons have not yielded, the hoop stiffness after cracking is approximately that of elastic rebar, liner and tendons acting alone. Therefore:

$\varepsilon_{ry} = \sigma_{ry} / E_{steel} = 470 / 200000 = 0.00235$

$P_{ry} = (0.00235 * (0.02361 - 0.004615) * 1300 * 200\ 000) / 24050 + (0.00135 * 0.004615 * 1300 * 200000) / 224050 + 13.8 * 1300 / 24050 = 1.17$ MPa

Rebar yield displacement $d_{rebar,\ yield} \approx 0.002 * 24050 \approx 48.1$ mm

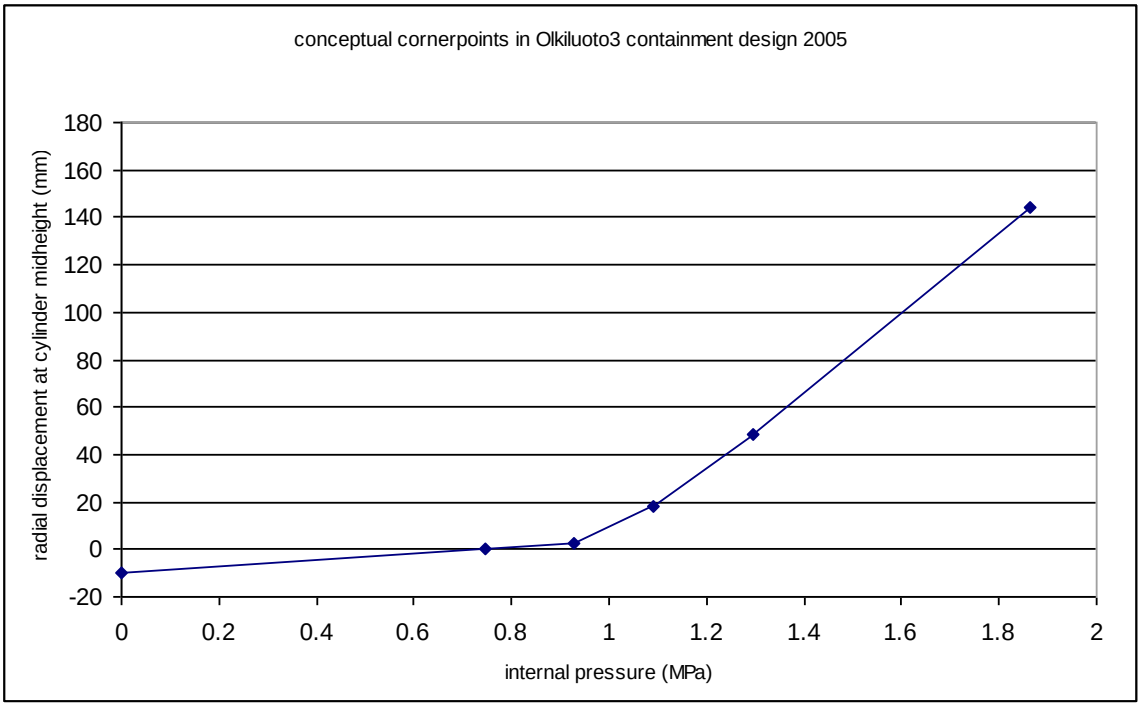
(To calculate the rebar yield displacement the steel strain is assumed to be 35% higher than the average shell strain, which means that the average shell strain calculated for whole circumference is less than the steel strain at cracks because of the stiffness increase in un-cracked portions of shell wall).

Ultimate cylinder membrane failure based on ultimate strengths of steel components, P_{ult}

$P_{ult} = (0.006 * (0.02361 - 0.004615 - 0.004615) * 1300 * 200000) / 24050 + (0.00235 * 0.004615 * 1300 * 200000) / 24050 + (0.00135 * 0.004615 * 1300 * 200000) / 24050 + 13.8 * 1300 / 24050 = 1.86\ MPa \approx 4.13 * P_d$

Ultimate radial displacement $d_{ultimate} \approx 0.006 * 24050 \approx 144.3$ mm

Figure 0.15 Conceptual corner points of Olkiluoto3 containment design, the design pressure for OL3 containment was Pd = 0.53 MPa (abs)



6.3.6.2 Ultimate capacity by 3D modelling

6.3.6.2.1 Tianwan unit 1 & 2, 1998 [58]

The Tianwan inner containment is a prestressed concrete shell structure that consists of a cylindrical part and a hemispherical dome. The inner surface of the containment is covered with a 6 mm thick carbon steel plate to secure the tightness. The inside diameter of the cylinder is 44.0 m. The height of the cylinder part is 41.6 m and the top of the dome is at level +71.60 m. The thickness of the cylinder and dome are 1.2 m and 1.0 m respectively. Concrete characteristics are given in Table 0.4 below.

Table 0.4 Concrete characteristics for Tianwan containment.

Notation	Description	Grade B45
f_c'	Specified compressive strength of concrete	37.5 MPa
E_c	Modulus of elasticity of concrete	29 000 MPa
ν_c	Poisson's ratio of concrete	0.2
α_c	Coefficient of thermal expansion of concrete	$1.0 \cdot 10^{-5}$ /°C

In the non-linear analysis the isotropic material of the concrete of grade B45 is described using the stress-strain relationship presented in Table 0.5. The liner plate and reinforcement has been taking into account by giving the tension strength for the concrete. Design internal pressure for Tianwan containment was 0.4 MPa and the beyond design check was made for 1.5 times the design pressure.

Table 0.5 Non-linear concrete stress-strain characteristics for Tianwan containment.

Strain [10^{-3}]	-1000	-2.0	-1.5	-1.0	-0.5	0.0	1.95	1000
Stress [MPa]	-37.5	-37.5	-35.5	-28.0	-14.5	0.0	2.613	2.613

Tianwan containment prestressing tendons are Freyssinet type tendons consisting of 55 strands of 15.7 mm nominal diameter, made of high strength steel SUPER St 1630/1860. Nominal cross-section area of a tendon is 8250 mm^2. Strength and elastic characteristics for the prestressing steel is given in Table 0.6.

Table 0.6 Prestressing steel characteristics for Tianwan containment.

Notation	Description	St 1630/1860
f_{pu}	Specified tensile strength of tendon	1860
f_{py}	Specified yield strength of tendon, 1% strain	1630
E_p	Modulus of elasticity of tendon	199 000
α_p	Coefficient of thermal expansion of tendon	$1.0 \cdot 10^{-5}$ /°C

The extreme values of stresses in tendons without elastic losses are given Table 0.7.

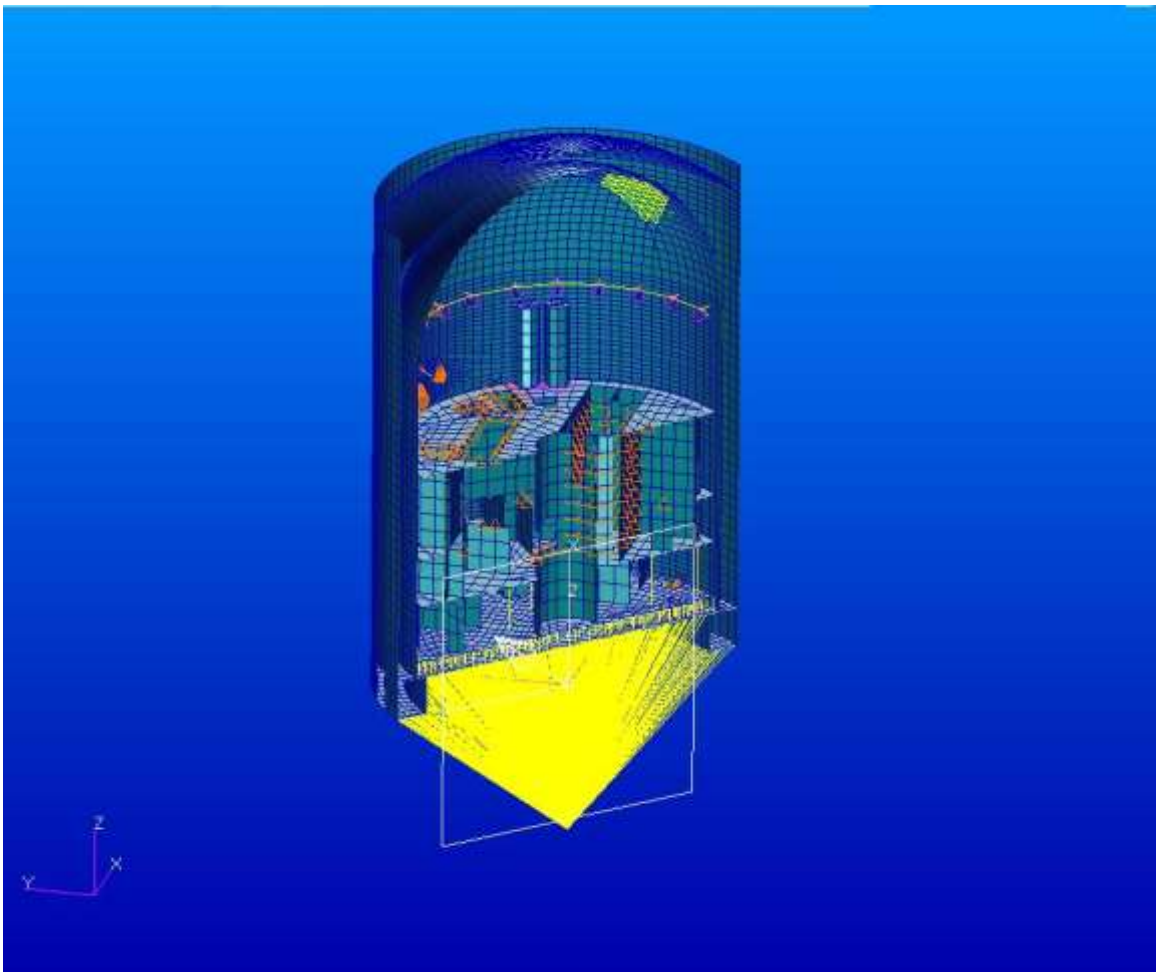
Table 0.7 The extreme values of stresses in tendons without elastic losses in Tianwan containment.

	Horizontal tendons [MPa]		Vertical tendons [MPa]	
	Group 1	Group 2	Group 1	Group 2
After prestressing, max.	1452	1452	1413	1413
After prestressing, min.	603	649	740	672
After 40 years, max.	1347	1347	1315	1315
After 40 years, min.	540	583	674	610

In

Figure 0.16 the cross-section view of the Tianwan reactor building model is given. The total number of degrees of freedom in the model is 140 000. The beyond design analysis for the containment was carried out with the aid of the whole reactor building model. The yellow rod elements in the model represent the foundation soil. The tendons in Tianwan containments were unbonded.

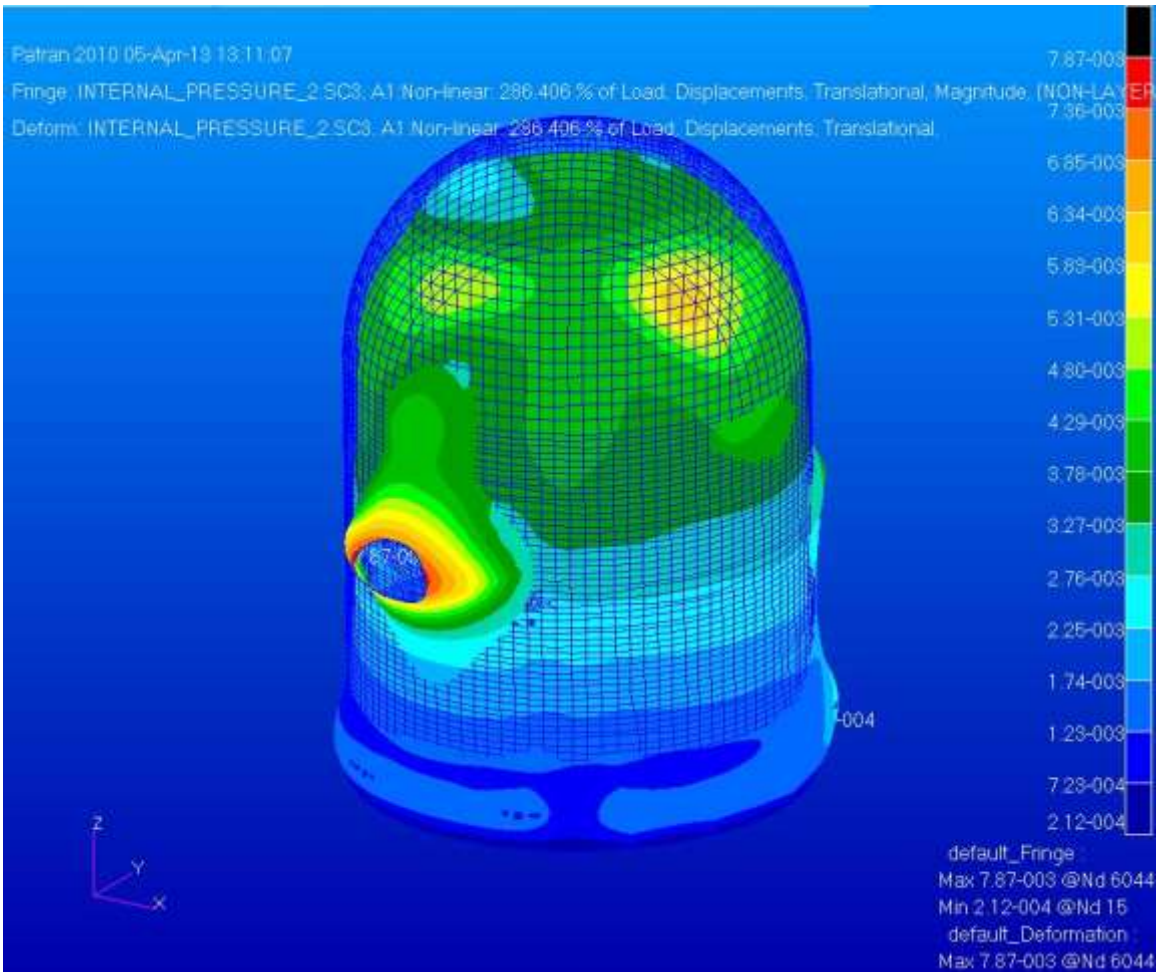
Figure 0.16 The cross-section view of the Tianwan reactor building model



The displacements (in meters) of the Tianwan inner containment shell for the internal pressure of 0.6 MPa are given in

Figure 0.17.

Figure 0.17 The displacements (in meters) of the Tianwan inner containment shell for the internal pressure of 0.6 MPa



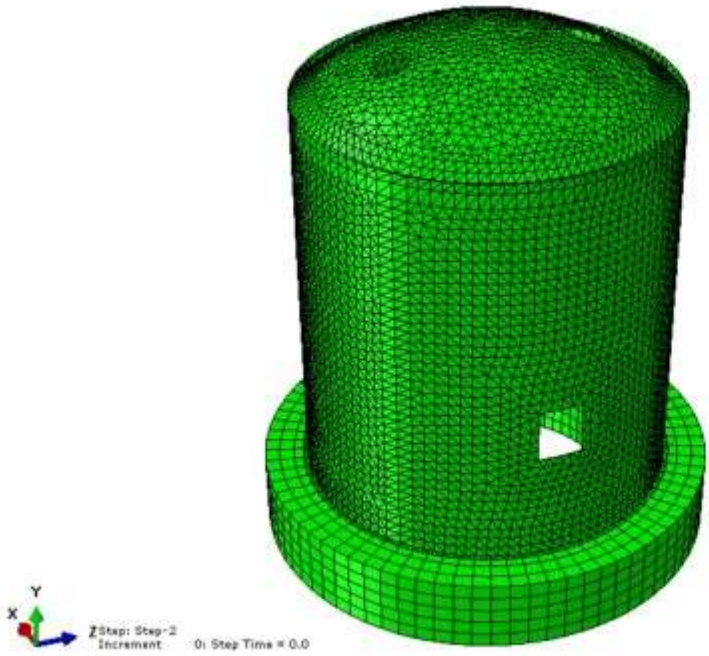
6.3.6.2.2 BARCOM containment model tests [59]

In this subsection a preliminary mid-term analysis of the BARCOM test model is presented. The BARCOM test model is a 1:4 scale of an existing pressurized heavy water reactor (PHWR) prestressed concrete inner containment of 540 MW Tarapur Atomic Power Station 3 & 4 units in India. The analysis was carried out using ABAQUS software. In the analysis explicit time integration was used in quasi static setting. For the concrete material presentation the brittle cracking model was adopted. In the brittle cracking model the value used for the ultimate tensile stress was 3.6 MPa which is slightly higher than the value specified in the BARCOM documentation. The Young's modulus and Poisson's ratio used for concrete were 30 GPa and 0.2, respectively. The tendons were assigned ideal plastic von Mises material model with 1.683 GPa yield stress. The Young's modulus and Poisson's ratio used for tendons were 190 GPa and 0.3, respectively.

The mesh of the containment consists of 4 -node quadrilateral and 3 -node triangle general purpose elements. In the analysis the bottom edge of the containment is fixed. The tendons in BARCOM specimen were unbonded. The BARCOM specimen finite element model used in [59] is given in

Figure 0.18.

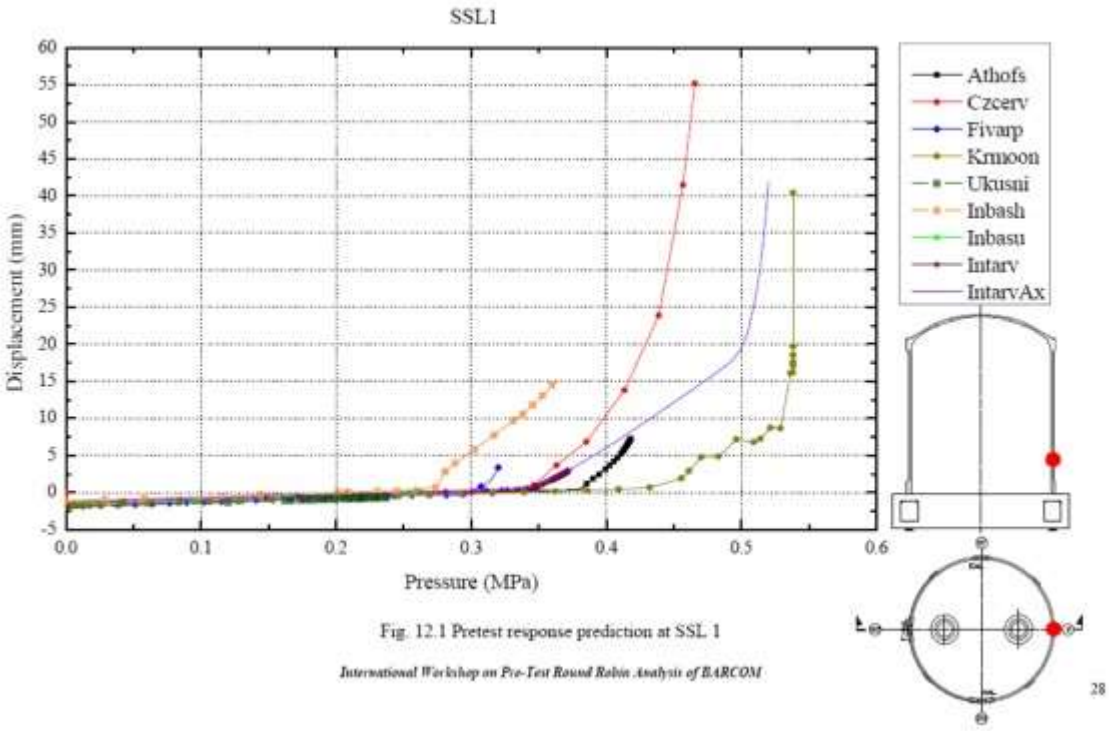
Figure 0.18 BARCOM specimen finite element model used in [59]



The ultimate capacity predictions from various participants in BARCOM benchmark in measuring location SSL-1 are given in

Figure 0.19.

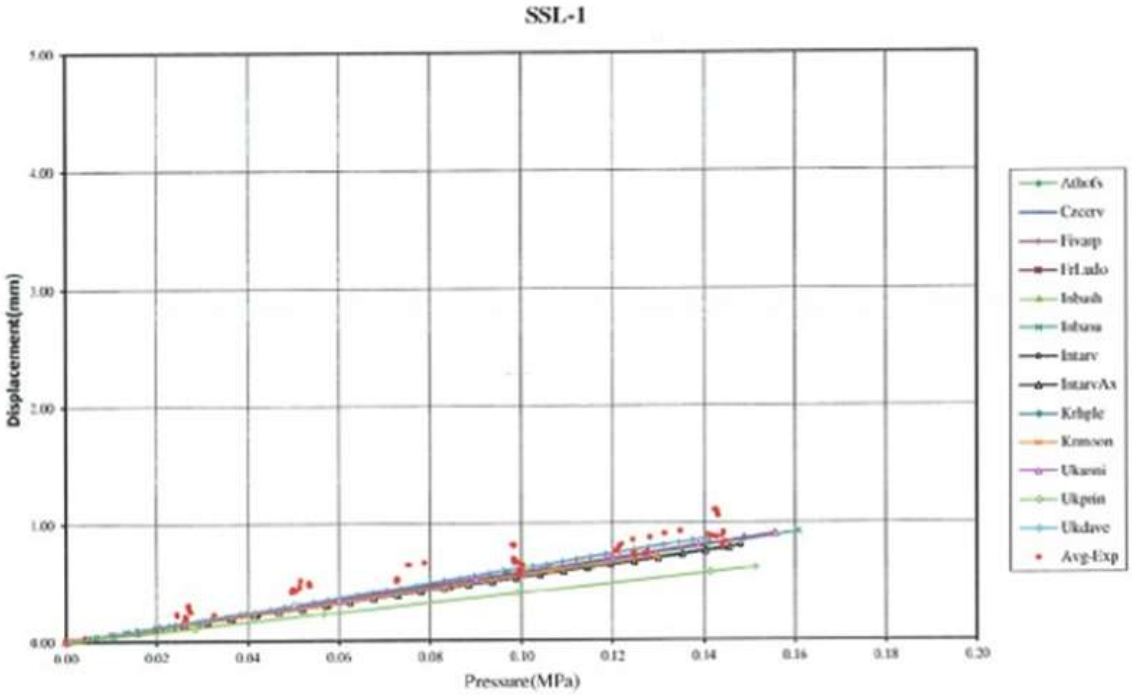
Figure 0.19 Ultimate capacity predictions in BARCOM benchmark. Fortum contribution [59] is denoted by notation Fivarp



The comparison between measured displacement and simulated displacement by various participants in the benchmark in BARCOM leak-rate test are given in

Figure 0.20. The ultimate capacity test in the BARCOM program is not carried out, April 2013.

Figure 0.20 Simulated and experimental results from BARCOM leak-rate test



6.3.7 Probabilistic methods for ultimate pressure capacity assessment

The objective is to assess the pressure capacity of the containment at which the structural integrity is retained, and a failure leading to a significant release of fission products does not occur. For estimating the pressure capacity of the containment structure by deterministic structural simulation with the aid of the static nonlinear 3D finite element analysis will be needed for predicting the global response. It is possible also to use the linear Finite Element model which is developed during the containment design process at the design basis level.

6.3.7.1 Assessment of the mechanical capacity of the containment

The three-dimensional model of the containment wall, developed as part of the initial detailed design is adapted for this assessment. The modelled structure represents the entire containment, from the base of the gusset to the top of the dome. The model includes the tendon buttresses and the main openings of the different hatches (equipment hatch and personal airlock) and associated metal sleeves. Prestressing is accurately modelled by coding, discretisation of tendons and the application of loads induced on the finite element model by prestressing or by modelled the tendons as steel sections in the FE model. The rebar are not modelled as resistant bars because the model is used at the initial design stage for the rebar calculation. The calculations performed are elastic and linear. The elementary load cases are: self-weight, pressure and prestressing.

After the rebar calculation, the rebar drawings are established.

The beyond design assessment consists of determining possible containment failure modes and the pressure associated with fracture. Several standard areas of the inner containment are assessed (Cylinder general section at mid-height, central part of the dome, bottom part of the containment, base of the dome,

top of the cylinder beneath the dome). The analysis is finalised by an assessment of several singular zones (Reinforcement of the equipment hatch, personnel airlocks and some through wall small penetrations).

The failure mode and median pressure associated with ultimate loads resulting in the failure of the section (rupture pressure at 50% confidence: $P_{ult,50}$) is estimated with the following methodology.

The methodology adopted for the failure analysis of the different areas assessed, consists of making a direct comparison between the loads induced by a pressure P_{ult} exerted on the inner surface of the containment wall and the ultimate strength due to the various resistant elements present in the wall of the section under assessment. Failure is reached when one of these elements ruptures (usually when it reaches its maximum elongation).

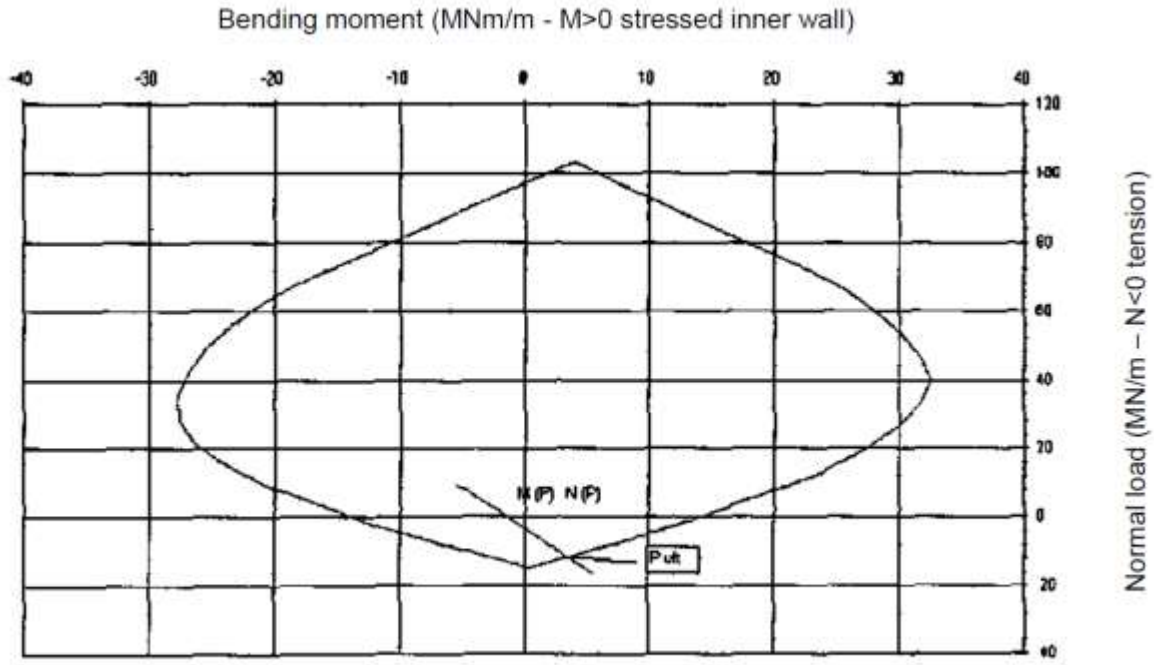
For each standard area and each singular zone which are modelled, the loads induced in the concrete wall are calculated on the basis of the results of the elementary cases from the three-dimensional finite element model of the inner containment wall. For the sections around small penetrations, the loads are determined using analytical formulae.

Changes in elastic loads N(P) and M(P) (Normal forces and bending moment according to the pressure P) in the wall are then modelled on the assumption that they vary linearly as a function of the pressure for each section. In singular zones, it is assumed that the appearance of locally non-linear behaviour does not significantly affect the load distribution.

For each section, a resistance diagram is constructed using the elementary resistances of each constituent of the wall and the pre-existing stresses that exist in some materials (liner compression, tension in the prestressing tendons). The ultimate resistance domain of the section is then determined, ignoring the tensile strength of the concrete.

The line defining the variation in loads N(P), M(P) as a function of the pressure is then superimposed on the resistance diagram characterising the ultimate strength capacities of the section. The ultimate pressure P_{ult} is defined by the point of intersection of the line N(P), M(P) with the resistance diagram.

Figure 0.21 Example of resistance diagram



This deterministic approach is generally completed by a probabilistic assessment of the ultimate resistance of the containment to determine the mechanical strength limit pressures, allowing for intrinsic variability of the materials and models. The method used to determine the fragility curve is taken from the EPRI report [18].

The analysis consists of establishing the variability linked to the mechanical characteristics taken into consideration and the modelling considered in the analysis of the strength capacity of the structure and the mechanical loads induced by the internal pressure. This variability, specific to each failure mode, is used to determine the rupture pressure at a 95% confidence level ($P_{ult,95}$).

The probabilistic assessment is performed considering the variable "pressure capacity" as a random variable resulting from the product or quotient of independent random variables characterised by log-normal probability laws. This hypothesis has two advantages: it can correctly describe the spread of the mechanical characteristics of the materials (log-normal law) and associate the variable P with a probability law of the same type using the mathematical properties of the independent random variables and the log-normal law.

The "pressure capacity" variable is thus completely defined by three parameters:

- Median ultimate pressure Pm,
- Modelling variability βM,
- Material variability βS.

The logarithmic βM and βS variability quantify the uncertainty due to the lack of information arising from the differences between the analytical model and the actual structure and material properties. Modelling uncertainties are introduced by the hypotheses and simplifications used to produce the analytical models and their ability to provide an adequate representation of the failure conditions. Uncertainties in

terms of the strength of the section are linked to those regarding the materials (behaviour laws, yield and tensile strength, influence of the temperature on the mechanical characteristics).

The ultimate rupture pressure at the 95% confidence level, corresponding to a probability of occurrence equal to 5%, is therefore estimated using the following formula:

$P_{ult,95} = P_m \exp(-1.645\ \beta)$ with $\beta^2 = \beta_M^2 + \beta_S^2$ Eq. 0.8

6.3.8 Assessment of the leak tightness of the containment

6.3.8.1 Beyond Design with nonlinear Finite Element Model

Large penetrations are usually included in the finite element model; smaller penetrations and penetration closure components are analysed using a local finite element models either separate or as parts of modified global finite element model.

For cylindrical containment structures the use of analytical solution formulas to estimate the strain field in the wall in order to estimate leak tightness of the containment.

The initial condition for the nonlinear analysis of the containment structure should be the linear elastic response caused by dead load and design pressure, at the design temperature. The internal pressure is incrementally increased until a failure is reached.

The localization phenomena can be induced by:

- Concrete cracking (cracks patterns is greatly depending of the choice bonded/unbonded tendons),
- Steel liner deformation at anchorage vicinity or other local layout (e.g.: penetrations, change in thickness).

7. CONSTRUCTION

7.1 General principles

The general principle of construction of post prestressing reinforced concrete containment of nuclear reactors is a complex process but is almost the same for all types of envisaged prestressing technology:

- Construction of reinforced concrete containment and metallic liner including instrumentation placed into concrete walls,
- Placing of the prestressing cables,
- Tensioning of the cables,
- Protection of cables by adequate system - grout or grease.

The prestressing operations are on the critical path of the global planning of plant construction. The entire process takes around 2- third of a year, with roughly one-third of this time allocated for each of: placing the cables, tensioning, and cement grouting. The non-grouting of the tendons can save some time in the prestressing sequence, however this gain is theoretical as other factors must also be taken into account (for example, the ducts are more encumbering and difficult to install). It is important to note that all the available time is used – that is to say that the process is continuous.

Due to crucial importance of prestressing system for nuclear safety, Constructor (or Constructor's prestressing system subcontractor) shall be certified for all kind of activities linked with prestressing system from manufacturing till finishing after application of prestressing. Process of certification is driven by requirements of Authorities in the country of construction site.

Generally, two kind of certification are required. First one is focused on nuclear safety and deals with independent check of all construction processes including certification of design, used materials and products. The goal of this certification is to demonstrate that design and its application meets required level of nuclear safety. Second one is focused on technical safety during construction, especially during application of prestressing. The constructor proofs that he has knowledge, qualification and practice for required work.

7.2 Requirements for containment structure

7.2.1 Installation of instrumentation devices inside containment wall

Due to high importance of stress-strain response of prestressed concrete structures (both short and long term), instrumentation measuring deformations and forces inside containment walls is installed. Generally, sensors placed into the wall before concrete casting are used.

Individual sensors as well as complex measuring systems (compound from sensors, wires, electronics, SW) shall be verified for assumed application and certified for using in NPP conditions. The kind of certification shall follow expected role of measuring system in design – e.g. the design shall define if the function of the system is required during and after LOCA.

The installation of containment monitoring systems should be performed by highly-qualified company and experienced staff.

Before being installed within or on RC containment, monitoring instruments and devices should be carefully stored and protected from extreme weather conditions, water ingress, corrosion, impact and shocks.

Technical specifications should be provided for the mounting and installation of the sensors as well as for their protection during the concrete pouring. Particular attention shall be paid to the following points:

- Installation procedures should be approved by the Client,
- If necessary, (e.g. delay between installation and operational use of the devices), temporary protection should be implemented,
- Just before and after mounting, checks and tests should be undertaken to verify that the instruments are functional and suitable for the intended use. The results of these checks and tests should be recorded in a Site Acceptance Test file, submitted to the Client for acceptance of the installation,
- Power and data wires should be protected during installation and concreting. This point is of critical importance for sensors embedded within concrete members,
- For embedded sensors, the exact position of the sensors should be recorded (tolerance about 5 cm to the design drawings for the positioning of a measurement point). The direction of embedded extensometers should be verified carefully.

The presence of monitoring devices (sensors, wire, junction and marshalling boxes) should be marked properly on site.

Additional considerations can be proposed for the VWSG, which are commonly used for containment monitoring. These sensors are set into the concrete in one of two ways: either by casting the devices into the mix directly or by pre-casting into concrete “briquettes” that are subsequently cast into the structure.

If the VWSG are directly cast into the structure, the ends of the sensors can be attached to the reinforcement rebars or attached to a specific supporting frame (see

Figure 0.1). Precautions should be taken to avoid damage to the cable or VWSG from vibrators. It is also essential that large pebbles or aggregate do not rest against the gauge, as these will cause localized strain discontinuities that may influence the gauge readings.

Figure 0.1 Example of VWSG mounting directly attached to reinforcement rebar (left) or with specific supporting frame (right)

If the VWSG are pre-cast into briquettes of the same concrete mix as the mass concrete, the fabrication of a briquette should be close to the concrete pouring and it should be continuously cured with water until the placement into the structure.

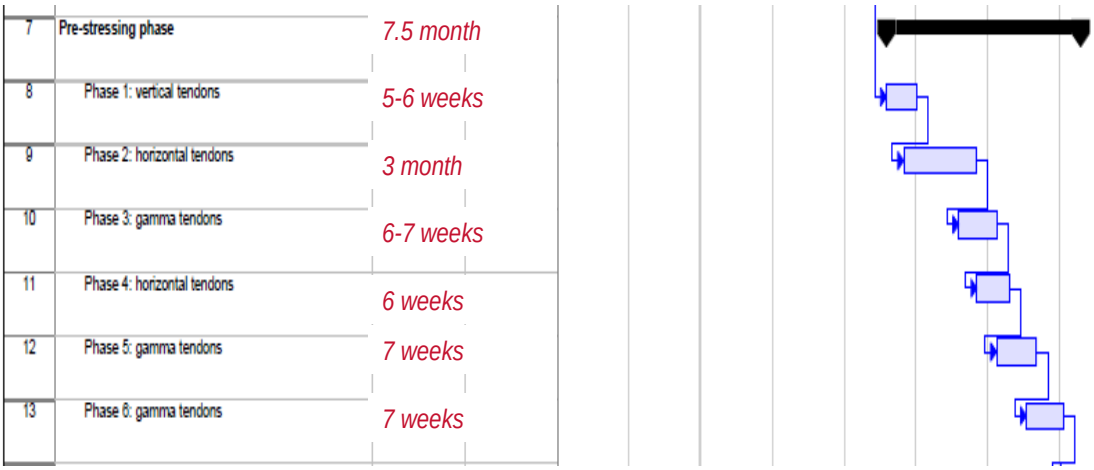
7.2.2 Requirements for detailed design of containment structure

Before construction, detailed studies are carried out by the civil engineering office responsible for of the preliminary design of the containment of the reactor building and the metallic liner.

During the prestressing phase of the horizontal cables, a stress analysis of the internal wall of the containment is undertaken, considering that the internal wall was entirely concreted.

The stresses induced by the tensioning of the cables are calculated for all the sections studied in the internal containment. The corresponding reinforcement required is then calculated. The rebar sections obtained are then compared with that resulting from previous dimensioning calculations.

Figure 0.2 Example of planning for prestressing phase



It is not the objective of the document to give a detailed description of the cable tensioning process. This can be defined in the detailed specifications by the Constructor in accordance with the construction schedule and with the general instructions given in this report.

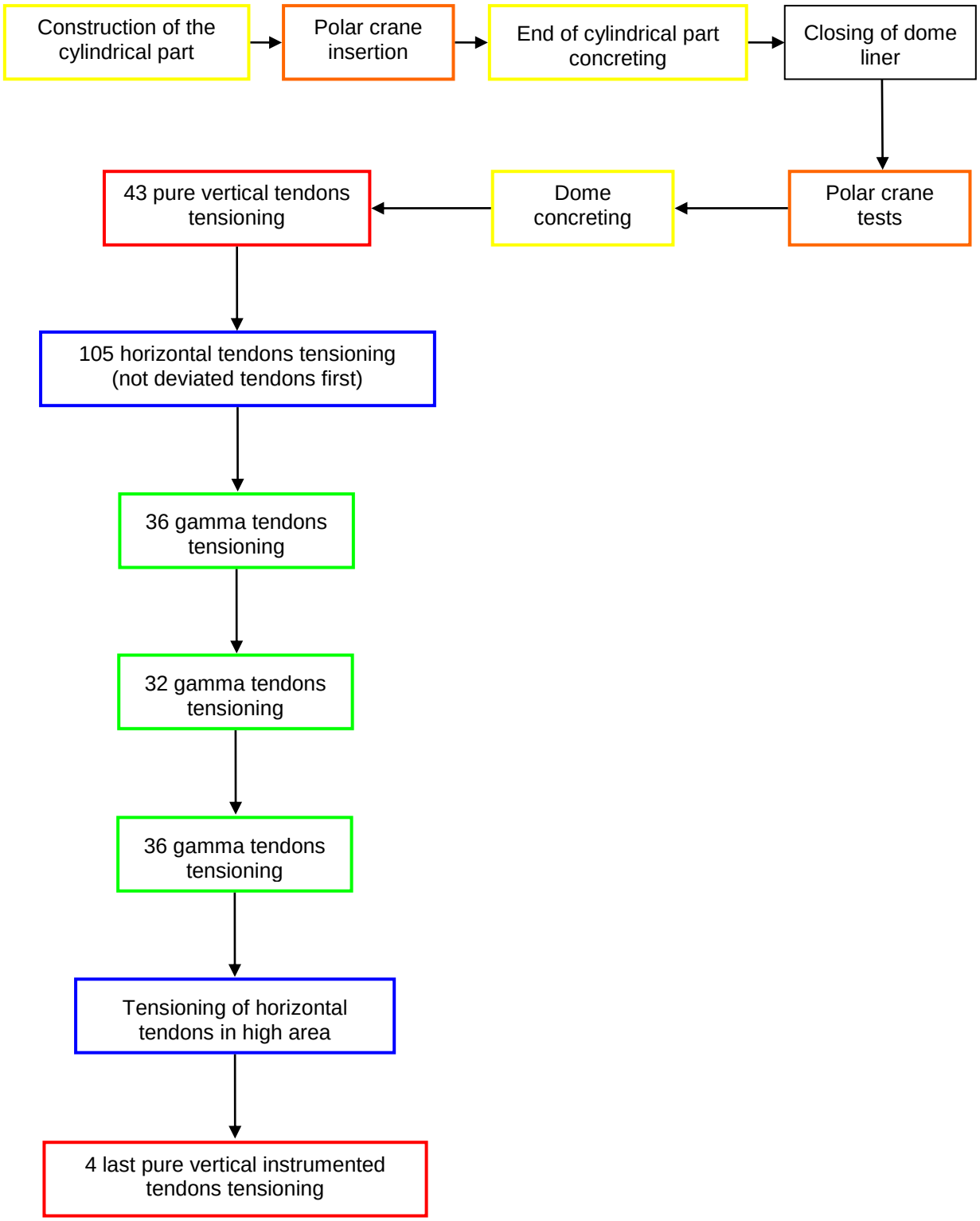
The tensioning of horizontal cables takes place after the tensioning of the vertical cables in order to avoid excessive flexural effects.

7.2.3 Requirements for preparedness of containment structure for prestressing

The tensioning of the tendons should be carried out no earlier than 28 days after concreting in order to limit early age concrete creep. As an example, for the case of the EPR, the tensioning of the gamma cables is carried out when concreting of the dome has been completed and when the concrete of the last layer has reached more than 28 days and attained the specified compressive strength. This is in order to ensure that the concrete has sufficient mechanical resistance to support the tensioning of the cables and thus limit the concrete creep deformations.

Designer should specify to contractor times for prestressing in consideration to creep (prestressing for each part should be correlated to the time of casting).

Figure 0.3 Example of general program of cable tensioning of a nuclear containment



7.3 Material handling, traceability and tests

7.3.1 General requirements

The construction procedures should specify the manner in which material is to be received, stored, and handled in the Fabricator's plant and at the construction site.

The Constructor's, Fabricator's, or Material Supplier's Quality Assurance Program shall provide for material identification and segregation. Based on these programs, primary checks should be executed during material take over procedure. Checks are focused on material specification, geometry parameters and validity of authorization.

Whenever possible, standard procedures should be applied to reduce the risk.

In case of design according European standards, the post-tensioning system should have a European Technical Approval (ETA) and a corresponding certificate of product conformity delivered respectively by the relevant bodies as defined in ETAG 013 [17]. All post-tensioning system components should be referenced in the ETA, and jacks should comply with additional requirements. The post-tensioning system should satisfy the following documents (in order of priority):

- The requirements of the present section,
- The ETA.

Further to the demonstrations carried out under the scope of European Technical Approval, additional specifications shall be required to prove that the post-tensioning system is suitable for use in the containment of a nuclear power plant. These requirements are based on the feedback experience of the construction of nuclear containments using bonded post-tensioning systems, and take into account:

- The material and positioning of the ducts in order to control the friction coefficient inside the ducts,
- Strands threading, tensioning, and jack’s specifications in order to control possible deviation between tension of the strands,
- The injection of cement grouts to ensure ducts filling and strands protection in compliance with the principle of the bonded post-tensioning system on the one hand and the control of corrosion risks corresponding to the exposure conditions of the tendons on the other hand.

The Construction Specification should establish requirements for manipulation and storage of all material to ensure that it is protected from detrimental mechanical damage and corrosion. Limits of detrimental mechanical damage and corrosion should be specified in the Construction Specification.

Requirements for storage protection against corrosion should take into account different stage of material operation during manufacturing of tendons:

- Transport – short-time protection for outdoor conditions,
- Manufacturing – short-time protection in interior with possibility to do manipulation easily,
- Storage – long-time protection, material is waiting for application in structure.

The type of corrosion protection, the detailed procedure for its application, and the pertinent time limitations should be specified in the construction procedures. For all these stage, effectiveness of protection should be provided and validity time have to be assessed including periodical testing, if necessary.

A temporary corrosion prevention coating should be applied to all unbonded tendons during or after fabrication. The coating material should be compatible with the permanent corrosion preventing coating. The temporary coating should be specified in the Construction Specification. It should be made to be easily removable in the field with the use of nonchlorinated petroleum solvents for the installation of field attached anchorages.

In case of Greased Sheathed Strands (GSS), the strands should be protected from direct sunlight to which they may be exposed during transport and storage.

In case of bonded tendons using Cement Grouting Technology, a satisfactory quality of grouting work can only be achieved if grouting equipment of a suitable capacity adapted to the particular project is used. Such equipment should be confirmed prior to the actual grouting work during suitability testing to be able to produce a sufficiently homogeneous grout mix. The quality and reliability of the filling of the tendon duct and anchorages depends on the suitability of the chosen grouting procedure. Therefore, only grouting procedures should be used which have been proven through sufficient experience and / or representative testing.

7.3.2 Wire and strand

Wire or strand is delivered in packages. Each package shall be provided by identification indicating:

- Manufacturer identification,
- Basic geometric characteristics (wire diameter, strand composition),
- Heat no. or other Fabricator's ID which enables to link package with previous steps of manufacturing,
- Package number.

Each package shall be complement by wire material certificate (based on Material Supplier's tests) indicating:

- Chemical composition,
- Geometry of cross-section (check of diameter and marginal deviations, check of ovality),
- Modulus of elasticity,
- Nominal strength,
- Yield strength,
- Elongation,
- Bend test (no. of reverse bends at specified diameter),

- Relaxation (under different temperatures: 20°C, 40°C, 60°C recommended).

In case of Greased Sheathed Strands (GSS), the inner diameter of the coil must be greater than or equal to 60 times the nominal diameter of the strand. To improve the ducting phase the winding should allow easy unwinding on relevant device. Static leak tightness control and friction between sheaths and strands should be carried out on samplings taken in factory. Each unit of strands delivered on site should be subjected to a visual inspection of it surface condition (any visible damage, bubbles and traces of filling material, etc.). In addition to the controls specified by the regulation, the following tests shall be performed:

- External diameter and thickness of the sheathing,
- Mass of strand, sheath and grease per linear meter.

According to requirements of Authority in the country of construction site, Construction Specification can also specify the extent of on-site tests of above mentioned material parameters including acceptance criteria.

7.3.3 Anchor

Anchor parts Fabricator's Quality Assurance Program should specify all checks and tests of materials and final products with respect to requirements on NI suppliers. According to requirements of Authority in the country of construction site, Construction Specification can also specify additional on-site tests including acceptance criteria.

Checks and tests, executed by anchor parts Fabricator, should include:

- material tests (chemical composition, tensile strength, impact strength, hardness)
- check of geometry
- check of surface defects
- check of internal defects (i.e. ultrasound method)
- loading tests of individual parts and/or assemblies (i.e. application of loading in hydraulic press)

7.3.4 Grease

The permanent corrosion prevention coating applied to the tendons should guarantee long-term protection for design service life of containment. According ASME Boiler & Pressure Vessel Code Section III, Divisions 2, grease used for permanent corrosion prevention coating should be a microcrystalline wax (petrolatum) base material containing additives to enhance the corrosion inhibiting, wetting, and moisture displacing properties, as well as the ability to form a polar bond with the tendon steel.

In case of design according European standards, these soft products used should satisfy the specifications of ETAG 013. The products should have a European Technical Approval (ETA) and a corresponding certificate of conformity delivered by a notified body. Wax could be used. Other products may be used with a specific justification in order to show that these products are stable and are able to protect the constituents of the post tensioning system for the applicable working life.

Grease is delivered in packages. Each package should be provided by identification indicating:

- Manufacturer and product identification,
- Package number,
- Date of production.

Manufacturer should specify storage policy and methodology of periodical tests including their criteria for verification of protection function of grease. It should be verified that these products have a certificate of compliance with the specifications of the supplier and the results of the internal inspections performed.

In case of Greased Sheathed Strands (GSS), the origin of grease should be identified and test reports on specified characteristics should be validated by the project before manufacturing. On each unit of manufacture of grease following test should conducted as the measure of dropping point at beginning and end of manufacturing, the measure of oil separation at beginning and end of manufacturing, the analysis of the chemical contents. Before manufacturing, the origin of sheathing base material has to be identified and test reports on specified characteristics should be validated by the project. On each representative sample following tests are conducted as the tensile strength, the ultimate elongation at different temperatures.

7.3.5 Cement grout

Constituents of the grout should comply with the specifications presented in section 0 through 0 below.

7.3.5.1 Cement

Cements should comply with EN 197-1-A1-A3 and be certified for conformity by the marking “NF – Liants hydrauliques”.

In specific cases, when non-standard special cements are used, an inspection equivalent to the regulation of the marking “NF – Liants hydrauliques” should be carried out by a laboratory accredited by an independent third party such as COFRAC (or equivalent in other country than France).

Cements used in grout should be CEM I, comply with the requirements of EN 197-1-A1-A3. Cements should also comply with the following specifications:

- Cl- < 0.05%,
- S2- < 0.01%.

Cement should not contain any other element which leads to corrosion of the steel.

- Secondary constituents < 3%,
- Additives (Chrome VI reducing agent not included) < 0.1%.

Only the grinding agent additive is permissible.

Cements grouts should not contain any other element which leads to corrosion of steel.

7.3.5.2 Admixtures

Admixture products should be certified for conformity by an approved body in compliance with standards EN 934-1 and EN 934-4 or EN 934-2.

The conditions of use of admixtures should comply with those given by the manufacturer in the data sheet. When the use-by date on the manufacturer's certificate is exceeded, the products shall no longer be used.

Admixtures should be stored in an area protected from adverse weather and particularly frost, observing the conditions of storage required by the manufacturer's data sheet.

Admixtures should not contain any element which leads to corrosion of steel.

7.3.5.3 Mixing water

Mixing water should comply with EN 1008 and should also comply with the following specifications:

- Chlorides Cl- ≤ 250 mg.l-1,
- Sulphates SO42- ≤ 250 mg.l-1,
- Magnesium Mg2+ ≤ 125 mg.l-1.

7.3.5.4 Final product of cement grout

Cement grout should comply with the requirements of EN 445 and EN 447, with the following additional specifications:

- The fluidity range should comply with the post-tensioning specialist company's specifications which should be justified by the tests performed on a mock-up,
- For thixotropic grouts, the viscosity (shear rate) range should comply with the post-tensioning specialist company's specifications which should be justified by the tests performed on a mock-up,
- The working time should comply with the post-tensioning specialist company's specifications which should specify required fluidity ranges,
- The bleeding of grouts should comply with EN 447 after a period of 3h, 24h and until the end of the setting of the grout,
- The volume change and bleeding of the grout should comply with EN 447 at 3 h, at 24 h and until the end of setting of the grout.

Excess water in the grout has been confirmed as a major cause of grouting and durability problems. Hence, control of the water added to the grout is essential including the water eventually present in the duct system which should be sealed on site at all times to avoid ingress of rain or other water before grouting.

7.3.5.5 Qualification of permanent protection products

Before placing, cement grouts should be subject to the following:

- For grouts which do not have an ETA: qualification tests with the chosen cement should be carried out, so as to determine the optimum composition of the grout. The reliability of the nominal formula should be justified. This justification should include an analysis of the content variation of the constituents and an analysis of temperature change;

- Qualification tests for grouts which have an ETA supplied by a notified body are limited to specifications and criteria added to certification referential;

- Suitability tests (carried out on the nominal formula adopted during the design study for grouts) in order to ensure that the grouting equipment used on site will produce a grout in compliance with the required characteristics.

At the end of this step, modifications according to those defined in the justification of the robustness of the nominal formula may be necessary due to scale effects and efficiency of industrial equipment used.

Upon the delivery of the grout constituents, the following checks and controls should be conducted:

1. For each lorry or delivery container:
    - A check of the packaging, markings and delivery note,
    - A rapid identification test of the cement, to detect any errors in the delivery,
    - A Tusschenbroeck test, in accordance with P 18-363, or equivalent (the cement should not exhibit false setting),
    - A control of the following characteristics once per delivery:
        - Determination of the setting and soundness, in accordance with EN 196-3,
        - Determination of the density, in accordance with EN 196-6.

These checks and controls should be conducted on the site upon arrival of the cement and before its transfer to the silo.

Methods for taking and preparing samples of cement should be in accordance with EN 196-7 upon arrival of the cement on the site. Five kilograms of tested cement should be stored for conservation.

The results should meet the specifications of EN 197-1 for the cement grade considered.

2. A test of the 28-day compressive strength, twice for each tendon category:
    - These tests should be conducted in accordance with EN 196-1. The result should comply with the requirements of EN 197-1 for the cement grade considered.
    - For each batch of admixtures delivered on site, the following controls should be performed in addition to the checking of the delivery note:
        - A control of the relative density,
        - A control of the conventional dry extract.

The tests and results should comply with EN 934-1.

An analysis of the mixing water used should be performed every six months or before each production run of grouts.

7.4 Manufacturing of tendons

The construction procedures should specify the methods and procedures for cutting and cutting tolerances of prestressing elements.

A detailed fabrication procedure, including a checklist of work and information as required by the construction procedures, should be prepared before tendon fabrication. The checklist information for each tendon should include traceability data such as heat number or element coil number, anchorage component serial numbers, etc. It should also include length, location, numerical designations of the tendons, and the temporary corrosion protection of the tendon.

Attention should be given to specific aspects such as dimensions, geometry, connections, concentricity, alignment, angularity, and surface conditions. Limits and tolerances of these aspects should be specified in the construction procedures.

According type of prestressing system and placement of tendon in structure, construction procedures should specify requirements for twisting of tendon during manufacturing.

Coiling, when required for transportation, should be performed in a manner not to cause damage to the tendon. Coil diameter should be specified in the construction procedures.

Upon completion of fabrication into a whole or partial tendon, the tendon should be identified with a tendon number. The materials in the tendon should be recorded so that they can be traced to the tests that have determined their quality.

7.5 Installation

7.5.1 General requirements

A detailed installation procedure, including a checklist of work and information as required by the construction procedures, should be prepared before the tendon installation. The checklist should include lengths, locations, and numerical designations of the tendons; inspection and preparation of the tendon ducts; method and materials for the corrosion protection of the tendons; requirements for welding or burning where tendons are handled and installed; and sequencing of installation. Prestressing system components should not be installed, or shall be removed and replaced, if handling or storage causes their characteristics to change beyond the tolerances established in the Construction Specification.

The post-tensioning specialist company should be certified according to AC CWA 14646 by a notified body. Their personnel should be trained and qualified. These qualification requirements include the following procedures:

- Placing of embedded anchor parts,
- Placing of anchor blocks for dynamometric systems,
- Placing and connection of ducts,
- Tendon installation and tensioning,
- Injection of protection products.

All of these activities should be carried out by the post-tensioning specialist companies.

7.5.2 Tendon ducts and embedded anchorage components

The Construction Specification should specify the tolerances for position and alignment of tendon ducts and embedded anchorage components. Tendon ducts and channels should be adequately supported

against displacement during concreting. Open ducts should be protected by capping or plugging to prevent entry of concrete or other deleterious material. All joints should be made tight against the inleakage of mortar or appreciable water from the fresh concrete. The Construction Specification should specify the temporary corrosion protection system, if any, and the construction procedures should define the method for its application.

The quality level of tendon duct construction must be rigorous in order to meet designer requirements.

For concrete joints and construction joints, the duct length exiting the concreted section should be sufficient to enable the execution of the connection of the duct under good conditions.

The supports should be rigid and in sufficiently close proximity to avoid unintentional angular deviation (sagging) and damage to the ducts during concreting. Specific supports should be placed near the anchorages to reinforce the restraining of the ducts before or during concreting.

The restraining of the ducts should take into account concreting and vibration-induced loads. The bursting reinforcement and bearing plates should be correctly positioned.

The upper extremities of the vertical tubes as well as the extremities of the horizontal ducts should be sealed by a temporary stopper during the intermediate phases in order to avoid foreign bodies or concrete entering the duct.

The position, shape and diameter of vents, drains, injection and re-injection points should be defined according to the ETA and EN 446 by a procedure and methods which comply with design requirements and with the specifications on the execution drawings.

In case of bonded tendons, a post-tensioning tendon can be only reliably and completely filled if the entire tendon and duct system, including anchorages, hoses, etc. is leak tight. The leak tightness of the tendon system may be confirmed by air pressure testing.

7.5.3 Transport of tendons

Tendons and anchors should be protected against weather during transport from manufacturing hall to the site. Time of exposure should be minimized.

Packaging should provide mechanical protection and enable transfer by crane.

7.5.4 Duct checks

All water and debris should be removed from ducts prior to installation of tendons. Before installation of each tendon, check of duct continuity should be done. Used equipment checks not only continuity of duct but also sufficient diameter and radius of bends need for pulling the tendon through. Visual inspection of internal surface of duct by camera is recommended.

7.5.5 Inserting into the structure

Suitable measures should be provided concerning weather protection and protection against mechanical damage (both prestressing system and containment structure) during tendon installation and prestressing.

In case of Greased Sheathed Strands (GSS), the strands threading must assure none damage on the individual sheaths during the operation. Thanks to the protection against corrosion ensured by the

individual sheath and grease, the threading phase can be anticipated without specific arrangements. Usually, during this phase, the end of the strand being threaded is caulked to avoid damaging the other sheaths already in the duct. Indeed after ducts grouting and tendons tensioning, sheaths of strands must allow slippage of strand in the sheath. The ability of strands threading should be demonstrated on full scale mock-up representative of highly deviated horizontal duct and another representative of horizontal non deviated duct to check the ability to thread the maximum number of strands expected. The complete visual examination of the sheathing should show no visible damage and any visible filler grease. This mock-up can be also an opportunity to check the procedure of replacement of a strand. Different methods can be used to thread the tendons:

- Threaded each strand one by one,
- Precast the tendon and full threaded in one try,
- Threaded by pack of some strands.

The method depends on the ducting of the containment, the unit of the tendons and the experience feedback of the company in charge of the prestressing.

After installation of each tendon, detail visual inspection should be executed. Inspection is focused on completion of tendon and presence of mechanical damage. Suitable protocol from this inspection is base for approval for application of prestressing.

7.5.6 Grouting of GSS tendons

Preliminary tests should be performed on full scale mock-up representative of deviated duct threaded with the maximum of strands expected in order to qualify different parameters and tolerances of grouting methods as the cement grout characteristic. The ability to replace strands and the ability at the re-tensioning in the anchors environment during construction and service life structure has to be checked. Preliminary studies of the ducting can allow avoiding difficulties in relation to a specific geometry of the project.

The nature of cement grout can be different between bonded tendons and GSS. The criteria linked with the quality of the cement grout (filling, rate of acceptable space, etc.) can also be adapted. For bonded tendons, the aim is to assure the efficiency of the protection against corrosion whereas for the GSS, the main goal is to fix the geometry of the tendon before the tensioning (severe strains on the cement grout).

Detail information concerning grouting are presented in section 0 in this report.

7.6 Tensioning

The required sequence of prestressing shall be specified in the Construction Specification. Selected prestressing sequence shall minimize the risk of damage of containment structure due to irregular distribution of stress during prestressing process. Therefore, loading steps and corresponding groups of tendons should be set based on numerical analysis of whole process of application of prestressing.

Tensioning should be performed with the aid of a hydraulic jack, fully compatible with the anchorage block of the system, in accordance with the program defined by the designer, who specifies the order of tensioning and the calculated elongation. The prestressing system, including jacks, should be checked in order to prove it is suitable for works and to limit the differential loading between strands. The following aspects should be checked:

- Its implementation in all situations encountered during works,
- Initial simultaneous tensioning of each strand of a tendon, with individual displacements of each strand under a load between 10 and 15 kN for each strand at the anchorage,
- Tensioning of the tendons in a single stroke of the jacks (at least 500 mm) without recovery of tensioning,
- The suitability for works of the jacks to satisfy the specifications should be justified. This justification should be based on tests and include friction loss measurements in the jacks.

Pure vertical tendons should be tensioned at one end. Other tendons should be simultaneously tensioned at both ends.

For each tendon, measurements and various observations should be recorded on the tensioning record sheet, including any incidents that may have occurred.

The strand overlength shall not be cut before the acceptance of the tendon tensioning sheets and thorough check of the compliance with the requirements.

All prestressing operations should be under the direction of an experienced supervisor and should be carried out only by trained operators. Components of the tensioning equipment should be accurately set and supported in line with the axis of the tendon to which they are fitted, and they should be squarely seated on each other and on the anchorage before stressing is commenced. All anchorages, temporary connections, and jacks should be checked for alignment before transmitting any significant load to the tendon. Hydraulic pressure gages or dynamometers should be calibrated against standards traceable to the National Standards before their use in the prestressing operation. Pressure gages and jacks should be calibrated as a unit and shall always be used together.

In case of Greased Sheathed Strands (GSS), the tensioning of GSS tendons is performed after ducts grouting.

Tendon extension should be measured during the application of the jacking force. The measurement of extension should commence at a specified load of approximately 10% of the ultimate load of the tendon. The calculated elongation due to the initial force applied prior to commencing measurement should be added to the measured elongation. In addition, proper allowance should be made for any significant elongations within the stressing equipment if they are included in the measured elongation. The load applied to any tendon immediately prior to anchoring should not exceed the limits prescribed in the Construction Specification. According ASME Boiler & Pressure Vessel Code Section III, Divisions 2, the tensioning load shall be measured by load cells or equivalent means having an accuracy not less than ±2% of the required tensioning force.

During stressing, records should be made of elongations as well as forces obtained. Dynamometer or gage readings shall be checked against elongation of the tendons. According ASME Boiler & Pressure Vessel Code Section III, Divisions 2, the cause of any discrepancy exceeding ±10% for individual tendons and ±5% for the average of all tendons of one family (configuration; for example, hoop, dome, inverted U-shaped) of tendons of that predicted by calculations, using average load elongation curves, shall be resolved in consultation with the Designer. Final elongation and stress shall be recorded.

Deformation of containment during application of prestressing should be monitored. At least, change of diameter of cylinder, total vertical deformation of cylinder and vertical deformation in the middle of

dome shall be measured. Evaluation of deformation changes should be done after each step of application of prestressing. According ASME Boiler & Pressure Vessel Code Section III, Divisions 2, the cause of any discrepancy exceeding ±30% of that predicted by calculations should be resolved in consultation with the Designer. Final deformations of containment structure after finishing of prestressing should be measured and recorded.

The acceptable damage of tendon due to application of prestressing force or loss of prestressing force due to unreplaced, broken, or damaged prestressing elements shall be specified in the Construction Specification. Reach of required level of prestressing force in all tendons and fulfilment of criteria for acceptable damage of tendons are fundamental background data for final protocol from prestressing.

7.7 Injection of tendons by cement grout

7.7.1 General requirements

Grouting work on site is a complex activity. It needs to be well prepared. Once it has started it should not be interrupted. The assessment of the quality of grout during injection is still based. Most, if not all, grouting activities are on the critical path, in particular for grout mixes which show an early start of setting. For all the above reasons, grouting work needs to be planned, and supervised by experienced technicians, with a thorough understanding of the behaviour of grout, and awareness of the potential implications of poor grouting on the durability of a posttensioned structure.

Following specific methods of grouting with cement grout adapted to these methods of injection are required for all type of tendons:

- For horizontal and vertical tendons, by injection from one anchorage to the other in one phase, without the use of the different openings placed at the low points of the deviated ducts (these openings are required to drain away either water before threading or oil pulverization before injection when the delays between threading, tensioning and injection are not respected),

- For gamma tendons, by injection in two phases (vertical part then deviated part).

Cement grouts should be delayed and/or thixotropic.

Preliminary tests on several mock-ups should be performed to enable quantification of the different parameters and tolerances of grouting methods and the cement grout characteristics.

Specifications of EN 446 should be applied where they are not inconsistent with the requirements of ETC-C.

Grouting procedures shall be documented and approved by the Project. They shall include at least:

- The description of the equipment used and personnel resources,

- The composition, characteristics and manufacturing process of the grout, the composition and characteristics of the wax,

- The injection method,

- The order of injection of the ducts,

- The injection parameters (rate and pressure),
- The special orders and instructions to follow in case of an incident or when unfavourable climatic conditions occur.

The ducts should allow free passage of the grouting products and should be free from foreign bodies which may cause obstruction.

7.7.2 Execution

Three mock-ups for each tendon type should be tested (lightly and heavily diverted horizontal tendons, pure verticals, "gamma" tendons) see section 0.

The injection pumps should be equipped with a pressure gauge with an accuracy of □0.1 MPa and a pre-adjusted pressure switch. A pressure gauge is placed at the entry of the tendon.

The flow capacity of the grout pumping station is between 15 and 20 metres per minute. After setting of the grout, the mock-ups should be cut in order to check the filling level.

The nature and frequency of the inspections to be undertaken has to be defined.

Acceptance criteria are listed below:

- For all tendons and anchorage caps: wedges, anchor heads and strands should not be visible in any voids.
- In addition, for tendons:
  - Ducts shall not contain free water or display potentially damaging cracks when opened,
  - For pure vertical tendons and the vertical part of gamma tendons: volume voids should be limited. Criteria should be submitted to the Project approval,
  - For pure horizontal tendons, deviated horizontal tendons and dome sections of gamma tendons: defects should not have dimensions (depth, width, length) that expose strands,
  - For the descending part of deviated tendons: the volume of voids in the descending part of the duct should be limited.

7.7.3 Controls

7.7.3.1 On the tendons before injection

The leak tightness of the ducts should be checked as follows:

- For ducts of horizontal tendons: once the orifices in the duct are all sealed, except for the injection vent, oil-free compressed air at a pressure between 0.5 and 0.7 MPa shall be blown into the duct through this vent. It shall be checked that the pressure decrease does not exceed the values established during the preliminary tests, conducted in accordance with the procedure (losses shall be less than 0.1 MPa in 3 minutes).
- For the ducts of injected tendons under partial vacuum: once the orifices in the tendon are all sealed, except for the exit vent, the duct should be placed under vacuum through this vent. It

should be checked that the pressure increase does not exceed the values established during the preliminary tests, conducted in accordance with the procedure.

For the two types of verification, in case of non-compliance or failure indicating a local leak and the possibility of connection with other ducts, effects on neighbouring ducts should be investigated.

7.7.3.2 On the grout

Controls should be conducted in accordance with specifications and the measurement results should observe the criteria given in the specifications.

7.7.3.3 During fabrication

Fluidity:

- A measurement after mixing of each batch,
- A measurement after re-mixing of grout in each tank.

Viscosity (for thixotropic grout): a measurement after addition of a thixotropic agent in the mixing tank.

Bleeding and temperature:

- A measurement at each start of the batching plant,
- A measurement every ten batches.

In addition, measurements of the temperature of the water, cement and ambient air should be performed at each start of the grout station and every ten batches.

The results should comply with the allowable range determined during the suitability test.

Any mix that does not satisfy one of the criteria should be rejected.

The mechanical strengths Rc and Rt, as well as the capillary absorption and shrinkage, should be measured twice for each type of tendon.

7.7.3.4 During injection

Fluidity or viscosity and temperature:

- A measurement at the start of pumping, for each grout transport tank,
- A measurement before the restart of injection, in case of a pause of longer than one hour during the grout pumping operation,
- A measurement at the entry and exit of each tendon,
- A measurement at specific exit vents for gamma tendons.

The measurement results should comply with the allowable range determined during the suitability test.

Pressure: the injection pressure should be constantly monitored so as not to exceed the pressure limit at the entry, which for each type of tendon should be as follows:

- 2.0 MPa for vertical tendons or vertical parts of gamma tendons,
- 1.0 MPa for other tendons.

For each duct, the progression of the quantities of injected grouts and the total quantity injected as well as the duration of the operation should be recorded. An injection record sheet should be established as and when the operations take place. The sheet should record the results of the measurements taken, diverse observations and incidents encountered.

Injection should be stopped if any of the criteria are not observed.

7.8 Injection of tendons by grease

The Construction Specification should specify the permanent corrosion protection system, if any, and the construction procedures should define the method for its application.

In case of bonded tendons, this injection process should be applied to the tendons instrumented with dynamometers (usually vertical tendons).

7.8.1 Execution

The leak tightness of the ducts should be checked before injection, in accordance with the process used for bonded tendons. The injection products should be melted as follows:

- Either on the site, following an established procedure,
- Or in the workshop, and then transported to site in an equipped tanker: the tank should be insulated with a system for keeping at temperature and/or heating.

The product temperature before injection should be sufficiently high to avoid the formation of a stopper, but should not exceed 100°C.

The injection should be performed using a pump with a flow capacity of at least 10 meters/minute.

7.8.2 Controls

The injection should be subject to the following controls:

- Measurement of temperatures at the entry and exit,
- Measurement of pressure during injection, and control with respect to the value prescribed,
- Control of the quantity injected with respect to the volume of the duct.

7.9 Installation of tendon instrumentation

There should be installed an instrumentation enables the tension monitoring of the instrumented tendon. There are different kinds monitoring of force in the tendons:

- Dynamometric systems - these systems should be interposed between the anchor block (or anchor head) and its support plate,
- Magneto-elastic systems - these systems are installed on the duct and measure changes of the stress in tendon steel,
- Tensiometer systems - these systems are installed on the anchor body and measure changes of anchor strain.

According kind of instrumentation, its installation should be done in appropriate period of construction (before casting of concrete, before or after installation of tendons).

Requirements for measuring systems qualification and verification are the same as in case of containment instrumentation – see section 0 in this report.

7.10 Anchor protection

All elements of prestressing system exposed to weather condition should be protected by removable cover.

Tendon ends should be equipped with permanent or temporary caps. Protection of the anchorage should be performed:

- By injection of anchorage caps with a cement grout, at the same time as the ducts (in this case, the external faces of the anchorage caps should be painted),
- By injection of anchorage caps with a grease,
- By covering of the anchor block with concrete, connected to the supporting concrete by joint treatment involving scrabbling, and if necessary involving gluing with the product specified in the operating procedure.

The dimensions of the caps should take into account the strand overlength if necessary. Furthermore, on the dynamometer side, a specific watertight cap should protect the tendon head/dynamometer assembly from externally generated hazards.

The injection hoses and the anchorage cap vent should have an internal diameter of at least 38 mm.

In case of Greased Sheathed Strands (GSS), the tendon ends should be equipped with permanent or temporary specific caps: GSS restressable, measurable, detensionable tendons during the operating life of the structure must keep sufficient excess length to allow performing these operations at a later date. This excess length, depending on the planned operations for the tendons (measurement and retension, detension and retension) requires establishing long caps (plastic or cast iron between 850 mm and 2.00 m), filled with petroleum wax or grease as temporary and permanent protection. Grease rather than wax is recommended to fill the long caps to avoid heating the anchorage and strands at high temperature. These caps are flame retardant due to the presence of flammable material (HDPE and grease).

7.11 Summary report

All of the tensioning and injection operations should be described in an analysis and summary report, which should include the results from the tensioning and injection record sheets.

As a part of this report, there should be an evaluation of the real tensioning process in the view of design requirements and presumptions. The conclusion of this evaluation is a confirmation of fulfilment of the design.

7.12 Full-scale mock-up for qualification – an example for EPR

Injection techniques should be qualified by the undertaking of tests based on full-scale mock-ups of the prestressing system. Full-scale mock-ups should be created for the entire prestressing system, see **Figure 0.4** below showing a full-scale mock-up.

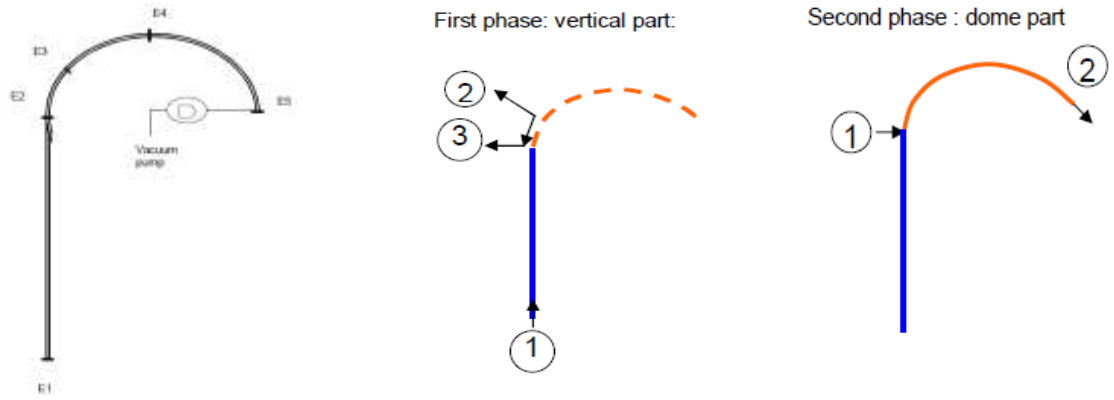
Figure 0.4 Example of a full scale mock-up

The injection technique should be considered to be qualified if three successively samples compliant with the conditions of this section (successful samples) are obtained from each type of tendon described below:

- Non or slightly deviated horizontal tendon,
- Highly deviated horizontal tendon,
- Pure vertical tendon,
- Vertical tendon returned on the dome (also referred to as a gamma tendon).

The representative mock-ups of ducts injected in two distinct phases may be carried out as partial full-scale mock-ups. In this case the partial mock-ups should be representative of the whole duct injected in a single phase.

Figure 0.5 Principles of grouting methods for gamma tendon



The interface between two injection phases should be included in the partial mock-up which is representative of the second injection phase.

Where the injection grouts and the injection techniques are identical for several groups of tendons, the mock up should be carried out on the worst case group of tendons.

Filling of the ducts should conform to the following criteria:

- For all tendons and injection caps:
  - Wedges, anchor blocks and strands should not be visible in any voids. In addition, for no free water and no potentially damaging cracks should be present during the opening of the ducts.
- Pure vertical tendon:
  - No voids should be present.
- Gamma tendon
  - Vertical part: the conformity criteria should be identical to those for pure vertical cables.
  - Dome part: The depth of a defect should not be greater than 15 mm, the dimensions of a defect (apart from the depth) should not be more than a few centimetres.

The volume of voids in each deviated part should be less than 0.3% of the theoretical volume of the deviations concerned. The volume of voids should include bubbles with a diameter greater than 10 mm.

- Non-deviated horizontal tendon
  - Conformity criteria are identical to those for the dome part of gamma tendons, except for the depth of voids, which should not be greater than to 2 mm.
- Deviated horizontal tendon

- Parts with a downward radius of curvature: no voids should be present,
- Other parts: the conformity criteria should be identical to those for the dome part of gamma tendons.

After the grout has hardened the mock-ups should be cut into sections in order to check the level of filling.

7.12.1 Inclined tube test

7.12.1.1 Objective

This test serves to determine the bleed properties and stability of a grout, at full scale and includes the filtering effect of strands. It also allows confirmation of the proposed grouting procedures, in particular the effect of time between ending an initial grouting and starting of re-grouting on site, if specified, and equipment used on site.

The intent of the test is to confirm that a duct on site can be completely filled with the proposed grout, equipment and procedure, without unacceptable bleed and segregation of the grout.

7.12.1.2 Test method

In a first test phase, the bleed water and air accumulated on top of a tube filled with grout should be determined. The grout is injected under pressure and is setting such that water losses due to evaporation are prevented.

In a second phase, the effect of re-grouting of a tube on bleed water and air accumulated should be determined, if such a procedure is envisaged by the PT specialist contractor in the grouting method statement.

7.12.1.3 Test equipment and set-up

- Two transparent PVC tubes, of approximately 80 mm diameter and 5 m long, equipped with caps at each end including grout inlet at the lower end, and grout vent at the top. The tubes should be able to sustain a grout pressure of at least 1 MPa.
- 12 prestressing strands Ø0.6” per tube, i.e. a total of 24.
- Grouting equipment as per the grouting method statement.
- A thermometer with automatic recording.

7.12.1.4 Test procedure

- The two tubes are fixed on their supports such as to avoid noticeable deflections, at an inclination of 30° ± 2° against a horizontal reference line. 12 strands should be installed in each tube. The caps are subsequently installed on the tube ends (fixed with glue).
- The grout is prepared as per the grouting method statement. Specimens should be taken from the grout mix to confirm flow time according to the specification. In case of a thixotropic grout, other suitable methods should be used.

7.12.1.4.1 Grouting of first tube

Grout is injected into the first tube (Tube 1) from the bottom end. When the grout exits from the vent at the top with the same consistency as it enters at the bottom, the valve should be closed, and the grout pressure should be maintained for the duration specified in the method statement.

Subsequently, the valve at the bottom is closed, and grouting of Tube 1 is considered complete.

The level of air, water, and any other eventual liquid on top of the grout should be measured. Such eventual liquid on top of the grout can be distinguished from the grout by its whitish to yellowish colour, usually clearer than the grout. A minimum of 4 measurements of levels should be taken between 0 and 24 hours after completion of grouting, with one measurement just before re-grouting of Tube 2 is started. The following 4 measuring intervals are suggested: 30 minutes, 1 hour, 2 hours, and 24 hours after grouting.

7.12.1.4.2 Grouting of second tube

Grouting of Tube 2 shall follow the same procedure as used for Tube 1, and shall be done quasi simultaneously with Tube 1. At a time specified in the method statement for re-grouting, the mixing of grout in the equipment is started again, and the flow time of the grout is determined again.

Subsequently, the valves of inlet and vent of Tube 2 are opened again, and grouting is started again. This will allow any liquid accumulated on top to be replaced by grout. When grout exits from the vent on top, the valve is closed, and the grout pressure is maintained for the duration specified in the method statement. Subsequently, the valve at the bottom is closed, and regrouting of Tube 2 is considered complete.

The time between initial grouting and re-grouting, and the duration for the second mixing activity, should comply with the grouting method statement. Typically, this time will be between 30 minutes and 2 hours.

Similar to Tube 1, the measurement of levels are done between 0 and 24 hours after completion of the initial grouting. One of the measurements should be taken just prior of re-grouting of Tube 2, followed by measurements 30 minutes, 1 hour, and 2 hours after completion of re-grouting.

7.12.1.5 Measurements and observations

The following measurements and observations should be made and recorded:

- Description of test set-up,
- Grout mix design, origin and certificates of all grout constituents,
- Mixing procedure of grout,
- Flow time of grout mix before initial grouting, and before re-grouting (or viscosity of a thixotropic grout),
- Method statement for grouting specified by the PT specialist contractor,
- Measurements of level of air, water, and eventual liquid on top of the grout,
- Any observations and comments on the formation of bleed or liquid, or on difficulties encountered during the test,

- Any observations and comments on cracking of the grout, with location, orientation, and approximate widths of cracks,
- Development of air temperature during the entire test period,
- Photos illustrating test setup, and details of top end of tube with air, water and eventual liquid.

7.12.2 Wick-induced bleed test

7.12.2.1 Objective

This test serves to determine the bleed properties of a special grout. It is considered to be more representative than the bleed test as per EN 445.

7.12.2.2 Test method

Bleed is expressed as the percentage of the bleed water depth on top of the grout column divided by the original grout column height, up to 3 hours and after 24 hours.

7.12.2.3 Test equipment

- One transparent PVC tube, of approximately 60 to 80 mm internal diameter, and 1 m long, equipped with caps at each end, as used in the sedimentation test.
- One 7-wire strand Ø0.6” of one meter length such as to fit inside the tube.
- Grouting equipment as per the grouting method statement.
- A thermometer with automatic recording.

7.12.2.4 Test procedure

The grout mix specified by the PT specialist contractor is prepared in the grout mixer intended to be used on site. The transparent tube is placed and held vertically on a surface free from shocks or vibrations.

The strand is placed standing inside the tube and held concentrically.

The tube is filled with grout to about 10mm below the top and sealed to prevent evaporation. Up to 3 hours and after 24 hours the bleed water depth on top of the grout column is measured.

7.12.2.5 Measurements and observations

The following measurements and observations should be made and recorded:

- Description of test set-up,
- Grout mix design, origin and certificates of all grout constituents,
- Mixing procedure of grout,
- Flow time of grout mix before filling of tube (or viscosity of a thixotropic grout),
- Record temperature of grout constituents before testing, and air temperature during test period,
- Record type and size of strand installed in column,

- Record the original grout column height,
- Record bleed water depth at the top of the grout column up to 3 hours and after 24 hours,
- Determine the bleed ratio of the grout column as the depth of bleed water divided by the original height of the grout column,
- Photographic documentation, and comments (not required for testing on site).

7.12.3 Sedimentation test

7.12.3.1 Objective

This test serves to determine the sedimentation properties of a grout. It is considered as a measurement of the homogeneity of the grout mixed in the equipment intended to be used on site.

7.12.3.2 Test method

Sedimentation is measured as a percentage difference in density of the grout between the samples taken from the top and bottom of the test specimen.

7.12.3.3 Test equipment

- Two transparent PVC tubes, of approximately 60 to 80 mm internal diameter, and 1 m long, equipped with caps at each end.
- Grouting equipment as per the grouting method statement.
- A thermometer with automatic recording.

7.12.3.4 Test procedure

The grout mix specified by the PT specialist contractor is prepared in the grout mixer intended to be used on site. The two transparent tubes are placed and held vertically on a surface free from shocks or vibrations.

The two tubes are filled with grout to the top and sealed to prevent evaporation.

At least 24 hours after filling, but after setting of the grout, the grout columns should be removed gently from the tubes. The grout columns should be marked and subsequently cut into equal slices of about 50mm each over the entire height. The relative position of each slice in the column should be recorded. The density of each slice should be measured by an approved method.

7.12.3.5 Measurements and observations

The following measurements and observations should be made and recorded:

- Description of test set-up
- Grout mix design, origin and certificates of all grout constituents
- Mixing procedure of grout
- Flow time of grout mix before filling of column

- Record temperature of grout constituents before testing, and air temperature during test period
- Record the density of each slice of both grout columns
- Determine the sedimentation ratio, R, of each of the grout columns as the variation of grout density between the bottom, D Bot , to the top, D Top , of the column as follows:
- R = 1 – (D Top / D Bot)
- Report any particular observation such as eventual bleed water on top of the grout column at the time of removing the grout column (presence of water and quantity), or discoloration of grout columns.
- Photographic documentation, and comments

7.12.4 Full Scale Mock-up with large deviation

7.12.4.1 Principles

In nuclear industry, due to the particular shape of ducts (dome of reactor building, deviation of horizontal and vertical cables around the equipment hatch and some other), injection techniques should be qualified on entire or partial full-scale mock-up, three models for each tendon type should be tested (lightly and heavily diverted horizontal tendons, pure verticals, domes and verticals turned over on the dome, also designated as “gamma” tendons).

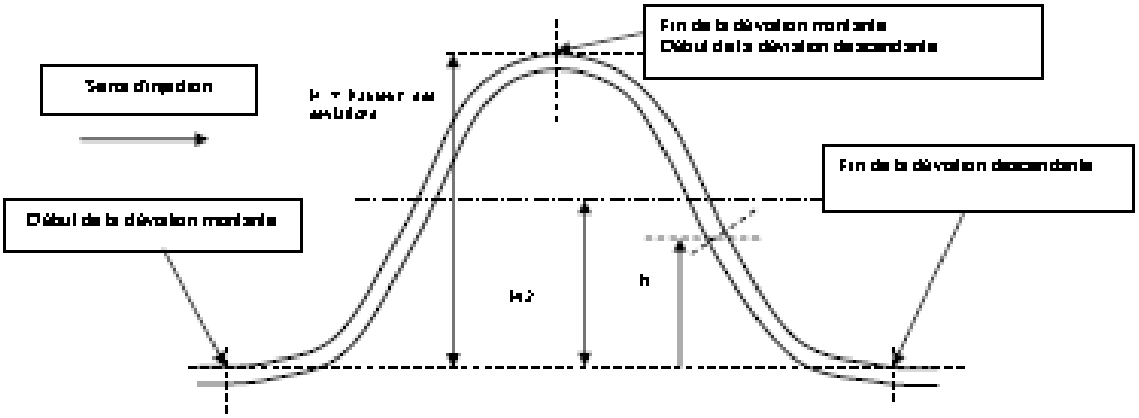
As the bonded technology is a “one shot” technology, this kind of full-scale mock-up is considered as necessary.

7.12.4.2 Experience realized at Flamanville and Quinshan for European project EPR

Figure 0.6 and **Figure 0.7** show a mock-up tested for the EPR reactors.

Figure 0.6 Mock-up tested for the last reactors EPR in Finland, China and France for deviated tendons

Figure 0.7 Schematic sketch of the mock-up for deviated tendons



The injection pumps should be equipped with a pressure gauge with an accuracy of □0.1 MPa and a pre-adjusted pressure switch. A pressure gauge is placed at the entry of the tendon.

The flow capacity of the grout pumping station is between 15 and 20 metres per minute. After setting of the grout, the mock-ups should be cut, as specified in Table 0.1, in order to check the filling level following the requirements of **Error! Reference source not found.** below.

Table 0.1 Control of mock-up.

Type of control	Non-deviated or vertical parts of the tendon	Slightly or highly-deviated part of the tendon
Cross section of ducts	Every 5 m *	Ascending and descending part of the deviation: beginning, middle and top *
Longitudinal window on upper part 17x40 cm	Every 10 m *	Descending part of the deviation
Anchorage caps	If, upon removal of anchorage caps, defects are found which do not satisfy the criteria, a bearing device shall be cut transversely to characterize the defects	

* If a defect is detected, additional windows shall be opened to characterise the defects.

Acceptance criteria are listed in **Error! Reference source not found.** (acceptance criteria used for the lkiluoto and Tinshan NPP constructions). There are some elements difficult to obtain without any safety reason and, to this fact, not really relevant between them and EDF tried to clarify.

Table 7.2 Filling requirements.

Parts of ducts	Type of defect	Acceptance Criteria following references		
		ETC-C [4]	EDTGC100457	CCTG 2010
Horizontal	Bubbles	-	-	-
	Voids	No visible strand at opening at opening No harmful voids	No void with radial size > 15 mm	No visible strand at opening Depth of voids < 2 mm, size cms, Cumulated volume<0.3% (included bubbles with d > 10 mm)
	Cracks	No harmful crack	No longitudinal crack	No harmful crack
	Bleed Water	None	-	None
Vertical	Bubbles	-	-	-
	Voids	No visible strand at opening No voids	No voids	No visible strand at opening No voids
	Cracks	No harmful crack		No harmful crack
	Bleed Water	None	-	None
Rising deviating part downward	Bubbles	-	-	Identification of bubbles d > 10mm
	Voids	No visible strand at opening No harmful voids	In upper part, no void radial size > 15 mm In lower part, no void Cumulated volume<0.3%	No visible strand at opening Depth of voids < 15 mm, size cms, Cumulated volume<0.3%
	Cracks	No harmful crack	No longitudinal crack	No harmful crack
	Bleed Water	None	-	None
Downward deviating part	Bubbles	-	-	Identification of bubbles d > 10mm
	Voids	No visible strand at opening No harmful voids Voids volume limited	In upper part, no void radial size > 15 mm In lower part, no void Cumulated volume<0.3%	No visible strand at opening Depth of voids < 15 mm, size of some cms, Cumulated volume<0.3%
	Cracks	No harmful crack	No longitudinal crack	No harmful crack
	Bleed Water	None	-	None
Cap	Bubbles	-	-	-
	Voids	No strand at opening head of anchorage or key visible	No strand at opening head of anchorage or key visible	No strand at opening head of anchorage or key visible
	Cracks	-	-	-

EDF has proposed as a consensus between the different projects to state the following acceptance criteria synthetized in Table 0.2 below where:

- *P* is the depth of the void considered approximately as spherical,
- v_i is the volume of an elementary void (windows) / Va which is the volume of grout delimited by a window,
- *Si* is the surface of an elementary void (section) / Sa which is the internal section of the duct.

The minimal frequencies of controls after hardening of the grout are shown in Table 0.3 below.

Table 0.2 Acceptance criteria defined by EDF.

Parts of ducts	Example of duct or part of duct	Visible strands in voids	Keys or anchorage blocks visible in voids	Voids		Bleed water
				Windows	Section	
Horizontal parts	Pure horizontal ducts	None	-	p< 7mm and Σv_i < 1%V_a	p < 7mm and Σs_i < 2% S_a	None
Vertical deviation	Gamma ducts on dome Deviated part of horizontal ducts	None	-	For h> H/2: p < 16 mm and Σv_i < 1% V_a	For h> H/2: p < 16 mm and Vs_i < 5% S_a	None
				For h < H/2: p < 18h/H+7 mm and Σv_i < 1% V_a	For h < H/2 Σs_i < 5% S_a	
				For h =0: p < 7 mm and Σv_i < 1% V_a	For h =0: p < 7 mm and Σs_i < 2% S_a	
Vertical parts	Vertical ducts and vertical part of gamma ducts	None	-			None
Parts near the anchorages	Anchorage area of horizontal ducts and gamma	None	-	None		None
Anchorage caps	All ducts	None	None	-	-	None

Table 0.3 The minimal frequencies of controls after hardening of the grout are the following.

Part of ducts	Frequency of windows	Frequency of sections
Horizontal parts	Every 10m whom one in each part close to the anchorages	Every 5m
Vertical deviation	One in lower part, one in the middle and one at the top	One in lower part, one in the middle and one at the top
Vertical rising deviation in the injection sens	On the entire part	5m
Vertical parts down in the injection sens	Every 10m	5m
Parts near the anchorages	One in each part close to the anchorages	One in each part close to the anchorages
Anchorage caps	Each cap is inspected	Each cap is inspected

8. IN-SERVICE INSPECTION FOR CONTAINMENTS

8.1 Unbonded tendons

8.1.1 Introduction

Prestressed concrete containment structure is commonly used for nuclear power plants all over the world. The principal functions of containment structures are to (1) protect the reactor coolant system from potentially disastrous events such as, tornado, earthquake, wind-generated missiles, aircraft impact etc. and (2) prevent the release of radio-nuclides in the event of a loss-of-coolant accident. Because of such a critical safety-related function of the containment structure, an effective in-service inspection (ISI) and monitoring program is required by regulation.

Within the United States almost all the prestressed concrete containments make use of post-tensioned unbonded (greased) tendons with the exception of one operating plant, H. B. Robinson where the vertical tendons are bonded.

Within the Czech Republic all the prestressed concrete containments make use of post-tensioned unbonded (greased) tendons. Every tendon is formed by 450 wires featuring a diameter of 5 mm (wire goes over anchors forming an independent loop). The initial nominal prestressing force according to the design is 10 MN. Tendons corrosion protection is made with grease during production, preservation of anchors was made after prestressing. There is no filling of ducts by grease after prestressing.

8.1.2 Overview

The use of unbonded tendons provides the opportunity for direct in-service inspection of the tendons. Furthermore unbonded tendons can be replaced if found defective or degraded.

In the USA, containments with unbonded tendons are inspected in accordance with ASME section XI “Rules for In-service Inspection of Nuclear Power Plant Components”, subsection IWL“ Requirements for Class CC Concrete Components of Light-Water Cooled Plants” [6] and NRC Regulatory Guide 1.35, “In-service Inspection of unbonded Tendons in Prestressed Concrete Containments.” [37].

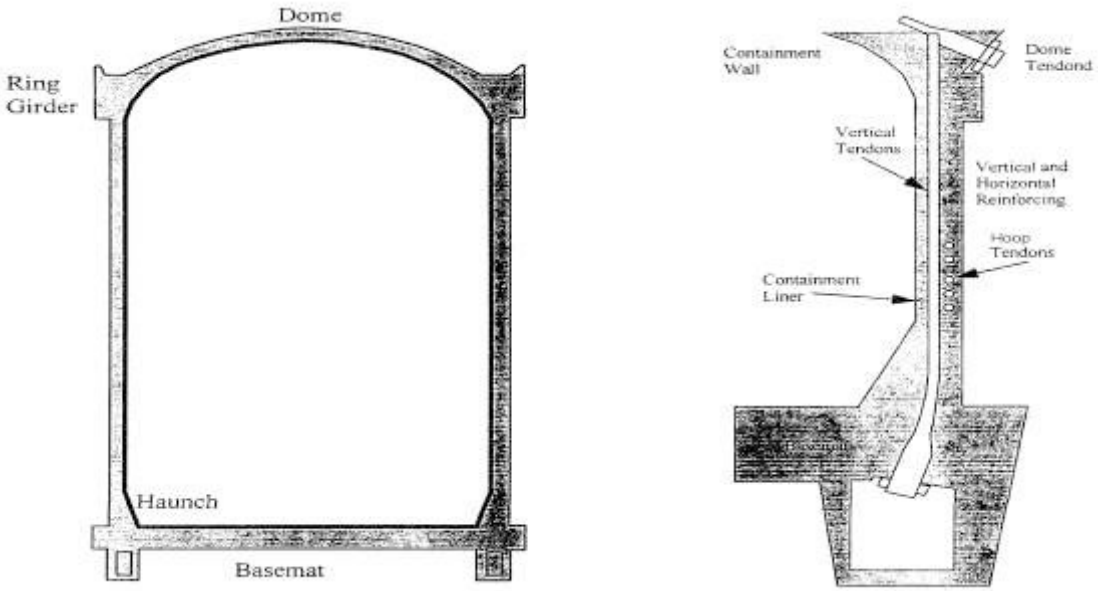
Within the CZ, the program of inspections of prestressing system is based on original Russian ISI program for this type of prestressing system. Application of original ISI program for CZ containments takes into account changes in the area of prestressing forces measurement added onto all anchors (frequency and extend of lift-off tests is decreased). The principles of NRC Regulatory Guide 1.35 [37] have been also incorporated into ISI program. ISI program exists in the form of NPP Operator’s ISI program. ISI program is represented by set of operator’s guides and it is approved by national authority.

During ISI of posttensioned concrete containments, all concrete surface areas (accessible and any area suspect of degradation), tendon, wire or strand, anchorage hardware and surrounding concrete, corrosion protection medium (grease), and free water are inspected. The approach for performing ISI of a posttensioning system consists of a random sample of tendons selected from the accessible tendon population. Anchorage components for each tendon in the sample are visually inspected for defects or abnormalities. The prestressing force for each sample tendon is measured (with corresponding elongation measurements), a grease sample is obtained for laboratory analysis of impurities, and a wire sample is removed for material property determinations. The prestressing force, elongation, grease impurities, and material properties are determined and compared with allowable limits. In the case of the prestressing forces, these limits vary with time and must be calculated for the time of the surveillance.

8.1.3 Visual examination of concrete

Concrete surface areas are visually examined for evidence of conditions that indicate an existing or potential damage and degradation. The degradation may be caused by chemical and physico-chemical processes in the concrete and deterioration due to environmental conditions including, but not limited to, aggressive chemical attack, erosion and cavitation, corrosion of embedded steel, freeze-thaw, settlement, leaching of calcium hydroxide, reaction with aggregates, staining, grease leakage, increase in permeability or porosity, and combined effects. ACI 201.1R, "Guide for Conducting a Visual Inspection of Concrete in Service" [1] and ACI 349.3R, "Evaluation of Existing Nuclear Safety-Related Concrete Structures" [2] provide guidance on visual examination. Cracking, spalling, excessive strain, loss of strength and stiffness are some of the major factors that are looked for to estimate overall integrity of the concrete.

Figure 0.1 Vertical section through containment showing the different structural parts included



8.1.4 Examination of unbonded post-tensioning systems

8.1.4.1 Tendon selection – US practice

During an inspection tendons to be examined are selected on a random basis except if it is inaccessible or exempt from inspection. The population from which the random sample is drawn consists of:

a) All tendons of a particular type (as defined in Table IWL-2521-1) not examined during earlier inspections. The number of tendons to be examined during an inspection is also specified in ASME section XI, Table IWL-2521-1 and Table IWL-2521-2.
b) One tendon of each type (as defined in Table IWL-2521-1) is selected from the first year inspection sample and designated as a common tendon. Each common tendon is examined during each inspection. A common tendon is not detensioned unless required for evaluation for another reason. If a common tendon is detensioned, another common tendon of the same type is selected from the first year inspection sample.

c) If a containment with a stranded post-tensioning system is constructed with a predesignated number of detensionable tendons, one tendon of each type shall be selected from among those that are detensionable. The remaining tendons are selected from among those that cannot be detensioned.
d) The population of tendons from which a random sample is drawn for examination in accordance with Table IWL-2521-1 need not include tendons subject to augmented examination in accordance with Table IWL- 2521-2.

8.1.4.2 Remarks on sampling principle (NUREG/CR-2719 [34])

Sampling requirements proposed by both the NRC and ASME with respect to the number of the tendons to be investigated during the surveillances appear to be inadequate when evaluated on a purely statistical basis. However, this is somewhat misleading because the statistical study did not factor into the analysis either the evaluation of prestressed concrete technology, quality control, and quality assurance programs, or the history of proven performance of prestressing tendons in civil engineering structures over the past several decades. For this reason an overview of the performance of prestressing in structural application was conducted. The results of the overview revealed that the probability of prestressing tendon corrosion was extremely low, i.e., on the order of 0.0007% which indicated that the proposed criteria of monitoring 4% of the population of each tendon family (minimum of 4 from each group) for the first 3 surveillance, followed by a drop to 2% in future surveillances are reasonable, if no abnormal degradation is uncovered.

The first few inspections are conducted primarily to identify construction defects, the 4% criteria seems reasonable for the first 5 years after the Initial Structural Integrity Test.

8.1.4.3 Tendon selection – Czech practice

Tendons are divided into two groups - tendons of dome part and tendons of cylindrical part of containment. Each group has specific limit criteria according to different characteristic length of tendons and their trajectory.

For each tendon, history of inspections and force measurement is maintained. Inspections are documented in form of inspection reports (stored in operators inspection report database) and in form of individual detail reports from prestressing or lift-off tests. Force measurement is documented by individual reports for each performing of measurement and full history of measured forces is maintained in database of measurement (is used as an input for periodical containment evaluation).

In general, tendons to be examined are selected on a random basis but the history of each tendon is taken into account and the most critical tendons are selected usually (e.g. tendons with considerable change of prestressing force, tendons with higher no. of broken wires or tendons with risk of corrosion due to repeated presence of water in their duct). Also a number of inspected tendons or period of inspections could by modified according the results of previous inspections. In each inspection, approx. 30% of anchors are selected for visual inspection and approximately 10% of anchors are selected for lift-off test.

8.1.4.4 Tendon force measurements

Within the USA, the prestressing forces in all inspection sample tendons are measured by lift-off or an equivalent test. Equipment used to measure tendon force are calibrated in accordance with a calibration procedure prior to the first tendon force measurement and following the final tendon force measurement of the inspection period. Accuracy of the calibration is maintained within 1.5% of the specified minimum

ultimate strength of the tendon. If the post-test calibration differs from the pretest calibration by more than the specified accuracy tolerance, the results of the examination are evaluated.

Within the CZ, the measurement of prestressing forces is installed on all anchors of tendons. Measurement and its basic evaluation are executed every month. This measurement is used for demonstration of sufficient level of prestressing of containment structure. Lift-off tests are used only as an independent way of verification of measurement system. Accuracy of the calibration of the equipment used to measure tendon force is maintained within 1% of the nominal prestressing force of the tendon.

There is also measurement of prestressing forces in several points along tendon length installed on two tendons of dome part and two tendons of cylindrical part of containment. This measurement was used mainly during application of prestressing for a verification of friction of tendons in ducts.

8.1.4.5 Tendon wire and strand sample examination and testing

Within the USA, one sample tendon of each type is completely detensioned. A single wire or strand is removed from each detensioned tendon. Each removed wire or strand is examined over its entire length for corrosion and mechanical damage. Strand wires are also examined for wedge slippage marks. Tension test is performed on each removed wire or strand. Wire samples to be taken are one at each end, one at mid-length, and one in the location of the most corroded area, if any. Values for yield strength, ultimate tensile strength, and elongation are obtained from each test result.

Within the CZ, tendon structure (independent loop of wire) unable removing of single wires. Therefore removing of whole tendon is applied. Removed tendon is examined over its entire length for corrosion and mechanical damage. Verification of mechanical parameters of wires is executed on wire samples - rules for samples selection is similar as within the USA. Special attention is paid to anchor areas – these areas are critical for occurrence of corrosion and damage of wires.

8.1.4.6 Retensioning and elongation – US practice

Tendons that have been detensioned are retensioned to at least the force predicted for the tendon at the time of the test. However, the tendon force after retensioning should not exceed 70% of the specified minimum ultimate tensile strength of the tendon based on the number of effective wires or strands in the tendon at the time of retensioning. During retensioning, the tendon stresses are maintained within the limits of the construction code and the Nuclear Power Plant owner's requirements. During retensioning of a tendon, the tendon elongation is measured and recorded.

8.1.4.7 Tendon force and elongation acceptance by examination

Tendon forces and elongation are acceptable if the following conditions are met:

a) The average of all measured tendon forces, including those measured 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 (1) are not less than 95% of the predicted forces.

3. For tendons requiring augmented examination (additional examination after repair or replacement), the measured forces in two like tendons located nearest to but on opposite sides of the tendon described in (1) 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 during the inspection, 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%.

8.1.4.8 Retensioning and elongation – Czech practice

Tendons that have been detensioned are retensioned to the average force predicted for the same tendon type at the time of the retensioning. Because the force at anchor decreases due to clearance adjustment among anchor part, prestressing force is increased with aim to balance these losses. Elongation is measured and compared with the last measurement but there are no limiting criteria.

8.1.5 Corrosion protection medium (Petrolatum-based grease) NUREG/CR-6598 [35] - US practice

8.1.5.1 Visual examination

Bottom grease caps of all vertical tendons should be visually inspected to detect grease leakage or grease cap deformations. Removal of grease caps is not necessary for this inspection.

The use of organic petrolatum-based corrosion protection compounds (greases) gained prominence for use in prestressed concrete containments (PCCs) in the United States because of the relative ease with which the tendons could be inspected and tested. Furthermore grease covering provides an approximately 50-percent reduction in the friction factor, thus permitting the use of longer tendons and fewer buttresses and anchorages. Unbonded tendons may be relaxed, retensioned, and replaced as required. Also a corrosion-protection coating applied in the shop (before shipment) permits the efficient scheduling of installation and tensioning of tendons and the installation of grease during the construction sequence.

The petroleum-petrolatum wax type base plus additives filler greases have evolved over 30-40 years to make the products more suitable to the application of unbonded (greased) tendons in PCCs. Initially, the product consisted of a sheathing filler containing polar wetting agents, rust-prevention additives, microcrystalline waxes, and proprietary constituents formulated to displace water, self-heal, and resist electrical conductivity. The second generation of materials added a plugging agent to raise the low-flow point of the products (approximately 39° C (100° F)) to keep them from seeking loose sheathing joints and flowing into hairline cracks in concrete and eventually staining the concrete surface. A subsequent refinement involved the incorporation of a light base number to provide alkalinity (3 mg KOH/gm of product) for improved corrosion protection. The current generation of products has been formulated to increase the viscosity without sacrificing pumpability, raise the congealing point to 57-63°C (135-145°F), increase the resistance to flow from sheathing joints and increase water-displacing characteristics, and raise the base number (35 mg KOH/gm of product) to provide higher reserve alkalinity.

8.1.5.2 Examination of grease and free water samples

Samples of the grease are taken from each end of each tendon examined. Free water outside should not be included in the samples. Samples of free water are taken where water is present in grease in quantities sufficient for laboratory analysis.

Each corrosion protection medium sample needs to be thoroughly mixed and analysed for reserve alkalinity, water content, and concentrations of water soluble chlorides, nitrates, and sulphides. Analyses shall be performed in accordance with the procedures specified in Table IWL-2525-1. Free water samples are analysed to determine pH.

8.1.5.3 Removal and replacement of grease

After examining the tendon, wire/strand, anchorage hardware and surrounding concrete, grease (corrosion protection medium) shall be replaced to ensure sufficient coverage of anchorage hardware, wires, and strands. The amount of grease removed at each anchorage is measured and the total amount removed from each tendon sheath and end cap is recorded. The total amount replaced in each tendon sheath is recorded and differences between amount removed and amount replaced are documented. Corrosion protection medium may be replaced using a pressurized system or cold pack, by pouring or by non-pressurized pumping on each end. Maximum pressure to be used in a pressurized system and the installation method for corrosion protection medium should be carefully selected.

8.1.5.4 Corrosion protection medium acceptance standard

Corrosion protection medium is acceptable when the reserve alkalinity, water content, and soluble ion concentrations of all samples are within the limits specified in ASME, section XI, Table IWL-2525-1. The absolute difference between the amount removed and the amount replaced shall not exceed 10% of the tendon net duct volume.

8.1.6 Corrosion protection medium (petrolatum-based grease) – Czech practice

8.1.6.1 Visual examination

Visual examination is focused on integrity and uniformity of covering of anchor by grease and on visual evidences of grease degradation. . Visual examination of grease apart from anchorages is performed on removed tendons, examination is focused on grease degradation

8.1.6.2 Examination of grease and free water samples

Samples of the grease are taken from each removed tendon and tests focused on ability of grease to fulfil protection against corrosion are examined. In case of anchor inspection, grease samples are taken only in case that there are questions concerning grease degradation (e.g. change of colour or surface structure of grease). Each corrosion protection medium sample needs to be thoroughly mixed and analysed for reserve alkalinity, water content, and concentrations of water soluble chlorides, nitrates, and sulphides.

Due to not-filling of ducts by grease, periodical checks of presence of water in ducts are performed. In case of cylindrical tendons, the checks are performed once a year by means of drainage runoffs. In case of dome tendons, the checks are performed along with examination of anchorage zone of dome tendons. Samples of free water from ducts or from areas under anchor coverings are taken where water is present in quantities sufficient for laboratory analysis.

8.1.6.3 Removal and replacement of grease

After examining the tendon, wire and anchorage hardware, grease (corrosion protection medium) shall be replaced to ensure sufficient coverage of anchorage hardware and wires. Inspection after grease replacement is focused on sufficient thickness of coverage and integrity of coverage.

8.1.6.4 Corrosion protection medium acceptance standard

Corrosion protection medium is acceptable when the parameters meet minimum requirements of producer specified in the material certificate.

8.1.7 Anchorage zone

8.1.7.1 Visual examination

A detailed visual examination is performed on the tendon anchorage hardware, including bearing plates, anchor heads, wedges, buttonheads, shims, and the concrete extending outward a distance of 610 mm (2 ft.) from the edge of the bearing plate. Concrete cracks having widths greater than 0.25 mm (0.01 in.) are documented. Any corrosion, broken or protruding wires, missing buttonheads, broken strands, free water and cracks in tendon anchorage hardware are also recorded. Also broken wires or strands, protruding wires and detached buttonheads following retensioning of tendons which have been detensioned are documented.

Figure 0.2 Source – EPRI TM-102C

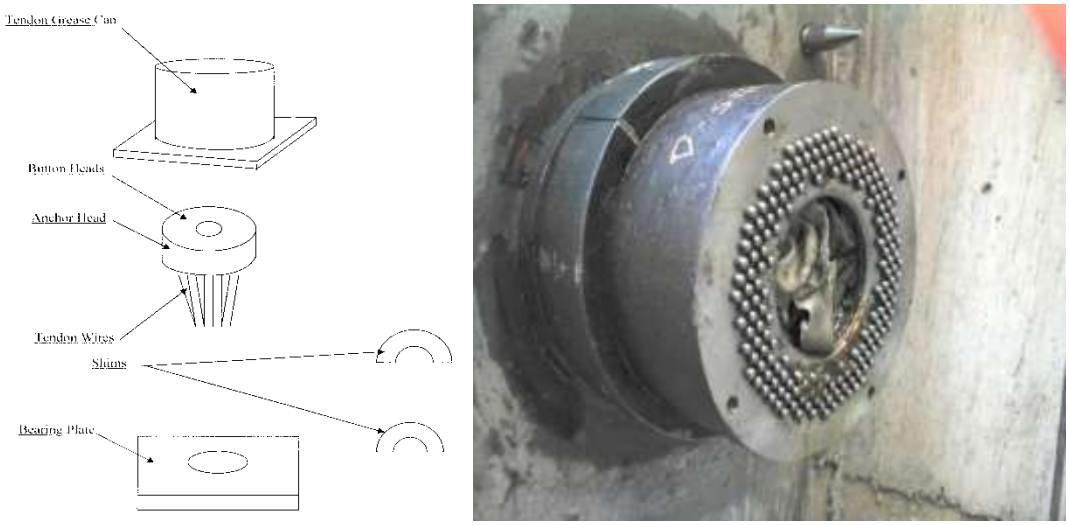


Figure 0.3 Anchor of VVER1000 type prestressing tendon

8.1.7.2 Tendon anchorage areas acceptable limits

The condition of tendon anchorage areas is acceptable if there is no evidence of cracking in anchor heads, shims, or bearing plates, no evidence of active corrosion, and no evidence of free water. In cases where broken or unseated wires, broken strands, and detached buttonheads were documented and accepted during a preservice examination or during a previous in-service examination; they will be acceptable in subsequent examinations. Cracks in the concrete adjacent to the bearing plates should not exceed 0.01 in. (0.25 mm) in width.

8.1.8 In-service inspection schedule - US practice

8.1.8.1 Concrete

Concrete is examined in accordance with at 1, 3, and 5 years following the completion of the containment Structural Integrity Test and every 5 years thereafter. The 1, 3, and 5 year examinations commence not more than 6 months prior to the specified dates and are completed not more than 6 months after such dates. If plant operating conditions are such that examination of portions of the concrete cannot be completed within this stated time interval, examination of those portions may be deferred until the next regularly-scheduled plant outage.

The 10 year and subsequent examinations commence not more than 1 year prior to the specified dates and should be completed not more than 1 year after such dates. If plant operating conditions are such that examination of portions of the concrete cannot be completed within this stated time interval, examination of those portions may be deferred until the next regularly-scheduled plant outage. Concrete surface areas affected by a repair/replacement activity shall be examined at 1 year (± 3 months) following completion of the repair/replacement activity. If plant operating conditions are such that examination of portions of the concrete cannot be completed within this time interval, examination of those portions may be deferred until the next regularly-scheduled plant outage.

8.1.8.2 Unbonded post-tensioning systems

Unbonded post-tensioning systems are examined at 1, 3, and 5 years following the completion of the containment Structural Integrity Test and every 5 years thereafter. The 1, 3, and 5 year examinations commence not more than 6 months prior to the specified dates and are completed not more than 6 months

after such dates. If plant operating conditions are such that examination of portions of the post-tensioning system cannot be completed within this stated time interval; examination of those portions may be deferred until the next regularly-scheduled plant outage.

The 10-year and subsequent examinations commence not more than 1 year prior to the specified dates and are completed not more than 1 year after such dates. If plant operating conditions are such that examination of portions of the post-tensioning system cannot be completed within this stated time interval, examination of those portions may be deferred until the next regularly-scheduled plant outage.

8.1.9 Sites with multiple plants

For sites with multiple plants, the examination requirements for the concrete containments may be modified if the containments utilize the same prestressing system and are essentially identical in design, if post-tensioning operations for each subsequent containment constructed at the site were completed not more than 2 years apart, and if the containments are similarly exposed to or protected from the outside environment. When these conditions are met, the inspection dates and examination requirements may be as follows:

- For the containment with the first Structural Integrity Test, all concrete surface areas (accessible and suspect), tendon, wire or strand, anchorage hardware and surrounding concrete, corrosion protection medium, free water shall be performed at 1, 3, and 10 years and every 10 years thereafter.

Only the examinations of tendon anchorage and corrosion protection medium and free water shall be performed at 5 and 15 years and every 10 years thereafter.

- For each subsequent containment constructed at the site, all concrete surface areas (accessible and suspect), tendon, wire or strand, anchorage hardware and surrounding concrete, corrosion protection medium, free water shall be performed at 1, 5, and 15 years and every 10 years thereafter.

Only the examinations of tendon anchorage and corrosion protection medium and free water shall be performed at 3 and 10 years and every 10 years thereafter.

8.1.10 In-service inspection schedule - Czech practice

The following ISI are done every year:

- Concrete - visual inspection of accessible concrete surface including coating and roof covering, non-destructive testing of concrete strength,

- Liner - visual inspection of accessible surface including coating, non-destructive testing of the thickness.

ISI of prestressing system is divided into two basic time periods. First period is the first five years of unit operation. During this period, inspections are done every year - primarily to identify construction defects. Second period takes till the end of unit operation. During this period, inspections are done every 6th year. Removing of tendons is done every 12th year beginning last year of the first period. Approx. 2% of tendons are removed for detail inspection, i.e. one tendon of dome part and 2 tendons of cylindrical part of containment.

No rules for Multiple Plants are applied due to low number of units operated in the Czech Republic.

8.1.11 Dry air ventilated tendons and greased tendons – Swedish practise

In Sweden there are ten reactors in operation. All the reactor containments consists of an outer bearing prestressed concrete part, an inner sealing of tight-welded steel liner and an reinforced concrete part to protect the steel liner. Three of the reactor containments are provided with greased tendons (PWRs Ringhals 2,3 and 4) and three with dry air ventilated tendons (BWRs Forsmark 1, 2 and 3. The periodic verification of leak-tight integrity of all the primary reactor containments is performed in accordance with option A in NRC 10CFR50 appendix J to part 50 (Type A tests). The test interval is fixed and set to three tests equally distributed over ten years.

The program used for inspection of reactor containments provided with dry air ventilated tendons and greased tendons is based on NRC Regulatory Guide 1.35. The tendon forces are measured through the lift off technique for a number of randomly (2-4 % from each group) selected cables every 5th year for the dry air ventilated tendons and since 1989 every 10th year for the greased tendons. A few individual wires are taken out to control changes in material properties. Analyses are made on the corrosion protective grease.

8.1.12 Trending prestressing force

RG 1.35.1 [38] provides one method for constructing the upper and lower bounds of prestressing forces, based on the expected variation in the systemic time-dependent losses in prestressing forces. Appendix B of RG 1.35.1 provides the upper and lower bounds and the minimum required prestressing forces as a function of time. To be acceptable, the measured prestressing forces should exceed the minimum required prestressing force derived from the design requirement to counteract the tensile stresses in the containment shell induced by the postulated design-basis accident. Another way of predicting prestressing force is to develop procedures that would enable the utilities to develop trend lines based on measured forces recorded during all prior inspections.

Title 10, Section 50.55a(b)(2)(viii)(B), of the Code of Federal Regulation (USA) mandates “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”.

The provision essentially mandates corrective actions if, during an inspection, the trending of prestressing forces based on the prior tendon force measurements indicates that the prestressing force value is likely to fall below the minimum required value. In addition, in the license renewal applications, the owners of plants with PCCs are required to perform a time-limited aging analysis of the tendon prestressing forces to provide assurance that, at the end of the license renewal period, the tendon forces will meet the licensing basis requirements. These two provisions required the owners of the plants to perform a trending analysis of prestressing forces. The trend line only reflects the prestressing force values at tendon anchorages. Trend lines do not represent the minimum prestressing forces along the tendons. The minimum required prestressing force values must be adjusted to account for friction to compare them with the values obtained from the trend lines. To maintain consistency when comparing the projected prestressing forces from the trend lines with the minimum required prestressing forces, the tendons should be grouped so that they exhibit similar time-dependent characteristics.

The monitoring and trending of prestressing tendon forces assure that the containment has adequate compression to counteract the tension produced by various loadings for which the containment is designed. The tension generated by the pressure associated with various postulated accident conditions represents the

major loading for which the prestressing force in a prestressed concrete containment is provided. The trending of prestressing forces is essential for projecting the potential behaviour of prestressing forces during plant-life extension. A statistically valid approach recommended for developing a trend line is essential to performing the regression analysis of the measured prestressing forces.

8.1.13 Future action

The NRC is currently reviewing RG 1.35, RG 1.35.1 and RG 1.90 to assess the need for improvement. This applies to containments in existing and future plants. The overall objective of this review is to ensure there is no reduction of the safety margins of concrete containments under operating and environmental conditions. The review includes operational experience of actual ISI, the latest codes and standards for post-tensioned containments worldwide, and the latest research results that affect prestressed losses. The outcome of the review will provide input for an evaluation of the appropriateness of changes in the three regulatory guides.

8.1.14 Tables from ASME section XI, subsection IWL [6]

TABLE IWL-2500-1
EXAMINATION CATEGORIES
EXAMINATION CATEGORY L-A, CONCRETE

Item No.	Parts Examined	Test or Examination Requirement	Test or Examination Method	Acceptance Standard	Extent of Examination	Frequency of Examination	Deferral of Examination
L1.10	Concrete surface						
L1.11	All accessible surface areas [Note (1)]	IWL-2510	General visual	IWL-3210	IWL-2510	IWL-2410	NA
L1.12	Suspect areas	IWL-2510	Detailed visual	IWL-3210	IWL-2510	IWL-2410	NA
L1.13	Inaccessible Below-Grade Areas Note (2)]	IWL-2512(c)	IWL-2512(c) [Note (3)]	IWL-3210	IWL-2512(a)	IWL-2512(c)	NA

NOTES:
(1) Includes concrete surfaces at tendon anchorage areas not selected by IWL-2521 or exempted by IWL-1220(a).
(2) Concrete surfaces exposed to foundation soil, backfill, or ground water.
(3) Method of examination as defined by the Responsible Engineer, based on IWL-2512(b) evaluation.

EXAMINATION CATEGORY L-B, UNBONDED POST-TENSIONING SYSTEM

Item No.	Parts Examined	Test or Examination Requirement	Test or Examination Method	Acceptance Standard	Extent of Examination	Frequency of Examination	Deferral of Examination
L2.10	Tendon	IWL-2522	IWL-2522	IWL-3221.1	IWL-2521	IWL-2420	NA
L2.20	Wire or strand	IWL-2523	IWL-2523.2	IWL-3221.2	IWL-2523.1	IWL-2420	NA
L2.30	Anchorage hardware and surrounding concrete	IWL-2524	Detailed visual	IWL-3221.3	IWL-2524.1	IWL-2420	NA
L2.40	Corrosion protection medium	IWL-2525, IWL-2526	IWL-2525.2(a), IWL-2526	IWL-3221.4	IWL-2525.1(a), IWL-2526	IWL-2420	NA
L2.50	Free water	IWL-2525	IWL-2525.2(b)		IWL-2525.1(b)	IWL-2420	NA

TABLE IWL-2521-1
NUMBER OF TENDONS FOR EXAMINATION

Inspection Period	Percentage [1,2] of all Tendons of Each Type[3]	Required Minimum [1] Number of Each Type	Maximum Required Number of Each Type
1st Year	4	4	10
3rd Year	4	4	10
5th Year	4	4	10
10th Year4 [4]	2	3	5

NOTES:

(1) Fractional tendon numbers shall be rounded to the next higher integer. Actual number examined shall not be less than the minimum required number and need not be more than the maximum required number.

(2) The reduced sample size listed for the 10th year and subsequent inspections is applicable only if the acceptable criteria of IWL-3221.1 have been met for the last three inspections.

(3) A tendon type is defined by its geometry and position in the containment: e.g., hoop, vertical, dome, helical, and inverted U.

(4) The number and percentage of tendons to be examined every 5th year thereafter shall remain the same.

TABLE IWL-2521-2

AUGMENTED EXAMINATION REQUIREMENTS FOLLOWING POST-TENSIONING SYSTEM REPAIR/

REPLACEMENT ACTIVITIES

Examination Frequency	Number (N) of Tendons of Each Type[1] Affected by Repair/Replacement Activity	Required Minimum Percentage of Tendons of Each Type[1] Affected by Repair/ Replacement Activity To Be Examined	Augmented Examination Requirement [2,3]
Initial Inspection: 1 year (± 3 months) following completion of the Repair/Replacement Activity [4]	3 < N < 5% N ≥ 5%	4%[5] Lesser of 4% or 10 tendons	L2.10, L2.30, L2.40 & L2.50 L2.10, L2.20, L2.30, L2.40 & L2.50
Subsequent In-service Inspections scheduled to coincide with IWL-2420[6] following completion of the Repair/Replacement Activity	3 < N < 5% N ≥ 5%	4% Lesser of 4% or 10 tendons	L2.10,L2.30,L2.40 & L2.50 L2.10,L2.20,L2.30,L2.40 & L2.50

NOTES:

(1) The tendon type is defined by its geometry and position in the containment: e.g., hoop, vertical, dome, helical, and inverted U. If a repair/replacement activity affects a group of tendons, and differing actions are performed on individual tendons in the group (e.g. tendon replacement, retensioning, and detensioning and retensioning), each type of action performed on the tendons need not be considered separately when calculating the number (N) of tendons affected.

(2) A common tendon need not be selected for examination as specified in IWL-2521(b).

(3) Examination requirements are identified in Table IWL-2500-1

(4) If plant operating conditions are such that examination of portions of the post-tensioning system cannot be completed within this stated time interval, examination of those portions may be deferred until the next regularly scheduled plant outage.

(5) Where the minimum number of tendons is given as a percentage, fractional tendon numbers shall be rounded to the next highest integer and shall be considered the minimum number of tendons to be examined. The percentage is to be applied separately to each type of tendon affected.

(6) The required minimum number of affected tendons of each type to be examined may be reduced to the lesser of 2% or 5 tendons, if the acceptance criteria of IWL-3221.1 have been met for the last 2 (two) inspections.

TABLE IWL-2525-1
CORROSION PROTECTION MEDIUM ANALYSIS

Characteristic	Test Method	Acceptance Limit
Water content	ASTM D 95	10 percent maximum
Water soluble chlorides	ASTM D 512 [Note (1)] or ASTM D 4327 [Note (1)]	10 ppm maximum
Water soluble nitrates	ASTM D 992 [Note (1)] or ASTM D 3867 [Note (1)] or ASTM D 4327 [Note (1)] or 4110 [Notes (1), (2)] or 4500-NO3 − [Notes (1), (2)]	10 ppm maximum
Water soluble sulphides	APHA 427 [Note (1)] or APHA 4500-S2− [Note (1)] or 4500-S2− [Notes (1), (2)]	10 ppm maximum
Reserve alkalinity (Base number)	ASTM D 974 Modified [Note (3)]	[Note (4)]

NOTES:

(1) Water Soluble Ion Tests. The inside (bottom and sides) of a one (1) litre beaker, approx. OD 105 mm, height 145 mm is thoroughly coated with 100 ±10 grams of the sample. The coated beaker is filled with approximately 900 ml of distilled water and heated in an oven at a controlled temperature of 100°F (38°C) ±2°F (1°C) for 4 hours. The water extraction is tested by the noted test procedures for the appropriate water soluble ions. Results are reported as PPM (parts/million) in the extracted water.

(2) These referenced test methods are published in "Standard Methods for the Examination of Water and Wastewater," published jointly by APHA, AWWA, and WEF. The following specific test methods are approved for use:

(a) 4110 B. — Ion Chromatography with Chemical Suppression of Effluent Conductivity

(b) 4110 C. — Single-Column Ion Chromatography with Direct Conductivity Detection

(c) 4500-NO_3 − E. — Cadmium Reduction Method

(d) 4500-NO_3 − F. — Automated Cadmium Reduction Method

(e) 4500-NO_3 − H. — Automated Hydrazine Reduction Method

(f) 4500-NO_3 − I. — Cadmium Reduction Flow Injection Method

(g) 4500-S2− D. — Methylene Blue Method

(h) 4500-S2− I. — Distillation, Methylene Blue Flow Injection Method

(3) ASTM D 974 Modified. Place 10 g of sample in a 500 ml Erlenmeyer flask. Add 10 cc isopropyl alcohol and 5 cc toluene. Heat until sample goes into solution. Add 90 cc distilled water and 20 cc 1 Normal (1N) H2SO4. Place solution on a steam bath for 1⁄2 hour. Stir well. Add a few drops of indicator (1% phenolphthalein) and titrate with 1 Normal (1N) NaOH until the lower layer just turns pink. If acid or base solutions are not exactly 1N, the exact normalities should be used when calculating the base number. The Total Base Number (TBN), expressed as milligrams of KOH per gram of sample, is calculated as follows:

$TBN = [20 (N_A) - (B)(N_B)]56.1/W$

Where

B = millilitres NaOH

N_A = normality of H2SO4 solution

N_B = normality of NaOH solution

W = weight of sample in grams

(4) The base number shall be at least 50% of the as-installed value, unless the as-installed value is 5 or less, in which case the base number shall be no less than zero. If the tendon duct is filled with a mixture of materials having various as-installed base numbers, the lowest number shall govern acceptance.

8.2 Bonded tendons

8.2.1 Technical background

This section deals with rationale, operating experience, current practices and further development related to In-Service Inspection and Surveillance systems for containment with bonded tendons.

8.2.2 Introduction

In concrete structures with unbonded tendons, lift-off tests or other technologies (see EPRI report [20]) can be used to measure the effective stress in the tendons. For containment prestressing design involving tendons bonded with cement, it is obvious that the strands cannot be directly inspected, re-tensioned, or removed. Furthermore, the force in the bonded tendons can't be directly measured with current monitoring systems. Following the IAEA guidelines for Surveillance and In-Service Inspection [24] and the requirements from the concerned Regulators in the world, the choice of bonded ducts for prestressed concrete containment shall be associated with a dedicated program, including specific and appropriate instrumentation and pressure test during the operating life of the plant. The aim of this instrumentation is to provide relevant and quantified data on the remaining prestressing forces in the tendons and on the integrity of the strands over time.

In addition to periodic visual inspections of concrete surfaces and anchorage hardware, the surveillance system shall be designed to monitor the following phenomena:

- Stress relaxation of prestressing strands,
- Delayed concrete strain due to shrinkage and creep,
- Decrease of cross sectional area of wire or possible failure of strands, due to corrosion.

We note here that some specific methods dedicated to direct force monitoring for prestressing tendons were proposed in the past, or are still under development. We can mention systems based on the magneto-elastic couplings in the steel of the strands (such as TENSMOIMAG® proposed by Freyssinet or the DYNAMAG [51]). These techniques rely on the monitoring of magnetic properties which are meant to change with stress. If they have given satisfactory results in laboratory conditions and, for example, on bridge structures, there is still a need for technical development when it comes to install the electromagnetic sensors within a concrete wall for bonded tendon monitoring.

The following sections intent to:

- Give an overview of the existing feedback from different countries,
- Present some guidelines for implementing a In-Service Inspection and Surveillance program,
- Address further needs in term of technical development.

8.2.3 Feedback on bonded tendons monitoring

Reference [19] provides a list of selected nuclear power plants fit with embedded sensors for prestressing level monitoring:

- Temelin (Czech Republic, VVER),
- EDF fleet (France, PWR),
- Tihange and Doel (Belgium, PWR),
- Pont Lepreau (Canada, CANDU),
- Uljin (Korea, PWR),
- Wolsong (Korea, CANDU).

8.2.3.1 U.S.NRC guidance for bonded tendons for PWR and BWR (USA)

For In-Service Inspection of containment with bonded tendons, U.S.NRC provided the regulatory guides RG 1.90 [39] which has been updated recently with a 30 year-feedback from several countries. It entails two acceptable alternative methods for monitoring the containment:

- Alternative A: a surveillance program based on monitoring the prestressed level by monitoring of the tensile strains in the wires of bonded tendons and evaluating the prestress level at a section in the containment structure from readings of appropriately located strain gauges or strain or stress meters at the section and a pressure test at 1, 3, 5 and then at 10 year-intervals; with a comparison with Initial Structural Integrity Test (ISIT), performed during the pre-operational test of the containment and predicted prestress force.

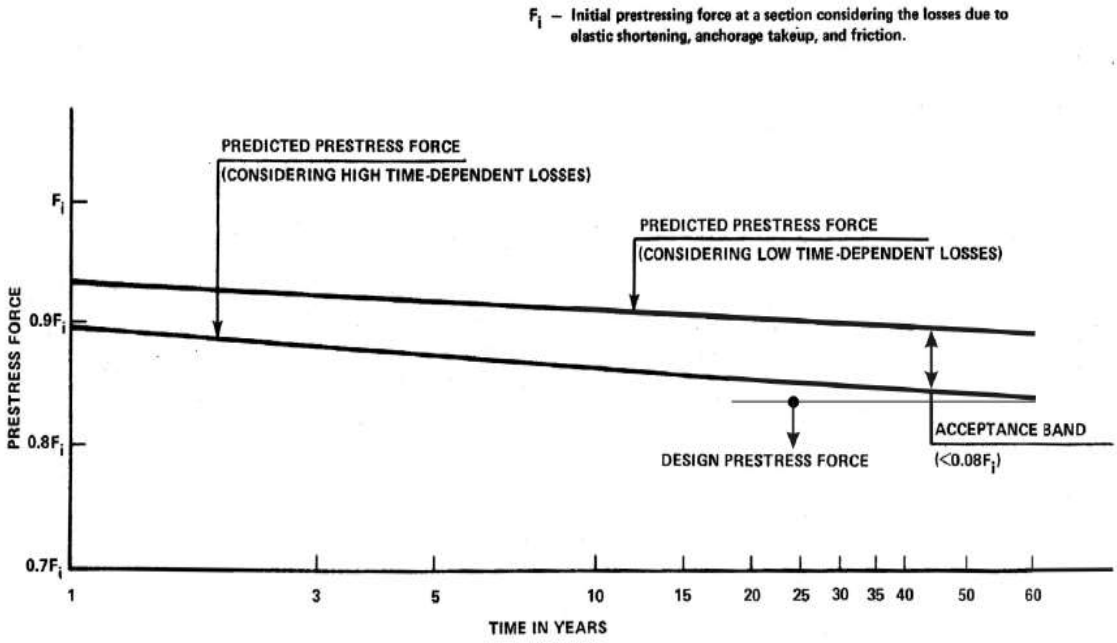
- Alternative B: Evaluating elastic response of containment subjected to pressure testing at 1, 3, 5 and then at 5 year-intervals, with a comparison with ISIT, performed during the pre-operational test of the containment and predicted prestress force.

For both alternatives, the pressure test in the peak pressure P_A representative for accidental conditions.

The prestress force inferred from monitoring data is compared with predicted forces calculated for licensing by taking time-dependant losses of prestress (relaxation of steel and shrinkage/creep of concrete) into account. Typically, records shall comply with acceptance band of prestress forces over time, which take overall uncertainties into account (see

Figure 0.4).

Figure 0.4 Typical bands for acceptable prestress levels [39]



In addition, [39] recommends:

- A force monitoring for 3 types of tendons left unbonded(in vertical, dome and hoop directions),
- Periodic visual examinations of visible parts of the containment and of the prestressing systems.

8.2.3.2 EDF practice for PWR (France)

For French PWR in operation, the technical specifications are based on the code RCC-G [3]. For EPR series, the technical requirements are stated in the code ETC-C [4].

The French methodology is rather similar with the alternative A of US NRC RG 1.90 [39] but includes some differences also:

- Force monitoring of unbonded test tendons (4 vertical tendons, whereas [39] specifies unbonded test tendon for hoop and dome tendons as well),
- Monitoring of the prestress level from the prestressing phase to the end of life of the plant, by using concrete strains as primary records to access to the force in the tendons by calculation,
- Monitoring of deformation under pressure (10-year periodic pressure tests at P_A, the pressure of Design-Basis Loss of Coolant Accident),
- Visual inspection of the concrete accessible surfaces and tendon anchorage systems, during pressure tests and in operation. Crack width measurement systems are set in place to monitor the main cracks during pressure tests. Moreover, in operation, a five-year inspection dedicated to corrosion marks detection is carried out on seaside power stations.

In France, the pressure test is performed every ten years, in conjunction with the Integrated Leak Rate Test (ILRT), at the peak pressure representative for an internal accident P_A. As a matter of fact, in France, the decennial outage is also the time for the primary coolant system pressure test, which is mandatory. But pressure tests related to ILRT and prestress monitoring can be performed at different time and with different frequency, depending on national regulations.

Technical specifications give some information on the type of sensors to use:

- Pendulums and Invar wires for measuring the global deformation of the containment cylinder part,
- Vibrating wire strain gages (VWSG) and temperature probes which are embedded in the concrete wall,
- Dynamometers for the force monitoring of the 4 unbonded tendons.

The data which are the most used for containment analysis are the VWSG measurements (with the associated temperature records).

Additional sensors or systems can be implemented to improve the surveillance system or to test new technologies of devices. For example, EDF have tested several types of embedded sensor to monitor the concrete moisture over time and try to highlight the relationship with drying shrinkage and creep.

The need to monitor the prestressing level led EDF to define an Optimum Surveillance System (OSS), dedicated to the long term monitoring of the containment. The OSS shall be maintained throughout the lifetime of the structure. In the event of failure, the sensors have to be replaced. Then, alternative solutions have to be developed to replace embedded sensors (mainly VWSG). That is the reason why EDF has designed a dedicated surface strain meter [55], which is able to deliver readings consistent with VWSG measurements, during test and in operation. The surface strain meter is made up of a 1m length Invar bar, to limit the sensitivity to temperature variation. Displacement is measured using a linear variable-differential transformer (LVDT) sensor. A photograph of the device is shown below (

Figure 0.5).

Figure 0.5 Surface strain meter installed on site. The displacement sensor is installed on the left hand end of the 1 m Invar bar (EDF)

In operation as well as during pressure tests, obtained measurements are compared with the results of predictive calculations taking pressure, temperature, creep, shrinkage and relaxation into account. It is to be checked that the observed strain amplitude and rate are always below the expected values from the blind analysis. Any deviation beyond measurement and calculation uncertainties shall be reported, and the operator shall be required to justify the long term safety.

Further details on the French practice are provided in section 0.

Once the readings are collected, monitoring data shall be processed to allow a relevant analysis of the structural behaviour and shall be stored, in a way so that it is possible to extract easily old measurements and perform new analysis. For that purpose, information on a reading should include: the date and time of the reading, the physical measurement (current voltage or intensity, resistance, frequency, etc.), the formula or the process to derive readings into the monitored quantity (strain, displacement, temperature…), the derived quantity and a possible comment on the reading operation (signal shape or stability, sound quality for VWSG, etc.).

Typically, a two-level analysis should be considered for monitoring data management. The first level is a data analysis, with little knowledge on the “expected” behaviour of the structure. It is performed by the staff in charge of the measurement. The second level consists of more or less sophisticated structural analysis and is performed by specialist of civil structure design or modelling. The experience shows that a straight discussion is needed between teams in charge of measurement and the teams in charge of structural analysis. Unfortunately, it is not obvious to manage monitoring data without any help from people who achieved them.

Typically, level 1 analysis should be undertaken within 5 days after the measurement operation. The analysis aims to:

- Detect any delay in measurement or measurement omission,
- Identify measurement errors and correct them,
- Perform thermal effects correction,

- Update and maintain the database,
- Identify any failed devices,
- Identify any faulty mechanical behaviour.

Statistics tools can be used to assess the dispersion of data and to identify possible outliers.

When investigations confirm that a measurement point deviates from the normal dispersion range by more than the allowed limit a warning or an alert is issued.

The level 2 analysis is dedicated to periodic safety analyses, which are fed by monitoring data and to alert management, if required. If an alert is raised, an initial set of basic analyses is undertaken to identify more accurately the structural causes and consequences of the detected anomaly. The Regulator should be officially informed of the alert as soon as possible and provided with the subsequent action plan within adequate time period. An assessment is made as to justify the compliance of the containment behaviour with safety criteria. If required, it can be considered to strengthen the structure, to increase the frequency of measurement acquisition or to monitor other part or new phenomenon.

8.2.3.3 Canadian approach for CANDU reactors

CSA N287.7 [16] provides requirements related to containment in-service examination and testing for CANDU reactors. Monitoring of the containment structure during Pre-operational Proof Test (PPT) and Leakage Rate Test (LRT) is addressed in CSA N287.6 [15]. It is required to instrument containment structure to enable evaluation of its behaviour during the PPT.

CSA N287.7 [16] requires the containment post-tensioning system that is used as principal reinforcement to be a subject to an integrity evaluation. Guidelines for instrumented monitoring including monitoring of the strain, temperature and deformation of the containment structure are provided in the Standard. The in-service examination program includes visual examinations for visible parts of the concrete and anchorage systems. The frequency of the in-service examination is at a minimum equal to the frequency of the ILRT.

The standard also provides guidelines for an indirect method to monitor the post-tensioning system behaviour, by the use of test beams. The beams are built at the time of containment structure construction and use the same materials and the same post-tensioning system as the containment structure; concrete cover and level of prestressing in the beams replicate that of the containment structure. They should be stored in a manner so that they are exposed to the same environmental conditions. Number of beams that need to be cast should be calculated to last the service life of the structure as well as to cover possibility of life extensions. The beams are tested periodically, typically with a minimum frequency equal to that of ILRT. The tests include flexure tests and destructive tests to confirm effectiveness of corrosion preventive medium. Also, the lift-off tests are performed on the beams cast with unbonded tendons to estimate time depending prestressing force losses. The Test beam method is also used in other countries (e.g. Finland, Belgium) to evaluate integrity of the bonded post-tensioning system.

8.2.3.4 Chinese practice

Chinese experiences related to monitoring of containments with bonded tendons can be found in [57] and [63]. Both articles deal with pre-operational integrity test reports on Chinese PWR, and show applications of French and American practices, referring to [3], [5] and [39].

8.2.3.5 Belgian practice

Belgian practice, for instance reported in [28], refers to the 1977 version of [39] (alternative "A" for the prestress level monitoring, to [5] and [6]. It includes also the use of test beams, such as in Canadian specifications. Due to the embedded sensors failure (mostly VWSG), surface strain meters have also been installed.

8.2.3.6 South African practice

The design of the ESKOM containment structures at Koeberg is similar to the French 900 MW PWR series. The concrete structures are monitored and are required to meet the criteria of ASME XI [6] subsection IWL with some exceptions due the cement grouting of the tendons, which makes it impossible to comply with the prescribed method of inspection as described in ASME XI [6]. So, the Koeberg monitoring program is a kind of mix between ASME XI [6] requirements and French practice.

The containment structures are subject to regular monitoring, which enable the performance of the containment structures to be evaluated during routine in-service tests. The monitoring system includes the same devices as used in France: topographical reference marks on the upper and lower rafts (since Koeberg station is built on seismic isolators), vertical invar wires, pendulums, embedded VWSG and thermocouples, dynamometers set on unbonded tendons.

The external concrete surfaces of the containment structures at Koeberg station are subject to examination in accordance with ASME XI [6] subsections IWL and IWE. Due to the accelerated degradation of the concrete integrity at Koeberg power station, this examination shall be performed during every refuelling outage. The metallic liner is visually inspected during every second refuelling outage.

In-service testing of the containment structure is monitored by performing leak rate tests based on ASME Operation and Maintenance code [7] and 10CFR50 Appendix J requirements. The behaviour of the containment is also assessed by recording measurements from the monitoring devices acquired during the pressure tests.

8.2.3.7 Swiss practice

With one exception all concrete structures of the Swiss NPPS are conventionally reinforced. For bearing the roof loads the uppermost 5 metres of the cylindrical outer wall of the reactor buildings Beznau I and II (secondary containments) are prestressed. The secondary containments are not designed to resist internal pressure, these loads will be absorbed by the primary steel containments.

14 Groups of 4 parallel bonded tendons (system BBRV) are embedded all around the secondary containments. The tendons consist of 43 wires with diameter 6 mm per wire. The original design bases for the reactor buildings were:

- ASME code, Section III [5],
- Safety code for Design, Fabrication and Maintenance of Containment Structures for Stationary Atomic Power Reactors, ASME,
- Codes SIA 160/1956, SIA 161/1956, SIA 162/1956, Swiss Society of Engineers and Architects.

The tendons are examined within the scope of the ageing management program. Main inspections are done every 10 years with interim inspections in between after 5 years. Amongst others the following tests are carried out:

- Visual inspections of the concrete surfaces,
- Removing of concrete drill cores and testing of the concrete characteristics in the lab,
- Potential field measurements to detect corrosion in the tendons,
- Selective exposing and visual inspections of the anchor heads.

Up to now no signs for ongoing corrosion in the tendons were found. The inspected exposed anchor heads were in faultless state.

8.2.3.8 Finnish practice

8.2.3.8.1 Operating plants in Olkiluoto

Currently there are two operating units with prestressed containments with unbonded tendons OL1 (1978) and OL2 (1980) Asea-Atom Ab BWR 660 MW (updated to 880MW). The units have different prestressing systems: The OL1-unit containment has a VSL-system with Bridon Supa LR of 13 mm strands and the OL2-unit has a BBRV system with tendons of single 7 mm wires. The tendons have been bonded with cement grout.

Containment buildings are tested (A-test) with 0.4 MPa of overpressure. In addition to pressure, temperature and humidity are measured during the test.

Strains are measured with an ISI system (strain gauges MDD 53 a). The measured strains are compared with theoretical values acquired from finite element analysis. Before the test and during maximum pressure, a visual inspection is made and a cracking map is created. Over 0.05 mm cracks are recorded. Initial pressure test was at 0.525 MPa ABS (1.15 times the design over pressure) and the first A-test with 0.4 MPa overpressure.

Integrity test details such as frequency and pressure have been set according to principles stipulated in American regulations 10 CFR, Part 50, Appendix J.

Additional tests and inspections have been done with NDT equipment. 3D FEM analyses have been done in order to verify test results.

8.2.3.8.2 OL3 under construction

The ISI-system of the containment of the OL3 EPR unit in Olkiluoto is based on French practice with displacement, strain, temperature, humidity measurement of containment and force measurement of the four unbonded tendons. An additional optical system was implemented to detect possible excess strains of the liner. Humidity measurements of the containment wall will be verified with test samples stored in reactor building annulus. One test sample consists 1.3 m long 150 mm diameter cylinder with 100 mm long parts. One end of cylinder is left open while other surfaces are connected to tight mould to avoid loss of moisture. Humidity of length on containment wall will be determined by weighting each part.

8.2.3.8.3 Regulations for new plants

Finnish legislation for nuclear power plants is based on the Nuclear Energy Act [30]. Design and construction of buildings and structures shall adhere to Land Use and the Building Act [21]. Radiation and Nuclear Safety Authority (STUK) is authorized to set safety regulations for design, construction and operation. The Regulatory Guides on Nuclear Safety (YVL-Guides) have been completely updated in 2013. Guide [31] sets forth requirements and instructions for containment buildings. Guide [32] sets

requirements for design, materials and construction and ISI for containment building. Other standards mentioned in YVL codes are:

- Eurocode standard for loads, concrete and steel structures EN 1990 [9], EN 1991 [10], EN 1992 [11], EN 1993 [14],
- SFS-EN 206-1 Concrete. Part 1: Specification, performance, production and conformity [29],
- ETAG 013, Guideline for European Technical Approval of Post-Tensioning Kits for Prestressing of Structures [17],
- ASME Boiler and Pressure Vessel Code III Division 2, Code for Concrete Containments [5],
- USNRC Regulatory Guide 1.90, In-service inspection of prestressed concrete containment structures with unbonded tendons [39].

General rules for containment building according to new regulatory guides [31] and [32] are as follows:

- A test plan shall be made for pressure and leak tightness test and shall include:
  - The deformations and the strains in different pressure levels (predictive analysis)
  - Retrieving of the deformations and the strains (measurements)
  - Mapping of the cracks
  - Temperatures, amounts of leaks
- Pressure test for $1.15P_d$ before commissioning
- Periodical leak tightness test at accident pressure for containment building and penetrations.
- Adequate instrumentation for retrieving deformations, strains, temperatures and moisture of base slab and containment during pressure and leak tightness tests.
- Tendon force for some tendons must be measurable.

ISI program shall be delivered to regulator for approval and shall include:

- Periodical and during testing inspections for deformations, strains and leak tightness
- Periodical inspections of tendons and anchors
- Assessment for essential structures with testing or with other reliable means (core drill samples, environmental samples, NDT, etc.)

ASME III Div.2 [5] and USNRC Regulatory Guide 1.90 [39] are mentioned as useful reference for pressure and leak tightness program.

8.2.3.9 Swedish Practise

In Sweden there are ten reactors in operation. All the reactor containments consists of an outer bearing prestressed concrete part, an inner sealing of tight-welded steel liner and an reinforced concrete part to protect the steel liner. Four of the reactor containments are provided with bonded tendons (BWRs Ringhals 1 and Oskarshamn 1, 2 and 3).

The periodic verification of leak-tight integrity of all the primary reactor containments is performed in accordance with option A in NRC 10CFR50 appendix J to part 50 (Type A tests). The test interval is fixed and set to three tests equally distributed over ten years.

The visual inspections of reactor containments provided with bonded tendons are supplemented with crack mapping and in some cases also with the “hammer-method”. The anchor and the concrete surrounding the anchor are inspected visually. The tendon forces can’t be measured.

8.2.3.10 Summary - bonded tendons monitoring

When the country is adopting bonded tendons, instrumentation of the different parts of the containment is one of the keys to demonstrate, to the satisfaction of the regulator, that the prestressing forces comply with the design requirements. The reliability and longevity of the instruments and also the data collection and interpretation are the critical aspects of monitoring.

Full pressure tests are often considered to complement the monitoring of the delayed strain. It is then important to keep carefully all the data acquired during the previous test to be able to get the overall trend of the observed phenomena.

The acceptance criteria are often related to the design calculations or blind predictions results. It is necessary to confirm the accuracy of the predictive analysis including ensuring that the codes used are appropriate, ambient conditions (i.e. temperature, humidity) and material properties used are representative. The consistency between the expected and observed containment behaviours is considered as a satisfying proof of compliance with safety requirement.

Periodic visual inspection and direct force monitoring (on a few unbonded tendon) are considered as well.

When interpreting results of the beams testing and projecting them to concrete containment structure, the following should be well understood and taken into account:

- Environment of exposure for the beams is not the same as environment that containment structure is exposed to as the beams can only be exposed to environmental conditions outside or operating condition inside the containment structure;

- The major contributors to the prestressing force loss such as creep and shrinkage greatly depend on the size of the concrete element; therefore prestressing force loss estimated by testing the beams do not directly represent the prestressing force losses of the massive concrete containment structure.

As an illustration, Table 0.1 below presents an outlook of the specifications for American and French in-service inspection.

Table 0.1 French and American In-service Inspection guidance for prestressed concrete containment with steel liner and bonded or unbonded tendons.

Tendon system	**Bonded**		**Unbonded**
Country	**USA**	**France**	**USA**
Applied regulation, guide, code or standard	10CFR50.55a; RG 1.90; ASME section XI-IWL; App J of 10CFR50	RCC-G; EDF routine maintenance program	10CFR50.55a; RG 1.35; ASME section XI-IWL; App J of 10CFR50
Inspection method	Force monitoring of unbonded test tendons (3 vertical, 3 hoop and 3 dome). Visual examination of concrete and anchorage hardware. Two alternatives for determining bonded tendon performance: Alternative A : monitor prestress level by instrumentation and pressure test (P_A), compare deformation with ISIT Alternative B: pressure test (P_A), compare deformation with ISIT	Force monitoring of unbonded test tendons (4 vertical) Visual examination. Pressure test (P_A) Permanent monitor of the concrete prestress level by instrumentation, including embedded sensors	Based on inspection of a sampling of tendons of each type (about 4% of total tendon population): Visual inspection of concrete and post-tensioning system; Lift off test or equivalent; Removal of one tendon of each type for tests on steel Testing grease and free water.
Frequency of tests	Alternative A: Instruments read 2 months until 1st ISI then at 6 months; Pressure test at P_A 1, 3, 5 years after ISIT then at 10 year intervals. Alternative B: Pressure test at P_A at 1, 3, 5 years after ISIT then at 5 year intervals. (ILRT may be combined with ISI pressure test for both alternatives)	1st refuelling outage and then every 10 years, in conjunction with ILRT	1, 3, 5 years after ISIT then every 5 years. No pressure test associated with the surveillance test. ILRT is a separate requirement per 10CFR50 Appendix J

Tendon system	Bonded		Unbonded
Country	**USA**	**France**	**USA**
Acceptance criteria	Alternative A: Verify prestress forces are within the predicted range (8% band). Check unbonded tendon forces. Linear and reversible deformation Limited cracking and other defects Report abnormal trend and implement corrective action Alternative B (during test): Limited increase of containment deformation as compared to ISIT Verify prestress force are within the predicted range Check unbonded tendon forces Limited cracking and other defects Linear and reversible deformation Report abnormal trend and implement corrective action Verify unbonded tendon force	In operation: No abnormal trend in the records consistency with delayed strain predictions Pressure test: Linearity, reversibility consistent with design predictions Limited cracks and other defects	Prestress force within predictive limit (average of tendon forces and individual force) Tensile strength tests on removed tendon (greater than ultimate tensile strength) Grease test criteria from ASME

8.2.4 Guidelines for bonded tendons monitoring

The recommendations provided for a typical ISI program for containment with bonded cables are based on the current international practice. One should keep in mind that, if needed, designer and/or operator will have to adapt the surveillance method to the national context (nuclear regulation and industrial background) of the country where the NPP is built. Typically, the determination of the schedule of the ISI program and the associated acceptance criteria have to be formally approved by the Safety Authority/Regulatory body involved in the assessment of the power plant. Requirements may vary from one country to another.

All the recommendations hereafter are to be considered as technical proposals derived from an international feedback.

8.2.4.1 General provisions

In case of containment prestressing system with cement bonded tendons, it is recommended to include in the ISI program the following items:

- Force monitoring of some tendons left unbonded;
- Concrete containment structure monitoring for performance of bonded tendons;
- Visual inspection of concrete and visible part of the prestressing system.

If the performance of bonded tendons is assessed through a monitoring containment prestress level during operating time (close to the Alternative A in [39]), it is recommended to start data collection as soon as possible during the construction and especially during the tensioning phase of the cables. Despite the additional cost involved, applying this option provides baseline information (as recommended in [24]) and more reliable insights on the actual behaviour of the structure [54]. For that purpose, a permanent real-time monitoring system seems more convenient to acquire and to process the data.

If case of an existing plant showing some gaps in its monitoring records (period without measurement, for example), the following possible solutions might be considered:

- Use available information from other similar plants (same structural design and similar materials), if any;
- Fill the gap using structural analysis, fed with available monitoring data;
- Carry out laboratory tests on representative samples to determine the actual creep and shrinkage laws and incorporate through modelling (see [47] for an example). Without any data at the containment scale, the prediction capability of such a method might be limited to a few years, due to the current performances of materials and structural analysis tools.

As a reminder, in every situation for civil structure long term surveillance, we recommend to select the monitoring devices based on their performance, ease of use, robustness and reliability. Feedback is of course of great interest to check if a system meets these specifications. [8] may be used as guidelines to specify and to select new instrumentation devices.

Before being set in place, each sensor should be calibrated (if applicable) with a dedicated calibration report. The minimum requirement is to get from the vendor a data sheet stating a relevant metrological control for each sensor.

Data management is an issue to be addressed with caution. All the data relative to sensor fabrication and all the records from the devices shall be stored for the entire operating life of the plant. The attention is drawn to the fact that this requirement is not so easy to meet, because the operating life could reach 60 years or more. A dedicated system has to be developed by the owner for that purpose, with regular backup and upgrade procedure.

Moreover, each sensor or acquisition system should be regularly checked and cleaned and, if possible, calibrated, repaired or changed. When the repair is not possible (for embedded sensors, for instance), an alternative system should be set in place to provide an information consistent with that which was delivered by the failed device (for example the surface strain meter developed by EDF, see section 0).

8.2.4.2 Force monitoring in unbonded tendons

A few tendons can be left unbonded to allow for direct force measurements.

Apart from lift-off testing, alternative methods can be used to measure a force in a tendon, such as hydraulic load cells or dynamometer based vibrating wire sensing or on linear variable-differential transformer (LVDT). Generally, the time before failure allows acquiring enough data so that a relaxation law fitting is possible. But it is difficult to rely on these sensors for all the expected operation time of the plant. It is to point out that these dynamometers cannot be replaced in case of failure.

If needed, lift off can be considered to check the actual force in the unbonded tendons.

8.2.4.2.1 Location

Typically, minimum requirement is to have 3 vertical tendons along the axis of the cylinder available for force monitoring. Following the way the monitoring data are used in the overall safety assessment, more tendons can be considered, including tendons in the hoop direction or in the dome.

8.2.4.2.2 Frequency

The owner should define the frequency at which the force monitoring will be conducted. This frequency should be based on the final use of the results of the measurements and the capability of the measurement system. Of course, dynamometers enable more frequent measurements with fewer disturbances for the structure.

The established frequency shall be submitted to the national Regulator for approval.

8.2.4.2.3 Instrument specifications

Requirements on the range and the precision of the measurement system mainly depend on the expected tension variation in the tendon. The system should be operated normally within the range of expected site temperature (typically -20°C /+60°C).

Temperature in the containment cylinder shall be measured also, for further data processing.

Accuracy below 1.5% of the minimum ultimate force is recommended.

8.2.4.2.4 Data processing

Raw data requires interpretation. A first step is to assess the effect of volumetric changes in concrete due to temperature on the prestress force.

8.2.4.2.5 Possible acceptance criteria

The criteria listed below are given as examples. Acceptance criteria shall be based on the relevant regulations for each country and on regulator’s review and assessment.

It is recommended to establish a band of acceptable prestress levels similar to that illustrated in

Figure 0.4. A possible criterion proposed in [39] is not to exceed 8% of the initial prestressing force at a section after considering the short term losses (resulting from elastic shortening, anchorage take-up, and friction). The minimum criterion is to establish at the design stage a lower bound of the acceptable prestress force. Whatever the case, the comparison between readings and acceptable force should account for measurement uncertainties.

The owner should report any force monitoring output indicating a prestress force below the lower acceptable limit.

8.2.4.3 Monitoring for performance of bonded tendons

As the prestress force cannot be directly measured with current technologies in cement bonded tendons, their performances are assessed through indirect method.

Evaluating directly strand strain requires careful attention during instrumentation installation. There is little feedback information related to this technique (accuracy, drift, ease of use, reliability, robustness or possible repair during operation). For the reason, it is recommended to assess tendon prestress level by means of two types of instrumentation:

- Embedded strain meters (or stress meters if sufficient accuracy is demonstrated), set up in different sections of the concrete containment;
- Displacement measuring instrumentation (pendulum, plumb lines, “long range” extensometers, etc.) attached directly to the containment shell.

Embedded and “external” sensors should be used together. This redundancy will help to validate measurements by crossed controls and to confirm possible measurement error during test or over operating time.

The use of evaluation method based on strain measurement involves a good understanding of the contributing factors to the observed behaviour: elastic shortening, response under test pressure, shrinkage, creep and thermal strain.

Generally, the basic interpretation of the results relies on the devices located at sections away from structural discontinuities.

8.2.4.3.1 Location and quantity of instrumentation

Strain or stress meter and temperature probes should be installed in sufficient numbers at strategic locations and relevant direction in the containment structure so that loss in prestress levels can be assessed in an accurate and reliable way. The temperature monitoring is useful for isolating thermal effects and inferring delayed or elastic strains. The containment designer should be involved in the determination of the layout of the sensors. Moreover, sufficient redundancy in the instrumentation should be provided to evaluate anomalous readings and to isolate malfunctioning gauges. Currently, the selection of instrumentation types, locations and quantities are under the responsibility of the plant owner, but this task should be done in close association with the designer of the containment.

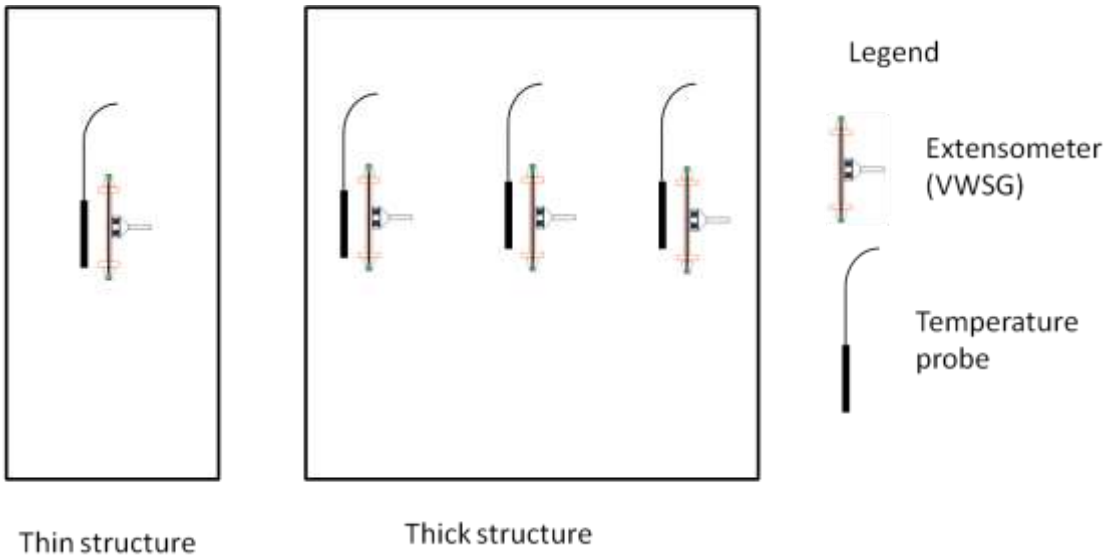
It is recommended to combine temperature probes (or thermocouples) with strain meters so that a temperature reading is available for all of the strain records.

It is recommended that the directions of the monitored strains match the directions of the prestressing cables, to get a better evaluation of the principal strains in the structure. The distribution of sensors through a section depends on its thickness and on the expected distribution of strain within the structure at this

section. Basically, the sensors will be set in the middle of a wall section. But if needed, one can considered that sensors are installed near the inner and the outer face of the wall (see

Figure 0.6).

Figure 0.6 Example of VWSG and temperature probe arrangement within a containment wall



These local measurement devices are complemented by other "external" instruments, such as pendulum, wire or inclinometer (tilt meter).

The minimum recommendations for the sensors quantities and locations are detailed hereafter:

The containment should be instrumented at different levels and in different areas: basemat, gusset, cylinder, ring girder, buttresses for prestressing, equipment hatch, large penetrations and dome (see

- **Figure 0.7** and **Figure 0.8**);

- The embedded sensors should be installed along at least 3 vertical lines of the cylinder, at 3 approximately equally spaced azimuths. These lines are extended to the dome and the basemat, and are away from structural discontinuities. They are complemented with other vertical measurement lines along buttresses (**Figure 0.8**);

- It is recommended to measure the "overall" (in contrast to "local" strain) displacements and deflection along the 3 vertical lines mentioned just above;

The basemat is instrumented at its centre and at different radii, as shown at **Figure 0.9**. It is recommended to have two layers of sensors within the thickness of the slab, in order to assess any possible deflection under settlement and test pressure (see

- **Figure 0.7**);

The dome and the ring girder are instrumented as shown at

- **Figure 0.7** and **Figure 0.9**;

The cylinder and the gusset are instrumented as shown at

- **Figure 0.7** and **Figure 0.8**;
- The equipment hatch is monitored through extensometers installed following the vertical and the horizontal axis.

Figure 0.7 Typical embedded sensors layout in a vertical cross section of a containment structure

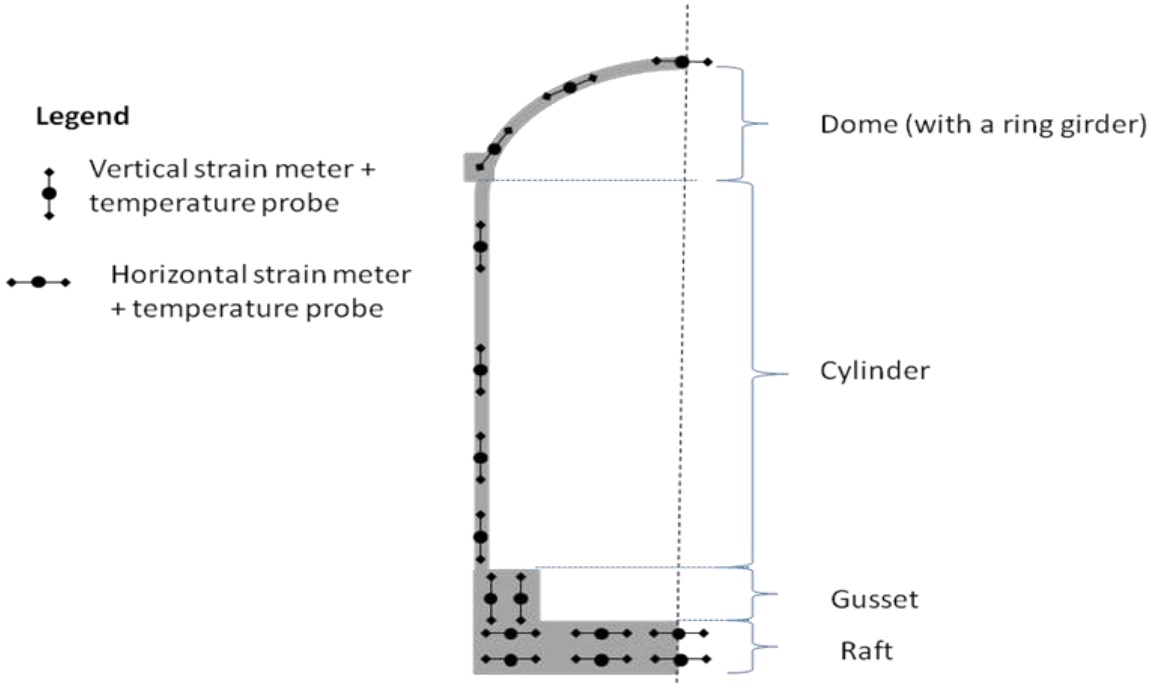


Figure 0.8 Typical embedded sensors layout on a developed view of the cylinder

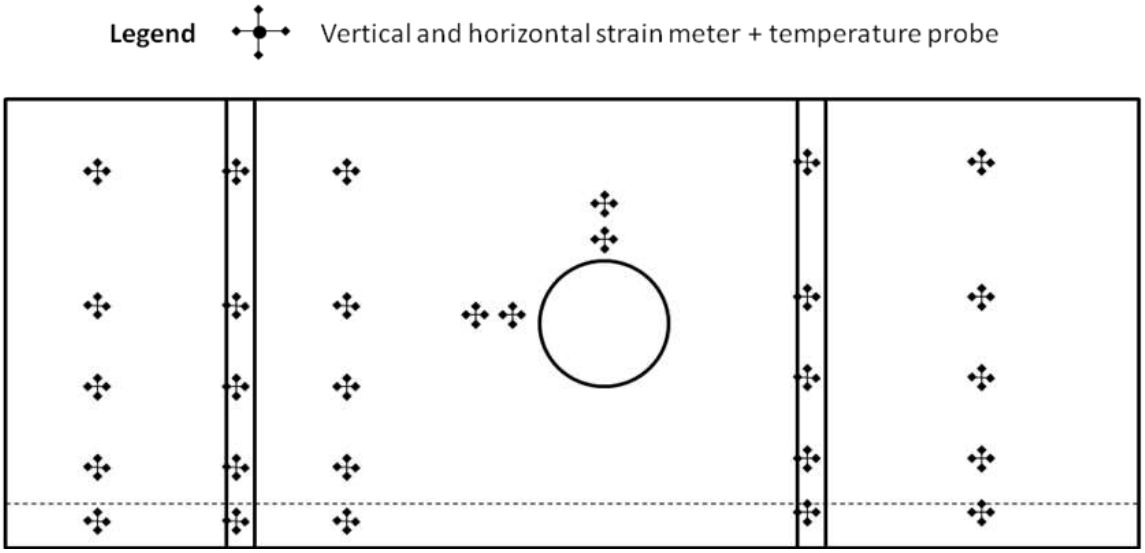


Figure 0.9 Typical embedded sensors layout in the dome (top view)

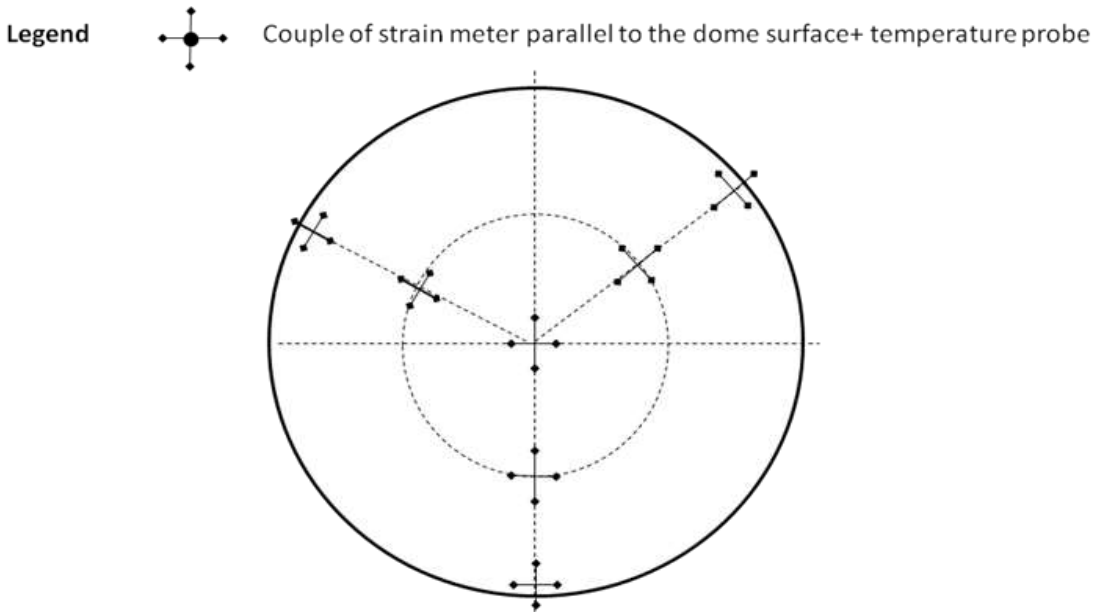
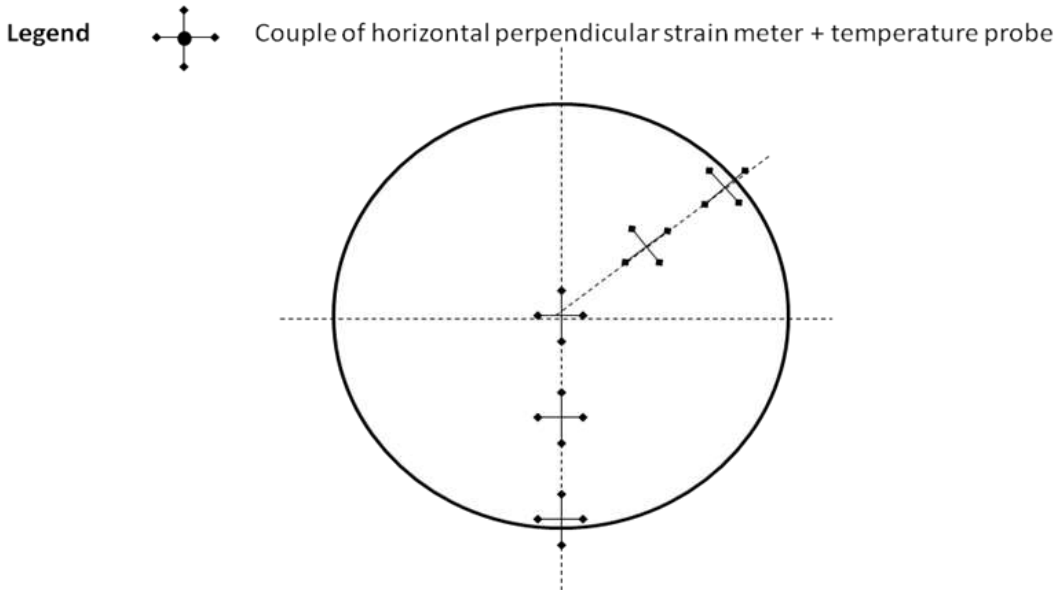


Figure 0.10 Typical embedded sensors layout in the raft (top view)



)

8.2.4.3.2 Frequency

The periodicity of measurement is to adapt to the monitored phenomena. The required frequncy for data acquisition is higher in test conditions than in operating conditions. Of course, a real time monitoring enhances the capability to manage the data acquisition and then allows a faster analysis of the behaviour of the containment in any condition.

Pressure test

10-year period is the maximum acceptable interval between two consecutive pressure tests dedicated to prestress level monitoring (ILRT program may include different time frames). During the test, it is recommended to acquire data at intermediate pressure levels, and not to limit the acquisition to P_A stage. Typically, records for $P_A/2$ level should be done during inflating and deflating of the containment.

Delayed strain assessment

It is recommended to adapt the measurement frequency to the phenomena, for example to creep and shrinkage rate. In normal operating conditions, the minimum recommendation to collect readings from all the monitoring system is:

- Every 3 months if the measurement are performed manually;
- Every 2 weeks if the data are automatically acquired.

8.2.4.3.3 Instrument specifications

The requirements over instrumental range and precision are derived from:

- The expected behaviour of the structure;
- The ambient conditions of the sensors;
- The criteria to apply;
- The provisions provided by the numerical calculations results with which the measurements would be compared.

Materials used in the strain or stress meters are specifically chosen to minimize or to self-compensate the temperature effects on the measurements. However, slight temperature effects can persist, and a temperature correction is to apply. The correction factor shall be specified by the manufacturer.

The regular maintenance checks of the monitoring system (involved in pressure tests preparation but also during normal operation) help to ensure satisfactory instrument performances are kept over time.

Some minimal guidance on range and accuracy of the measurement system (not only the sensors) is provided in Table 0.2 below.

Table 0.2 Minimum instrument specifications.

Phenomenon	Range of measurement	Accuracy of the measurement system
Pressure (for pressure test)	Including 0 bar_g and P_A, no more than $4xP_A$	+/- 2% of P_A
Temperature	-20°C / 60°C	+/- 1°C
Strain (ex: embedded strain meter)	About 3000.E-6	+/-5% of the maximum expected or observed value or 10.E-6, whichever is greater
Vertical displacement (ex: Invar wire)	80 mm	+/- 1 mm
Horizontal or radial displacement (ex: pendulum)	40 mm	+/- 1 mm

8.2.4.3.4 Data processing

After appropriate methods and instruments are employed for measurement, the measured data should be analysed to determine an average stress and an average prestressing force at a section.

The surveillance results obtained are related to the conditions, involving thermal conditions, in which the raw measurements were achieved. In complement to the direct temperature effect on the measurement system, the engineer in charge of the structure behaviour assessment has to evaluate accurately the reversible thermal volumetric changes of the structure (through the temperature readings), and to subtract them from the associated raw records.

Various procedures can be proposed for this thermal correction. The first and simplest approach is the thermo-differential correction method. The corrected values of strain ε_{corr} are derived from the raw strain ε_{raw} (deduced from readings) by using the following formula:

$$\varepsilon_{corr} = \varepsilon_{raw} + \alpha_s.(T-T_0) - \alpha_c.(T_m - T_{m0}) \qquad \text{Eq. 0.1}$$

With:

T: temperature recorded with the associated thermocouple at the time t (°C)

T_m: mean temperature of the wall (°C)

T_0: initial temperature recorded with the associated thermocouple (°C)

T_{m0}: mean temperature of the wall at the initial time (°C)

α_s: thermal expansion factor of the sensors (°C^{-1}). Typically, α_s ranges between 10 and 12.E-6 °C^{-1}

α_c: concrete thermal expansion factor (°C^{-1}). Typically, α_c ranges between 7 and 13.E-6 °C^{-1}

Other methods can be proposed. We can mention the EDF's practice, inspired by dam surveillance (see [28] pp 105-106). This method consists in a statistical data processing, based on a multiple linear regression analysis in which temperature and time are the explanatory variables.

If the measurements made on test beams are used as complement to the structural monitoring, those measurements should be included in the raw data processing.

When extrapolations are needed, different approaches can be undertaken: rather simple, relying on creep and shrinkage laws from standards [54], [48] or by finite element analysis [46].

It is recommended to provide a periodic report including the measurements and a diagnosis on the actual conditions of the containment, with a comparison between measurements and design criteria assumptions. The minimum outline of the report should include:

- The layout of the instrumentation;
- A description of the pressure test procedures, if any;
- A comparison between measured values and response predicted by analysis including tolerance and accuracy of both measurements and numerical model;
- An evaluation of any deviation (such as test results exceeding allowable limits) and need for correctives actions.

8.2.4.3.5 Possible acceptance criteria

The criteria listed below are given as examples. Acceptance criteria shall be based on the relevant regulations for each country and on regulator's review and assessment.

Limits of acceptable prestress levels, similar to that illustrated in

Figure 0.4, should be established for all the operation time of the plant.

The difference between strains measured during periodic pressure test and strains observed during the Initial Structural Integrity Test (ISIT) should not exceed an acceptable threshold. This limit may vary over the countries. The various sources of uncertainties should be taken into account in the data analysis.

The owner should report any measurement indicating a strain exceeding the pre-set acceptable limit.

8.2.4.4 Visual inspections

In addition to monitoring of the concrete containment structure using strategically located instrumentation, visual inspection of accessible areas inside and outside of the containment structure should be conducted periodically.

8.2.4.4.1 Location

Visual inspections should concentrate on critical areas, such as anchor regions of post-tensioning system, joints, areas of high stress concentration as identified by analysis. In addition, those areas where leakage was detected during ILRT and areas where problems were found previously should be visually inspected. Areas where repairs were performed should also form part of periodic inspection to ensure integrity of repairs (e.g. ASME XI section IWL requires “augmented” inspection for such areas).

8.2.4.4.2 Frequency

It is prudent to perform visual inspection during every outage. As a minimum, periodic inspection of concrete containment structure are typically performed every 5 or 6 years, (e.g. [2] recommends visual inspection of safety related structures to be performed every 5 years, while [16] has a minimum requirement of 6 years between inspections. For seaside sites, it is recommended to adapt the frequency to the risk of corrosion which is more likely to occur in a more aggressive atmosphere.

It is beneficial to perform visual inspection of the external surfaces of the containment structure during the Proof Pressure Test and Integrated Leakage Rate Tests in order to observe the behaviour of the structure under pressure. Detection of the leak paths associated with concrete structure or its components may be associated with cracks, areas of unconsolidated concrete or other discontinuities or defects in the structure.

8.2.4.4.3 Specifications

Some useful guidance for conducting a visual inspection of concrete structures: definitions, illustrations, checklist for routine inspection, etc. is provided in [1].

The system should be able to detect and characterize the crack patterns on the visible concrete surfaces of the containment. Remote systems should be considered in case of difficult access of areas by the inspectors.

Depending on the configuration, remote visual inspection should be considered to monitor the critical parts of the containment which are not easily accessible.

Pictures with appropriate resolution should be taken of the main cracks and visible defects, in order to record the actual conditions and to get a relevant database for the patterns evolution. Crack maps should be created.

Cracks and cracks patterns should be reported based on the observed lengths, spacing and widths. The scope of the concrete conditions to report has to be defined prior to the inspection. For example, any cracks with a width greater than 0.3 mm or a length greater than 200 mm may be considered as reportable.

In complement to cracks, defects such as stains, spalling, pop outs, corrosion marks and visible rusted rebar may also be reportable.

During pressure tests, the widths of the main cracks should be monitored with temporary displacement sensors with adequate accuracy (less than 0.1 mm).

8.2.4.4.4 Data processing

The reportable cracks and defects should be mapped to provide an overview of the containment conditions.

Basic data processing can be considered to assess the cracks or defects distribution and their evolution over time.

8.2.4.4.5 Possible acceptance criteria and reportable situations

The criteria listed below are given as examples. Acceptance criteria shall be based on the relevant regulations for each country and on regulator's review and assessment.

Guidelines provided by [2] for acceptance criteria for nuclear safety related structures can be applied for containment structure. However, these are generic and more precise criteria may need to be established so that functions of the structure as well as exposure conditions of a particular part of the containment structure are taken into consideration.

[2] provides tiered approach; if considered necessary based on results of visual inspection, other techniques, such as non-destructive / destructive tests or analysis may be used to identify the cause and extent of the degradation.

The following situations should be considered as reportable by the owner of the plant:

- At the structurally critical areas, any significant decrease in the spacing or any significant increase in the widths of cracks compared to those observed during the ISIT at zero pressure after depressurization;

- Any significant degradation or movement of the anchor hardware.

8.2.4.5 Corrosion detection

Heat-treated steel is sensitive to stress-induced corrosion, which may lead to localized, non-ductile failure of the tendon. Main functions of the grout are to protect prestressing steel from corrosion and to bond the tendons to the surrounding concrete. Due to the bond between tendon and grout, bonded tendons are less vulnerable to local damage than unbonded ones because if tendon breaks, part of the prestress remains transmitted to the concrete.

The possibility of tendon corrosion is low if the grout and concrete surrounding tendons are in good condition. The presence of voids within the tendon duct might be due to blockages, inadequate grouting procedures, grouting material problems, and construction oversight. Inadequate grouting may not fully protect the tendons against corrosion and hence reduce the durability, possibly causing the loss of the cross-section and breakage of the tendons. In grouts with high water/cement ratio, bleeding may occur, causing formation of pockets of water, particularly in the elevated areas of the prestressing ducts. High operating and ambient temperatures will tend to facilitate corrosion once it is active.

Techniques for detection of corrosion in post-tensioning tendons are available as discussed below and have been successfully used on civil structures (e.g. bridges). However, corrosion detection methods are not widely used in nuclear industry. Large areas of the containment structure would have to be covered to perform thorough investigation. If investigation is performed locally, detecting corrosion would raise concern for the rest of the structure; however, if no corrosion is detected it would only be applicable to the area tested. With instrumented monitoring, local capacity reduction would be detected by the sensors of sufficient accuracy that are located strategically in appropriate quantities. Then corrosion detection methods can be applied to further investigate identified location. Corrosion detection can also be used if there is a concern with the quality of grouting.

Ultrasonic tomography is successfully used to detect voids in the grout of the post-tensioning system. It is based on the ultrasonic echo method. The transducers applied to the concrete surface introduce low frequency pulses of shear waves. A 3-D tomographer system with transducers sending and receiving the reflected pulse is able to scan and detect the voids in the grout at a distance of up to about 0.5 m from the surface depending on the amount of reinforcing steel (i.e. heavy reinforcement reduces “visibility”).

Post-Tech® Corrosion Evaluation method used by Vector Corrosion Technologies is based on the fact that the possibility of corrosion depends on relative humidity at the location of tendons. The method is partially destructive as small holes (about 16 mm diameter) need to be drilled into the structure to access the tendon sheath. The dry gas is injected at a low pressure at an inlet. Measurements of relative humidity and temperature are taken at the outlet port(s) to determine possibility of corrosion. Other methods for corrosion detection in bonded prestressed concrete are described elsewhere (see [44] and [42]).

8.2.5 The French practice as an illustration

The purpose of this section is to show an example of existing monitoring system currently used for containment building with bonded prestressing tendons. Obviously, it does not intend to enforce such a system for all the operators and all the countries.

In France, EDF operates 58 PWR originally derived from Westinghouse design. Based on its experience in the monitoring of large dams, EDF have applied a generic surveillance system which aims at measuring the following parameters:

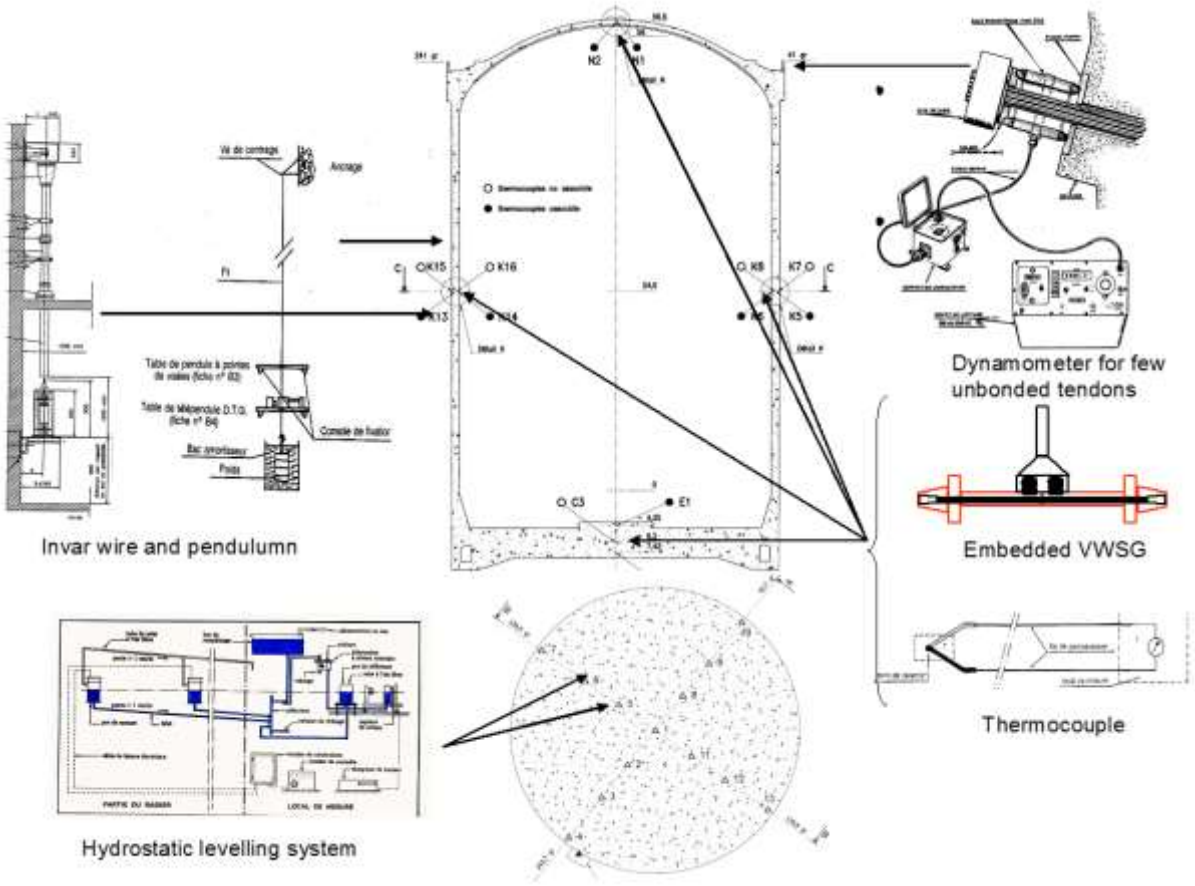
- Horizontal displacements of the structure, along 4 vertical lines of the cylinder (devices: pendulums);
- Vertical displacements, along 4 vertical lines of the cylinder (Invar wires for the cylinder, levelling pots for the raft);
- Local strain measurements, in vertical and horizontal directions, at various locations inside the prestressed concrete structure (embedded extensometers, such as VWSG and associated with temperature probes, such as thermocouples);
- Force in the tendons (dynamometers on 4 vertical unbonded tendons).

Pendulums provide measurements of the overall deformation of the containment whereas extensometers readings give information on local measurements. Theoretical relationships between both may be useful to make a diagnosis.

A sketch of the monitoring system is shown in

Figure 0.11 below.

Figure 0.11 Illustration of the sensors used by EDF for its containment monitoring (EDF)



8.2.5.1 Force monitoring system of the 4 unbonded tendons

4 unbonded vertical tendons are fitted with dynamometers to monitor transfer losses and tension losses throughout the NPP operating life. The choice to limit the monitoring to vertical tendons has been made due to the rather crude precision induced by the deviations of the non-vertical tendons (uncertainties concerning the friction behaviour of the grease or wax along the time).

The dynamometer measurements are compared with theoretical calculation to validate or to calibrate the effective relaxation law of strand steel. Once the relaxation law determined during the first years of operation, it can be used to assess the associated time-dependant prestress losses.

In France, no periodic measurement throughout the life of the plant is required (dynamometers are not included in the OSS). Actually, some containment structures have lost all their dynamometers, and the force monitoring is no longer applied. However, lift-off tests on these tendons remain an option if necessary.

8.2.5.2 Monitoring for performance of bonded tendons

The containment is fitted with embedded and external sensors for the monitoring of the delayed strain (which enables the monitoring of the prestress level) and for the pressure test (related to the overall integrity of the structure and its evolution over time).

The layout of the sensors is driven by some basic principles: adequate instrumentation performance, robustness, redundancy, diversity of techniques, ability to interpret the measurement in term of structural behaviour. From a metrological point of view, it would be required to be able to check and calibrate periodically all the sensors. For embedded sensors, accurate calibration is no longer possible, of course. But long term experiments have been undertaken in controlled conditions to determine the possible instrumental drift or other uncertainties sources for the VWSG embedded in containment concrete.

EDF takes advantage of the standardization of its nuclear fleet for the monitoring system as well. A “first-of-a-kind” containment may include about 400 to 500 embedded sensors, the subsequent plants requiring fewer devices, due to the acquired feedback from the prototype. To fully benefit from feedback, a similar layout of the sensors applies among the different plants of a given series of plant.

The sensors are strategically placed within the structure:

- In areas where the expected behaviour is representative of the overall response of the containment and where the interpretation of measurements is relatively simple (mid-height of the cylinder, centre of dome);

- In areas where some tensions can occurs due to bending induced by pressure (gusset, cylinder-dome junction, near the large penetrations, etc.).

If needed for dedicated analysis, other sensors can be added, such as inclinometers to monitor the structure rotations in the areas which can undergo significant bending (gusset, dome ring girder). The displacement between the flange of the equipment hatch and the surround concrete has also been measured during pressure test in specific plant, where data were needed to feed numerical analysis or to assess the effectiveness of maintenance operation (bolt replacement, for example).

An example of possible layout for pendulums, invar wires, dynamometers, VWSG, thermometers and levelling pots for a standard case is shown in

Figure 0.12 below.

Figure 0.12 Example of a typical monitoring system used by EDF (sensors installed around penetrations are not represented)

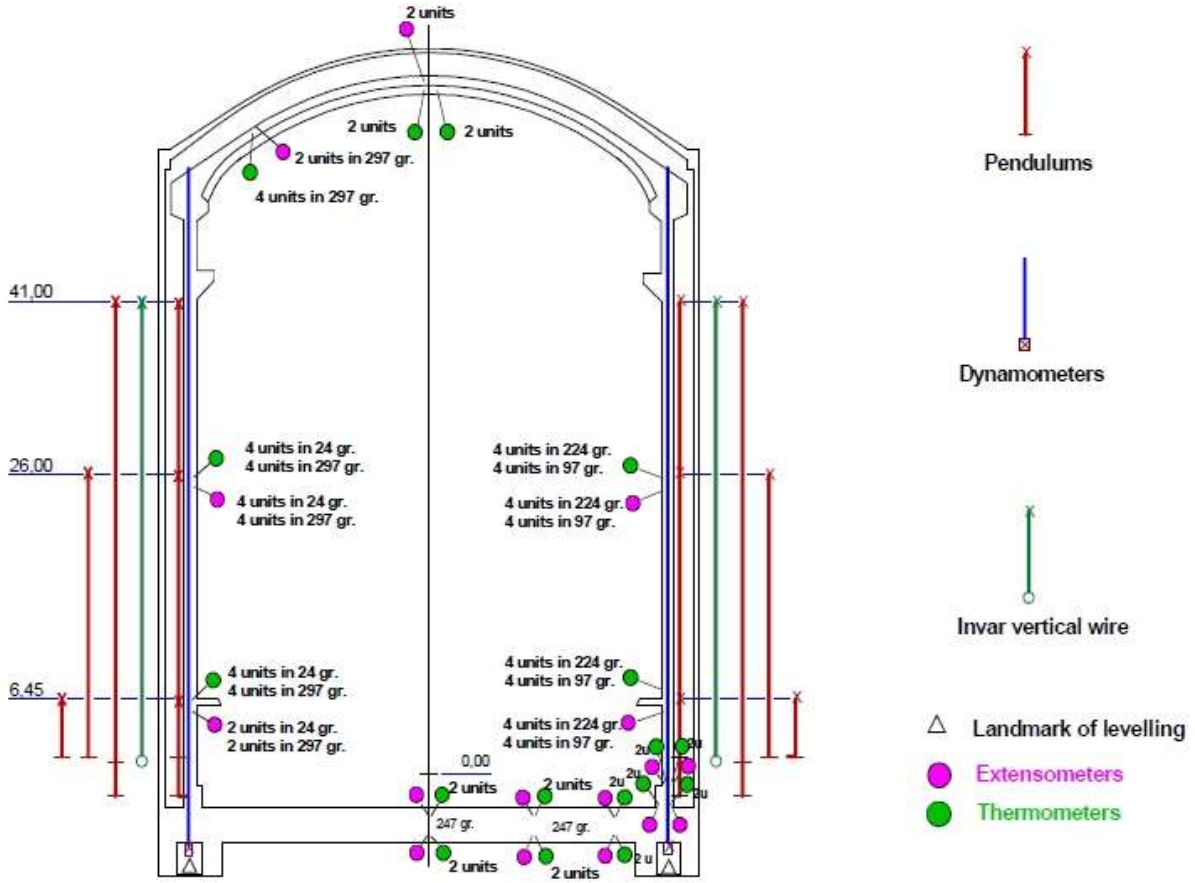


Table 0.3 below provides a list of the devices used for containment monitoring, with their specifications and locations.

Table 0.3 Example of specifications for sensors included in EPR Flamanville 3 containment.

Sensor	Monitored zone	Number	Property measured	Comments
Dynamometers	tendons injected with grease	4	Cable tension	Vibrating wire technology (Telemac or Geoinstrumentation)
Embedded strain gauges	Foundation raft	30	Local strains	VWSG (Telemac C110) Sensitivity = 0.35 µm/m Range = 2900 µm/m
	Gusset plate	72		
	Cylinder wall	114		
	Ring belt	30		
	Dome	39		
	Equipment hatch	12		
Vertical invar wires	Cylinder wall	3	Vertical deformation	Placed on 3 vertical axes:
Pendulums	Cylinder wall	9	Vertical displacement	Placed on 3 vertical axes
Levelling pots	Foundation raft	17	Settlement	Water level type. 2 * 9 pots on the diameters 20°-200° and 60°-240° corresponding to the radii: 23m, 20m, 12m, 3m and the centre.
Temperature sensors	Coupled with levelling pots (foundation raft)	17	Temperature	PT100 probe
	Foundation raft	10		
	Gusset plate	24		
	Cylinder wall	55		
	Ring belt	15		
	Dome	20		
	Equipment hatch	6		

The rate of failure encountered on EDF containment is about 1% per year for the embedded VWSG, and less for the temperature probes. In some cases, advanced measurement methods enable to recover weak or disturbed signals from the sensors [56] and [55]. When it is no longer possible to retrieve measurement from embedded sensors, EDF has defined a suited instrumentation dedicated to the long term monitoring of the containment, called in French “Dispositif d’Auscultation Optimal” (DAO) or “Optimal Surveillance System” (OSS). The OSS shall be maintained throughout the life of the structure. It includes on one hand the sensors which deliver global displacement and which can be replaced (such as pendulum) and, on the other hand, the sensors dedicated to local phenomena, initially embedded in concrete (such as VWSG) and thus inaccessible (see Table 0.4). An equivalent system is then required to maintain the OSS in case of failure of an embedded sensor. For that purpose EDF has developed new devices.

The OSS includes the following embedded sensors:

- 2 vertical and 2 hoop strain gages located at mid-height of the cylinder. To get a rather simple interpretation and evaluation of the results, the instrument is located at sections away from structural discontinuities (hatch, buttress, etc.);
- 2 strain gages at the centre of the dome, in two horizontal perpendicular directions;
- 1 vertical strain gage in the gusset (thick junction between raft and cylinder), available at least on one plant of a site.

Table 0.4 Sensors included in the OSS of the EDF's NPP fleet.

Quantity	Original monitoring instruments	Part of the OSS	Replaceable nature of the equipment
Displacement	Survey markers	YES	YES
Displacement	Plumb lines and Invar wires	YES	YES
Strain	Vibratory strain meters embedded in the concrete	YES	NO
Temperature	Thermocouples	YES	YES

A maintenance plan is carried out throughout the operating life of the plant. The systems that can be inspected (such as pendulum or Invar wire, but also data logger and cabling) are regularly checked and calibrated with specific procedures by experienced technicians, with significant skill in measurement devices. The usual period for calibration is one year for any measurement device, but thanks to the feedback gained over the hardware and the sensors, the periodicity has been extended up to 4 years.

For normal operation period, the standard frequency of acquisition ranges from 15 days to 3 months, depending on the acquisition system and on the rate of observed deformations. During pressure tests, the data measurements can be performed every hour, with mandatory stages (baseline before inflating or beginning and end of pressure plateaus, for example).

The acquired data are collected by specialist technical team, which is in charge to check the validity of the measurement, from a metrological point of view and by comparing the new records with the overall trend of the time history curves. If a reading is considered as an outlier, a new measurement process is launched. If the reading is accepted from a metrological point of view but denotes an unexpected behaviour of the structure, it gives rise to an alert, and further analysis and/or monitoring are undertaken to better understand the observed phenomenon and to assess the impact on safety.

Whatever the behaviour of the containment, a monitoring report is issue every two to five years, depending on the observed strain rate. The purpose is to propose an overview of the behaviour, to check if the observed deformation and temperature are consistent with the prediction and to state a structural health diagnosis.

For the data processing relative to strain or displacement, the measured temperatures are used to remove the effect of thermal dilation (seasonal variation for example) from the measured total strains. The following figures (**Figure 0.13** and

Figure 0.14) provide an illustration of the thermal effect on strain measurement and on the subsequent data processing by the temperature records.

Figure 0.13 Example of concrete strain time history, with raw data affected by seasonal effects – vertical strain in blue, hoop strain in red (EDF)

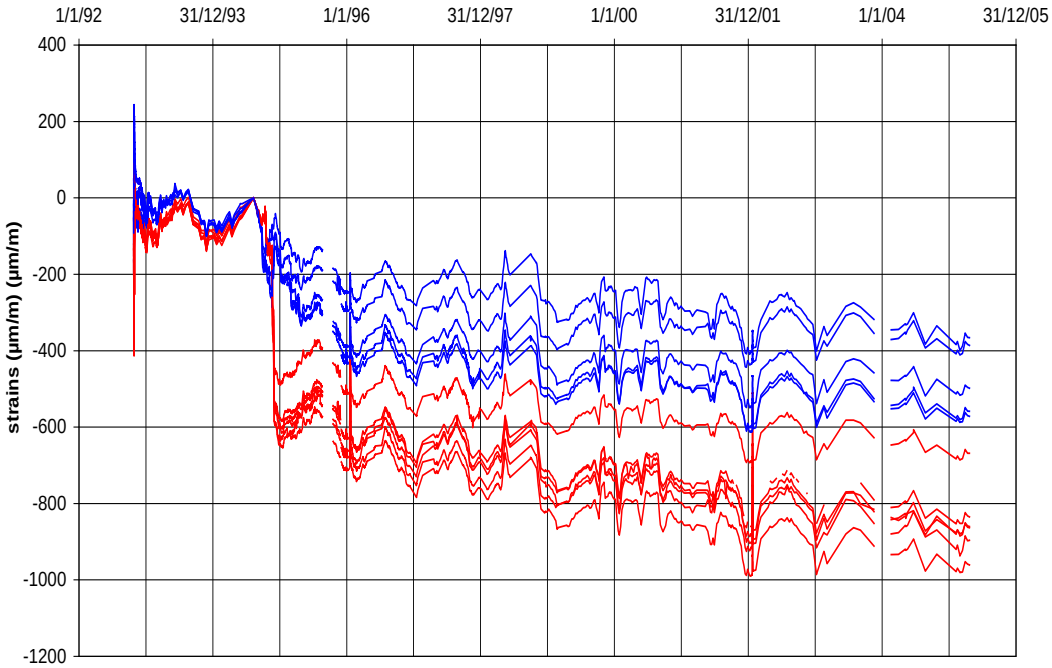
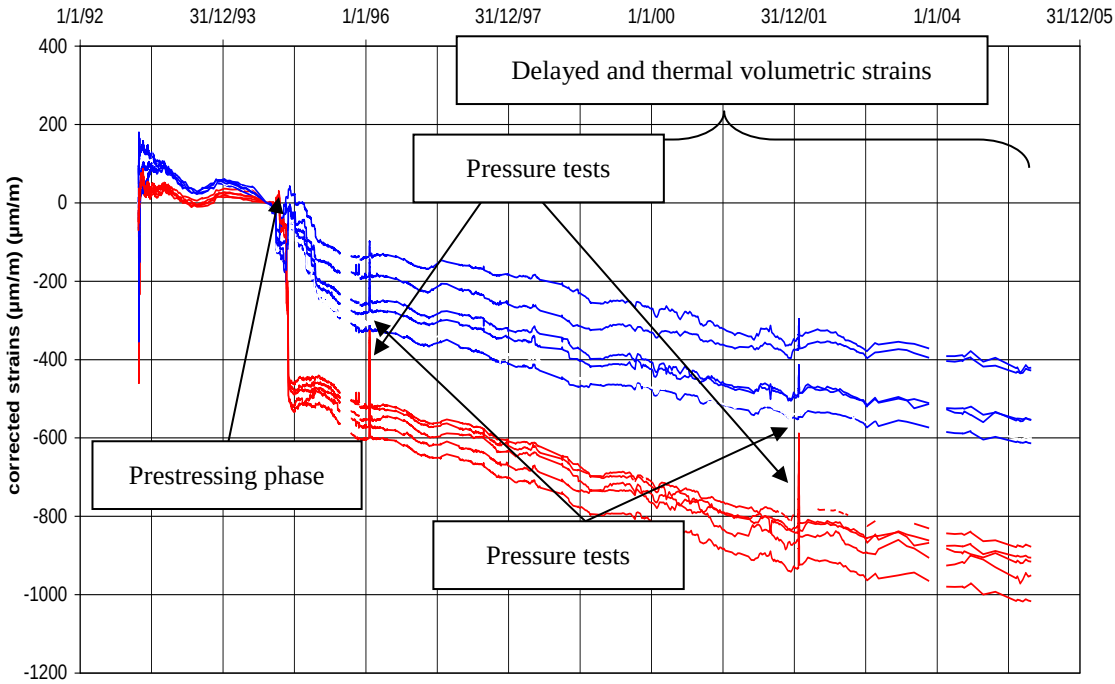


Figure 0.14 Same data as Figure 0.13, after data processing– vertical strain in blue colour, hoop strain in red colour (EDF)



To go further in the data analysis, it is necessary to use models, from simple equations to more complexes structural models (FEM), in order to assess the theoretical behaviour. The comparison between modelling results and monitoring data aims at determining some parameters, such as concrete stiffness (Young modulus), or at predicting the evolution of delayed strain until the end of the operation time. The consistency between previous model predictions and updated monitoring data is checked regularly, typically when a monitoring report is issued.

During pressure test, linearity and reversibility of the strain curve against pressure shall be checked in the tension zones (typically, mid height of the cylinder or centre of the dome). Sometimes, due to short term creep of concrete, the reversibility criterion is not met just after the come back to the normal pressure but some hours or days later (see examples in

Figure 0.15). In some areas of the containment, where bending and tension can occur during pressure tests, non-linear strain curves may be observed, but, in most cases, the phenomenon is reversible and the cracks openings are limited (see **Figure 0.16** for an illustration).

Figure 0.15 Typical strain-pressure curve observed during a containment pressure test at mid height of the cylinder

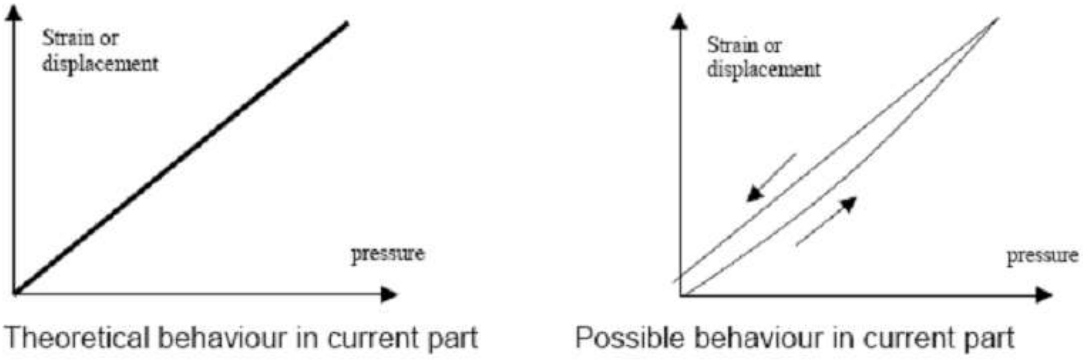
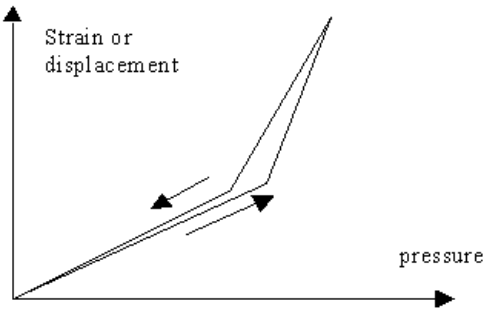


Figure 0.16 Synthetic example of strain-pressure curve during test, with a reversible non-linear behaviour



The possible variations of local strains over the different tests are assessed to lead to an overall diagnosis.

8.2.5.3 Visual inspection

Just before a pressure test, an overall examination of the surface of the containment is undertaken with respect to cracks and defects. At peak pressure stage, visual inspections will focus on the most critical areas, with monitoring of the longest (longer than 1 meter), widest cracks (wider than 0.3 millimetres), and detection and mapping of any significant defect (spalling, delaminations, etc.). After completion of the test, a second examination is carried out, with special attention given to checking the propagation and closure of cracks. Residual defects are mapped and recorded, to give a reference state for later tests.

Any unexpected observation will be reported to the Design Authority of the Operating Company and to the Regulator, and further investigation and remedial actions will be taken as necessary. Further testing may be required to check the effectiveness of any remedial actions.

In addition to the Periodic Tests, 5-year period visual examinations are also carried out to monitor ageing of concrete (defects and cracks) and steel (corrosion of rebar). In these tests comparison is made with results of previous visual inspections.

8.2.6 Further needs

Reliable and efficient system that would allow the direct measurement of the force of a bonded tendon would be considered as a significant progress. Some research works are currently performed in this direction [50].

Monitoring system that would allow an enhanced multiplexing capability would help to get a more accurate overview of the containment behaviour. In this respect, distributed fibre optics is a rather promising technique [49].

Shrinkage and creep of concrete, as well as rebar corrosion and alkali aggregate reaction are related to water content variation in concrete. Moreover, evaluation of water content in concrete members used for biological or radiological shield are needed for safety analysis of beyond design accidental conditions. Further research for concrete water content monitoring can help for that purpose [45].

For crack monitoring and visual inspection, a valuable support could be provided by the use of ultrasonic sensors, acoustic emission or digital image correlation.

Whatever the novel techniques, it is recommended to assess it by the procedure proposed in [8].

8.2.7 Conclusions and recommendations

Although, direct monitoring of the bonded post-tensioning system is currently not possible, proven reliable indirect methods can be used to ensure continuous integrity of the post-tensioning system over the service life of the plant.

Preferably embedded or otherwise attached (retrofitted) instrumentation should be used to monitor the response of the concrete containment structure to external loads (i.e. pressure loads during PPT and ILRT). Where possible, a few tendons should be left unbonded to allow for direct monitoring of prestressing force.

For new structures, data collection should start during construction (at least at the beginning of the tensioning of the cables) so that the initial state, i.e. baseline parameters could be established at the end of the construction period.

Instrumentation should be proven, calibrated and reliable and should have sufficient redundancy.

Instrumentation should be of sufficient accuracy to be able to identify failure of small enough number of strands so that the integrity of the concrete containment structure is not compromised.

Instrumentation should be strategically located in the areas of high stresses and deformations; location of instrumentation should allow for easy comparison with the theoretically estimated values.

Care should be taken when interpreting the results of the monitoring. Response of the structure to environmental and operating conditions (e.g. temperature, humidity) as well as volumetric changes in concrete and long term relaxation of the post-tensioning tendons in the given environment should be well understood in order to confirm design assumptions and ensure that prestressing force losses are within acceptance limits and sufficient margins are maintained at all times during service life of the plant. If necessary, structural analysis should be updated with actual ambient conditions and material properties.

The methods of testing the beams cast during construction of the containment structure can be used to ensure integrity of the post-tensioning system; however, limitations of this method should be well

understood and considered. It is recommended to use this method in conjunction with monitoring instrumentation method and visual inspection.

Visual inspection of accessible areas of concrete containment structure should focus on critical areas, such as, for example anchorage regions, construction joints, areas of stress concentration as well as those areas where leakage is detected during ILRT.

It is recommended to encourage and support any R&D effort to develop new devices that are able to measure directly prestressing force in bonded tendon.

8.3 GSS technology

To this day, international standards do not propose an ISI program specifically adapted to GSS tendons. The available documents only include prestressed containments with tendons injected with grease or wax (ASME section XI, RG 1.35, etc.) for which local measurements of the tension in tendons are generally carried out, and prestressed containments with bonded tendons (ETC-C, RG 1.90, etc.) for which a follow-up program on overall behaviour of the structure is generally carried out.

Therefore, the purpose of this document is to present guide lines allowing the definition of an ISI program for a containment consistent with GSS tendons specifications, and thus ensure the structure's durability over of its operation time.

The recommendations presented in this document can be applied to a containment prestressed entirely or partially with GSS tendons, with a torospheric or hemispheric dome (proposal not yet implemented).

8.3.1 GSS tendon specifications - Inspection principles

An ISI program of a prestressed containment with GSS tendons must allow:

- The analysis of the evolution of the structure's mechanical behaviour in relationship with the main phenomena of ageing (shrinkage, creep and relaxation) during its operation time,
- To carry out an extrapolation of the containment's behaviour at the end of its operation time, in order to plan, if needed, an adapted maintenance process (tendons retensioning operation, for example),
- To check the presence of specific pathologies (corrosion, cracking, delayed ettringite formation (DEF), alkali aggregate reaction (AAR...) and deterioration.

The evolution of the containment's mechanical behaviour during its operation time is directly linked to prestressing losses induced by the shrinkage and creep of the concrete and the relaxation of tendons. These phenomena are taken into consideration during the design stage, and are taken into account during prestressing design (determination of the unit capabilities and tendons spacing). However, as GSS tendons are unbonded, they can be retensioned. Thus, in the event of shrinkage or creep higher than the values considered during design (a typical concrete behaviour) and relaxation of tendon, a retensioning operation can be planned. Thus, the containment is not critical in terms of the plant unit's operation time.

In terms of corrosion risk, which can lead to ruptures of wires or strands, GSS tendons have particular features that render them less sensitive to these phenomena. GSS tendons benefit from more effective protection from corrosion than tendons injected with grease, wax, or cement grout. GSS tendons have the particular feature of being composed of greased strands individually sheathed at the manufacturing stage by a layer of extruded high density polyethylene (HDPE). All strands are then gathered into a general duct

made of thick HDPE or steel, which totally isolates them from the environment. Thus, their protection from corrosion comes from both the individual HDPE sheath and the grease as well as the injection of cement grout in the general duct providing a sealed barrier completely surrounding the peripheral surfaces of the individual sheaths. This protection is provided starting from the manufacturing of the strands until the implementation on the site, then during the entire operational phase of the structure.

However the GSS prestressing system presents an area not better protected against the corrosion risk than the other prestressing system: the anchorages and tendons excess lengths have to be protected with a means of protection appropriate and sustainable.

Generally, the pathologies associated with problematic corrosion of tendons are induced by:

- Poor storage conditions during the construction phase (tendons injected with grease, wax, or cement grout),
- Migration of grease during the operation time (tendons injected with grease).

However, concerning GSS tendons:

- Strands are not directly in contact with the environment during storage and threading phases (protection provided by grease and individual sheath).
- During the operation stage, the general duct injected with cement grout and the individual strand protection, provides total isolation from the environment.
- The tendon ends (excess length) are protected by a long cap injected with grease or wax, offering protection comparable to tendons injected with grease or wax. Therefore, all caps can be inspected during the overall operating life of the plant unit.

Note that the phenomena of stress corrosion, not dealt with here, which mainly depends on the grade of steel provided, is mainly independent of the selected technology.

8.3.2 Proposal of an In-Service Inspection (ISI) program

8.3.2.1 Direct measurement of prestressing force

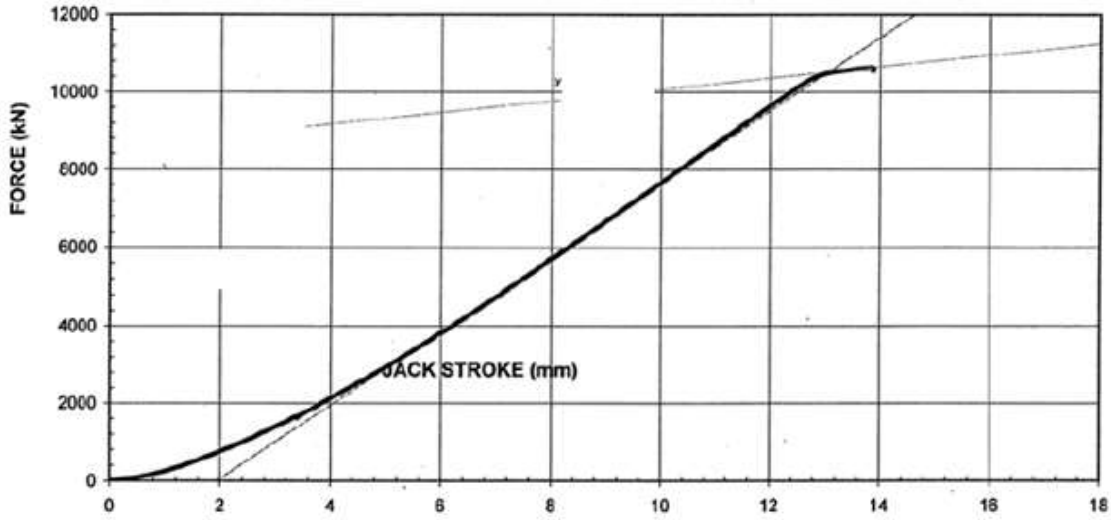
Two main ways can be used in order to perform the direct measurement of the prestressing force. The measurement is either directly measured at reference tendons equipped with dynamometers (load cells) or other measured by lift-off in order to:

- Measure the relaxation of tendons and isolate the part attributable to creep and shrinkage,
- Verify consistency between measured prestressing losses and design hypotheses.

Note that the purpose of a lift-off is to verify the force of a tendon after it has been stressed. This test may be conducted with a hydraulic stressing jack by lifting the anchor head off the bearing plate to determine the residual effective force in the tendon at the anchorage (see

Figure 0.17 for an example of a force/displacement curve obtained during a lift-off test).

Figure 0.17 Example of a force/displacement curve obtained during a lift-off test



In order to limit any measurement disturbances induced by friction, it is preferable to consider non-deviated GSS tendons (ducting not affected by a close penetration) as reference tendons. Based on RG 1.90, it is recommended to consider reference tendons for each group (depending on the design):

- 3 pure vertical tendons,
- 3 horizontal tendons,
- 3 dome tendons,
- 4 gamma tendons,
- 3 inverted U tendons (hemispheric dome).

If the measurement of force in the reference tendons is performed by lift-off or equivalent test, sufficient precautions must be taken in order to guarantee that these tension measurement operations are not detrimental to the tendons. Therefore, it is preferable to carry out lift-off operations of complete blocks (threaded blocks) with a dedicated annular jack in order to limit block displacement, necessary for the measurement.

Lift-off test frequency is to be determined depending on the prestressing systems implemented as well as the available experience feedback. Prior qualification tests on models can also be performed.

For the site's first plant unit, it is recommended to equip at least 4 vertical tendons (or inverted U tendons) for the Initial Structural Integrity Test (ISIT). This provision allows for regular follow-up of the relaxation phenomenon, for at least the first three years of operation in order to have an ample database on the deferred behaviour of GSS tendons.

If the structure considered is prestressed with both, GSS tendons and tendons injected with cement grout and it lacks non-deviated vertical GSS tendons able to be measured, only 4 vertical tendons (or inverted U tendons) should be left ungrouted so that it could be equipped with dynamometers. The ducts for these tendons shall be filled with grease in order to limit the number of tendons with different prestressing technologies in the structure.

8.3.2.2 Overall monitoring of the containment's behaviour

A monitoring system of the containment must be implemented in order to analyse the evolution of the overall behaviour of the structure in operation stage and during tests.

The monitoring device will be composed of a redundant and diversified instrumentation in order to allow:

- Verifying the containment's behaviour is consistent in comparison with the design hypotheses,
- Verifying the evolution in the containment's behaviour over time, in operation (evolution of deferred deformations, localised deformations) and during the tests (module evolution, localised deformations, linear and reversible behaviour).

Thus, the monitoring system will consist at least of:

- Pendulums (variation in diameter, inclination)
- Invar wires (elongation, overall deformations),
- A sufficient number of extensometers and thermocouples placed in specific points of the containment, far from discontinuity areas (in the cylinder wall and in the dome).

The measurements and interpretations are carried out with an appropriate frequency that can evolve with time, depending on the kinetics of the phenomena to study during the operation time. Based on RG 1.90 (alternative A), it is recommended to perform a measurement every 2 months until the first test, then every 6 months after. In the case of specific needs, this frequency can be increased.

All of the sensors included in the monitoring system, necessary and sufficient to optimally follow-up on the structure's behaviour during its entire operating life and to produce any necessary justifications, must be defined. The functionality of the whole reduced number of sensors included in the optimal monitoring system (called “OSS”, in ETC-C- Part 3) must be ensured during the structure's entire operating life (cf. §”Monitoring for performance of bonded tendons”). Additional sensors can be added to follow the structure's behaviour at a young age. However, the loss of sensors will not have an impact on the follow-up of the structure's behaviour. Therefore, they won't necessarily be replaced, contrary to the sensors included in the optimal monitoring system. These will need to be installed by verifying, as much as possible, the redundancy and diversification of measurement means.

Thus, in the case of inoperative embedded extensometers, included in the OSS, during the structure's operating life, they can be replaced by surface mounted extensometers, commonly used for substitution.

8.3.2.3 Tests

This document does not deal with test cases (independent of the type of prestressing technology) which must be carried out frequently and at a sufficient pressure level. During the pressure tests, the mechanical behaviour of the containment will be analysed based on a reinforced monitoring system (cf. §0).

8.3.2.4 Visual examinations

Visual examinations must be regularly performed on the containment's surface (regular and specific areas) and prestressing anchors (caps and tendon heads).

- Periodical visual examinations of the containment surface must be performed (during operation and pressure tests) in order to detect any deteriorations of the concrete (flaking, cracking) in the common area. The most critical areas will also be inspected, particularly, the specific areas or those under important solicitation, such as the areas near the prestressing anchors or the main deviations (near air locks, ribs).

- Periodical visual examinations of the prestressing anchor caps must be performed in order to detect any pathology such as abnormal caps deformations, grease leaks, corrosion or other deteriorations.

In this event, a few caps will be removed in order to inspect the anchor head and also to verify the lack of corrosion marks, abnormal ageing or other deteriorations. The tendon samples subject to cap removal will have a minimum of 4 caps per group of tendons, based on rules of RG 1.35, ASME section XI IWL, concerning random selection of tendons to inspect. Thus, all the prestressing anchors must remain accessible during the plant unit's entire operating life.

The frequency of visual inspections could be carried out 1, 3 and 5 years after the ISIT, then every 5 years (based on RG 1.35).

In the field of bridges industry, GSS tendons are often used for internal or external prestressing. Inspection programs are defined to check the integrity of the prestressing system. The main practices implemented are:

- Measurement of the prestressing force with load cells set in anchorage blocs or using magnetostrictive sensors,

- Direct measurement of the strands stress by transversal pulling of the strand when the tendons are accessible,

- Lift off test,

- Permanent acoustic monitoring on specific area,

- X-ray or gamma-ray control to check the duct filling and detect strands failures,

- Ultrasonic test to detect defects on strands near the measurement point,

- Visual examination of the anchorages areas.

8.3.3 Assessment and outlook

This document provides elements which define an in-service inspection program for a prestressed containment with GSS tendons, based on the adaptation of principles described in the ASME section XI, RG 1.35, RG 1.90 and ETC-C. Thus, alternatives are proposed in order to avoid requiring operations inappropriate for GSS tendons such as:

- To require to remove entirely a tendon. Pulling out the strands of the wedges, necessary to carry out this operation, can induce significant excess stress in the tendon (low tension losses of GSS), which could lead to limit the initial tension (penalising for the design) or to change the whole tendon (penalising for the operation). In addition, protection against corrosion of the strands which have been re-threaded can be not as efficient as before, because the grease might be less well spread along the strand than in the initial situation (operation carried out in the factory during the manufacturing stage).

- To carry out lift-off operations with pulling out the strands of the wedges. It can induce excess tension in the tendon, inaccuracies in force measurement, initiate strand rupture in the case of a wedge tightening in an area near the initial position. Therefore, this operation is to be avoided to measure tendon force. However lift-off operations of complete threaded blocks can be carried out without impact on the design.

- To carry out stress measurements directly on the strands. This seems to present difficulties in terms of setting up sensors and durability.

The elements proposed in this document fitted to containments with GSS tendons, are based rather on the requirements for containments prestressed with bonded tendons than unbonded. If a sufficient level of knowledge is reached on the GSS prestressing system and on non-destructive inspection means, allowing individual integrity check of the tendons, an alternative could be proposed based on inspection procedures habitually implemented for unbonded tendons. This would require testing and mock-up in order to validate the consistency and representativeness of results obtained as well as the lack of detrimental consequences for the tendons. This alternative could also be applied by using prestressed test beams stored in environmental conditions to best duplicate that the containment structure in order to test the evolution of the mechanical characteristics (concrete and tendons) and to check the ageing phenomenon.

To this day, an overall monitoring of the structure rather than a direct inspection of the prestressing seems to be the best compromise.

9. IN-SERVICE INSPECTION; CZECH EXPERIENCE

9.1 Check of cables prestressing

Checks of prestressing are implemented by regular round measuring, when the connector of tensometric sensors is connected to the connector of the measuring system and the tests is started. Measuring is implemented regularly once in a month. Thus, every drag anchor is measured individually. Cables of cylinder and cables of the dome are measured. Measuring on the cables of the dome is carried out by the worker, who is in a cage lifted by the handling crane. Thus, every tensometric place is measured. In case it is not possible to use the handling crate, there is a worker with climbing certification and equipment, who is ready to rope down from the dome to each place measured. The data obtained and measured are saved and recorded. The proper evaluation is carried out on computer.

For this purpose a one-purpose measuring system has been designed and produced. It serves only for round checking of prestress.

The source of literature for this chapter is the Report on advanced tendons prestress measurement systems, No. 201200766 [41].

9.1.1 Check of cables functionality

Checks of the wider scope, i.e. above the scope of the planned operating checks of prestressing cables, were carried out on two cables, including the tests for assessment of their residual life. Checks of functionality of cables were solved and carried out within implementation of the change of the cables 1.80S and 1.35K in 2009. When the original cable, which was installed approx. 16 years ago, was taken out, the condition of the surface for presence of the corrosion and the places with possible occurrence of corrosion in the nearest years were monitored. Further, the condition of the preservation layer along the entire length of the cable was monitored. Places affected with corrosion and places with disrupted and degraded preservation layer were ascertained during this monitoring process. After this situation was ascertained, samples were taken from the cables - from the bends of the cables, i.e. bends with eye rings and with 15 metres of wires. These segments were left for performance of tests.

After the performance of tests were suggested and agreed by the operator - NPP Temelin - both ends of the cables were tested in 2010/2011 in the Test and verification training stend in the premises of ÚJV Řež, a. s. (Nuclear Research Institute).

9.1.2 Calibration of tensometric measuring of prestress

Calibration of tensometric measuring is always performed when a significant deviation of the values measured is ascertained. However, the proper performance is preceded by assessment of the cause. The most frequent cause of this situation is the malfunction of the measuring elements - tensometric measuring or cable output. If the malfunction ascertained is caused by the tensometric measuring elements, they are always reinstalled.

The tensioning set is then used by means of which the real value of force of prestressing in cable is ascertained. This moment - the point of the prestressing force - is ascertained by means of the nut position sensors. The force is measured in the moment of separation of the nut from the washer, or separation of the nut together with the washer. The point of separation is monitored on the control computer display. The measured force is then assigned to the tensometric measuring so that the measuring of prestress of the anchor equipment shows the right value of prestress.

Figure 0.1 Nut position sensors at the calibration of the tensometric measuring

9.2 Check of cables condition

9.2.1 Visual check of cables prestressing

Visual check of prestress of cables lies in check of integrity of the covers of drag anchors. Any defect of these covers could indicate defect of the cable. The check of dome cables is carried out by means of binoculars. All the covers are viewed from the area of concrete lining (so that it is not necessary to erect scaffolding). Check of wall cables is already carried out as a round check. Any non-standard situation ascertained is solved in detail. The protective cover of the drag anchor is removed and the condition is examined.

9.2.2 Taking samples of preservative grease

The grease prescribed and used as the anticorrosive protection of prestressing cables of the containment in the operating documentation is the grease used for arms and declared as “grease gun“. In accordance with the Russian standard GOST 19537-83 [26], based on which the below (see Table 0.1) specified quality certificates are issued and characteristics are prescribed.

Table 0.1 Analysing the preservative grease.

	Indicator	Value	Testing methods
1	Appearance	Homogeneous yellow up to brown colour	Paragraph. 4.2 [26]
2	Dripping point	min. 60°C	GOST 6793
3	Softening temperature	min. 50°C	GOST 6037 p. 4.3
4	Impacts of corrosion on metal	fulfilled	GOST 9.080 p. 4.4
5	Acidity number	0.50 to 1.00	GOST 5985 p. 4.5
6	Content of water	no	GOST 2477
7	Content of mechanical dirt	not more than 0.07 hm%	GOST 6370 p. 4.7
8	Content of acids and alkalis	Slightly acidic	GOST 6307 p. 4.7
9	Test of protective properties	fulfilled	GOST 9.054 (method 1)
10	Penetration	90 to 150 mm-1	GOST 5346 (method B)

Sampling of the preservative grease from the surface of the eye ring is carried out (once a year) on the group of cables selected in advance. Part of the protective layer of the anchor equipment is taken for the purpose of analyses at which the ability to provide reliably the protective function is ascertained. The analysis is carried out in the process control laboratory of NPP Temelin. In case it is ascertained that the sample already doesn't correspond to the properties required, a work order is issued for implementation of renewal of preservation. Methods of performance of repairs are specified in chapter 0.

9.2.3 Analyses of the preservative grease and repairs of preservation

9.2.3.1 Analyses of the preservative grease

Analyses of the preservative grease may be divided into three parts according to the place where the grease is found. One sample is taken from the anchor equipment, one sample is taken from the preservative bath, and one sample is taken from the new grease from cans.

Analyses of the samples from the anchor equipment are carried out in accordance with the NPP internal procedures and are performed in the process control laboratory of Temelin power plant. Other analyses are implemented irregularly.

Based on the checks of the exposed cables taken out (1.80S and 1.35K) it is evident that the corrosion damage of individual wires was caused always in places of cable bends on the eye rings of drag anchors. The reason is probably the highest intensity of condensation of air humidity at changes of open-air temperature and humidity just in these localities. Closing of metal covers of drag anchors significantly extends the time of humidification of the cables and increases corrosion risks.

To ascertain, whether the properties of the preservative grease in the bath and of the new grease in cans provide sufficient protection, it is necessary to take the samples which will be subject to analyses. The analyses will be carried out in two workplaces, i.e. in the process control laboratory of the power plant and in the accredited laboratory in the Ukraine. The process control laboratory will carry out the tests which will determine, whether the grease meets the requirements or not. The accredited laboratory in Ukraine will examine the samples in accordance with the original and still valid standard GOST, based on which the grease was supplied years ago. Thus, it will be ascertained whether it meets the properties declared or not.

Sets of samples were taken for the purpose of analyses. They were two sets by four samples (0.2 litres).

The samples from the preservative bath were taken when the temperature of the hot bath was 70°C.

The sampling was carried out:

- From the preservative bath - from the surface;
- From the preservative bath - from the bottom;
- From the store of grease – from 20 L can;
- From the store of grease – from 200 L barrel.

9.2.3.2 Results of the samples from the anchor equipment

The laboratory of the Temelin power plant uses spectrometry in infra-red area of wave lengths - the technology of attenuated absolute reflection - for the purpose of evaluation of the quality of preservative grease. Based on the spectra measured it is possible to ascertain presence of absorption belts pertaining to the types of chemical compounds occurring thanks to the oxidative degradation of hydrocarbons (wave numbers approximately 1700 (1660-1800) and 1150 cm^{-1}); then it is possible to assess the rate of the degradation according to the intensity of the absorption belt.

Presence of water is determined by means of absorption belts - OH groups in the area of the approximate wave number of 3400 cm^{-1}.

The samples of the preservative grease from the cables of the prestressing system of the containment supplied for the purpose of check to the laboratory often contain significant lack of homogeneity. That is, only a small surface layer is degraded in many cases, in which also the increased content of water is ascertained. It is probably the cause of the degradation. In some cases there are also small solid particles of dirt. However, the grease is usually in good condition under this surface layer showing no significant indications of degradation and without content of humidity or dirt. Quantification of e.g. content of water for the whole sample delivered has no sense. The same applies also for quantitative determination of solid particles of dirt. If any dirt is present in the grease, their occurrence is determined by smearing the sample on the crystal using the technology of attenuated absolute reflection, the principle of which is simple or multiple absolute reflection of radiation on the phase interface of the measured sample and the measuring crystal with sufficiently high index refraction.

Routine infrared of the spectroscopic analysis carried out by the laboratory of the Temelin power plant showed good condition of the grease in warehouse stock. The grease in the preservation bath showed traces of oxidation degradation products, but it will be necessary to examine their extent by a complex analysis carried out in accordance with GOST 19537-83 [26].

Taking into account the small quantity of samples the laboratory is not able to carry out the tests of physical parameters.

9.2.3.3 Results of the samples from the preservative bath and from cans

Within the assessment the analyses of the following samples were carried out:

Sample no. 1 – grease from the surface of the preservative bath

Sample no. 2 – grease in the store - big silver barrel

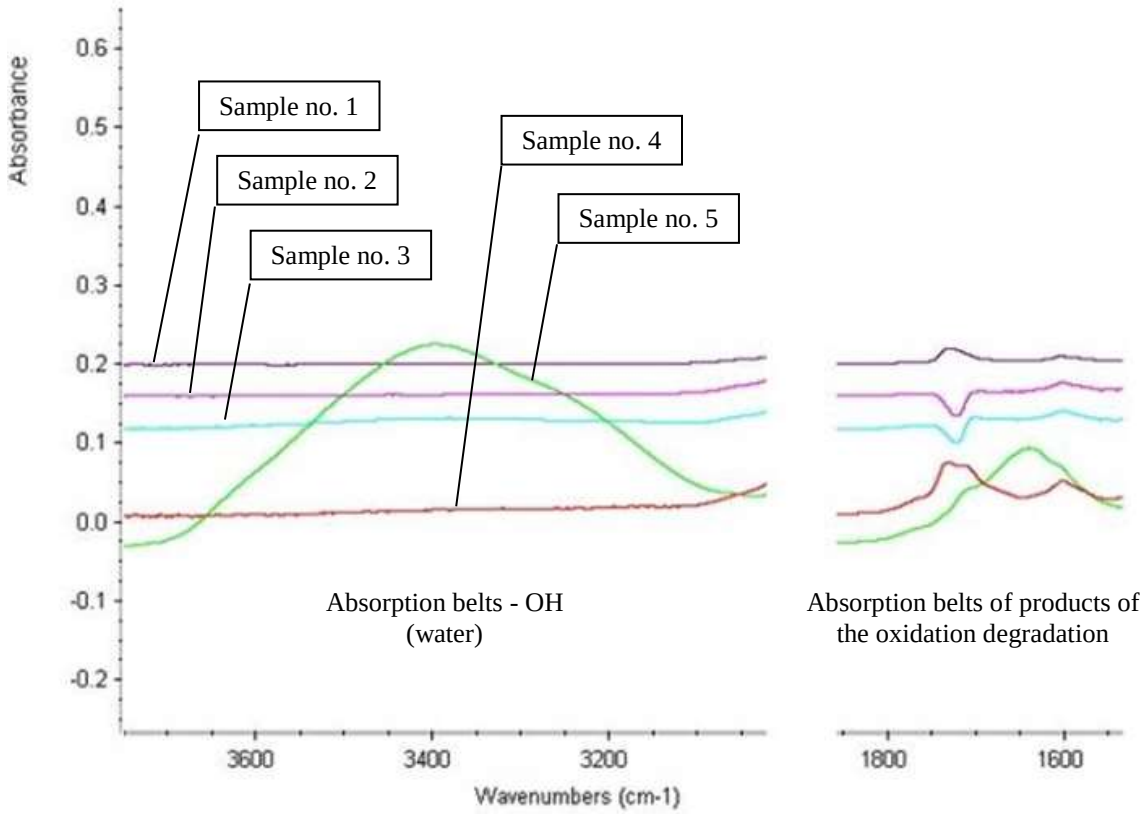
Sample no. 3 – grease in the store - little barrel

Sample no. 4 – sample spectrum of grease with increased degree of degradation (without water)

Sample no. 5 – sample spectrum of grease with high content of water

The spectroscopic analysis was carried out. The following picture includes the selected characteristic parts of spectra measured.

Figure 0.2 Characteristic parts of infrared spectrum of the samples of grease



Evaluation of spectra

Sample no. 1: Moderate degree of degradation, absence of water, solid particles absent.

Sample no. 2: No traces of degradation, water and solid particles absent.

Sample no. 3: No traces of degradation, water present (slight moisture), solid particles absent.

Sample no. 4: Increased degree of degradation, absence of water.

Sample no. 5: Strong degradation of grease with high content of water.

The results of the analyses carried out from the preservative bath and from cans are not available yet; the analyses are in process.

9.2.3.4 Repairs of preservation

Repairs of preservation of the anchor equipment of the containment are performed regularly, always when checks of anchor equipment are carried out or tensometric sensors of force are calibrated. Repairs of preservation are carried out in the following cases:

- Damage of preservation layer of anchor equipment;
- If degradation of preservation layer is ascertained.

In the first case, only the damaged place is repaired by warming up the grease in a vessel and its application on the surface preserved using a brush. In the second case, it is necessary to remove the degraded layer using a plastic spatula, mostly as far as to the metal. Then, new grease is warmed up and it is applied on the surface preserved using a brush. For application it is recommended to warm up the parts by hot air, so that the new preservation layer mingles with the layer on the surface.

9.3 Tests of strain-gauge measuring systems

Tests of tensometric measuring systems were carried out so that tensometric sensors were installed on the bolt of the drag anchor installed in the stend. Depending on the tested tensometric sensors also the scope of load tests was proposed – standard, temperature tests (temperature tests -30 to +60°C in the area around the drag anchor). The first tests were carried out on all tested variants. The second tests were carried out only on two selected.

9.3.1 Summary of the requirements for sensors measuring the prestressing force

a) Requirements regarding utilization of measured values:
   - Measuring of sensor in the scope of force in the cable from 0 to 12MN;
   - Distinctiveness of measuring ±100kN;
   - Accuracy of determination of the value of prestressing force ±200kN;
   - The sensors must reliable measure during the whole period of life of the prestressing cable (block).

b) Requirements regarding environment and conditions of action:
   - Resistance against repeated changes of the force in the cable in the range from 0 to 10MN in the minimum number 5 cycles ("big cycle");
   - Resistance against cyclical changes of the force in the cable in the scope of ±100kN in the order of 10^4 cycles (changes of stress caused by changes of the ambient temperature);
   - Measuring in the scope of the temperatures from –30°C to +60°C;
   - Measuring at the relative humidity from 10% to 100% with the option of surface condensation;
   - Functionality of the sensor for seismic events up to the level of the maximum calculating earthquake.

c) Requirements regarding installation and operation of the measuring system:
   - Installation and calibration without necessity to release prestress in the cable;
   - Continuous renovation of sensors without the necessity to handle the prestressing cable.

9.3.2 Summary of the requirements for sensors proving functionality of the cable

a) Requirements regarding utilization of measured values:
   - Measuring of sensor in the scope of force in the cable from 0 to 12MN;
   - Guaranteed signalisation of change of prestressing force by more than 500kN;
   - The sensors must reliable measure during the whole period of life of the prestressing cable (block).

b) Requirements regarding environment and conditions of action:
   - Resistance against cyclical changes of the force in the cable in the scope of ±100kN in the order of 10^4 cycles (changes of stress caused by changes of the ambient temperature);
   - Measuring in the scope of the temperatures from –30°C to +60°C;
   - Measuring at the relative humidity from 10% to 100% with the option of surface condensation;
   - Functionality of the sensor for seismic events up to the level of the maximum calculating earthquake.

c) Requirements regarding installation and operation of the measuring system:
   - Installation and calibration without necessity to release prestress in the cable;
   - Continuous renovation of sensors without the necessity to handle the prestressing cable.

9.4 Tests of the cables taken out

The tests created part of the technical aid of ÚJV Řež, a. s. within the project Creation and application of the long-term strategy for stabilisation of the prestressing system of Temelin envelopes.

The goal of these tests was to perform a series of load tests of the cable taken out, evaluate the process of these tests and take the determined sections of the cable tested for the purpose of subsequent analyses and tests. The goal of the mentioned and the subsequent tests was the suggestion of technical and organisational measures for minimisation of operational failures of prestressing cables of the containment.

The test starts in the moment of installation in the test stend.

9.4.1 Installation procedures

After the already tested sample of the cable with the length of 10m has been disassembled, the auxiliary steel rope from 478 pcs of 5 mm diameter wires was pulled through the stend. The nuts of the drag anchor were unscrewed to the maximum possible degree, so that the bolt of the drag anchor is inserted in the stend as much as possible. The pulling clamp was attached to the cut end of the sample of the cable. The pulling clamp was attached to the end of the rope pulled through. The cable was pulled through the stend by means of the rope and the manual winch. In this phase the pulling bracket was installed. Installation tapes were gradually removed during the installation of wires. After all wires had been installed the wedge was installed, 2 pieces of hydraulic cylinders were assembled, and clamps were attached. Wires of the cable were straightened by means of two linear hydraulic cylinders. The clamp was tightened after all wires of the cable had been straightened.

The stressing device, which is placed on the movable undercarriage, was connected to the test stend. Standard pushing device was used for connection, which interconnected the entire system with the eye ring.

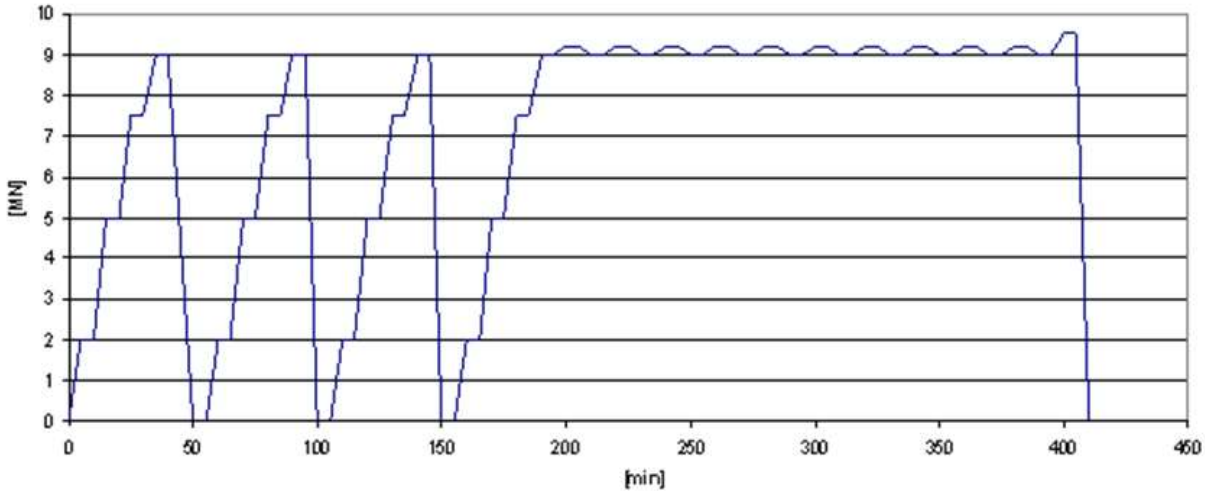
By subsequent slow drawing of individual pistons of the stressing gun out the cable was straightened by gradual sitting down of the piston rod on the supporting platform of OSS. The maximum force in the system not exceeding 0.1 MN is assumed there.

Figure 0.3 Fixed loose end of the cable

9.4.2 Load test procedure

The prestressing to the force of 9.5 MN was carried out gradually with the holding time of 5 minutes at the values of 2 MN, 5 MN, 7.5 MN, and for 20 minutes with the force of 9 MN. The whole load application was repeated 3 times. After the holding time on the 4th cycle of the third repeating the load was applied on the test cable in another 10 cycles with the value of F_{perzik} = 9.2 MN and with the holding time of 5 minutes. The holding time between the individual cycles was 3 minutes. After 10 cycles were completed the cable was stabilized on the force of 9 MN. The holding time of 5 minutes was before the last load application to the value of F_{max} = 9.5 MN.

Figure 0.4 The overview of the load cycles of the test procedure



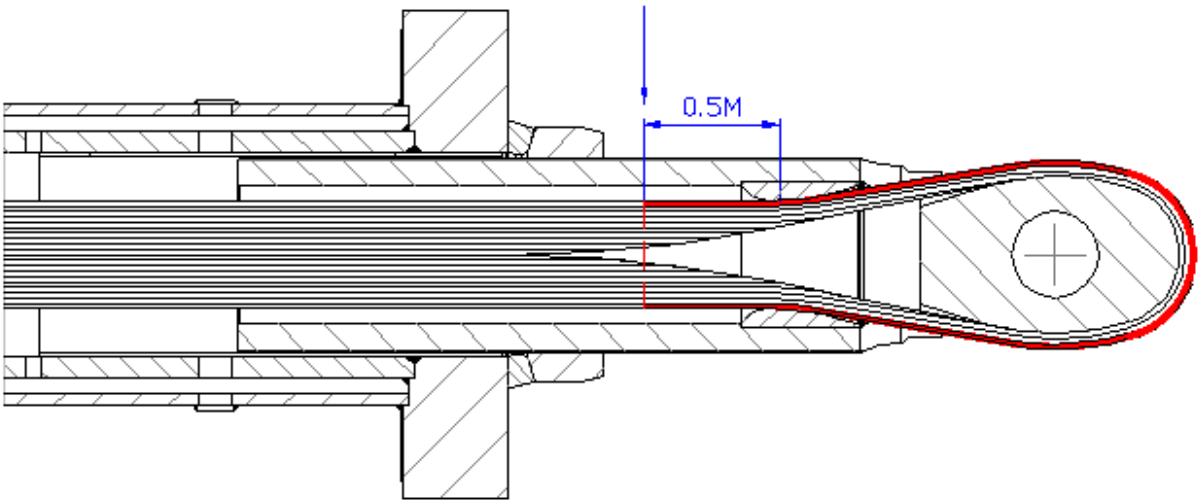
9.4.3 Completion of the load test

After the series of the load cycles were performed the fastening clamp was dismantled, the tested cable was released and taken out, and the condition of the cable was recorded focusing particularly on:

- Burst wires in the eye ring bend;
- Burst wires in the active length of the cable;
- Depth of cuts and scratches in wires.

Samples were taken from the cable tested this way for the purpose of the subsequent analysis.

Figure 0.5 Draft of the sample taking of the eye ring segment



9.5 Tests of drag anchors and eye rings

The goal of the tests is performance of load tests of Drag anchors and Eye rings in accordance with the work operation approved. The tests of the drag anchors were performed on the prestressing base of Temelin nuclear power plant in the Test press ZK 12.5, where the 100% scope of tests was performed for all tested drag anchors, including recording of the data in the plan of checks and tests with the attendance of the authorized person. Each drag anchor went through the test assembly. Based on the tests successfully performed the inspection report was issued.

9.5.1 Test procedure

Each drag anchor went through test assembly, successfully went through the Load test 1 – the load of 12.5MN was applied on the sets during the test (100% of the sets), through the Load test 2 – this test serves for testing of connection of the drag anchor with the eye ring (by means of wedges), which must stand load for tearing off with the force of 50 kN (10% of the sets), and disassembly with eye ring.

The drag anchor and the eye ring which met all the requirements, the compliance of which was confirmed by PKZ (the plan of checks and tests), Final test, Inspection report, and Declaration on conformity, shall be regarded as the set which complied with the test. The procedure of the Load tests no. 1 and no. 2 is the same as for eye rings testing.

Figure 0.6 Tests of sets of Drag anchor - Eye ring

Figure 0.7 Test of set of Eye ring - Drag anchor

9.6 Qualification of measuring systems (sensors of force)

The force sensors to be used for measuring on the prestressing tendon of the containment had to go through vast tests first, which were performed on OSS, on Test press ZK 12.5, and on the Tensile machine TS3. After the both phases of tests have been successfully performed, the third phase will be performed in year 2012, during which concurrence of the current and the new measuring method will take place on the selected ends of the cylinder cables of Temelin containment.

9.6.1 Requirements for newly installed sensors

a) Tests regarding utilization of measured values:
   - Scope of measuring; its distinctiveness and accuracy;
   - Impact of changes of the ambient temperature on measuring of the sensor - elimination of the impact of changes of the ambient temperature on the value of the prestressing force measured.

b) Tests regarding environment and conditions of action:
   - Resistance against repeated changes of the force in the anchor as a result of easing of the anchor;
   - Resistance of the sensor against cyclical changes of the force in the anchor as a result of changes of the ambient temperature;
   - Resistance of the sensor against external impacts (humidity, changing temperature) and the requirements for protection of the sensor against these impacts.

c) Tests regarding installation and operation of the measuring system:
   - Determination and verification of the requirements for handling with the prestressing cable at the installation of the sensor;
   - Determination and examination of the requirements for installation of the sensors on already prestressed cables in the construction (protection against weather, limiting conditions of installation, time demandingness of installation, and handling with the cable during installation);
   - Examination of the ability of the sensor to measure changes of the force in the cable at the failure of some measuring components of the sensor, possibilities of repairs of the sensor without necessary calibration requiring handling with the prestressing cable;
   - Life span of individual components of the sensor and requirements for maintenance and continuous restoration of the sensor during operation of the system (intervals between the replacement of the components, necessary maintenance, and calibration).

The tests and examinations performed for the individual types of sensors shall verify observance of the requirements for new sensors. They are performed based on the data provided by the manufacturers of individual components supplemented, as needed, by the tests of the entire sensor and its components. For the tests of the entire sensor it is assumed that OSS and the test press ZK 12.5 will be used. They will allow simulation of behaviour of the anchor of the prestressing cable with real placement of individual components of the sensor. It is possible to test individual components of the sensor on individual devices (e.g. fatigue tests and climatic impacts tests).

After the tests and examinations are completed in all areas for the selected types of sensors determined for future installation on the construction, the certificates proving the fitness of the sensors to perform the required function will be issued.

See Table 0.2 below for explanations of abbreviations and vocabulary used in this chapter.

Table 0.2 Specific abbreviations and vocabulary used in this chapter.

NPP Temelin	Nuclear Power Plant Temelin
ÚJV Řež, a. s.	ÚJV Řež, a. s., http://www.ujv.cz/en
OH	OH group (in chemistry)
GOST	Russian standard
OSS	10 meters long testing stend
PERZIK	Periodic Leaking Test of the containment integrity
ZK 12,5	Test press no. 12,5
PKZ	Plan of checks and tests
TS3	Tensile machine no.3

10. SUMMARY AND RECOMMENDATIONS

This activity of the concrete sub-group of OECD/NEA WGIAGE concrete subgroup: study on post-tensioning methodologies in containments, was approved by CSNI in June 2009.

In this study the two post-tensioning methodologies: bonded and unbonded methods and their technological features are analysed. The mechanical behaviour of the containment is directly influenced by the adherence of the tendons to the concrete, locally and under high stresses in case of severe accident. The main goal of this work has been to clarify the consequences and necessary procedures when choosing the post-tensioning technologies in terms of design basis, in terms of behaviour during severe accident, in term of construction requirements as well as in term of monitoring and in-service inspection of the containment. The choice of the post-tensioning technology is related to the life time extension procedures of old plants as wells as to the construction methods of new NPP's.

The most important results of this work are as follows:

- For new structures, data collection should start during construction (at least at the beginning of the tensioning of the cables) so that the initial state, i.e. baseline parameters could be established at the end of the construction period.

- Differences of the unbonded and for bonded tendons to the maintenance aspects are taken into account in the early design of new construction to be sure that all aspects to the accessibility, inspections and preventive maintenance are managed in a relevant way.

- Procedures of monitoring and In-Service Inspection methods of containment should carefully defined and approved with special features of chosen technology for the new and existing plants to make sure that all safety goals are fulfilled.

- Direct monitoring of the bonded post-tensioning system is currently not possible, proven reliable indirect methods should be used for the new and existing plants to ensure continuous integrity of the post-tensioning system over the service life of the plant.

One important recommendation of this study is that the correct modelling of the tendon bonding condition with concrete around the tendons is important, especially when the Design Extension Conditions are taken into account in the design of new constructions. For the new and existing plants the lifetime monitoring, maintenance and testing procedures must be designed and reviewed according to the choice of the protective system for tendons, namely, bonded or un-bonded protective system.

The objective of this CAPS was not to establish guidelines for countries to follow but mainly to clarify the consequences of the chosen containment tension technology in terms of design basis, in term of behaviour during severe accident, in term of construction requirements, in term of monitoring and in-service inspection of containment in various operating conditions due to the high importance of the tendons for the containment capacity. This objective has been satisfactorily achieved in this report.

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