Testing of five 30-year-old prestressed concrete beams

Peter Lundqvist
Juha Riihimäki

INTRODUCTION

The most common way to enclose a nuclear reactor is by using a concrete structure, the so-called reactor containment, which is prestressed both horizontally and vertically. The main function of the containment is to prevent radioactive discharge to the environment in case of a severe internal accident in the reactor. In addition, the containment is also expected to protect the reactor from external actions such as airplane crashes and explosions. The most severe accident in a nuclear power plant is the "LOCA", the loss of coolant accident, which e.g. can be initiated by a pipe rupture in the cooling system thus both the pressure and the temperature will increase inside of the containment. The increased internal pressure at a LOCA is referred to as the "design pressure". The function of the prestressing system is to counterbalance the tensile stresses in the concrete which occurs at the design pressure thus maintaining the structural integrity of the containment. However, due to creep and shrinkage of the concrete and relaxation of the prestressing steel, the effective prestress forces in the containment will decrease with time.

The corrosion protection of the prestressing system, i.e. the tendons, can be arranged in two different ways, either by cement grouting (bonded tendons) or e.g. by grease injection (unbonded tendons). One major disadvantage using bonded tendons is the difficulty of assessing the status of the tendons, e.g. measuring the remaining tendon forces or detecting corrosion damages.

When the nuclear power plant Olkiluoto was built in Finland in the mid-seventies several prestressed concrete beams were fabricated. The purpose of the beams was to monitor the prestress losses in the containment by testing one beam approximately every third year. The testing results were however deemed unreliable and the entire testing program was cancelled and the remaining beams have been stored inside of the reactor containment building ever since. The remaining tendon forces in five of these beams have been tested and the prestress losses obtained from the tests were compared to several models for predicting creep and shrinkage of the concrete and relaxation of the prestressing steel.

RESEARCH SIGNIFICANCE

In many countries, e.g. Sweden and USA the nuclear reactors have been licensed for an operating period of 40 years. Within 10 years the first of the Swedish reactors, which were built in the seventies, will reach the end of their operating license and the license holders now wish to extend the service life of the reactors with some 20 years. One requirement for the renewal of the licenses is that the safety of the reactor can be verified and guaranteed during the prolonged operating period, i.e. the effective prestress forces will still be sufficient to maintain the integrity of the structure when subjected to the design pressure. To monitor, or at least to estimate, the prestress losses in the containments is therefore of utmost importance. Furthermore, to be able to estimate prestress losses, which significantly affects the structural safety, is also of great importance in other prestressed concrete structures with bonded tendons, such as bridges and hydro power dams.

DESCRIPTION OF THE BEAMS

A total of five beams was tested, two from Olkiluoto reactor 1 (OL1) and three from reactor 2 (OL2), see figure 1 for beam details. The beams were manufactured between 1975 (OL1) and 1977 (OL2), respectively, at the same time as casting of the containment walls. The lengths of the beams are 9 ft 10 in. (3 m) with a square cross-section 19.7 x 19.7 in. (0.5 m). Two different post-tensioning systems were used, for the beams from reactor 1 VSL type 19 Ø 0.5 in. (13 mm) has been used and for the beams from reactor 2 BBRV type R 238 (72 Ø 0.24 in. (6 mm)) has been used. The tendon in the VSL system consists of a number of strands which in turn consists of 7 wires each. The tensioning is performed by pulling directly in the strands which are fixed by inserting wedges in the anchor head. The tendon in the BBRV system consists of a number of individual wires fixed directly to the anchor head. The tensioning is performed by lifting the anchor head and placing shims between the anchor head and the anchor plate. In each beam one tendon is placed in the center of the cross-section. The anchorage system consists, in both cases, of a square bearing plate with split holding rings and a circular anchor head with openings for passage of the wires. The initial tensioning forces in the beams were 550 kips (2.44 MN) and 567 kips (2.52 MN), respectively. After the initial tensioning, the anchor set losses were eliminated by lifting the anchor head and placing shims between the anchor head and the anchor plate. After tensioning the ducts were injected with cement grout.

The water-cement ratio of the concrete ranged between 0.49 and 0.54, slow-hardening cement was used, i.e. type IV cement (according to ASTM C150). The beams were cast with the same concrete that was used for the containment walls. Unfortunately, information regarding the curing method and curing time was not available.

The beams were stored inside of the containment building at approximately 90°F (32°C) and 21 % relative humidity (RH).

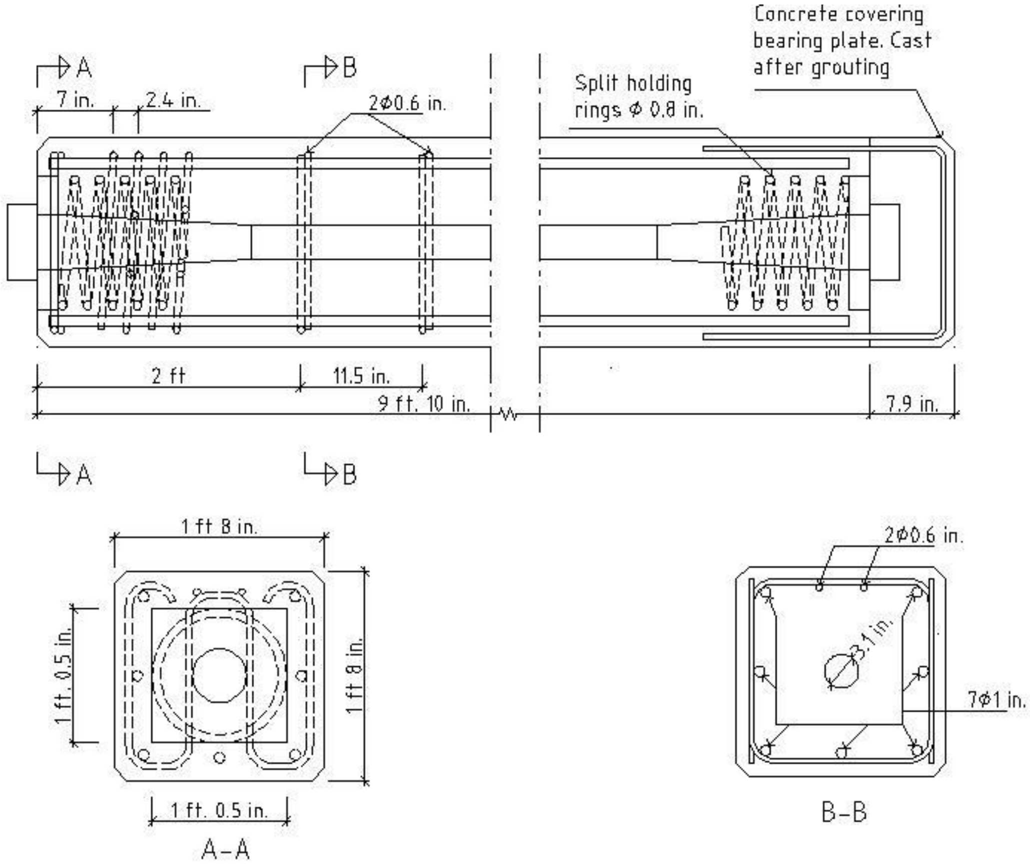

Figure 1. Details for the beams with the VSL post-tensioning system. The arrangement of reinforcement and tendons are the same for all beams. The only difference between the beams with the two post-tensioning systems is the dimensions of bearing plates and ducts. Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm.

TESTING PROCEDURE

The beams were arranged as simply supported in the testing machine and subjected to a single point load at midspan, see figure 2. The beams were loaded in deflection control with increments of $3.9 \cdot 10^{-4}$ in. (0.01 mm) per second until flexural cracks appeared at the bottom of the beam. The initial crack was marked and the beam unloaded and reloaded again until the crack re-opened. In order to determine the so-called decompression load, one LVDT-gauge was mounted across the crack, see figure 3. The beams were loaded and unloaded three times in order to monitor the accuracy of the measurements.

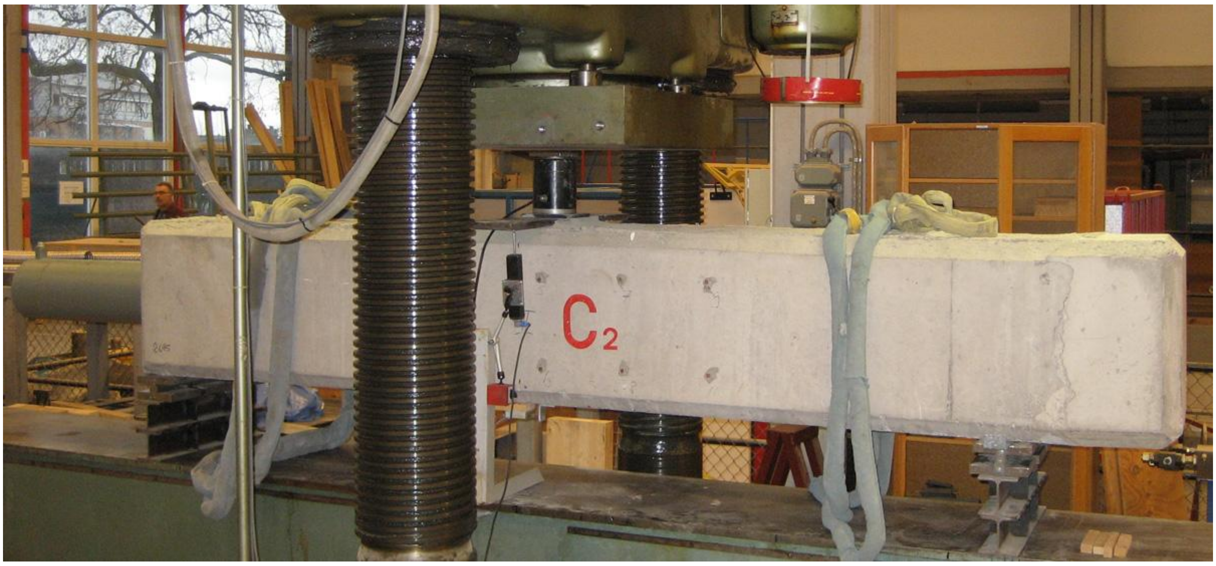

Figure 2. Shown is the arrangement of the beams in the testing machine.

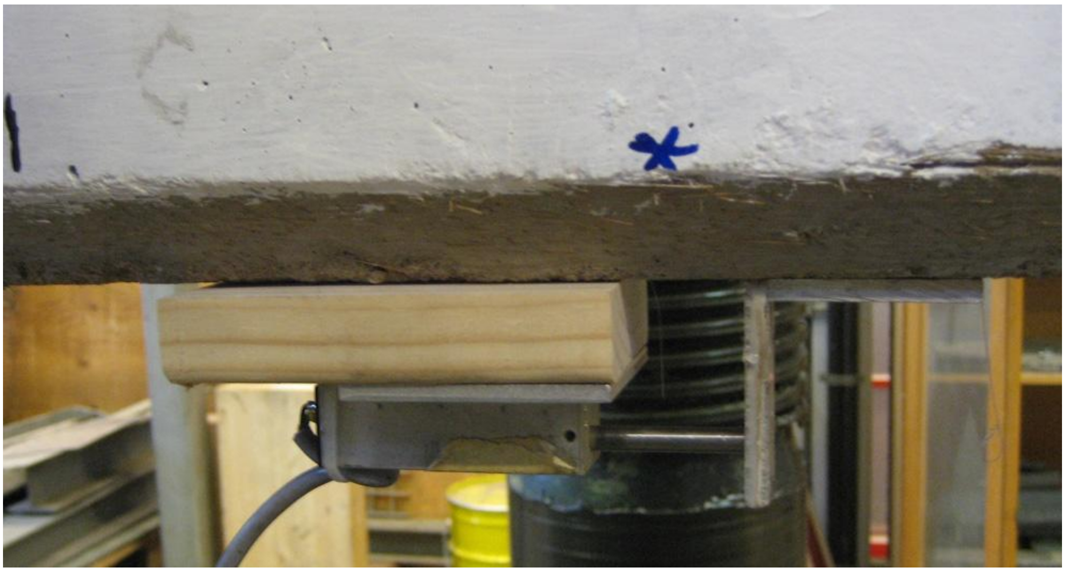

Figure 3. Shown is the LVDT-gauge mounted across the crack.

The decompression load was determined from the load versus crack-width diagram where a significant change in the displacement, recorded by the LVDT gauge, occurred after the crack re-opened. The decompression load was determined by intersecting the tangents of the two slopes, see figure 4 where the crack-width versus load diagram for one of the loading cycles for beam number 1 is shown. The final decompression load was calculated as the mean value of the decompression loads of the three loading cycles. The intersection point was found using the following procedure: the intersection point, δ, was estimated visually (i.e. "by hand"), regression lines were adapted to the upper and lower slopes of the curve, respectively. For the lower slope, the regression line was adapted from the point where the displacement was 4.10^{-4} in. (0.01 mm) (avoiding the irregularities in the region 0-4.10^{-4} in.) to the point on the slope corresponding to the displacement $\delta - 2.10^{-4}$ in. (0.005 mm). For the upper slope the regression line was adapted between the point corresponding to the displacement $\delta + 2.10^{-4}$ in. (0.005 mm) to the endpoint of the slope, $7.3.10^{-4}$ in. (0.02 mm). This procedure, i.e. adding/subtracting 2.10^{-4} in. (0.005 mm) to/from the intersection point, was used in order to avoid the irregularities when the change in slope occurs, which is clearly seen in figure 4. Finally, the final intersection point was found by intersecting the upper and lower regression lines.

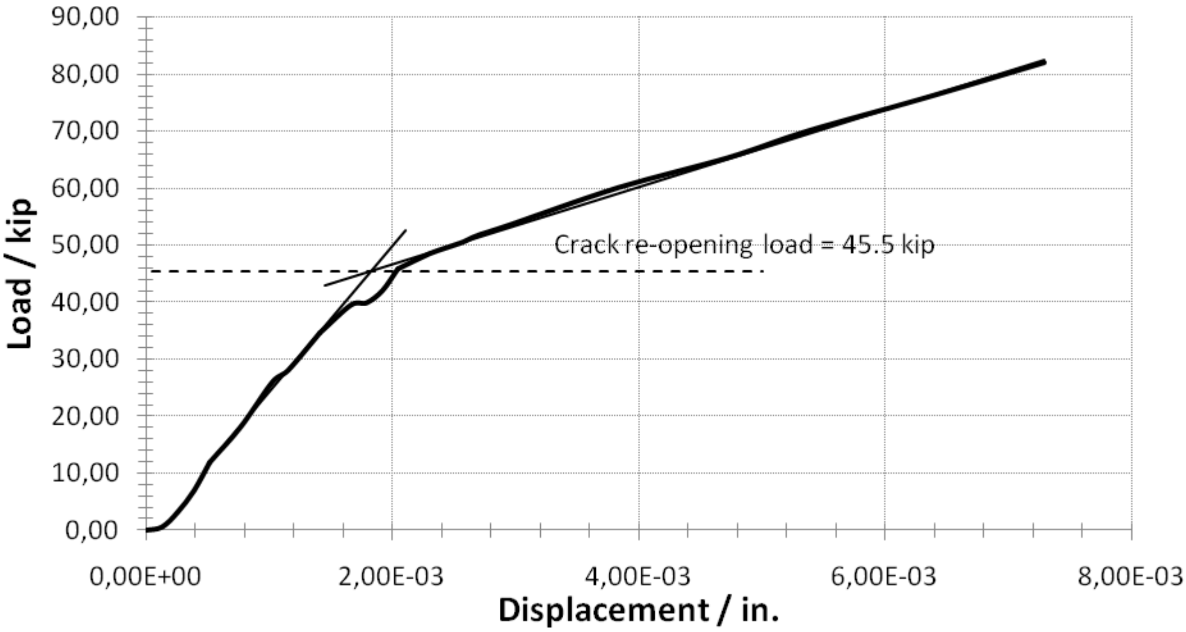

Figure 4. The graph shows the crack-width-vertical load diagram for one of the loading cycles for beam number 1. The decompression load is determined by intersecting the two slopes. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN

Since the stress at the bottom of the beam is zero at the decompression load, the remaining tendon force can be calculated using Naviers formula, which gives the following equation:

$$0 = \frac{P_{eff}}{A_c} - \frac{M}{S}$$

where:

P_{eff} = remaining tendon force

A_c = cross-section area of beam

M = applied moment

S = section modulus of the beam

The cross-section area of the beams is 2.7 ft^2 (0.25 m^2) and the section modulus is 0.74 ft^3 (0.021 m^3).

TESTING RESULTS

The measured and initial prestressing forces and prestress losses for each beam are presented in table 1.

Table 1. Measured prestress forces and prestress losses.

Beam #	Initial prestress force, kips	Measured prestress force, kips		Prestress Losses, %
		Average	Range	
1	567	349	343 – 358	38
2	567	351	346 – 356	38
3	567	215	211- 226	61
4	550	346	343 – 353	37
5	550	281	276 - 284	48

Note: 1 kip = 4.448 kN. Beam number 1 to 3 are from OL2 with the BBRV-system, beam number 4 and 5 are from OL1 with the VSL-system.

The variation in the results can be explained by the fact that during the testing of beam number 3 and 5, splitting cracks occurred and propagated at both ends and did not close after unloading. In addition, extensive cracking was observed in the zones surrounding the bearing-plates in these two beams. This indicates that the anchors have moved inwards during the testing, which causes a shortening of the tendon thus increasing the prestress losses. Due to the short length of the tendons a movement of 0.04 in. (1 mm) will result in prestress losses of approximately 5 percent.

Furthermore, for beam number 3 the time between casting and tensioning was 63 days, while for the others, it was more than 135 days. In all of the beams slow-hardening cement, i.e. type IV, has been used, which means that the strength development of the concrete is slow. The age at loading is an important parameter influencing creep, in particular if the loading occurs before the concrete is fully hardened, the creep will increase the earlier the load is applied and this will affect the prestress losses in the beam significantly. The early loading and the movement of the bearing plates are the most probable explanations for the high losses in beam number 3. The high prestress losses in beam number 5 are probably due to the movement of the bearing plates.

The prestress losses in the beams are relatively high compared to results from tests of old bridge beams in the literature[1,2,3], where the measured prestress losses ranged between 17-20 percent. This is probably due to that the beams have been stored indoors at approximately 90°F (32°C) and 21 % RH and not been subjected to any varying climate. Since both drying-creep and shrinkage is due to the drying of the concrete, the ambient climate, i.e. RH and to some extent the temperature, strongly affects these phenomena and thus increasing the prestress losses in the beams. In addition, both the relaxation of the tendons and the basic creep of the concrete are strongly affected by the temperature and thus the high ambient temperature is another factor increasing the prestress losses in the beams.

PREDICTION MODELS

The prestress losses in the beams have been calculated using various prediction models for creep and shrinkage of concrete and one model for relaxation of the tendons. The models that were used for calculating creep and shrinkage are the CEB/ FIP Model Code 1990[4] and 1999[5], ACI 209[6], Model B3[7], GL2000[8] and the PCI Committee on Prestress Losses[9]. The PCI model was also used for calculating the relaxation in the tendons. The method in the CEB/FIP model code was used, even though it is stated that the creep and shrinkage model is only valid for concrete subjected to a mean relative humidity ranging between 40 to 100 percent. The tendon forces, thus the concrete stress, will decrease with time due to creep, shrinkage and relaxation, since creep is proportional to the stresses in the concrete the creep rate will therefore decrease with time. In addition, the change in tendon force will also affect the relaxation rate. These effects are taken into account by using the procedure in Eurocode 2[10] where a mean value over time of the concrete stress is used when calculating the prestress losses, the effects on relaxation is also taken into account.

The creep and shrinkage models are empirical and each model is based on a huge amount of data from shrinkage and creep tests. The procedure for estimating the strains is similar for both creep and shrinkage: a final creep coefficient/shrinkage strain is calculated from different parameters, e.g. compressive strength, water-cement ratio and ambient relative humidity. The development of strains over a certain period of time is described by a time function which is calculated from e.g. concrete age, age at loading and the volume to surface ratio of the structure.

The input parameters for the models are given in table 2. Most of the data regarding the beams were available, however some of the most important information, e.g. the modulus of elasticity and the compressive strength of the concrete was missing and therefore the following assumptions had to be made.

- On two different occasions, in 1980 and 1983 respectively, tests have been performed on concrete cylinders which were cast simultaneously as some of the beams from reactor 1. The age of the concrete at the time of testing was 3 years and 3 months and 6 years and 2 months, respectively. The 28-day compressive strength and the modulus of elasticity both at 28 days and at loading were estimated from the test values by calculating backwards using the equations regarding the development of strength and modulus of elasticity with time according to CEB/FIP[5], see table 2.
- It was assumed that the beams were cured by membrane curing, the same method that was used for the containments.
- The removal of the curing membrane, i.e. the onset of drying, was assumed to have occurred simultaneously as the tensioning of the tendons.

Three cores were extracted from beam number 3 in order to determine the compressive strength of the concrete. The results from the different cores were scattered and the strength, 3.26 ksi (22.5 MPa), was lower than that measured on cylinders in 1980 and 1983. A closer examination of the cores revealed numerous air voids in the concrete, which indicates a poor workmanship when casting the concrete. Due to the unrealistic results the measured strength was not used in the prediction models.

Table 2. Input parameters for prediction models.

Parameter	**Beam #**			
	1,2	**3**	**4**	**5**
Curing method	Membrane	Membrane	Membrane	Membrane
Curing time, days	135	63	647	687
Age of concrete, days	11,063	11,056	11,922	11,961
Age at loading, days	135	63	647	687
Age at onset of drying, days	135	63	647	687
Compressive strength at 28 days, ksi	4.5	4.5	4.5	4.5
E-modulus at 28 days, ksi	2,815	2,815	2,815	2,815
E-modulus at loading, ksi	3,118	3,000	3,278	3,278
Cement type	Slow hard.	Slow hard.	Slow hard.	Slow hard.
Cement content, lb/ft^3	25	23.4	24.6	23.5
Water content, lb/ft^3	12.5	12.8	12.8	11.6
Aggregate content, lb/ft^3	108	108	109	120
Air content, %	1.3	1.5	0.9	1.6
Fine aggregate, %	53	55	52	50
Aggregate type	Norm. weight	Norm. weight	Norm. weight	Norm. weight
Slump, ft.	0.39	0.39	0.31	0.28
Ambient RH, %	20.9	20.9	20.7	20.7
Ambient temperature, °F	90	90	88	88
Type of prestressing steel	LR	LR	LR	LR

Note: 1 in. = 25.4 mm; 1 ft. = 0.3048 m; 1 ksi = 6.895 MPa; 1 lb = 0.453 kg

RESULTS FROM PREDICTION MODELS

Table 3. Prestress losses calculated with the various prediction models.

Beam #	Testing	MC 90	MC 99	ACI	PCI	B3	GL2000
1	38 %	23 %	25 %	17 %	24 %	37 %	31 %
2	38 %	23 %	25 %	17 %	24 %	37 %	31 %
3	61 %*	26 %	27 %	18 %	24 %	38 %	31 %
4	37 %	19 %	20 %	15 %	22 %	36 %	29 %
5	48 %*	21 %	20 %	14 %	22 %	36 %	29 %

* Test result may be misleading due to slip in anchors during tests.

A comparison between the results from the prediction models and the measured prestress losses, see table 3, show that model B3 was the most accurate of the prediction models and that most of the prediction models underestimate the measured prestress losses in the beams (disregarding the results for beams number 3 and 5). Possible explanations to the accuracy of model B3 is that it is the most complex of the models with the highest number of input parameters and that the approach for predicting creep is only semi-empirical and partly based on theoretical knowledge of the phenomena affecting creep. In addition, it is the only of the models which separates basic and drying creep.

Explanations for the difference between the predicted and measured losses can be found by considering different parameters. The models are developed to be applicable to a wide variety of structures under more or less normal climatic conditions, i.e. normally around 70°F (20°C). Since none of the models takes the effect of temperature on prestress losses into account the ambient climate is probably the parameter which has the biggest influence on the deviation between predicted and measured prestress losses. The models are developed from creep and shrinkage test data and normally the duration of these tests is only a couple of years. This is probably also a parameter which affects the accuracy of the models since the prestress losses in the beams have been developing for more than 30 years.

The fact that the prestress losses obtained from the tests in most cases are higher than those predicted by the models indicates that the assumption that the onset of drying occurs simultaneously as the tensioning is correct. Otherwise, due to the longer period of drying prior to the tensioning, in particular for beam number 4 and 5, both the shrinkage contributing to the prestress losses and the drying-creep strains of the concrete would decrease.

CONCLUSIONS

- The prestress losses in the beams are relatively high compared to results from similar tests found in the literature, which probably is due to the ambient climate in which the beams have been stored. An almost constant temperature of 90°F (32°C) and low relative humidity increases both the creep and shrinkage strains in the concrete.
- Model B3 was the most accurate of the prediction models and was in good agreement with the prestress losses obtained from the tests.
- Most of the prediction models underestimate the measured prestress losses, one possible explanation for the deviation between prediction models and measured prestress losses is the influence of the ambient climate.

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REFERENCES

1. Pessiki, S., M. Kaczinski, and H. H. Wescott. 1996. Evaluation of Effective Prestress Force in 28-Year-Old Prestressed Concrete Bridge Beams. *PCI Journal*, V.41, No. 6 (November-December): pp 78-89.
2. Czaderski, C. and M. Motavalli. 2006. Determining the Remaining Tendon Force of a Large-Scale, 38-Year-Old Prestressed Concrete Bridge Girder. *PCI Journal*, V.51, No. 4 (July-August): pp 56-68.
3. Shenoy, C. V. and G. C. Frantz. 1991. Structural Tests of 27-Year-Old Prestressed Concrete Bridge Beams. *PCI Journal*. V.36, No. 5 (September-October): pp 80-90.

4. CEB Bulletin d`Ínformation no. 199. 1991. *Evaluation of the Time Dependent Properties of Concrete*. Comite European du Beton/Federation Internationale de la Precontrainte: Lausanne, Switzerland.
5. CEB-FIP Bulletin 1. *Structural Concrete, Textbook on Behaviour, Design and Performance- Updated knowledge on the CEB/FIP Model code 1990*. 1999. The International Federation for Structural Concrete: Lausanne, Switzerland.
6. ACI Committee 209. 1992. *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures, ACI 209R-92.* American Concrete Institute: Detroit, MI.
7. Bazant Z. P. and W. P. Murphy. 1995. Creep and Shrinkage Prediction Model for Analysis and Design of Concrete Structures – Model B3. *Materials and Structures,* V. 28. No. 180: pp. 357-365.
8. Gardner N. J. and M. J. Lockman. 2001. Design Provisions for Drying Shrinkage and Creep of Normal Strength Concrete. *Canadian Journal for Civil Engineering.* V. 98, No. 2 (March-April): pp.159-167
9. PCI Committee of Prestress Losses. 1975. Recommendations for Estimating Prestress Losses. *PCI Journal*, V. 20, No. 4 (July-August): pp 45-70.
10. Eurocode 2: *Design of concrete structures*. 2002. European standard, prEN 1992-1-1. European Committee for Standardization.

NOTATIONS

A_c = cross-section area of concrete beam

M = bending moment applied from testing machine

P_{eff} = remaining tendon force

S = section modulus of the beam

δ = visually estimated intersection point in load-crack-width diagram

Ø = diameter of strand in tendon

ABOUT THE AUTHORS

Peter Lundqvist, M.Sc. Civ.Eng., is a PhD-student at the Division of Structural Engineering, at Lund University, Sweden.

Juha Riihimäki, M.Sc. Civ.Eng., is the manager at Teollisuuden Voima Oyj in Olkiluoto, Finland.

SYNOPSIS

The Swedish nuclear reactors are enclosed by a prestressed concrete containment. To prevent corrosion, the ducts in several containments have been grouted. When using these system problems arise in monitoring the prestress losses. When the nuclear power plant Olkiluoto was built in Finland in the mid-seventies several prestressed concrete beams were constructed. Five of these beams have recently been tested in order to determine the prestress losses. The prestress losses have also been calculated using models for predicting creep and shrinkage of the concrete and the relaxation in the tendons. This paper presents the results from both the tests and the models.

The measured prestress losses were quite high, ranging between 37-61 percent. The most accurate of the predictions models was model B3, which was in good agreement with the measured prestress losses. Explanations for the scatter in the measured results and the difference between the results obtained from the prediction models and the tests are proposed in the in the main body of this paper.

KEYWORDS

Prestressed concrete, concrete beams, prestress losses, bonded tendons, creep, shrinkage, relaxation.