Application of a Twin Model for Monitoring and Predictive Diagnosis of Calle Calle Bridge in Chile

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Abstract. This study has developed a twin model tool that allows evaluating the current state and future behaviour of the Calle Calle bridge in Chile. The development of this twin model began with a finite element numerical model that was subsequently calibrated with data from sensors installed on the bridge. ANSYS APDL software was used to represent the structure by means of finite elements. The parameters captured by the sensors were the inclination and settlement in the abutments, the instantaneous deflection, and the proper frequencies. Calibration was an iterative process that sought to obtain a model that behaved similarly to the real structure. The twin model developed describes the reality of the structure allowing the simulation of future scenarios, thus improving the efficiency of management tasks, and thus achieving predictive maintenance. From the analysis carried out, it is concluded that the structure is in an acceptable state, requiring some maintenance. A great sensitivity of the bridge beams was also observed, which did not meet the current design criteria.

Keywords: Twin Model; Finite element simulation; Bridge field measurements; Structural response to loading.

1. Introduction

The collapse of the Morandi Bridge in Genoa, Italy, highlights the debate on the effectiveness of methods used in the maintenance of structures. With the current technological advances in the engineering world, it seems logical to update the concept of maintenance to act before damage occurs. For this purpose, there are two options: preventive maintenance and predictive maintenance. Whereas, preventive maintenance forces you to act in situations where it would really not be necessary, predictive maintenance allows you to act only when it is necessary, being a better option [1].

The objective of this study is to achieve a working tool that allows to evaluate the current state and future behavior of specific structure, with the intention of optimizing maintenance and control plans, in terms of safety and economy. This tool is based on the development of a twin model, which starts from a finite element numerical model and is subsequently calibrated with data registered by sensors installed in the real structure. This twin model is based on a work philosophy based on a constant and bidirectional flow of information, from reality to the model and from model to reality. The model describes reality, allowing the simulation of future scenarios that will improve the efficiency of maintenance tasks, preventing the appearance of future pathologies, thus obtaining predictive maintenance [2].

The methodology carried out in this study has been tested on the Calle Calle bridge, located at PK 53.94 of route U-911 in the Los Lagos region of Chile. With the completion of this study it has been...
possible to know the current state of the bridge and what future behaviors it may have when faced with complex scenarios that include earthquakes and heavy traffic, among others.

2. Method

This section details the work carried out to achieve the twin model that allowed managing the maintenance of the Calle Calle bridge. First, all the information that was necessary to compile a first numerical model was collected, selecting the parameters to monitor, and choosing the location, inside the bridge, where the monitoring sensors could be installed. Then the sensors were installed to be able to calibrate the model, and finally, with the validation and calibration of the model (twin model), different analyzes were carried out with which the structure could be evaluated. The evaluation of the current state of the bridge has been called as a primary diagnosis and the evaluation of future scenarios as an advanced diagnosis. Figure 1 illustrates the development of the method carried out.

![Figure 1. Development of the method carried out in this study.](image)

2.1. Previous Information

The first step was the acquisition of previous data such as: geometric parameters, existing damage, dimensions of structural elements, photographic report, etc.

The bridge was determined to have a total length of 46.7 m (including abutments). It was formed by a single isostatic span 37.3 m long, and a total width of 8.60 m. It was a bridge whose deck is made up of a 28 cm thick reinforced concrete top slab and two 1.78 m thick double T shaped steel beams. The longitudinal beams were connected to each other, at the supports, by reinforced concrete diaphragm walls, and along the beams by San Andrés steel crosses. With reference to the pavement, the circulation was carried out on the same reinforced concrete slab of the bridge.

After the visual inspection carried out, it was observed that the integrity of the resistant mechanism was not compromised despite requiring protective actions and/or replacement of certain elements. In addition, it was possible to start designing the geometric model of the bridge as the basis of the numerical model.

Figure 2 provides an overview of the Calle Calle bridge.

![Figure 2. Overview of the Calle Calle bridge.](image)

2.2. Instrumentalization

The instrumentalization was carried out using only PCE-VDL 24I triaxial accelerometer sensors with a sampling frequency of 100 Hz. A total of 10 accelerometers were installed: two for each abutment and another six throughout the span, this distribution can be seen in Figure 3. The parameters that were interested in monitoring the structure were the following: abutment inclination, differential settlement in abutments, instantaneous deflection, and natural frequencies on a bridge deck.

The first three parameters provided geometric information on the state of the bridge, and the last on the current operation of the structure. The determination of the natural frequencies of the bridge deck and the instantaneous deflection were made by treating the records of the accelerations obtained.
inclinations were measured from the variation between the readings of the accelerations in their axes, determining the existing relationship between the readings of the horizontal axes and the gravitational axis.

Figure 3. (Right) Distribution of installed sensors and (left) detail of the accelerometer.

2.3. Numerical Model and Calibration

ANSYS APDL v17.0 software was used to represent the structure in the finite element environment. The structural elements represented in the model and their constituent materials were as follows:

- Slab → Reinforced concrete.
- Longitudinal beams → Steel.
- Diaphragms wall → Reinforced concrete.
- Vertical triangulations → Steel.
- Horizontal triangulations → Steel.

The slab of the structure was modeled using SOLID-type three-dimensional elements, the longitudinal beams, and diaphragms walls in the support sections, using SHELL-type flat elements, and vertical and horizontal triangulations using BEAM-type one-dimensional elements. Finally, the interaction of the bridge deck with the abutments was performed by simulating simple bearings at the ends as boundary conditions. Figure 4 shows how the structure has been modeled in the software.

Figure 4. Structure modelling.

With the data from the instrumentalization of the bridge, the calibration of the model began. The objective of the calibration was to obtain a twin model of the structure, which reproduced the real static and dynamic response in the model. To do this, we sought to adjust the parameters that govern the behavior of the structure according to the general equation of dynamic equilibrium in its matrix form (1).

\[
[M]\ddot{X} + [C]\dot{X} + [K]X = [F(t)]
\]  

(1)

Where:

- \([M]\) = mass matrix of the system
- \([C]\) = damping matrix of the system
- \([K]\) = stiffness matrix of the system
- \(\ddot{X}\) = vector acceleration at each point in the system
- \(\dot{X}\) = velocity vector at each point in the system
- \(X\) = displacement vector at each point of the system
The obtaining of the Twin Model is based on the premise that, despite not knowing exactly the quality, geometry and distribution of the elements and materials that form the structure, it is possible to obtain a model that correctly represents reality as long as there is a similarity between the natural frequencies of the numerical model and the real structure. To validate the twin model, it is necessary to define all the parameters related to the mass, damping and stiffness of the structure. These parameters are:

- System masses: Dimensions of structural elements and density of materials.
- Damping of the system: Damping rate.
- System rigidity: Dimensions of structural elements and constitutive properties of materials.
- Links between structural elements.
- Boundary conditions: Links between the model and the exterior and stiffness of the terrain.

The dimensions and connections of the structural elements were known thanks to the plans and reports generated during the bridge's previous inspection campaign. The damping rate for conventional structures in which a high degree of damage is not appreciated can be estimated at 5%, as indicated in The Highway Manual of the Road Administration of the Government of Chile in section 3.1004 [3]. On the other hand, the parameters that were necessary to determine in the calibration process were: The constitutive properties of the materials and the stiffness of the ground.

Calibration was an iterative process that sought to obtain a model that behaved similarly to the real structure in its current state. The calibration process wanted to obtain the same natural frequencies, in a series of vibration modes, between the modeled structure and reality. According to the Chilean Highway Manual standard, these vibration modes must be those that can mobilize 90% of the structure's mass. Therefore, one of the key aspects of the calibration process was to define the properties of the bridge materials in the model that can give similarity to all the necessary vibration modes captured by the sensors. In this case, given the simplicity of the bridge, it was only necessary to study the first bending vibration mode since it was the only one that mobilized 90% of the structure's mass.

Based on experience in projects and prototypes previously carried out [5], it has been verified that an adequate response is obtained from the model when the difference between the registered and calculated natural frequencies does not exceed a margin of + / - 0.5 Hz. The data obtained from the sensors established a first peak of the response spectrum of the bridge's natural frequencies, associated with the first mode of bending, in the interval of 2.8 - 3 Hz, so it was necessary adjust the parameters of the materials in the model until similar data is reached. This was achieved through only two iterations. After that, it was considered that the model was adequately reproducing the response of the structure and the developed twin model was considered validated.

2.4. Current State of the Bridge (Primary Diagnosis)

With the development of the twin model, the current state of the bridge was defined according to certain limits set by the regulations of the Highway Manual and the AASHTO Standard Specifications for Highway Bridges [5] about key parameters of the bridge. These parameters were the same as those monitored with the sensors installed on the bridge: abutment inclination, differential settlement in abutments, instantaneous deflection, and natural frequencies on a bridge deck. With the intention of facilitating the management of the structure and knowing the current state of the bridge, three possible states were defined for each of the above parameters:

- Excessive: Imminent effect on the global or local integrity of the structure. Requires the adoption of corrective measures.
- Acceptable: Affection to the use, comfort and, or perception of the structure. This condition also originates those conditions that can influence, with their persistence, the integrity of the structure. It requires the study of its evolution.
- Good: No abnormalities are detected in the structure response.

Next, the limits of each parameter are specified to be included in each of the previous states:

- **Abutment inclination**: The maximum admissible distortion in abutments and retaining walls follows the provisions for the geometric distortion of foundation elements according to Article 4.4.7.2.5. of the AASHTO Standard Specifications for Highway Bridges, which indicates a
limitation in the rotation of 0.29º, in this case. A minor distortion means a Good condition, however, exceeding this value would indicate the need for continuous monitoring. On the other hand, in order to guarantee the overall stability of the structure, an angular distortion that causes the inefficiency of the support elements, of the expansion joints, or of some other structural element must not be reached in any situation. In this case, the limit would be reached due to the breach of the admissible differential settlement, considering the overturning due to subsidence, of the support of the abutment. This value is defined in a rotation of 2.26º and would suppose an Excessive state of the structure.

- **Differential settlement in abutments:** According to Article 4.4.7.2.5. of the AASHTO Standard Specifications for Highway Bridges it is recommended that the angular distortion in isostatic spans does not exceed the limit of 0.005 between the ratio of the vertical displacement ($\delta$) and the length of the span (L). A lower value in this relationship assumes the Good state, while exceeding this value assumes an Acceptable state. Exceeding this limit does not imply the failure or collapse of the structure. However, once it has been overcome, it is recommended to evaluate the structural health of the same, as well as to carry out continuous monitoring, avoiding jeopardizing the stability of the structure. Given that the bridge is isostatic structure, it is not contemplated that a scenario that will exceed the Acceptable one can occur, since this scenario would be a symptom of catastrophic damage, which would be alarmed much earlier with the other state limits.

- **Instantaneous deflection:** Article 8.9.3.1 of the AASHTO Standard Specifications for Highway Bridges marks as the maximum value of the instantaneous deflection ($\Delta f_{\text{inst}}$) the relationship between the length of the span (L) and the value 800. Exceeding this value will suppose the assignment of the Acceptable state. On the other hand, the instantaneous deflection must never reach values that induce greater tensions in the sections than the resistance of the material. To obtain this limit, a vertical displacement is imposed on the calibrated model such that it causes the plasticization of the steel or exceeds the resistance of the concrete at some point, whichever comes first. Exceeding this limit will suppose the Excessive status. In this case, it has been obtained that with a 6.6 cm deflection, in the centre of the span, the concrete of the slab began to fail.

- **Natural frequencies on a bridge deck.** The variation in the natural frequency of an element is closely related to the variation in its stiffness. The dynamic response of the structure is limited by two aspects:
  - Insufficient stiffness of the deck against gravitational loads: The behaviour of the bridge will not be adequate if its stiffness is not sufficient to meet the regulatory criteria for maximum instantaneous deflection. The frequency associated with the minimum stiffness for meeting these criteria is established as an Acceptable limit.
  - Possible resonance effects with traffic loads: If the excitation intensity associated with the use of the bridge were high enough and, also, were around of the board's natural frequencies, the dynamic amplification effects of the load could quickly damage the supporting elements. It is determined in this study that the typical excitation frequencies of greater intensity caused by vehicle traffic occur in the range of 8 to 20 Hz. In addition, in this study the criteria established by Wenzel in 2009 [6] has been applied where correlates the intensity of the accelerations on the structure and the frequency spectrum with the expected level of affection. Therefore, if peak acceleration values above the range of perception established by Wenzel are reached, higher frequencies under such conditions will cause Excessive rating. Otherwise, it will be considered Good rating.

From the premises discussed above, the thresholds shown in Table 1 were obtained by way of synthesis.

**Table 1.** Limit values of the possible states of the bridge.

|                     | Abutment inclination | Settlement in abutments | Instantaneous deflection | Natural frequencies on a bridge deck |
|---------------------|----------------------|-------------------------|--------------------------|-------------------------------------|
| **Good**            | $\Theta_{\text{abutment}} < 0.29^\circ$ | $\delta/L < 0.005$      | $\Delta f_{\text{inst}} < L/800$ | $f(L/800) < f < 8 \text{ Hz}$       |
Acceptable  
0.29º ≤ Θ_{abutment} < 2.26º  
0.005 L ≥ δ/L > 6.6 cm  
L/800 < Δ_{f,inst} ≤ 6.6 cm  
f ≤ f(L/800)  

Excessive  
Θ_{abutment} ≥ 2.26º  
Δ_{f,inst} > L/800  
f ≥ 8 Hz

2.5. Scenarios Simulation (Advanced Diagnosis)

The scenarios that were studied corresponded to the combinations of normative actions of the Highway Manual of Chile and the AASHTO Standard Specifications for Highway Bridges. For this study, the numerical simulation of different scenarios was performed in the described calibrated model. The structural standards are updated over time, and it is likely that the regulations with which the bridge under study was designed was not as demanding as the current ones. That is why we want to determine if the bridge would be able to support the scenarios required by current regulations.

The methodology to simulate the scenarios began with the definition of the loads that could act on the bridge. Afterwards, the regulations indicating how to combine these loads to create demanding situations for the structure were studied. Finally, the evaluation criteria of the aptitude of the bridge in the scenarios were defined.

On one hand, the loads considered were the following: permanent loads (own weight of the structure, weight of pavement, railings, etc.), live loads (mobile vehicle loads and pedestrians), dynamic effects or impact of mobile loads, seismic loads and wind loads.

The sizing of the different elements of the structure was carried out by the method of service loads and allowable stresses (ASD: Allowable Stress Design). The load groups according to the selected method were defined according to Article 3.22 of the AASHTO.

Considering the loads considered and the combination criteria for the Orthogonal Seismic Forces established in the Highway Manual of Chile, the load groups that are applicable are the following:

- Group I: It considers permanent loads (D), and live loads (L + I)n.
- Group II: It considers the permanent loads (D) and the wind loads on the superstructure (W).
- Group III: It considers the permanent loads (D), the live loads (L + I)n, the wind loads on the superstructure (W) and the wind loads on the live loads (WL).
- Group VII: Considers permanent loads (D), longitudinal earthquake loads (EQL) and transverse earthquake loads (EQT)

Regarding the load groups, the bending check is called (M) and the shear check (V). If the application of 100% of the earthquake load in the longitudinal direction and 30% in the transverse direction is considered, the designation is (I), if it is the other way around (II).

On the other hand, the evaluation criteria used determined the aptitude of the structure from two points of view:

- Compliance with current design standards. For this, the method of service loads and allowable stresses (ASD) is used as indicated in the AASHTO.
- Exhaustion of resistant capacity. For this purpose, a comparison is made of the stresses obtained in the different load groups with the resistance of the sections of the structural elements through sectional use. On the other hand, the sectional use criterion relates the Von Mises stress (σ_{VM}) to which a certain resistant element is subjected to the maximum characteristic stress of the material of the element in question. Since it is an isostatic bridge, the most requested fiber will not be allowed to reach the plasticization since the formation of a plastic hinge would imply the risk of collapse of the structure.

3. Results

Next, the comparison of results obtained through measurements made with the established limits is developed.

3.1. Results of Current State of the Bridge (Primary Diagnosis)

Firstly, after analysing the data of the measurements given by the accelerometers placed in the stirrups, an inclination of + 0.4 º was obtained on the east side and -0.1 º on the west side, therefore the parameter was rated as Acceptable.
Results of differential settlement in abutments. Figure 5 indicates the records obtained with the sensors. It could be observed that the inclination of the west abutment was 0.176º, causing a 3.82 cm abutment seat and the east abutment inclination of 0.538 causing a 11.68 cm differential seat. The total differential seat was obtained as the difference, in absolute value, of the seats calculated for each stirrup. In this case, a total seat of 8 cm that does not exceed any of the defined limits, therefore, this parameter was classified as Good.

![Figure 5. Inclinations in the abutments obtained by the sensors.](image)

Results of instantaneous deflection. Figure 6 (left) shows the 45-minute record where the maximum instantaneous deflection occurred during the inspection campaign in the centre of the span. Figure 6 (right) also shows the 2.5-second fragment of the integrated and filtered signal where the maximum arrow occurred, with a value of 0.22 cm. Since the value does not exceed any of the proposed limits, it gets a Good rating.

![Figure 6. (Left) Accelerogram corresponding to the maximum instant deflection and (Right) Maximum deflection obtained.](image)

After analyzing the results of natural frequencies on a bridge deck obtained using the accelerometers installed in the structure, the board's own frequency was 3.05 Hz, not exceeding any of the established limits, thus achieving a Good rating.

3.2. Results of Scenarios Simulation (Advanced Diagnosis)

The results of the maximum stresses obtained in the twin model of the Calle Calle bridge are provided below for each of the structural elements in the sections that have been most requested. In the graphs of results (Figure 7), the load groups indicated in section 2.5 have been represented on the vertical axis. In these groups, each horizontal bar represents one of the structural elements that form the bridge. On the horizontal axis the resistance capacity of each element has been represented. That any element exceeds the vertical line of 100% assumes that the elastic limit of the element has been exceeded and begins to work in the plastic area. It was suggested that overcoming the elastic zone implies moving away from the safety zone.
The AASHTO standard (right on the Figure 7) for determining the exhaustion of bridge parts is more demanding than the sectional mechanical criterion (left on the Figure 7). On the other hand, it is also observed that the most demanded piece is the longitudinal beam, both in the centre and in the bearings, exceeding 100% of its capacity in various scenarios.

4. Conclusions

From the primary diagnosis it is concluded that, according to the data collected from the instrumentalization and the twin model, that the structure is in an Acceptable state. However, to ensure the proper functioning of the structure in the future, different premises should be considered regarding the maintenance of the expansion joints and the concrete of the substructure.

The following considerations are extracted from the advanced diagnosis:

- The most sensitive elements of the structure were the longitudinal beams, in the sections near the bearings and the centre of the bridge.
- The beams in the centre section did not comply the limits set by AASHTO for Groups I and III. These load groups were those referring to traffic loads. However, regarding the sectional use, the maximum calculated was 80%, which was not exhausted.
- The beams in the sections near the bearings did not comply the limits set by AASHTO in any of the load groups considered. Regarding the sectional use, the exhaustion of the section was reached for Groups I (V), III (V), VII (I) and VII (II). These load groups are those related to traffic loads (I and III) and the earthquake (VII (I) and VII (II)). The maximum sectional use was made for Group VII (I) (dominant transverse earthquake), with 164% of the use.
- The slabs and triangulations comply both the sectional use criteria and the tension criteria set by AASHTO. Regarding the diaphragm, it was observed that it did not comply with the tension limit defined by AASHTO for Group VII (II).

With all the above, a great sensitivity of the beams was observed in the sections near the bearings against traffic loads and seismic loads, especially in the event of an earthquake transverse over the bridge. It should be noted here the importance of maintaining the stresses in the longitudinal beams at levels below their elastic limit, since being an isostatic bridge the formation of a plastic hinge in them would imply the risk of collapse of the structure. With the results obtained from this report, it could be stated that the structure did not comply the current design criteria and that the regulatory actions related to traffic loads and earthquake may cause plasticization in some sections of the longitudinal beams in the future.
As a conclusion, this study has developed a tool that allows evaluating the current state and future behaviour of the Calle Calle bridge, which provides valuable information to optimize the maintenance and control plans for the structure. All this has been possible with the development of a twin model, which was based on a finite element numerical model and was calibrated with data from sensors installed on the bridge. This twin model has been able to describe reality by allowing the simulation of future scenarios.

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