ALARM SETTING METHODOLOGY FOR THE MONITORING OF CABLES IN A STAYED BRIDGE

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The Bridge and Intelligent Structures Monitoring Center in Mexico was planned for the SHM of the most important structures in highway system to assure their structural integrity, reduce rehabilitation costs, and to increase their service index. Among those, the Río Papaloapan Bridge, which is a 402 meters long cable stayed bridge, was selected for full scale instrumentation and remote permanent monitoring based on previous structural record and its location in one of the most important highways. A critical issue for permanent remote monitoring is the alarm system configuration, so warnings can be put on display on time and to allow effective damage identification or structural evaluation.

In this case, monitoring included accelerometers in some cables and deformation sensors along the bridge deck; therefore, the structural diagnosis of the cables is done indirectly from pre-processed data from different sets of sensors. To determine the alarm limits, Monte Carlo simulation was used for different loading configurations, including traffic and wind loads. A combination of statistical loads and the material's resistance was used to calculate the structural reliability of the cables and define the 3 levels alarm (green, yellow and red). This methodology proved to be an alternative for structural monitoring for parameters that are not directly measured, but can be indirectly calculated.

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ABSTRACT: The Bridge and Intelligent Structures Monitoring Center in Mexico was planned for the SHM of the most important structures in highway system to assure their structural integrity, reduce rehabilitation costs, and to increase their service index. Among those, the Río Papaloapan Bridge, which is a 402 meters long cable stayed bridge, was selected for full scale instrumentation and remote permanent monitoring based on previous structural record and its location in one of the most important highways. A critical issue for permanent remote monitoring is the alarm system configuration, so warnings can be put on display on time and to allow effective damage identification or structural evaluation.

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1 INTRODUCTION

The Río Papaloapan Bridge is a cable stayed structure located on the La Tinaja-Acayucan highway in the state of Veracruz in Mexico that was completed in 1994 and commissioned in 1995. The main span of this bridge is 203 meters, with a total length of 407 meters and it has 112 cables in 8 semi-harps. In 2000, the upper anchorage

element of one of the cables failed after a typical winter wind storm, where speed winds reached almost 100 km/h and left the bridge with 111 cables only, causing minor damage in the deck with some permanent deformation, López et al. (2009). Failure analysis determined that the main cause for failure was associated to defects within the constitutive material of the upper anchorage element, which were a consequence of an inadequate cast process that produced voids and inclusions, and a deficient heat treatment that resulted in low toughness steel; defects acted as stress concentration zones, where cracks were generated and then propagated from fatigue.

Although the failure of the upper anchorage element of the Rio Papaloapan Bridge was repaired, concern was raised on the remaining 111 anchorage elements, as these were manufactured with the same process. To evaluate these remaining elements, an ultrasonic nondestructive technique was developed to identify similar microstructural deficiencies within the steel, where the main challenge was that the elements were partially imbedded in the concrete. After inspection, 16 elements were classified as structurally deficient due to their porosity or microstructural characteristics; though, the 16 deficient elements were changed with 4 additional elements "in good condition" that were used as a reference for a long term reliability study of the remaining 92 elements in the bridge.

The Río Papaloapan case is an example of a large bridge with some particular structural problems, where large scale monitoring is an appropriate alternative to assure its structural integrity. Structural health monitoring (SHM) is increasingly being used to monitor important structures, not only for problems or specific structural conditions, but also because it is possible to use advanced techniques for early damage detection, life prediction and structural prognosis.

In general, SHM can be defined as a measure of the load and operational conditions, and critical responses of a structure, to track and evaluate abnormal symptoms or damage that affect structural service, safety and/or reliability, Aktan & Grimmelsman (1999). Nowadays, structural monitoring and reliability is not only possible, but necessary for the conservation of the modern infrastructure of a country.

Most application of Bridge SHM is for normal operation conditions, where structural evaluation, early damage detection, structural load capacity and structural prognosis, are the main maintenance objectives. For abnormal or extraordinary conditions, damage detection and structural evaluation, must be part of an early warning system to assure safety and users integrity. Additionally, the use of SHM is also necessary because the actual load conditions on the bridges, have gone beyond from those conditions considered when these were designed; in particular, traffic, weather conditions, vehicle gross weight and vehicles configurations, are much more demanding than before.

It has been in this context that the Mexican Ministry of Communications and Transport has decided to develop, through the Mexican Transport Institute, the Bridge Remote Monitoring Center (*Centro de Monitoreo de Puentes y Estructuras Inteligentes* , or CMPEI), with the main objective to monitor the large and most important bridges in the highway network in Mexico; but also, those with particular structural problems, such as scouring or major damage. In this case, the main instrumentation concept was based on the FBG optical sensors for their long term reliability and performance.

Key issue is the sensor distribution and the strategy for structural monitoring and evaluation, which in this case, considered modal analysis and wave propagation

phenomena. For the monitoring of the cables' tension, accelerometers were placed on pre-defined cables in the 8 semi-harps; where changes in tension would be identified indirectly through changes on the instrumented cable. For the early warning system, it was important to define, for each structural variable associated with a sensor or group of sensors, the alarm limits for good, satisfactory, unsatisfactory or unacceptable conditions. As a standard procedure, the alarm limits were defined from a statistical analysis considering different load scenarios and design structural parameters, as it is described along the following lines for the particular case of the tension on the cables of the Rio Papaloapan Bridge.

2 METHODOLOGY

A 5 step method was designed to obtain the statistical distributions for the 112 cables on the bridge and consequently, to calculate the first alarm level of each cable. The first step was the generation and calibration of a finite elements model of the bridge, to simulate the live loads due to traffic and wind, considering different loading scenarios, present and future. The second step was the analysis of the actual and future traffic to calculate the vehicle occupancy index of the bridge, the type and velocity of the vehicles, and the vehicles' gross weight. The third was the analysis of field data to calculate the statistical distributions of the wind velocity, vehicle type and vehicle gross weight. The forth step was the formation of the Monte Carlo algorithm to calculate the tension statistical distributions of the 112 cables on the bridge, from the live loads (traffic and wind) statistical distributions. Finally, the last step was the characterization of a first level alarm limits for the cables.

2.1 Calibration of the Bridge Simulation Model

The dynamic structural parameters of the finite element bridge model were calibrated from structural field data and dynamic measurements from a controlled impact test. Main field data was obtained from a permanent evaluation of the bridge where the tensions on the 112 cables was monitored for almost one year period, Quintana (2009). In this case, acceleration measurements on the cables gave the first three frequency modes that were used to calculate the tensions by means of a nonlinear model, Carrion et al (2007). As for the impact test, the dynamic acceleration responses were measured at 11 different points on the bridge deck under controlled conditions (without traffic) and vibration was produced with a Dynatest® falling weight deflectometer, normally used for pavement tests, but in this case only to excite dynamic vibration on the bridge with a known force, Quintana (2009).

The finite element model was developed using StaDyn Version 4.54, which is a FE dynamic general purpose software, Doyle (1997, 2004). The geometric model of the bridge was built with 8224 elements (cable, plate, and beam) and 4693 nodes. Boundary conditions considered the piles fixed at the foundations, and the bridge deck simply supported on one side and a mobile support on the other (expansion joint). For the calibration, the materials' properties were adjusted following a sequential process in which the properties were adjusted until the static and dynamic responses adjusted the experimental measurements up to a predefined error limit. The model was made up with 26 different materials for the cables, bridge deck, longitudinal and transversal beams, and piles.

2.2 Bridge Traffic Analysis

The daily traffic on the bridge was used to calculate the bridge occupancy, and from that, the probability function for finding, at a given time, one or more vehicles (up to 9). For this calculation it was assumed the average vehicle speed on the bridge of 90 km/h, which means that a vehicle takes 16,248 seconds to cross the bridge (Table 1).

For the traffic characterization, data was obtained from field measurements during two days and from the records taken at the toll booth by the operator of the highway; the first, to identify the lane probability occupancy for both directions (Table 2); and the second, to make out the traffic composition for the most common types of vehicles that were identified as: B2 (bus with 2 axles), B3 (bus with 3 axles), C2 (truck with 2 axles), C3 (truck with 3 axles), T3-S2 (3 axles truck with 2 axes semi-trailer), T3-S3 (3 axes truck with 3 axes semi-trailer) and the full or T3-S2-R4 (3 axles truck, 2 axles semi-trailer, and 4 axles trailer).

Table 1. Occupancy percentages on the Rio Papaloapan Bridge in 2009

Number of vehicles	0	1	2	3	4	5	6	7	8	9
Occurrence probability (%)	19.98	26.31	28.74	15.17	6.51	2.43	0.67	0.10	0.040	0.002

Table 2. Lane percentage occupancy for the Rio Papaloapan Bridge in 2009

Lane	Northbound Direction		Southbound Direction	
	Right	Left	Right	Left
Percentage (%)	35.61	14.23	35.82	14.32

Table 3. Traffic composition

Vehicle type	Cars and light trucks	B2	B3	C2	C3	T3-S2	T3-S3	T3-S2-R4
Percentage (%)	51,74	2.40	0.60	6.37	5.53	14.32	4.45	14.56

2.3 Statistical Distributions

The gross vehicle weights, for the different types of vehicles, were obtained from a statistical field study of the Mexican trucking transport, Gutiérrez (2002). For cars and light trucks, and buses (B2, B3), the weight was taken constant to an average value of 3.0, 17.5 and 26 tons, respectively. Loads on the bridge were taken considering two point loads per axle and the distance between axles was considering an average value. For all vehicles' configuration, the unloaded weight was constant, taking also average value for each type, according to table 4. For the study, the axles were identified as independent sets of axels, because the double axles are too close that can be considered as a single independent set with its corresponding total load. From the same field study, probability percentages of loaded and unloaded conditions were calculated for each vehicle (Table 5); for this condition, it is important to note that for the heaviest truck (T3-S2-R4), two overloaded condition were identified and considered independently as additional loaded conditions.

Table 4. Vehicles' unloaded gross weight and axles distances

Vehicle type	Light vehicles	B2	B3	C2	C3	T3-S2		T3-S3		T3-S2-R4			
Gross weight (unloaded)	3.0	17.5	26.0	4.0	8.0	20.0		26.0		41.0			
Independent sets of axles	2	2	2	2	2	3		3		5			
Distance between axels (m)	1.75	5.25	5.25	5.25	5.25	Set 1-2	Set 2-3	Set 1-2	Set 2-3	Set 1-2	Set 2-3	Set 3-4	Set 4-5
						1.75	10.5	1.75	10.5	1.75	8.75	1.75	8.75

Table 5. Loaded-unloaded percentage for truck vehicles

Vehicle type	C2	C3	T3-S2	T3-S3	T3-S2-R4		
Unloaded	38.0 %	22.5 %	20.7 %	36.3 %	39.7 %		
Loaded	62.0 %	77.5 %	79.3 %	63.7 %	44 to 74 Ton range	84 to 93 Ton range	94 to 100 Ton range
					44.7 %	9.3 %	6.3 %

Finally, from the statistical field study of the Mexican trucking transport, statistical distributions were obtained for each loaded vehicle, Quintana (2009). As to the wind live load, the National Water Commission has a weather station close to the bridge, where records of the wind direction and velocities of the past 20 years are available. In this latter case, an extreme value distribution was found to be the most representative for the field measured data.

2.4 Monte Carlo Simulation

The general procedure for the Monte Carlo simulation of a given traffic condition (number of vehicles per day), was based on the following sequence: 1) Random selection of a single live load condition on the bridge (number of vehicles at a given time); 2) Random designation of the vehicle types; 3) Random assignment of the vehicles' location on the bridge; 3) Random definition of the vehicles' gross weight; and 5) Random choice of the wind velocity. Once a specific load condition was defined, it was simulated using the finite elements model with StaDyn, Doyle (1997, 2004). The tensions on the 112 cables were the final result from a given condition, and thus, after a sufficiently large number of simulations, a representative statistical distribution of the tensions was obtained.

The traffic conditions were established starting with 9500 vehicles per day, which is the traffic report on the bridge in 2009, and then 3 different growth rates were considered in a 30 years scenario. A 4% growth rate scenario represented the growth according to the last 10 years records; a 2% growth rate typified a pessimistic scenario; and a 6% growth rate exemplified an optimistic condition. As a result, six different traffic conditions were identified: 11580, 14117, 17208, 20816, 30812, and 54563 vehicles per day; and from those, the bridge occupancy were calculated considering constant speed, Quintana (2009), and shown in Table 6.

Table 6. Bridge occupancy for different daily traffic scenarios

	Number of vehicles at a given time (occupancy)																	Daily Traffic
	0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	
Probability	10,00	24,76	28,74	20,76	10,44	3,87	1,10	0,24	0,04	-	-	-	-	-	-	-	-	11580
	5,76	17,99	26,36	24,02	15,24	7,14	2,55	0,71	0,15	0,02	-	-	-	-	-	-	-	14117
	2,86	11,39	21,24	24,65	19,92	11,89	5,42	1,92	0,53	0,11	0,02	-	-	-	-	-	-	17208
	1,21	6,17	14,68	21,75	22,43	17,09	9,94	4,50	1,60	0,45	0,10	0,01	-	-	-	-	-	20816
	0,08	0,76	3,17	8.22	14,80	19,69	20,01	15,85	9,88	4,87	1,88	0,57	0,13	0,02	-	-	-	30812
	-	-	-	0,03	0,18	0,74	2,34	5,73	11,06	16,85	20,22	18,90	13,50	7,11	2,61	0,59	0,06	54563

2.5 Alarm Level Settings

To define the statistical distribution formulas for the tensions on the cables obtained from the Monte Carlo simulation, the software program @Risk 4.5b, was used to compare different types of distributions and select the most representative. In general, it was found that the Generalized Extreme Value statistical distribution was the one that matched better for the simulation data, which, in each case, was described with the shape (k), location (μ) and scale (σ) parameters (as an example, Table 7 shows these parameters for all cables in the semi-harp number 1).

In this case and from these data, the first upper alarm level for the tension of a cable was selected to 99 percentile, while the lower alarm level was to 1 percentile. The second lower and upper alarm levels were selected according to design limits; for the lower a 5% over the minimum design value, and for the upper to 95% of the maximum design value. Finally, the third alarm upper level was set to 95% of the maximum allowable tension (45% of the ultimate resistance of the cable), and the third lower alarm level was set to 80% of the minimum design value. Alarm levels for semi-harp are shown in Table 7.

Table 7. Alarms limits for semi-harp 1 with 9500 vehicles per day.

Cable	Semi-harp 1			Alarm levels (Tons)					
	k	σ	μ	First alarm level		Second alarm level		Third Alarm level	
				Min	Max	Min	Max	Min	Max
1	-0.0030	1.8209	212.34	209.55	220.73	177.65	228.95	149.6	249.01
2	0.0209	0.9326	116.95	115.54	121.45	86.45	124.95	72.80	158.46
3	0.0499	0.8096	148.84	147.64	153.02	125.40	160.55	105.60	203.73
4	0.0894	0.6110	152.67	151.79	156.14	137.75	173.85	116.00	215.05
5	0.1644	0.4498	179.14	178.53	182.23	152.95	193.80	128.80	237.69
6	0.2748	0.3408	182.94	182.51	186.08	165.30	209.00	139.20	249.01
7	0.2635	0.3702	194.56	194.09	197.87	179.55	227.05	151.20	271.64
8	0.1679	0.5755	225.72	224.94	229.70	191.9	239.40	161.60	282.96

9	0.0236	1.1087	234.44	232.77	239.82	202.35	251.75	170.40	294.28
10	-0.0280	1.5551	252.04	249.61	258.75	211.85	263.15	178.40	305.60
11	-0.039	1.6805	259.71	257.06	266.77	219.45	272.65	184.80	316.92
12	-0.034	1.6226	260.49	257.94	267.39	221.35	275.50	186.40	316.92
13	-0.022	1.4155	227.16	224.96	233.39	215.65	268.85	181.60	316.92
14	-0.009	1.4836	267.39	265.10	274.07	245.10	304.00	206.40	339.56

3 CONCLUSIONS

The use of the Monte Carlo technique with a finite element calibrated model, permitted a reasonable approach to simulate different loading conditions on a bridge and to calculate the stress statistical distributions on structural elements, such as cables in a stayed bridge, and to compare these to operational conditions and design parameters and, from those, to define alarm limits for a structural health monitoring strategy. As it can be noted, the first alarm level is defined according to live loading conditions, while second and third levels to design limits and structural capacity; this condition can be maintained if loading conditions do not exceed the design limits; otherwise, additional considerations must be taken.

Although simulations can become cumbersome, once the finite element model is adjusted to a given structure, these information can be not only used for structural health monitoring, but also for structural prognosis, considering future loading conditions, and then, to define maintenance strategies.

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