Reviewing Prestress Losses in An Aged Spanish Concrete Bridge

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Abstract. Prestressed concrete structures have been widely used in Europe since the 1960s for their ability to support long spans. However, as these structures approach the end of their service life, it is crucial to assess their current stress state. Errors in prestressing force calculations and execution mistakes have resulted in the collapse of prestressed concrete bridges. In Spain, bridges built during the 1980s with similar designs are now facing time-dependent prestress losses that affect their prestressing state. Methods for calculating time-dependent prestress losses have evolved to enhance durability. An analysis comparing different regulations, including EP-77, Eurocode 2, and ASSTHO-2020, is proposed to validate calculations. The study focuses on a typical Spanish bridge, considering adjusted loads, time-dependent prestress losses, and current durability regulations. Project calculations and a comparative finite element model using Eurocode 2 parameters are presented. The objective is to identify deviations and errors in predicting short- and long-term prestress losses and meeting serviceability requirements. Advanced age analyses of prestressed concrete infrastructures are vital for ensuring future durability and functionality.

Keywords: Bridge, Concrete, Prestress, Losses, Code.

1 Introduction

A significant portion of the current prestressed concrete infrastructure in continental Europe and the United States has aged since its construction in the 1950s-1960s and is nearing the end of its service life [1]. It is crucial to adequately study the current condition of these elements. Most prestressed concrete structures (PCES) were designed according to outdated codes that did not consider certain considerations related to time-dependent prestress losses occurring in prestressed concrete elements. Additionally, various loading effects on the bridge and aspects of premature degradation of the elements must be taken into account. Therefore, due to the rapid aging and functional deterioration of the structures, this is a significant problem that needs to be addressed promptly.

Hence, accurate determination of residual prestress is essential for the assessment of prestressed concrete structures (PCES) since the prestressing effect has a significant impact on the stress-strain response and capacity in this type of structure. In the design process, the designer must determine the prestressing force and estimate the prestress loss of prestressing force for the structure to meet its requirements during its service life. That is why it is important to obtain an approximation of the theoretical condition of such structures. Therefore, this text proposes, for a specific case, the calculation based on the applicable regulations in force at the time of the bridge's construction, such as the Instruction for the Design and Execution of Prestressed Concrete Works (EP-77) [4], and how it would be if the most commonly used calculation techniques for bridge design in Europe, such as Eurocode 2 (EC2) [5], and the American Association of State Highway and Transportation Officials (AASHTO 2020) [6], were used.

Hence, due to the aging of infrastructure composed of PCES, there is a growing trend in the capacity to detect, quantify, and predict damages by bridge owners, allowing for an effective and safe assessment of structural condition. Traditionally, periodic visual inspections have dominated maintenance programs worldwide [2]. However, it is important to have knowledge of quantifying prestress losses provided by outdated codes and compare them with those offered by more current codes.

Therefore, this study aims to analyze a bridge built in the 1980s located in Spain, initially designed according to the applicable code at that time for prestressed concrete elements, EP-77, and compare the values with those obtained using current codes in Europe and the United States. The main objective is to determine the prestress losses obtained from each of the codes and thus identify the variations between them.

2 Description of Structures

2.1 Geometry

The specimen studied in this paper corresponds to a bridge with a span of 30 meters and a platform width of 12 meters. The bridge section consists of a semi-rigid reinforced concrete barrier and parapet, a 7-centimeter thick bituminous wearing course, and a 25-centimeter thick reinforced concrete slab supported by 6 double T-section beams (see Fig. 1) [3].

Regarding the beams that make up the support of the deck, we have beams with a depth of 2.1 meters and a width of 0.8 meters. The geometric characteristics of the beam can be observed in the left part of Figure 2. There are 5 tendons, all of the same cross-section, with different alignments in both plan and elevation.

As for the arrangement of passive reinforcement, it can be observed in the right part of Figure 2.

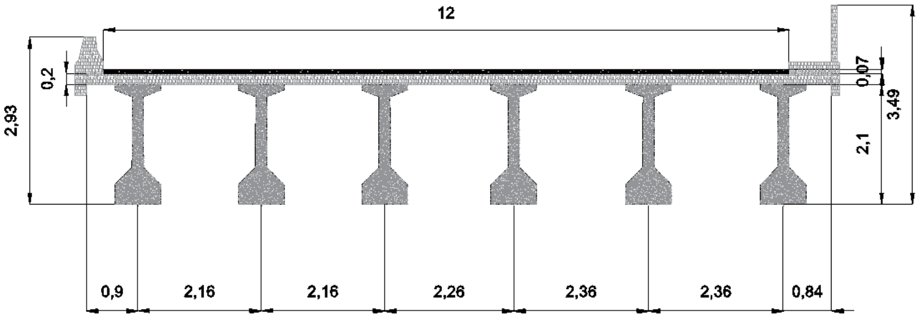

Fig. 1. Cross-section deck.

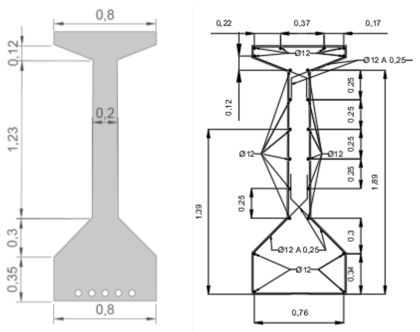

Fig. 2. Cross-section beam.

2.2 Materials Properties

The beams are made of concrete with a strength of 35 MPa, while the modulus of elasticity has been calculated for each of the different codes, resulting in a value of 35545,75 MPa for the design code (EP-77). For Eurocode 2, a value of 34000 MPa was obtained, and for the AASHTO 2020 code, a value of 29437 MPa was obtained.
The steel used for the passive reinforcement in the prestressed concrete elements is of type AEH-400N, with a modulus of elasticity of 210000 MPa and a yield strength of 400 MPa, which is currently referred to as B 400 S.

For the calculation of the layout and dimensioning of the steel cables that make up the active reinforcement, the following characteristics have been considered: a longitudinal deformation modulus of 190000 MPa, relaxation testing at 120 hours, at a temperature of 20°C, and a stress equivalent to 70% of the rupture of 1,35%, and relaxation testing at 1000 hours, at a temperature of 20°C, and a stress equivalent to 70% of the rupture of 2%. As for the cables used as active reinforcement, they have a net tendon area of 11,84 cm^2, a guaranteed breaking load of 1910 N/mm^2, and a load corresponding to the characteristic yield strength of 1718 N/mm^2.

2.3 Loads Pattern

The actions that affect the structure have been limited to permanent actions for the calculation of prestress losses and dimensioning of prestressing force. The permanent actions include the self-weight of the beam, the load of the slab, the wearing course, and the barriers at the ends, with the latter represented as linear loads. On the other hand, variable actions refer to the live load, which is considered as a surface load. In frequent use situations, 40% of the total maximum live load is taken as an additional load.

Considering all these factors, the beam with the highest load demand is chosen as the object of study, and therefore, the calculation of prestress losses is performed.

3 Prestress Losses

According to the initial design, each tendon was prestressed with a prestressing force of 1675 kN (1404 MPa), with the stress level in each tendon being 75% of the ultimate tensile strength (1980 MPa).

For the prestress losses due to friction and wedges' penetration, the same methodology was employed for the different codes, as the calculation for these prestress losses is similar. Based on this approach, the obtained results along the tendon for each of the mentioned codes are presented below.

3.1 Code-Based Analysis

In this section, graphs are presented for tendons 1, 2, and 5, as well as the current stress state, which significantly represent the behavior of the beam. Tendons 1 and 2 are most affected by elastic shortening as they are the first to be tensioned, while tendon 5, being the last in the tensioning sequence, does not experience any effect of elastic shortening. The graphs show the variations obtained according to the type of normative used.The graphs include instantaneous prestress losses, which take into account the elastic shortening, as well as total prestress losses, which reflect all the prestress losses in the tendon. In the calculation of instantaneous prestress losses, prior to elastic shortening, all the normatives used in this analysis share the calculation of prestress losses due to friction and wedging. Therefore, the variation in instantaneous prestress losses is solely due to elastic shortening.

Regarding tendon 5 (see Fig. 5), it can be observed that there is no difference between the different instantaneous prestress losses. This is because, due to the tensioning procedure, tendon 5 is the last one to be tensioned, and therefore it is not affected by prestress losses due to elastic shortening. As a result, the instantaneous prestress losses only take into account prestress losses due to friction and to wedge penetration.

In the graph corresponding to total stresses (see Fig. 6), it can be observed that the upper fiber ($\sigma 1$) is less compressed, while the lower fiber ($\sigma 2$) corresponds to the most compressed fiber in the beam for each of the different normatives used in the calculation.

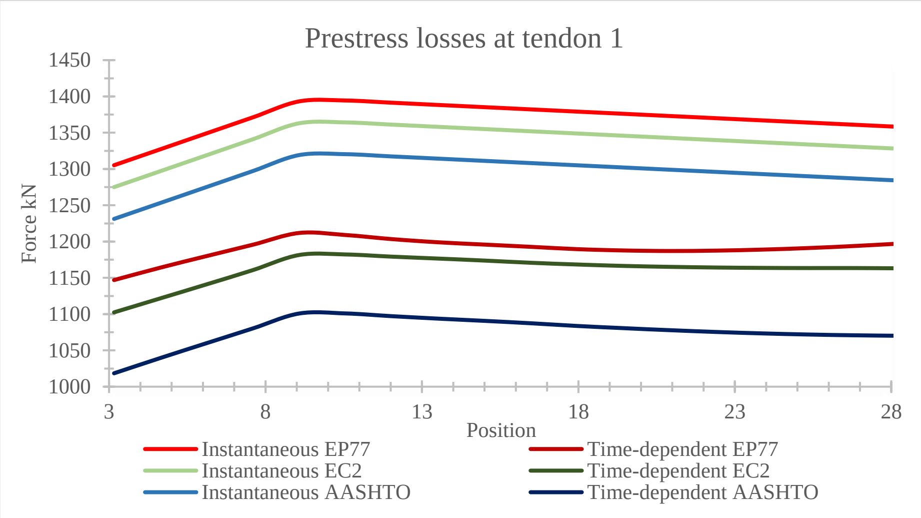

Fig. 3. Pretress losses at tendon 1.

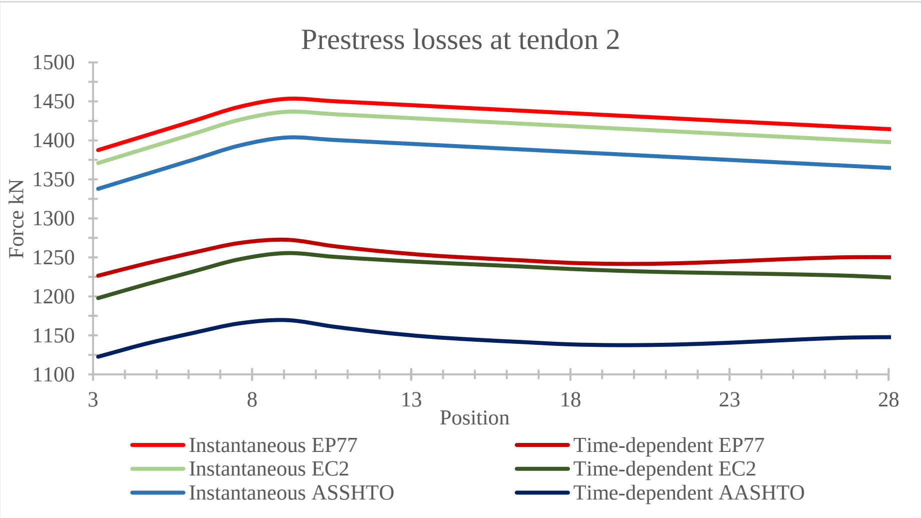

Fig. 4. Pretress losses at tendon 2.

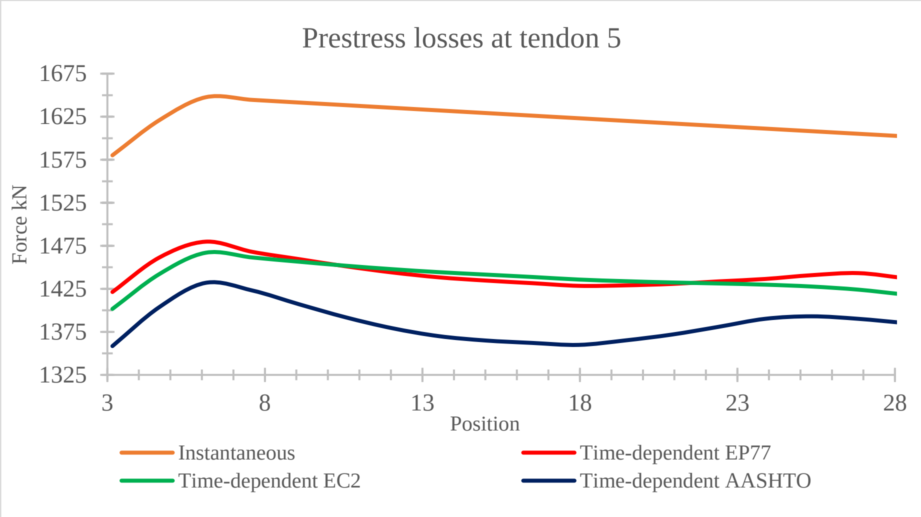

Fig. 5. Pretress losses at tendon 5.

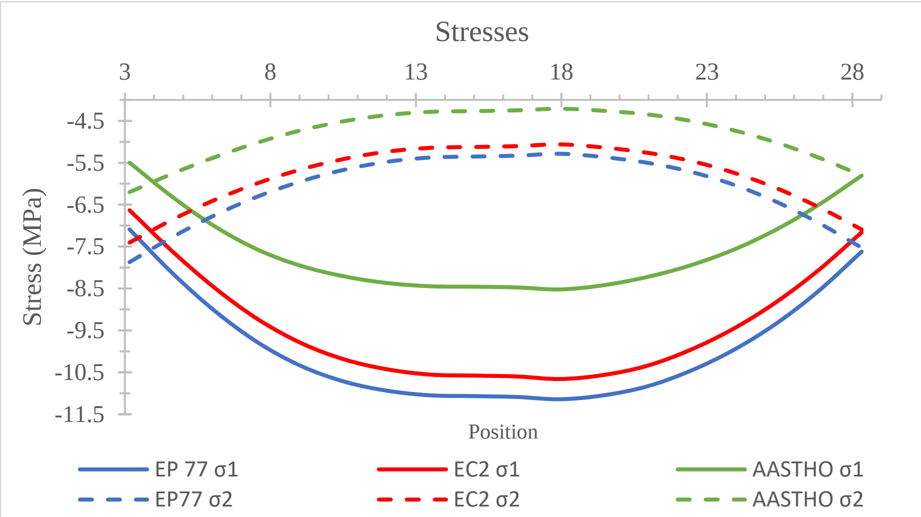

Fig. 6. Maximum and minimum stresses at beam.

3.2 Numerical Analysis

For the numerical analysis, the commercial structural analysis software SAP2000 was used. The staged construction analysis method was employed, which allows for long-term analysis of structural elements, as well as the sequencing of tendon stressing and consideration of loads introduced over the structure's service life [7].

To create the model, the elements were defined as tendon type, and the different tendon layouts were inputted. The calculation procedure followed the guidelines of the

Eurocode 2 (EC2) design code. As can be seen, a comparison was made between the results obtained from the EC2 calculation method and those provided by the finite element model (see Fig. 8).

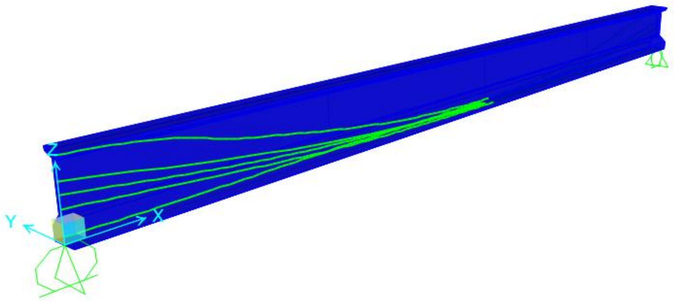

Fig. 7. Beam model from SAP2000.

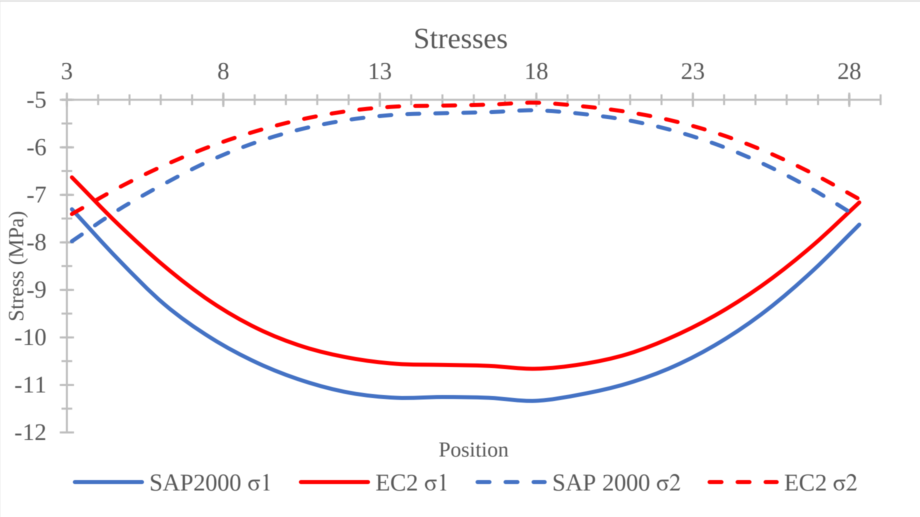

Fig. 8. Maximum and minimum stresses at beam.

4 Discussion

Regarding tendon 1, it can be observed that EP-77 predicts the lowest prestress losses, making it the least conservative option. It has an average prestress loss of 18%, with a maximum prestress loss of 22% in the beam. On the other hand, the EC2 accumulates average instantaneous prestress losses of 20%, with a maximum of 24%. As for the prestress losses calculated according to the AASTHO formulation, they are 23% on average and 26% at maximum. It is important to note that prestress losses due to wedge slip and friction were calculated using the same methodology and parameters provided by the project. The difference lies in the prestress losses due to elastic shortening.

Regarding time-dependent prestress losses in the concrete, EP-77 and EC2 show very similar total values of time-dependent prestress losses. However, it is worth noting that prestress losses due to shrinkage are much higher in EC2, while in EP-77, creep plays a more significant role. This is mainly due to the formulation used to calculate these prestress losses. EC2 takes into account the interaction between shrinkage, creep, and relaxation to obtain the total prestress losses, while EP-77 does not.

A similar pattern can be observed for tendon 2, where EP-77 is the least conservative in terms of instantaneous prestress losses, with an average prestress loss of 15% and a maximum of 17%. On the other hand, EC2 and AASTHO formulations have average prestress losses of 16% and 18%, respectively, with maximum prestress losses of 18% and 20%. The difference lies in the prestress losses due to elastic shortening.

Regarding time-dependent prestress losses in the concrete, EP-77 and EC2 show similar total values. However, EC2 (20%) has higher prestress losses due to shrinkage compared to EP-77 (17%). Additionally, the AASTHO code (23%) shows even higher prestress losses compared to the calculations made using the EC2 code.

For tendon 5, there is a greater difference in the calculation methods for time-dependent prestress losses. It can be observed that the highest time-dependent prestress losses, both in concrete and steel, occur according to the AASHTO code. On the other hand, EP-77 and EC2 yield quite similar results, with an average prestress loss of 14% for both and a maximum prestress loss of 15% and 17%, respectively. In contrast, the AASHTO code has an average prestress loss of 17% and a maximum prestress loss of 19%.

Regarding the time-dependent prestress losses, as previously observed in each of the tendons, the code that contributes the highest tension loss in the structure for each tendon can be seen in Figure 6. It is evident that the AASTHO code provides the highest tension loss. Consequently, after analyzing the stresses starting from the middle tendon, the AASTHO code again indicates a global state in the structure with less compression. In terms of comparing the ESP77 and EC2 codes, they provide very similar tension values. This similarity arises from the resemblance in calculating the time-dependent parameters among the different codes. However, the EC2 code is slightly more conservative due to modifications in obtaining parameters, leading to more conservative results than those obtained from the ESP77 code.

Regarding the comparison with the finite element model, the results obtained using the prestress loss formulation provided by the EC2 code reveal prestress losses around 11% compared to instantaneous prestress losses. On the other hand, the finite element method (FEM) yields a slightly higher value, with prestress losses of 13%. This is mainly because the values provided by the EC2 code are calculated based on approximate data regarding concrete elastic shortening values, while the FEM performs a more accurate calculation of elastic shortening.

As for time-dependent prestress losses in the concrete, the finite element method shows average total prestress loss values close to 27% of the initial prestress force. On the other hand, the calculation performed using the EC2 code provides values around 25%. This indicates that the finite element method offers very similar results to EC2 in terms of time-dependent prestress losses in the concrete, as the creep, shrinkage, and relaxation parameters of the concrete used are those provided by the EC2 code. Finally in figure 8, it can be observed that the tension values existing in the beam for the Eurocode 2 case, the compressive stresses in the beam are lower than those provided by the finite element model, specifically in the upper fiber (less tensile zone), the value is 6.8% lower than the calculated model, while for the most compressed fiber, it is 4.4% lower.

5 Conclusions and Future Lines of Research

This article has presented a specific case study on the evaluation of prestressing in a concrete beam. Three different design codes have been used to assess the prestressed state of the bridge based on the applicable regulations. It has been possible to verify the differences between modern regulations and those used during the bridge design, as well as the deviations from the calculation codes based on the age of these elements. Therefore, the conclusions drawn are as follows:

- During the 1980s, the design methods used in Spain for most structures tended to underestimate the prestress losses of prestressing compared to current codes, particularly in terms of time-dependent prestress losses.
- The American code method presents significant advantages, especially in terms of time-dependent parameters, as the prestress losses at this stage are higher than those provided by the EC2 code.
- In this particular case, a consistent trend is observed where the prestress losses according to the EC2 code tend to be approximately 4% lower compared to the AASHTO code. This suggests that the AASHTO code aligns more closely with the calculation obtained by the finite element method (FEM).
- Regarding the EP-77 code, the prestress losses obtained during the study were 10% for instantaneous prestress losses and 21% for time-dependent prestress losses. These values are lower than those obtained with other codes, indicating that this code may be indicating higher prestressing values than what actually exists in the structure. This could be potentially dangerous in some cases, as having less acting prestressing force increases the risk of failures if these parameters are not adequately considered.
- It is unclear whether the values provided by different codes are sufficient to adequately evaluate the condition of current bridges, as there are numerous variables that cannot be solely considered based on the codes.

The uncertainty of not knowing the actual prestressing force in the cables is a significant problem. If it were possible to accurately determine the current prestressing force through periodic inspections of prestressed beams, it would be much easier to develop maintenance and repair plans.

The future of this field lies in seeking techniques that can accurately assess the current prestressing force in structures. There is already a considerable amount of research that has made progress in this area, successfully obtaining tension measurements in the active reinforcement. In modern structures, there are sensor devices and monitoring systems that track the variation of prestressing force over time.

However, it is essential to have initial data on the structures and their evolution throughout their service life.

Other research avenues are focused on obtaining residual prestressing in structures through indirect measurement techniques [8]. These methods involve measuring the prestressing force at a specific moment. Techniques like Crack Opening [9,10] or Tendon Cutting [11,12] have shown promising results, but they are destructive and render the structure unusable.

Therefore, it would be valuable to develop non-destructive indirect measurement techniques for assessing prestressing force. Currently, numerous research groups are dedicated to interpreting results and parameterizing the current state of prestressed concrete elements based on experimental methods. This line of research appears to be appropriate for further investigation in order to accurately determine the tensional state of these structures.

Acknowledgments

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