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Cable tensioning during retrofitting of a suspension bridge: a case study on the use of updated nonlinear models

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ABSTRACT
Suspension bridges are highly nonlinear structures in which the live load distribution among its members is dependent on their current stress. As such, a proper, accurate model of the bridge in its current state is needed to adequately determine its ability to withstand the design loads. The combination of topographic measurements, impact testing on cables and OMA can be used to update the model used in design with real data, which can validate (or counter) the assumptions made.
This paper presents a case study of a suspension bridge in Argentina. The bridge had been retrofitted to carry higher dead loads, and the owner requested the cables to be measured and tensioned if necessary, following broad acceptance criteria. With no previous data nor design targets, an experimental campaign was carried out to determine the current cable tensions and the dynamic properties of the bridge, which were then used to update a numerical nonlinear model originally used for strength verification. Structural analysis using the updated model differed greatly from the results with the original version, highlighting the importance of considering as-built conditions and establishing appropriate acceptance criteria.
Further discrepancies and structural defects found during the experimental campaign also underscored certain limitations of models typically used for structural analysis in the design or verification stages. A higher level of detail may be needed in digital twin applications in order to fully incorporate all data collected, as otherwise crucial details which relate to structural safety may be ignored.

Keywords: Case Study, Nonlinear Analysis, Cable Structures, Bridge Maintenance, Model Updating

1. INTRODUCTION

1.1. General introduction

The estimation of cable tensions under design loads is of utmost importance when reviewing the load capacity of a suspension bridge. Given their highly nonlinear behavior (especially when loosely tightened), and large safety margin used in design, the current stress can have a significant impact on their tension at design loads and, in turn, on the bridge's safety.

Predicting the cable stresses in service is a tall task, which would require deep knowledge of the construction sequence and careful monitoring during said time. In many cases, this data is not properly defined during design, where many simplifying assumptions are made and, in turn, no real prescribed values are followed during construction nor are they measured once the bridge is in service, making posterior analysis and structural reviews harder.

This paper shows a case study of a suspension bridge for which, as part of a retrofit almost 15 years after the original construction, a complete first-time check of the cables was carried out, with the results leveraged to calibrate the structural model used for verification of the retrofit. The paper explores many of the lessons learned during the process and how they can help future projects of similar magnitude.

1.2. Background

In early 2024, a bridge over the Colorado River in the Cuyo region of Argentina was in the final stages of a major retrofit. The 2008 suspension bridge, with a 109 m long main span (Fig. 1), originally was a pipeline bridge suspended by two pairs of catenary cables (composed of 3 cables each) with 19 hangers each. Horizontal catenaries were also present to withstand the wind loads. Eventually, a truss girder was added to support more pipelines, while doubling the number of vertical hangers and adding an extra cable to each vertical catenary.

Figure 1: Bridge over the Colorado River.

As part of the retrofit, the owner requested an inspection to fully characterize the current state of the bridge and to ensure that the design documents were followed and the safety margins were as desired. However, the acceptance criteria for the cables was not clearly identified. The provisions required the cable tensions to be within 5% of the design values. Since the bridge was originally designed using hand calculations and pure statics in its final configuration, these were not available. In fact, for the horizontal hangers, for example, it was assumed that the cables were not initially prestressed and only the windward catenary would withstand the design wind load, a condition that in practice is impossible as cables need to be tightened somewhat to take the loading before there is significant bending or swaying of the pipelines. Similarly, vertical hangers were assumed to carry the dead load equally between them, despite being installed in two very distinct construction stages over the years and no measurements taking place when doing so.

In turn, there was no way to know what the "design values" were for the cables. A compromise approach to the acceptance criteria was eventually reached, where value ranges were adopted based on statics, similar to the original design. Based on the expected loading, and following the assumptions made in the original design, minimum and maximum values were agreed to for the hangers by the owner and the construction company. The maximum values were chosen to ensure a safety factor of 3 under the design live and wind loads following the methodology used in the original design documents (1.5 ton). The catenaries did not have prescribed values, as their tension was not a particular concern given the bridge geometry, though an explicit check was to be carried out afterward. Initially, a minimum value of 0.3 ton was agreed for the horizontal hangers, which meant to ensure some axial stiffness and ensure that all cables would contribute to withstand the wind loads. In the end, the construction company and the owner agreed to remove such lower bound on the basis that no such restriction was imposed in the design of the bridge.

2. METHODOLOGY

This section describes the techniques used during field testing to obtain the dynamic properties of the bridge, as well as how cable tensions were estimated based on vibration data. Finally, the structural model is described, along with the process followed for its calibration to measured data.

2.1. System identification

Typically, the dynamic properties of a structure (natural frequencies, mode shapes and damping ratios) are used to ensure the structural model reflects the true behavior of the as-built structure [1], and may also be leveraged to identify damage or changes during its lifetime [2]. In the particular case of a suspension bridge, the natural frequency may be quite accurately estimated simply by knowing the geometry of the main span and its total mass, as it behaves very similarly to an ideal taut string. In turn, as long as the model's geometry is relatively accurate (namely, the main span and the sag of the main cables), the natural frequency can be used for validation of the model's acting mass.

Given the above, it was considered enough to simply estimate the natural frequencies using the Peak-Picking method [3], placing sensors at the mid and quarter spans of the bridge. The low sensor density means that no mode-shapes were extracted.

2.2. Estimation of cable tension

The free vibrations $w\left(x,t\right)$ of a low-sag cable of length L, with mass along its length μ, accounting for its flexural stiffness EI and its tension T, is governed by the following differential equation [4]:

$$\mu\frac{\partial^2 w}{\partial t^2}+EI\frac{\partial^4 w}{\partial x^4}-\left(T+\Delta T\left(t\right)\right)\frac{\partial^2 w}{\partial x^2}=0 \quad (1)$$

Assuming the vibrations are small enough that tension is constant in time ($\Delta T=0$) and pinned connections at the end, it follows that the natural frequency f_n for mode n of the cable is:

$$f_n=\sqrt{n^2\frac{T}{4\mu L^2}+n^4\frac{\pi^2 EI}{4\mu L^4}} \quad (2)$$

The above equation can be rearranged to obtain a linear expression from which one can estimate both the cable tension T and its flexural stiffness EI given measured values of n and f_n:

$$\left(\frac{f_n}{n}\right)^2 = \frac{T}{4\mu L^2} + \frac{\pi^2 EI}{4\mu L^4}n^2 = A + Bn^2 \tag{3}$$

The process, then, consisted of measuring the free vibrations of the hangers and end catenaries (usually aided by an impact hammer), and via the Fourier Transform identifying the natural frequencies with the Peak-Picking method. Then, through a linear regression, the constants A and B were obtained, from which T and EI were finally estimated.

For the main catenaries the cable theory here presented does not hold as the hangers along their span alter its response significantly. In this case, it is easier to estimate their tension using statics, especially considering the total mass can be reasonably calibrated using the natural frequency of the bridge. Given the large number of closely spaced hangers, it is reasonable to approximate the geometry $\eta\left(x\right)$ as given by the catenary equation [5]:

$$\eta\left(x\right) = \frac{H}{\mu g}\left(\cosh\frac{\mu g L}{2H} - \cosh\frac{\mu g}{H}\left(x - \frac{L}{2}\right)\right) \tag{4}$$

with g being the acceleration of gravity. This means that by knowing the geometry of the bridge one can directly obtain the horizontal component of the tension H, and later the maximum tension at the ends T by considering the inclination angle.

2.3. Model updating

Both methods described above to determine cable tensions require accurate knowledge of the geometry of the bridge, which may be idealized in a structural model used for the design. In particular, the results are very sensitive to the measured sag of the catenary cable ($\eta\left(L/2\right)$) and the length L of each cable. To that end, a measurement campaign using a total station was carried out. Given the remote location of the bridge and large span, only the start and end points of the hangers were identified, from which the length was obtained. For the catenaries, information on the towers' height was then used to finalize their full geometry, though the true location of the hangers and the deformed shape of the truss girder was not available.

The measuring process took place several times, as the geometry of the bridge suffered small though noticeable changes as cables were retensioned or detensioned during the inspection process. Once the process was finished and within the agreed-to acceptance criteria, a structural model was updated to better reflect the measured conditions.

This model had been created in SAP2000 [6] for checking the design of the latest retrofit, and was not originally intended to be used as a digital twin. In turn, it had several simplifying assumptions typically used in design. Namely all catenaries (consisting of three cables each) were grouped into a single cable with the same total area, and hangers were equally spaced along the span. Similarly, as no design tension values were available, the model had no real defined starting point. Instead, a light pretension (modeled as a temperature change) was placed just to ensure horizontal cables were not completely loose against wind loads.

Given these limitations, the model updating process had two steps: first, ensuring the weight and mass included in the model was accurate by comparing the modeled and measured natural frequencies. Second, setting individual cable shortening load cases until the model with the formwork loading matched the hanger tensions measured on site. This was done through SAP2000's optimizer module, eventually reaching the load combination necessary to achieve this (that is, how much cable shortening was needed for each one).

Then, this calibrated model was used with the design loads to check the original goal of satisfying a safety margin of 3.

3. RESULTS

3.1. Cable tensions and natural frequencies

Figure 2 shows a typical plot of the measured accelerations on a cable during the field campaign, from which the linear regression as presented in Eq. 3 was performed. This process was followed for over 100 cables (76 vertical hangers, 38 horizontal hangers, 6 horizontal catenaries and 6 vertical catenaries), and had to be repeated several times during the tensioning process in order to meet the acceptance criteria. A total of 4 days were spent on site doing so, with most of the processing occurring in real time and refined overnight once the most accurate geometry estimates were provided by the total station.

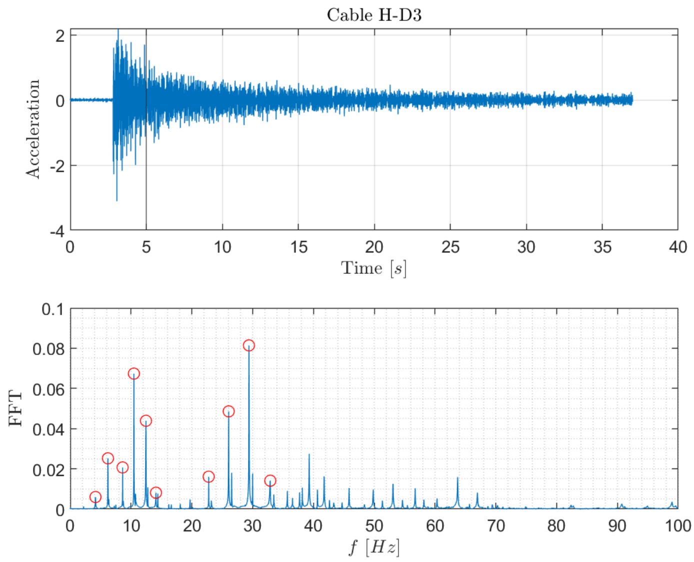

Figure 2: Measured vibrations and Fourier Transform for a vertical catenary cable.

In the interest of brevity, Table 1 shows the results for a small number of cables that will be of interest in this paper: The horizontal hangers near one of the piers (identified with the label H) and the horizontal catenaries (C). For the hangers, note that all of them are well within the maximum permitted value of 1.5 ton, meaning that the design hand calculations would result in a safety factor larger than 3. However, the cables were not carrying a similar load between them. A similar thing happens to the catenary. While it was assumed in design (and during the design checks) that each cable would carry approximately one third of the total catenary load, that is not the case, with a single cable accounting for 80% of the total tension. Note that while the values assumed during the retrofit are similar to those measured, that is mostly a coincidence as there was no basis to the initial tension in the model beyond avoiding "compressed" cables while modeling.

As for the natural frequency, it was found to be around 0.40 Hz in the vertical direction. Given the very low frequency and the relatively short (1 hr) testing time due to climate conditions, the estimate may have had some variance.

Table 1: Measured tension values for cables of interest.

Cable	Design [ton]	Retrofit [ton]	Measured [ton]	Acceptance [ton]
H1	0.00	0.26	0.20	1.50
H2	0.00	0.25	0.62	1.50
H3	0.00	0.24	0.21	1.50
H4	0.00	0.22	0.09	1.50
C1	N/A		0.14	
C2	N/A	3.64	0.35	$SF > 3$
C3	N/A		2.30	

3.2. Other findings during the inspection and retensioning process

The large disparities found in the cable tensions are not considered good design. Cables are highly nonlinear elements whose stiffness sharply rises as their stress increases. Such differences in stiffness can significantly alter the results of the analysis and should be avoided. However, if no tracking is done during construction and maintenance, the possibility of modifying the situation is largely restricted.

In this case, attempts to create more uniform conditions were largely affected by physical limitations of the anchorage system. Cable length is set once built, and the only way to retension them is changing the length of the connection by screwing or unscrewing the anchorage element along a rod fixed to the structure. In many cases, either the rod was not long enough to ease the tension any longer, or the anchorage element would clash with the added steel members that compose the truss, to the point cables could no longer be retensioned (Fig. 3a). In turn, this means that achieving a more uniform load distribution was physically impossible without outright replacing the current cables, which was not an option under short notice nor within the scope of the requested project.

Similarly, the connection between the horizontal hangers and the catenary systems (Fig. 3b) introduced a significant amount of friction, altering the tension distribution along its length. In fact, though Table 1 shows that cable C3 carried most of the load, the opposite happened on the other end of the bridge. Of course, this is unintended behavior, and essentially no common structural model would account for such possibility, especially one that lumps all cables into a single catenary.

(a) Cable Anchorage blocked by truss element. **(b)** Hanger-catenary joint introducing friction.

Figure 3: Details detected during inspection altering intended behavior.

3.3. Analysis of the updated model

With the above information, the model was updated in SAP2000 according to the methodology described before. As mentioned, this model was used for checking the design of the retrofit under new loading, and therefore had some typical simplifications. In turn, several of the details discovered during inspections could not be properly accounted for: especially the difference in tension between the different cables within a single catenary; and the friction introduced by the connection between the hangers and the catenary. The model, then, can only accurately cover the global behavior of the catenaries.

The calibration did not affect the natural frequency of the bridge, as the general geometry of the catenaries was accurate and so was the detailed calculation of the dead load. The resulting value of 0.38 Hz was within the uncertainty bounds obtained experimentally, and closer than the original estimate of 0.36 Hz, largely attributed to an increase in stiffness due to hanger tightening (mode shape can be seen in Fig. 4).

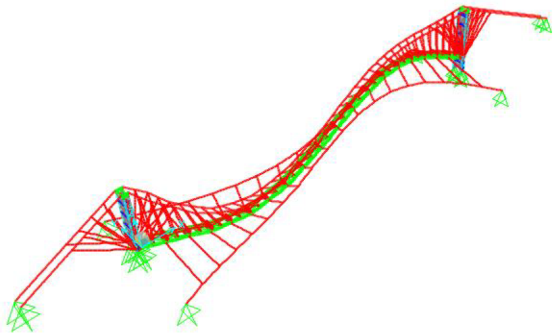

Figure 4: Structural model showing the shape of the first vertical mode.

However, the expected cable tensions under the design loading varied widely compared to those obtained during the original structural check, due to the difference in tension (and stiffness) not accounted for in the uncalibrated model. Table 2 shows the differences in the expected tensions between the uncalibrated model (as informed in the structural verification before the retrofit was done) and those resulting from the calibrated model once the retrofit was completed.

Table 2: Estimated tensions for design loads before and after calibration.

Cable	Before Calibration [ton]	After Calibration [ton]
H1	2.28	1.29
H2	**2.14**	**4.35**
H3	2.09	0.24
H4	2.01	0.22
C1		
C2	36.06	34.04
C3		

The estimated tension for Cable H2 almost doubles after calibration, while nearby cables completely unload compared to the uncalibrated model, due to the higher initial load (stiffness). Such an increase in tension could eventually compromise the structural safety, as it is not something the hand calculations used for determining the acceptance criteria could have predicted. Similarly, though the catenaries passed the safety check in a global sense, it is fair to wonder how their internal stress difference would affect the extrapolation to a higher global tension. If the ratio between cables were to stay the same, then the most stressed cable could be at a risk of failure. However, since friction seems to be the main culprit, it may disappear once the stresses get higher, causing a load redistribution that would protect the structure. Only a more detailed model that accounted for each cable separately and the behavior of the joint would be able to answer that, something which would never be considered at the design stage.

It seems as though the models used for design may not be fully adequate for use as a digital twin, in the sense that as more data is gathered, a much more complex model is needed to account for it and reduce uncertainties. In particular, in this case, had a digital twin model been used from the start, it could have modeled each catenary cable individually instead of as a group, and include the response of the hanger joints. A better approach to geometry extraction could have been used in order to properly account for the true hanger spacing and the deformed shape of the deck, which the design model could not have exploited. It could have also been continuously updated during the construction and retrofit stages, ensuring tensions were within acceptable ranges and performing mitigating actions at an early stage. When data is gathered late, as with this bridge, the design may not be flexible enough to later accommodate changes.

In any case, the results also show that the acceptance criteria for structures of this magnitude and complexity cannot be set at an individual level, but at a system level to account for nonlinear behavior. Similarly, the case study puts forward the difference between the level of detail one may use in design, where there are several unknown variables that require engineering judgment and simplifications, compared to a digital twin that tries to account for every measurable feature of the bridge with the goal of reducing uncertainty and improving predictive accuracy. Here, a design model was then updated based on data, but not all of it could be properly included due to limitations of the model itself.

4. CONCLUSIONS

This paper shows a case study on the use of model updating to assess the structural safety of a suspension bridge after undergoing a major retrofit to accommodate larger loads. A testing campaign was carried out to measure the cable tensions and natural frequency of the bridge, which was used to calibrate the design model with which the structural verification had been carried out.

The results show that highly nonlinear structures are very susceptible to changes in the initial configuration, and utmost care should be taken when making assumptions during design. Lack of monitoring during the construction stages led to a large variance in cable tensions that could not be reduced due to physical limitations of the anchorage systems by the time it was detected.

Also, although the calibrated model is a closer representation of the truth, it cannot cover every identified detail found during the inspection and evaluation, due to its simplified nature. This shows that the level of detail a digital twin may need to properly incorporate all the acquired data may exceed what is typically used or needed during design, when a lot more uncertainties are present and so adding detail cannot be justified. Otherwise, there is a risk of, even with an updated model, reaching unsafe conclusions about a structure's state. In this case, for example, catenary cables showed a large difference in tensions between them which could lead to failure of one of them, despite the group as a whole meeting the loading requirements. It is advisable for the digital twin model to be tailored-made for the data to be gathered and conceptualized early on in the project.

Finally, the limitations found during the retensioning process in order to create more uniform loading conditions for the cables underscores the importance of monitoring structures from as early as the construction stage, when early detection can help mitigate problems down the line and achieve design goals. Once the bridge is already built, it may be too late to find cost-effective solutions to any deviation found.

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