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Long-term deformation assessment of a long-span concrete bridge built by the cantilever erection method

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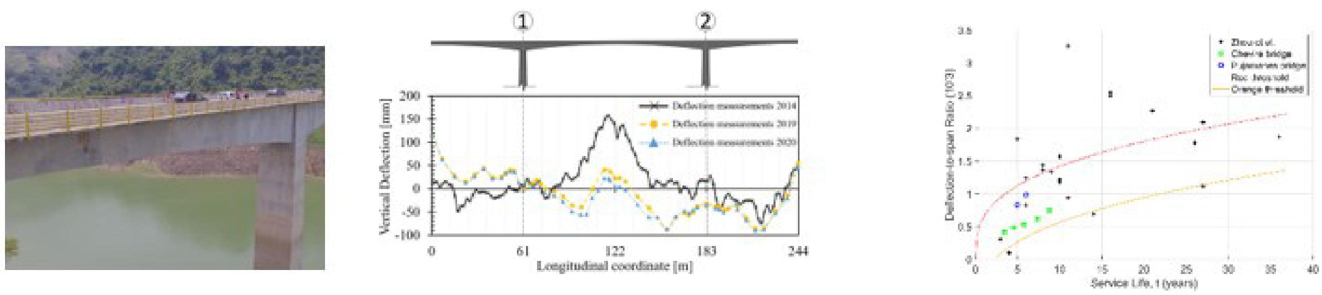
Abstract

The balanced cast-in-place cantilever erection method has become a popular construction bridge technique. The main advantages include an industrialized erection technique that prevents the use of intermediate supports and rapid construction. However, the long-term response of this bridge typology is not well understood as long-term deflections due to time-dependent phenomena (such as creep and shrinkage) are significantly simplified in bridge design codes. Existing prediction models commonly used in design tend to underestimate long-term deflections, and as a result, field measurements conducted in newly constructed bridges still report excessive long-term deflections. This paper shows the long-term deflection analysis of a long-span concrete bridge located in Colombia and opened to traffic in 2014. This study is conducted using field data collected within a one-year time interval and modelling results. Further comparisons using field data collected from bridges with similar structural configurations, but opened to traffic in the 90s and 2000s, show that the bridge's current situation is categorized as a major concern.

Keywords

Long-span box-girder concrete bridge; segmentally balanced cast-in-place cantilevers; long-term deflection; creep; shrinkage.

Graphical Abstract



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1 INTRODUCTION

In the Prestressed Concrete Box Girder Bridge (PSCB) typology, deck erection is conducted by constructing new segments supported in the previous ones. It is considered a suitable option for the main span range of 100 m – 300 m in high-altitude mountainous areas of difficult access (Pacheco, 2015). The cantilever erection method is carried out with the help of auxiliary cantilever structures called “travelling formworks”, which are used to compensate for the deflections generated during the cantilevered erection. Bridge sections are then cast with an upward vertical displacement. The first PSCB by segmentally balanced cast-in-place cantilevers was built in Brazil in 1930 with span arrangements of 26.8 m + 68.3 m + 26.8 m (Garrido Padilla, 2018). However, the boom of this bridge typology began thirty years later in countries such as Germany, France, and Holland, and from the 70s, its use was spread to the rest of the world (Barras et al., 2003). The construction of PSCB with larger spans has allowed the visualization of excessive deflections that contribute to the deterioration in the long term, thus leading to enormous economic, environmental, and social costs. Hołowaty stated that the problem of excessive deflection in PSCBs is directly associated with existing limitations of models to predict the behaviour of concrete over time (Hołowaty, 2015). The increment of deflections over time produces a redistribution of stresses that can trigger additional cracks in the concrete, accelerating the development of corrosion in the structural members and decreasing the safety margin. Such processes compromise the durability and sustainability of the concrete structure (Zdeněk P. Bažant, 2001; Gardner & Zhao, 1993; Tong et al., 2016). An accurate response forecast must consider mechanical and environmental loads from the construction until the end of the expected lifespan. The most influential factors that lead to the deterioration of these structures due to excessive deflections are creep and shrinkage on concrete. However, difficulties in modelling these phenomena are directly associated with a lack of accuracy in determining the concrete behaviour over time.

The creep and shrinkage prediction has focused mainly on empirical models that have been adopted in bridge codes. However, these phenomena are greatly affected by temperature and humidity changes. In addition, the databases of such empirical models consist of samples with no more than six years, but scientific evidence has shown that records of deflections in more extended periods are necessary to calibrate these models. The Model Code (MC) was published by the CEB/fip (Comité Euro-International du Béton / Fédération Internationale de Béton) and developed by Müller & Hilsdorf (1990). The model emerged from the RILEM (International Union of Laboratories and Experts in Building Materials, Systems, and Structures) database, and it is suitable for structures up to 70 years old with concrete of normal density, exposed to an average relative humidity between 40 and 100% and concrete compressive strength values less than 50 MPa. The main characteristic of the model is the low data requirement during the design process, but this aspect generates the impossibility of differentiating between the creep component due to drying and its primary component. In addition, the effects due to the variation in the curing conditions are not considered. The MC was updated in 1999 (hereinafter referred to as CEB-MC90-99) to meet the requirements of high-strength concrete (Mueller & Haist, 2009). Three dependent coefficients were introduced on the loading processes of the original model for creep calculation, and the calculation for shrinkage was divided into autogenous and drying components. In 2010, the development of high-strength concrete reinforced with metallic and non-metallic fibres motivated the development of an updated version of the model named MC 2010 (Fédération internationale du béton, 2012).

Branson and Christianson developed the model 209R-92 in 1971. It was adopted in the guide for modelling and calculation of shrinkage and creep of the American Concrete Institute (ACI 209.2R) in 1982 and updated in 1992. Since then, it has not been modified (ACI Committee 209, 1995). Nevertheless, the latest version has been widely used in the prefabricated industry due to the low data required for analysis and easy short-term calibration despite its limited accuracy. The GL2000 model developed by Gardner & Lockman (2001) was based on simplifying the Atlanta 97 model to adjust to the ACI regulations and was updated in 2004 (Gardner, 2004). It can be placed in similar conditions to the model 209R-92. The Bazant and Baweja model or B3 model (Zdeněk P. Bazant & Baweja, 1995a; Zdenêk P Bažant & Baweja, 1995b) is among the most used models in the scientific literature for the calculation of long-term effects on bridges; the model was the culmination of work started in the 70s (Z. P. Bažant et al., 1976). It is based on the mathematical description of more than ten physical phenomena that affect the development of creep and shrinkage (Z. P. Bazant, 2000) and has been widely used to calculate complex structures (Roman Wendner, Hubler, et al., 2015b). In addition, the model established that creep increases indefinitely over time and separates creep into drying and basic components (Zdeněk P. Bažant & Baweja, 1995a; Zdenêk P Bažant & Baweja, 1995c; Su et al., 2017; Tong et al., 2016; Roman Wendner, Hubler, et al., 2015b, 2015a). This aspect provides the B3 model with a significant difference in robustness from the previously developed models but imposes limitations in its practical implementation due to the required input data. It is essential to highlight that the B3 model was calibrated from extended laboratory database testing, adjusted and optimized statistical data, and a better understanding of the phenomena (Z. ˇk P. Bažant et al., 2012b; Z P Bažant et al., 2011; Zdeněk P. Bažant et al., 2012b; Tong et al., 2016; Roman Wendner, Tong, et al., 2015). Finally, lightweight aggregate concrete and concrete admixture technology motivated to recalibrate the prediction equations through a database compiled at Northwestern University that includes about 1,400 laboratory tests and deflection data collected from 69 bridges. This model adjustment was presented in 2014 as model B4 (R. Wendner et al., 2013, 2014), preceded by two articles that contained the optimization method and the statistics of the shrinkage formulation (Roman Wendner, Hubler, et al., 2015b, 2015a).

Current design practice for PSCBs calculates deflection using the recommendations provided by the ACI or the CEB/fip. However, data from field measurements collected from existing bridges show an underestimation of long-term bridge deflection, affecting the life span defined during the design of such bridges. A significant reason for this limitation is data scarcity used in developing, calibrating, and validating deflection prediction models. Despite the great efforts to improve such models (Zdeněk P. Bažant et al., 2012a; Z P Bažant et al., 2011; Zdeněk P. Bažant et al., 2012b; Maekawa et al., 2011; Roman Wendner, Tong, et al., 2015), few of them consider information that exceeds the scope of laboratory tests (R. Wendner et al., 2013). Recent studies using field data collected from existing bridges still show excessive deflections, and therefore, it is imperative to improve prediction models and approaches to measure and monitor long-term deflections for PSCBs. It is not surprising that the progress in the numerical modelling of these phenomena has stretched over several generations of researchers and is still incomplete (Roman Wendner, Hubler, et al., 2015b). Such affirmation is also supported by excessive deflections investigations on more than 80 bridges of this type around the world, with main spans ranging from 180 m to 301 m; where the Koror-Babeldaob bridge built in Palau in 1977 stands out, this bridge with a span arrangement of 72m + 241 m + 72 m reached a deflection of 1.61m after 18 years from its opening (Burgoyne & Scantlebury, 2006; Mcdonald et al., 2003). Such investigations have revealed that PSCBs suffer from excessive deflections associated with long-term effects between 20 and 40 years of being put into service, less than their lifespan, and, in some cases, they require a bridge replacement or a risky corrective prestressing. Due to current limitations in adequately predicting the long-term deflection of PSCBs, monitoring strategies have been developed in recent years. However, the cost associated with such implementations is prohibited, especially in developing economies, and thus a large number of PSCBs are currently constructed in such economies without strategies to guarantee early detection of excessive deflections. This paper aims to perform a deflection assessment using field data collected from a PSCB constructed in 2014. Three data sets are used: data collected in 2014 prior to the bridge's opening to traffic and data collected 5.42 and 6.33 years after the bridge's opening to traffic. Deflection results predicted by the model used during the design process of the bridge are compared with field measurements. Finally, deflection measurements obtained from similar PSCBs constructed in the 90s and 2000s are selected to conduct a comparative analysis using data collected from PSCB with an excessive long-term deflection to describe the current situation of the bridge and estimate a possible future scenario for the bridge. The structure of the paper consists of an analysis of existing prediction models for long-term-deflection and analysis of data collected from monitoring of PSCB to conduct numerical and field data analysis to show the current situation of the bridge.

2 STATE OF THE ART

2.1 Case studies on PSCBs deflection

One of the oldest investigations that report excessive deflections in PSCBs built by successive cantilevers was developed from data obtained by the National Swedish Road Board in 1970 (Keijer, 1970). This investigation analyzed the deflection measurements of four PSCBs for 15 years, determining a dominant influence of creep for the first years of service of the structures. The bridges analyzed were Tunsta, Stenungsun, Anlo and Kallosund, with main spans of 106.5 m, 94 m, 134 m, and 107 m, and maximum reported deflections of 110 mm, 80 mm, 110 mm, and 80 mm, respectively. The study determined that all cases have larger values of deflections than those expected by design. Subsequent works show a record of excessive deflections of 300 mm on the Grandmere bridge in a span of 181.4 m; 223 mm on the Humen bridge in a span of 270 m; 635 mm on the Parrotts bridge in a span of 195 m; 200 mm on the Stovset bridge in a span of 220 m and the Stolma bridge of 92 mm in a span of 301 m (currently, the second-longest midspan PSCB) (Hunter & Manzanarez, 1992; Massicotte & Picard, 1994; Xie et al., 2007). Further, Takács (2002) worked with the measurements of the Stovset, Stolma and includes the Norddalsfjord bridge to perform statistical analysis by Monte Carlo simulation to reduce the uncertainty in the prediction of long-term effects in lightweight aggregate concretes using known creep and shrinkage models. Subsequently, a monitoring program implemented in the Halawa North Valley viaduct on the Hawaiian island of Oahu by Robertson (2005) allowed a comparative analysis based on a Finite Element (FE) model incorporating recent creep and shrinkage models, thus improving long-term prediction. Another investigation started by decommissioning two PSCBs in Sweden due to extensive cracks (Malm & Sundquist, 2010). Then, research efforts focused on studying the effect of creep and shrinkage on such cracking conditions, concluding that to assemble a FE model of a bridge, in this case, the Grondal bridge, these two phenomena must be taken into account in order to avoid overestimating of cracking during the construction processes (Malm & Sundquist, 2010). Due to the facts mentioned above and other research findings, creep and shrinkage prediction models have received special attention due to their influence on calculating excessive deflections (Zdenek P. Bazant Guang-Hua Li, Gary J. Klein, and Vladimir Kristek, 2010). Then, a data collection of 69 bridges worldwide was analyzed using the most relevant prediction models, concluding that the B3 model provided reliable long-term prediction, but a recalibration was necessary (R Wendner, 2018).

The most critical case of excessive deflections was recorded on the PSCB built by successive cantilevers in 1977 in Palau, which held the world record span of 241 m (Yee, 1979). According to Mcdonald et al. (2003), its final deflection was expected to be between 710 mm and 737 mm, but it reached a value of 1.61 m 18 years after its opening to traffic. That is 5.3 times larger than the allowed deviation. A corrective prestressing was implemented, but a delay of three months led to the collapse of the structure (Burgoyne & Scantlebury, 2006; Mcdonald et al., 2003). More recent studies, such as the one carried out by Tong et al. (2016) on the Humen I and II bridges in China, proposed a constitutive model considering the coupled effects of viscoelastic behaviour, tensile cracking, and plastic softening of concrete. The model was validated from inspection reports (deflection measurements at midspan, deformed profile of the entire bridge, and distribution of concrete cracks). The model predicted maximum deflections of 180 mm and 200 mm, respectively, five years after its opening to traffic. The results supported a realistic modelling approach by considering the effects of cyclic creep and concrete cracking in PSCBs. On the other hand, the high-strength concrete approach implemented on the Sutong bridge determined the long-term material performance from a three-level experimental approach (Material, component, and structural) (Pan & Meng, 2016). The research presents a modified model that better reflects the characteristics of creep and shrinkage, improving the prediction of deflections of the bridge.

The Cheviré bridge in France, with a main span of 242 m and a total length of 1,563 m, experienced excessive vertical deflections detected in 1994 after three years of construction (Raphael et al., 2018). A series of deflection measurements aimed at detecting excessive vertical displacements showed that 1,275 days after construction, the bridge experienced a vertical displacement of 100 mm, reaching a vertical displacement of 182 mm after 3,200 days. It is important to note that the vertical displacements measured in the Cheviré bridge were also measured after 1,650, 2,125, and 2,709 days showing values of vertical displacements of 117 mm, 128 mm, and 149 mm, respectively. Based on the deflection values mentioned above, it is possible to state that the Cheviré bridge experienced an average deflection value at the main span of approximately 21 mm each year. Raphael et al. (2018) compared the results of long-term deformation in the Cheviré bridge predicted by the BPEL code and the Eurocode 2 using reliability analysis and FE modelling. Although long-term deformation in the Cheviré bridge computed from Eurocode 2 provided better prediction results than the results provided by BPEL, both design codes underestimated creep deformations. Raphael et al. (2018) concluded that concrete creep should be well predicted and controlled to guarantee structural safety.

Cao et al. (2018) stressed that in-situ experiments and laboratory model experiments show variations and limitations in simulating the bridge response. Therefore, Cao et al. (2018) conducted experiments with prestressed concrete box beams under sustained uniform load for 470 days showing that concrete cracks reduce the stiffness of the box beams and therefore decrease the creep effect by releasing concrete stress. Yang et al. (2020) conducted experimental work similar to Cao et al. (2018) but using prestressed concrete T-section beams with the four-point bending strategy loaded for 1,400 days. Yang et al. (2020) experimentally proved that the MC 2010 model underestimated midspan deflections and proposed an improved method to predict the creep of prestressed concrete structures. The above facts show that existing creep model prediction models still need further development. An equal amount of effort should be put into developing practical approaches to monitoring the deflection of existing and newly constructed bridges.

2.2 Deflection monitoring of PSCBs

The first step in establishing the service status of a structure is the attainment of proper instrumentation and monitoring to record possible excessive deflections from which the numerical models are subsequently validated. The measurement and monitoring of bridge deflections can be carried out through direct and indirect methods. The first is the most implemented in the scientific literature to study PSCBs. Generally, the simplest and most reliable way to perform monitoring using the direct method is by altimetry. Deflection measurements are collected using a precision level on the superstructure deck by considering the importance of recording temperature and humidity during the measurement process, such as in the study by Akl et al. (2017). This method is also used during the bridge's construction process, allowing control over the stresses of the structure, guaranteeing the design adjustment from the elevations in the constructed sections and the “travelling formworks”. Even when there is no subsequent monitoring plan, the control during construction should always be carried out. However, long-term monitoring is recommended for obtaining information that allows design verification, improvement of theoretical models, and construction processes.

Similarly, linear displacement transducers such as LVDT (linear variable differential transformers) have mainly been used to directly measure deflections in bridges during specific events such as load tests. It is important to note that the installation of these instruments requires detailed planning of their location, such as the one carried out on the Tauranga Harbor Link bridge (Ibrahim et al., 2008). For periodic measurement of deflections in this method, laser equipment such as LDV (Laser Doppler Vibrometer), photogrammetry, or microwave radars have been used (Board et al., 2005; Gentile & Bernardini, 2010; Ja et al., 2003; McCarthy et al., 2014). However, in some cases, such approaches lack the necessary precision and generate errors in the measurement (Helmi et al., 2015; Moschas & Stiros, 2013; Roberts et al., 2004; Watson et al., 2007; Yi et al., 2011). On the other hand, indirect methods calculate deflections from deformations, generally by using gauges welded to reinforcing steel during bridge construction. These gauges are constituted by an electrical or optical system susceptible to temperature variations and continuously collect information on specific structure locations. However, variations in the material's homogeneity or the sensor's degradation over time can generate incorrect readings, and therefore, it is recommended to be used together with other measuring instruments to ensure proper analysis. Takács (2002) implemented an indirect approach in the North Halawa Valley viaduct, where deformations were monitored continuously during the load tests of the structure and with periods of 2 hours for subsequent measurements. Another indirect method combines acceleration records with Multiple Impact References (MRIT) (Tian et al., 2017). This approach performs measurements of frequencies generated by impacts to identify structural flexibility and estimate deflections and excitations under specific loads, relying on adapted instruments, such as long gauges. More recently, fibre optic Bragg-grating strain sensors have been used due to their high resolution, which allows a better estimation of deflections; however, their use has been concentrated in laboratory tests (Chang & Kim, 2012; Helmi et al., 2015; Kim, 2004; Tian et al., 2017).

The previously mentioned approaches for deflection monitoring of PSCBs have proven to be expensive, mainly due to high maintenance costs. A cost-effective and real-time deflection monitoring system is still desirable. Therefore, an alternative direct measurement based on hydrostatic level measurements was firstly proposed by Burdet & Badoux (1999). Ye & Chen (2019) proposed a dynamic deflection monitoring approach based on liquid pressure transmitters installed on pipes named Liquid Level System (LLS). The dynamic pressure approach considers disturbances caused by atmospheric pressure, temperature, vibration, and noise. The Hanxi PSCB, with a main span of 160 m and opened to traffic in 2012, was instrumented using the LLS. Previous deflection measurements conducted in the Hanxi bridge showed values of 28 mm in midspan after three years of construction, showing an average deflection value of approximately 10 mm each year. A two-deflection level threshold approach was developed to determine static and dynamic deflections. More recently, Zhou et al. (2021) presented an improved LLS to monitor bridge deflections by considering the individual contribution of traffic-induced vibration, temperature variations, and long-term deformation. The proposed LLS was implemented on a PSCB located in the Guangdong province in China. The bridge having two main spans of 160 m, was opened to traffic in 1996, but during an inspection in 2014, structural problems such as concrete cracking, water seepage, and crystallization in several segment junctions were detected. Zhou et al. (2021) concluded that long-term deflections exceeded design specifications. Niu & Tang (2019), using deflection monitoring data from a PSCB, successfully demonstrated that traffic-induced vibration, temperature variations, and long-term deformation could be separated based on the corresponding distinctively features in the frequency domain. The authors highlighted that PSCBs currently in service are suitable to be instrumented to study long-term deformations during more extended periods and therefore perform robust long-term deflection measurements. In the following section, the facts mentioned above are used to conduct numerical and field data analysis to show the current situation of the bridge selected as a case study in this paper.

3 DEFLECTION ASSESSMENT OF THE PUJAMANES BRIDGE

A case study is selected to evaluate if the long-term deflections were or were not underestimated and the corresponding effects on the structure. Figure 1 shows the Pujamanes PSCB, located in Colombia. The bridge was opened to traffic in 2014 as part of a road connecting Bucaramanga with San Vicente de Chucurí (Municipality of Santander), has span arrangements of 60.5 m + 122 m + 60.5 m with a constant width of 10.55 m and a pylon height of 57 m. The superstructure consists of 72 cast-in-place concrete segments with a variable height ranging from 6.7 m (piers) to 2.4 m (mid-span). ASTM A416, Grade 270 is the design specification for the prestressing tendons (ultimate stress of prestressing strands fu = 1860 MPa). The compressive strength values of concrete used for the pylons and superstructure are 28 MPa and 35 MPa, respectively. The rectangular hollow-section pylons are connected to rectangular pile caps with a section of 8 m in length, and 4 m wide supported on 2.5 m diameter piles. The pile length varies from 21 m to 26 m.

Figure 1. View of the Pujamanes Bridge

Deflection measurements using a precision level on the superstructure deck were recorded prior to the bridge's opening to traffic in 2014, representing the starting deflection condition. The bridge code used for the design of the bridge was the Colombian bridge code (CCDSP-95) (Asociacion Colombiana de Ingenieria Sisimica, 1995), which is based on the standard AASHTO (1992). The loss in prestressing due to the creep of concrete specified as CR_c and the shrinkage strain factor SH adopted in the CCDSP-95 are presented in Eq. (1) and Eq. (2), respectively.

$$CR_c = 12f_{cir} - 7f_{cds} \quad (1)$$

$$SH = 0.80\left(1{,}190 - 10.5RH\right) \quad (2)$$

The concrete stress f_{cir} is computed at the centre of gravity of the pretensioning steel due to the pretensioning force and the dead load of the girder immediately after transfer, and the concrete stress f_{cds} is also computed at the centre of gravity of the prestressing steel due to all dead loads except the dead load present when the prestressing force is applied. The annual mean relative humidity RH is defined in percentage (see additional modelling parameters in Table 1). Design specifications of the Pujamanes bridge showed that the downward deflection based on shrinkage, creep, prestress loss and live loads was considered in the design of the prestress; therefore, the highway alignment was designed to be horizontal in the long term. It is essential to highlight that the objective of the present article is to analyze the long-term deflection prediction based on the shrinkage and creep models employed during the design of the bridge. Such models did not consider prestressed loss due to crack opening. Therefore, the FE model presented in this paper does not consider stiffness reduction due to concrete cracking. However, scientific evidence shows that models such as B3 consider complex phenomena' interaction and could provide reliable long-term deflections (Rincon et al., 2021).

3.1 Bridge modelling

The construction drawings provided by the bridge contractor are used to assemble a FE model of the Pujamanes bridge using the *MIDAS CIVIL Software* (2019). The FE model consists of 86 Timoshenko beam elements to consider the stiffness effects of tension/compression, shear, bending and torsional deformation. The segmental construction of the bridge adopted the cast-in-situ cantilever symmetric construction method. Therefore, the construction stages based on contractor reports are included during the assembly of the FE model. The initial load stage is related to the weight of the travelling formwork, followed by the pouring of concrete for giving curing conditions, and the final stage with the application of the prestress force once the concrete has reached the resistance of 28 MPa. The construction drawings specified the application of prestress to steel tendons after casting concrete of each segment and the mid-span closure segment. Construction load values are presented in Table 2. Long-term effects are incorporated in the FE model based on Eq. (1) and Eq. (2). Figure 2 shows the FE model of the Pujamanes bridge.

Table 1. Characteristics for creep and shrinkage models

Properties	Value
28-day strength	35 MPa
Maximum aggregate size	19 mm
Slump	75 mm
Curing time	5 days
Age at loading	5 days
Relative humidity	85%
Temperature	25 ºC

Table 2. Applied loads during the construction process

Load type	Load (kN)	Activation (Days)	Duration (Days)
Prestress tendons of the previous segment	1435-2460	0	Unlimited
Formwork traveler	1100	0	10
Wet concrete	450-1680	10	5
Segment Self weight	450-1680	15	Unlimited

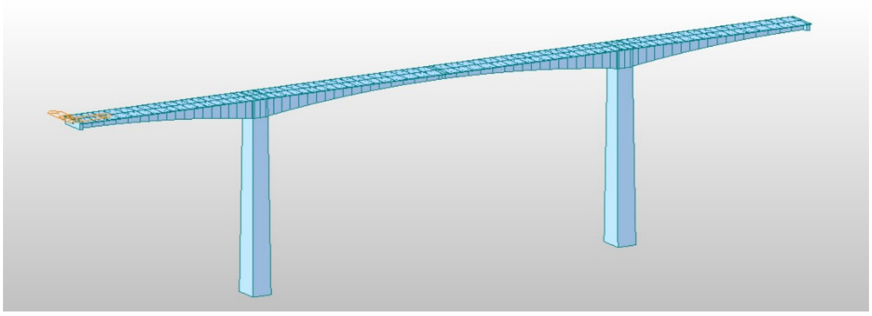

Figure 2. FE Model of the Pujamanes Bridge

3.2 Deflection measurements

The bridge contractor measured the construction levels in 2014 before the bridge's opening to traffic. The bridge deflection was measured at 18 points, 9 points placed at the bridge's pedestrian walkway and 9 points placed at the bridge centre line to evaluate the deflection of the bridge under controlled loads. The construction report did not show evidence of cracks appearance. No deflection measurements were collected during the time interval between the year of its opening in 2014 and 2019. In 2019, 5.42 years after the bridge's opening to traffic, a first deflection measurement campaign was conducted using a digital level (precision of 0.7 mm and maximum reading distance of 100 m). The digital level was located along the bridge's pedestrian walkway to collect 55 deflection measurements. A visual inspection of the bridge conducted in 2019 did not show evidence of cracks appearance. However, inspection access inside the box sections was not possible. In 2020, 6.33 years after the bridge's opening to traffic, a second deflection measurement campaign was conducted using the same procedure employed for the 2019 campaign. Figure 3 shows the deflection measurement results obtained in 2019 and 2020 and the construction levels measured in 2014. The measured deflections in the centre of the mid-span were 119 mm in 2019 and 138 mm in 2020. It is possible to state that the Pujamanes bridge experienced an average annual deflection value of approximately 19 mm. Figure 4 compares the numerically predicted deflection based on the FE model and the measured deflection results from the campaigns in 2019 and 2020. Significant differences are observed between the deflection numerically predicted at the centre of the mid-span and the results from the deflection measurement campaigns.

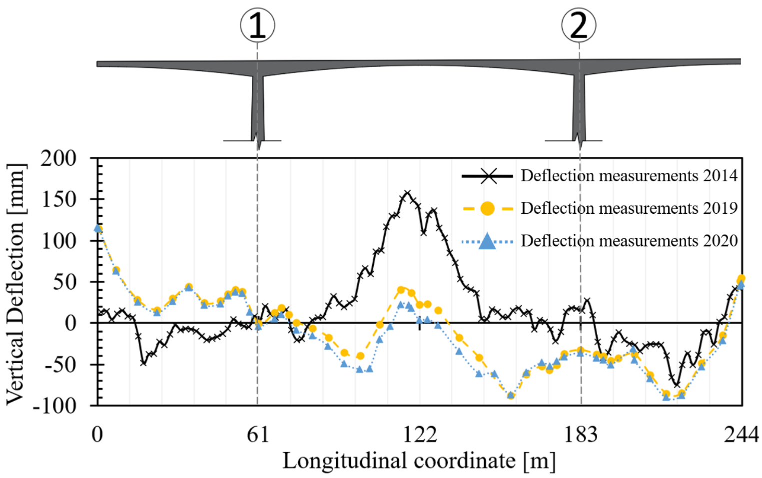

Figure 3. Evolution of the vertical deflection of the Pujamanes bridge

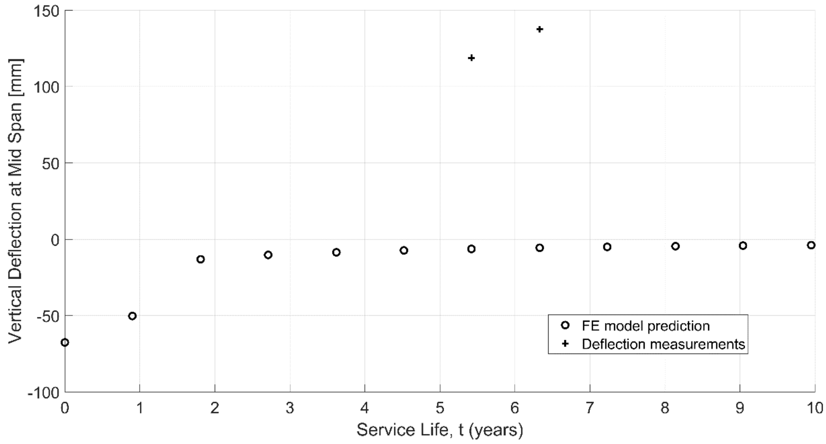

Figure 4. Numerical prediction and measured long-term deflection of the Pujamanes bridge at mid-span

3.3 Long-term deflection analysis

Based solely on average deflection values is not possible to assure potential risk in the future for the Pujamanes bridge. Therefore, the bridge deflection-based multilevel long-term deflection assessment developed by Zhou et al. (2021) is adopted to study the current deflection situation of the Pujamanes bridge. This assessment framework considers that a combination of not well-understood factors causes long-term deflection, and large deviations in model predictions are related to a complicated mechanism. A long-term deflection index was statistically developed using information from PSCBs with long-term deflection issues based on the deflection-to-span ratio. Eq. (3) corresponds to the red threshold. The orange and yellow thresholds defined as the average long-term deflection 10% and 1% quantiles are shown in Eq. (4) and Eq. (5), respectively. The service life of the bridge is defined as t.

$$0.000648t^{0.341} \tag{3}$$

$$0.000648t^{0.341} - 0.000857 \tag{4}$$

$$0.000648t^{0.341} - 0.001556 \tag{5}$$

Any value below the orange threshold indicates minor concern, below the red threshold indicates major concern, and above the red threshold indicates danger. Figure 5 compares the numerically predicted long-term deflections and the red, orange and yellow thresholds. Modelling results are located below the yellow threshold showing a tendency to rapidly increase in the early stage and, as time progresses, grow at a slower rate. The deflection-to-span ratios for the Pujamanes bridge computed from deflection measurements conducted in 2019 and 2020 are 0.975 10^{-3} and 1.131 10^{-3}, respectively. The 22 records collected from the PSCBs with long-term deflection issues used in deriving the multilevel index of long-term deflection proposed by Zhou et al. (2021) are also shown in Figure 6. It is observed from Figure 6 that the Pujamanes bridge's long-term deflection condition is located in between the orange and red thresholds. It implies that the bridge condition deserves special attention (major concern). The deflection-to-span ratios of the Cheviré bridge are also shown in Figure 6. This bridge is also categorized as a major concern but shows a closer tendency to the orange threshold.

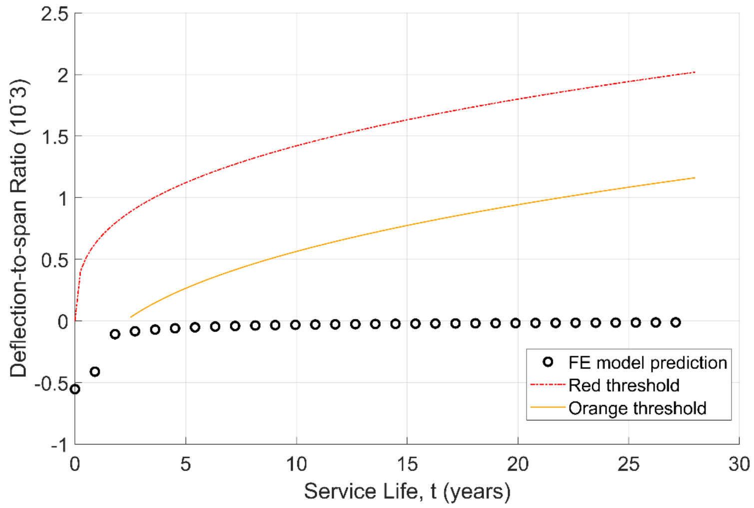

Figure 5. Numerical long-term deflection prediction of the Pujamanes bridge.

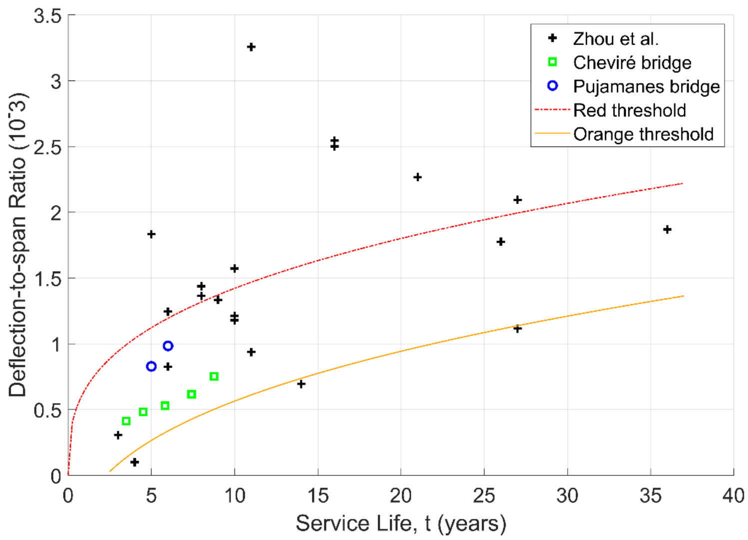

Figure 6. Long-term deflection index comparison of the Pujamanes bridge.

The length of the central span of the Cheviré bridge is double that of the Pujamanes bridge. This fact is vital due to prestressed loss caused by steel relaxation, which affects and is affected by creep and shrinkage, but it is important to stress that creep sensitivity is strongly related to self-weight (Raphael et al., 2018). The measured displacement in the Cheviré bridge after 2,125 days of bridge setup was 128 mm, in contrast with the measured displacement of 119 mm after 1,978 days (5.42 years) of the Pujamanes bridge setup. After 2,125 days of the Cheviré bridge setup, the measured displacement was 128 mm, and in the case of the Pujamanes bridge, the measured displacement was 137 mm after 2,310 days (6.33 years) of the bridge setup. As shown In Figure 6, the value of the annual increment of the deflection-to-span ratio for the Pujamanes bridge is larger than the values obtained for the Cheviré bridge. The study conducted by Jia et al. (2022) employed as a case study deflection measurements from a PSCB with span arrangements of 120 m + 4x225 m + 120 m. After eight years of bridge setup, the recorded deflection was approximately 115 mm, equivalent to a deflection-to-span ratio of 0.51 10^{-3}. In contrast, the deflection-to-span ratio value of 1.131 10^{-3} from the Pujamanes bridge after

6.33 years of bridge setup. Jia et al. (2022) proposed a Bayesian inference frame to address the current limitation of deterministic approaches for long-term deflection. The main factors affecting long-term deflection prediction are creep, shrinkage, dead load and prestress level incorporated in the Bayesian inference frame. However, the authors highlight the importance of approaching long-term deflection by developing a method allowing field measurements to be incorporated. Then, the necessity to conduct a comparative analysis with PSBCs previously constructed is essential, especially in-service PSCBs. Although the Cheviré bridge and the bridge presented by Jia et al. (2022) show values of deflection-to-span ratios lower than the Pujamanes bridge, the differences in the length of the central span are significant and, therefore, to further evaluate the condition of the Pujamanes bridge, an additional analysis is carried out using the deflection measurement data collected from the PSCB presented by Niu & Tang (2019). The selection of this PSCB is based on similar values of the main span lengths of the two bridges, but having a longer service life allows possible prediction of the long-term deflection condition of the Pujamanes bridge. The bridge is also located in Guangdong, China, with a span combination of 65 m + 100 m + 65 m and was opened to traffic in 1990. Long-term deflection values of 217 mm on the right and 171 mm on the left sides were measured after 11 years of bridge setup. The computed deflection-to-span ratios are 2.17 and 1.71, respectively. In 2001, driving comfort was affected, and it was necessary to implement a full-bridge deformation measurement system. The deflection increment analysis of the Pujamanes bridge from measurements collected in 2019 and 2020 shows that after 0.91 years, the deflection increment was 19 mm. Assuming that long-term deflection will increase at an average of 19 mm per year over the period 2021 -2025, it is expected that after 11 years of service in 2025, the Pujamanes bridge will have a deflection-to-span ratio value of 1.76. Showing a structural condition similar to the bridge (left side) presented by Niu & Tang (2019), its condition will be categorized as a danger, as shown in Figure 7. It is also possible to observe that the Pujamanes bridge will reach the red threshold approximately in 2022. As shown in Figure 4, the tendency of long-term deflection predicted by the FE model of the Pujamanes bridge is that long-term effects are not expected to increase in the coming years, but field measurements show that long-term deflection is still relevant; therefore, the implementation of monitoring strategies is essential.

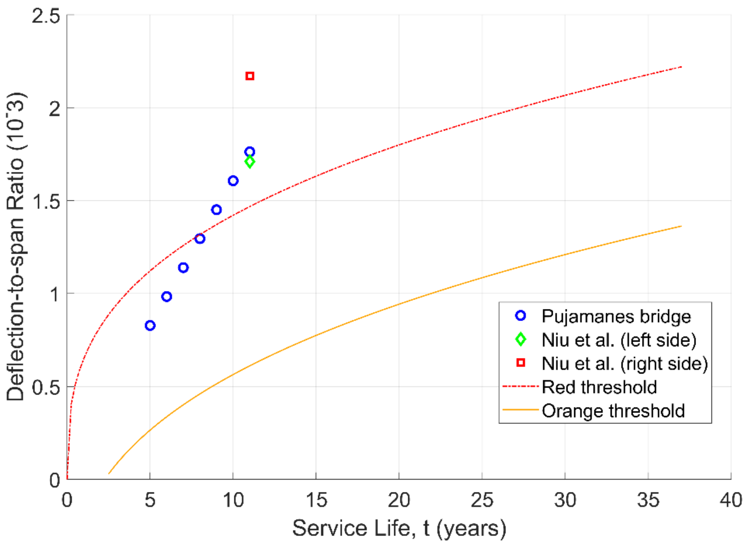

Figure 7. Long-term deflection index prediction of the Pujamanes bridge.

4 CONCLUSION

A deflection assessment of an existing PSCB is presented in this paper using the deflection-based multilevel long-term deflection assessment. Although, in this study, data collected at the approximately one-year interval was used to predict the long-term deflection of the bridge, it is not possible to affirm that long-term deflection equally increases in similar time intervals. The lack of bridge deflection between 2014 and 2019 imposes a limitation to predicting long-term deflection adequately. However, the approximately one-year interval adopted in this paper for deflection measurements allows comparisons with the tendencies found in similar bridges, where deflection measurements were taken more regularly. Due to this limitation, data collected from PSCBs constructed in the 90s and 2000s were used to show that the Pujamanes bridge is currently categorized as a major concern and, based on the deflection measurements collected by the authors, the bridge will reach the red threshold approximately in 2022. This finding emphasizes the necessity of implementing a full-bridge deformation observation system based on continuous deflection measurements. The authors consider that the LLS approach is a sustainable option to monitor long-term deflection due to the inherent economic limitations in Colombia to conduct field studies in PSCBs similar to the one presented in this paper. The findings presented in this paper lead to a current situation of uncertainty because from a total number of 20 PSCBs bridges in service in Colombia, 8 PSCBs built by cantilevers were opened to service between 2010 and 2020. No monitoring strategies for PSCBs are currently implemented or under development in Colombia. Finally, the authors recommend that the entities responsible for the construction and maintenance of PSCBs invest in studies of long-term deflections and implement a more conservative deflection prediction model.

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