SEISMIC ANALYSIS AND ASSESSMENT OF THE FRENCH SAINT-NAZAIRE CABLE-STAYED BRIDGE BASED ON A MULTIMODAL RESPONSE SPECTRUM ANALYSIS.

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Abstract

Since October 2010, a new seismic zoning has divided the French territory in five zones: from zone 1 with low seismic hazard, to zone 5 (Caribbean Islands) where seismic hazard is important. Return periods associated with design seismic action of bridges usually range from 475 years to 1250 years depending on their importance category. Then, probability of occurrence of a seism remains low, even in areas prone to earthquakes. The intensification of seismic prescriptions in France regarding existing structures results from a cost/benefit analysis between reinforcement cost of bridges and human, social or economic consequences of an earthquake. These indirect consequences frequently exceed bridges value, particularly for structures of great local significance.

Saint-Nazaire Bridge (France), built in 1975, has a total length of 3356 m. The structure is composed of two concrete approach viaducts with length of 1115 m (North) and 1521 m (South), and a main cable-stayed metal structure, 720 m in length, with a central span of 404 m. According to the new French seismic zoning, Saint-Nazaire Bridge is now located in zone 3, equivalent to moderate seismic hazard, whereas this location was considered as non-seismic in the previous zoning. Yet, the structure carries an average of 35 000 vehicles per day, but also electric and communications networks. The structure provides regional service, and no close (less than 3 km) alternative route is available. In view of these elements, the structure is ranked of major socio-economic significance: evaluation of its seismic performance is thus essential.

This paper provides a detailed description of the development of a three-dimensional linear elastic beam model to assess the seismic performance of the main cable-stayed structure and its approach spans. Two Cerema softwares are used in this study: PCP [1] to build the three-dimensional model and CDS [2] for sectional calculation. Effective rigidities of concrete piers and foundation piles are evaluated through an iterative process. Due to the uncertainty on soil dynamic properties and behavior, a range calculation is performed: maximum and minimum range values, with and without soil liquefaction. Modal properties of the bridge are evaluated, *i.e.* eigenfrequencies, mode shapes, modal masses and damping. A multimodal analysis is completed to assess the response of the structure to a spectrum defined by Eurocode 8. Efforts obtained are compared to resistance capacities of critical members of the bridge (piles, piers, cables, bearings, etc.), according to prescriptions of Eurocode 2 [3] and Eurocode 8 [4]. For every structural part, a resisting seismic level is assessed, as a percentage of the reference earthquake for a new structure. Shorter piers of approach viaducts show the most significant theoretical overruns, particularly under shear seismic demand. Finally, retrofit recommendations are presented to improve the Saint-Nazaire Bridge behavior regarding seismic hazard.

Keywords: bridge; cable-stayed; seismic hazard; modal analysis; Eurocode

1. Introduction

Since October 2010, a new seismic zoning has divided the French territory in five zones: from zone 1 with low seismic hazard, to zone 5 (Caribbean Islands) where seismic hazard is important. Return periods associated with design seismic action of bridges usually range from 475 years to 1250 years depending on the importance category of the structure. Probability of occurrence of a seism remains low, even in areas prone to earthquakes. The intensification of seismic prescriptions in France for existing structures results from a cost/benefit analysis between reinforcement cost of bridges and human, social or economic consequences of an earthquake. These indirect consequences frequently exceed bridges value, particularly for structures of great local significance.

Saint-Nazaire Bridge (France) is composed of two concrete approach viaducts with length of 1115 m (North) and 1521 m (South), and a main cable-stayed metal structure, 720 m in length, for a total length of 3356 m. The main structure is composed of a metallic box-girder, two metallic masts carrying 72 cable-stays and 4 concrete piers: 2 main piers (N1 and S1) and 2 abutment-piers (N2 and S2). The northern, center and southern spans are respectively 158 m, 404 m and 158 m long. Masts have an inverted V-shape and are 68 m high. Metallic box-girder is 15 m wide, have an internal height of 3.2 m and is composed of stiffened plate elements which thickness range from 12 mm to 16 mm. The North approach viaduct has 22 spans (piers N2 to N24) and the South approach has 30 spans (piers S2 to S32), coupled by group of 4. Each span is composed of 4 prestressed concrete girders, for a total height of 3.3 m and a width of 13.6 m. Transverse stoppers are located on piers N6, N10, N14, N18 and N22 for the North approach; on piers S2 and S6 for the South approach. Fig. *1* shows an overview of the main cable-stayed structure and its approaches.

Fig. 1 - Overview of Saint-Nazaire Bridge (Photo by Ludovic Péron)

According to the new French seismic zoning, Saint-Nazaire Bridge is located in zone 3, equivalent to moderate seismic hazard, whereas this location was considered as non-seismic in the previous zoning. The structure carries an average of 35 000 vehicles per day, but also electric and communications networks. The structure provides regional service, and no close (more than 3 km) alternative route is available. Based on these elements, the Cerema handbook [5] assigns a major socio-economic significance to the structure: evaluation of its seismic performance is thus essential. This paper is organized as follows. The following paragraph is dedicated to the modelling of the cable-stayed bridge and the South approach. Ground modelling and the methodology to evaluate effective rigidities of beam elements modelling piles and piers are presented, as well as the horizontal and vertical seismic spectra as prescribed by Eurocode 8 for equivalent new bridge structures. Number of necessary modes for the multimodal spectrum analysis is evaluated through a modal analysis. Results of the seismic analysis based on the model, including effective rigidities, are finally presented.

2. Modelling of the cable-stayed bridge and its approaches

This section provides a detailed description of three-dimensional linear elastic beam models to assess the seismic performance of the main cable-stayed structure and the South approach (same type of model has been repeated for the North approach but is not presented this paper). Since the sliding bearing at the end of the approach guarantees independence between the two structures, they are modelled separately with PCP software [1], developed by Cerema. All sectional calculations are performed with CDS software [2], also developed by Cerema.

2.1. Modelling of the cable-stayed bridge

The metallic deck of the cable-stayed bridge is modeled using beam elements. The deck beam elements are connected to the cable-stays anchor points through rigid elements. Pier "arms" and mast "legs" are connected through a hinge allowing rotation around the transversal axis. Rigid elements connect: mast legs and mast head; mast head and cable-stays; deck and abutment-pier; deck and masts. Modelling of the mast, cable-stays and the deck are presented in Fig. 2(a). Mast bearings are laterally blocked to model the lateral stoppers. Main piers are based on 18 piles of 1 800 mm diameter, divided in one-meter-long beam elements. Each pile head is connected to a node located at the mast longitudinal axis by a rigid element. A rigid element connects this node to the base of the mast. Modelling of the abutment-pier is presented in Fig. 2(b). Both abutment-piers are based on 6 piles of 1 800 mm diameter. Mechanical properties of main elements are summarized in Table 1. Material properties of the main deck and the metallic parts of the mast (leg and head) are: Young modulus $E = 210\,000$ MPa, density $\rho = 7\,850$ kg/m^3, Poisson coefficient $\nu = 0.3$ and thermal coefficient $\alpha_T = 1.10^{-5}$ K^{-1}. Concrete of the main piers and the abutment-piers is C35 and have the following properties: Young modulus $E = 35\,700$ MPa, density $\rho = 2\,500$ kg/m^3, Poisson coefficient $\nu = 0.2$ and thermal coefficient $\alpha_T = 1.10^{-5}$ K^{-1}. Cable-stays are divided into four "families" with various cross-sections and density, not detailed in this paper. Material properties of the cable-stays are: Young modulus $E = 160\,000$ MPa, Poisson coefficient $\nu = 0.2$ and thermal coefficient $\alpha_T = 1.10^{-5}$ K^{-1}. The material representing neoprene bearing elements has a Young modulus of 2 000 MPa, and a fictive Poisson coefficient of 999 to obtain a dynamic shear modulus G of 1 MPa as prescribed by Eurocode 8-2 [4].

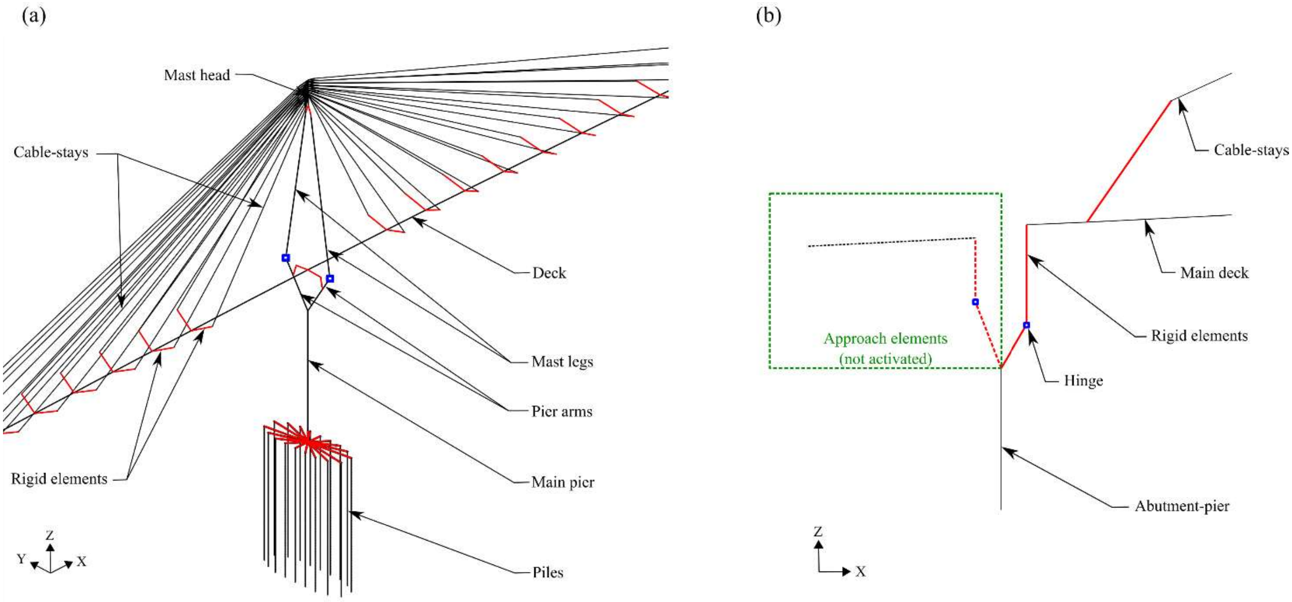

Fig. 2 - Modelling of: (a) the mast (isometric view) and metallic box; (b) abutment-pier (longitudinal view). Beam elements are represented in black, rigid elements in red.

Table 1 – Mechanical properties of the elements modelling the cable-stayed bridge.

Cross-section	Surface (m^2)	Torsional inertia (m^4)	Flexural inertia longitudinal axis (m^4)	Flexural inertia transversal axis (m^4)
Northern deck	0.605	2.558	1.208	12.806
Center deck	0.623	2.656	1.266	12.939
Southern deck	0.629	2.662	1.282	12.991
Main pier	28.975	373.5	1476.8	140.5
Mast (Leg)	0.194	0.200	0.128	0.194
Mast (Head)	0.387	0.498	0.387	0.377
Abutment-pier (Bottom)	3.6	0.258	9.7	2.252
Abutment-pier (Top)	12	19.10	16	9
Bearing	0.49	1	1	1

2.2. Modelling of the South approach

Each approach span deck is modeled using three beam elements. As prestressed girder widths differ at mid-span and at each end, two cross-sections are defined: S1 refers to approaches cross-section at the end of each span, and S2 to approaches cross-section at mid-span. Mechanical properties of sections S1 and S2 are presented in Table 2. As PCP affects mechanical properties to sections (*i.e.* nodes of beam elements), the first element mechanical properties are linearly interpolated between S1 and S2; the second element properties are constant and identical to S2 and the third element properties are linearly interpolated between S2 and S1. Approach spans are grouped by four, and expansion joints are modelled at each end of groups. Hitched spans, within a group of four, are connected through a beam element of low inertia. Bearings are modelled with beam elements. As five different types of bearing exist in the South approach, for the sake of brevity, bearings mechanical properties and material will not be detailed in this paper. Rigid elements connect the deck to bearings, and bearings to piers. Mechanical properties of beam elements representing piers are presented in Table 2. Each pier of the South approach is based on four piles of 1 800 mm diameter (S3 to S7) or 1 500 mm diameter (S8 to S29). An isometric view of the modelling of a group of four hitched spans is presented in Fig. 3.

Table 2 – Mechanical properties of elements of the South approach

Cross-section	Surface (m^2)	Torsional inertia (m^4)	Flexural inertia longitudinal axis (m^4)	Flexural inertia transversal axis (m^4)
S1	5.970	0.539	6.305	90.547
S2	7.102	0.753	6.831	107.906
Pier	3.6	0.258	2.252	9.7

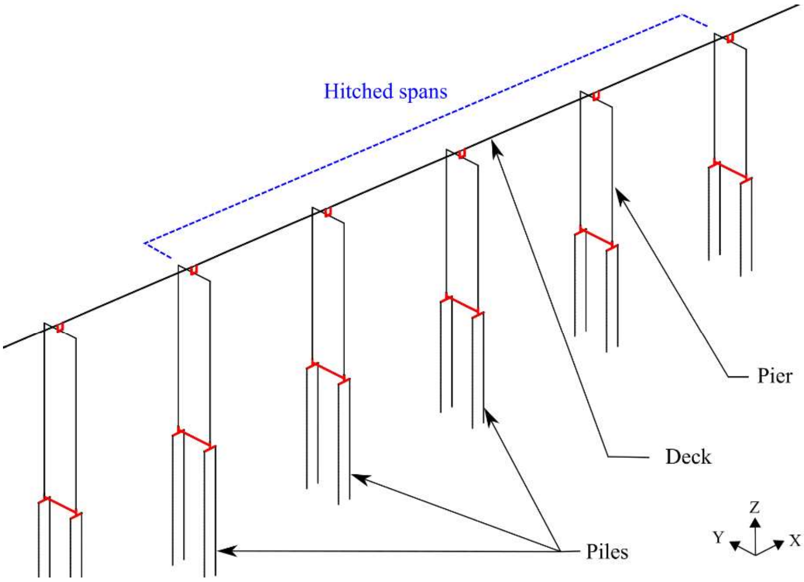

Fig. 3 – Isometric view of the modelling of a group of four hitched spans of the South approach. Beam elements are represented in black, rigid elements in red.

2.3. Ground Modelling and range calculation

Piles are reinforced by a sheath, which had an initial thickness of 16 mm for piles of 1 800 mm diameter, and 14 mm for piles of 1 500 mm diameter. No information is available regarding the residual thickness of these sheathes. Furthermore, soil liquefaction must be considered in case of a major earthquake. A range calculation is then performed, based on the two following assumptions: (i) the "soft foundations" assumption, where half of the sheath thickness is considered in the resisting section, soil liquefaction over 20 m is assumed and static reaction modulus of soil are used; (ii) the "rigid foundations" assumption, where the initial thickness of the sheath is considered in the resisting section, no soil liquefaction and dynamic reaction modulus of soil are used. Based on these assumptions, mechanical properties of piles are homogenized based on the sheath thickness considered. The results (cross-section S, torsional inertia I_x and flexural inertia I_y and I_z) are presented in Table 3. All piles have the following material properties: Young modulus is 32 100 MPa, density is 2 500 kg/m^3, Poisson coefficient is 0.2 and thermal coefficient is 10^{-5} K^{-1}. For the sake of simplification, N1, S1 to S8 and S22 to S31 are considered based on sand over the entire height of the piles; N2 and S9 to S21 are considered based on mud. In all cases, base of piles enters the substratum over approximately 50 cm, then embedding is considered in the modelling. Based on soil characteristics, reaction modulus K_f is calculated for each pile diameter thanks to the formula provided by [6]:

$$K_f = \frac{12E_M}{\frac{4}{3}\frac{B_0}{B}\left(2{,}65\frac{B}{B_0}\right)^{\alpha} + \alpha} \tag{1}$$

where B is the diameter of the pile, B_0 the reference diameter (0.6 m), α the rheological coefficient of the soil and E_M the pressiometric modulus. Table 4 presents reaction modulus for sand and mud, for the two diameters of piles.

Table 3 – Mechanical properties of piles based on the residual thickness of the metallic sheath

Diameter of pile (mm)	Soft foundations				Rigid foundations			
	S (m^2)	I_x (m^4)	I_y (m^4)	I_z (m^4)	S (m^2)	I_x (m^4)	I_y (m^4)	I_z (m^4)
1500	1.91	0.497	0.287	0.287	2.12	0.497	0.345	0.345
1800	2.74	1.031	0.591	0.591	3.02	1.031	0.706	0.706

Table 4 – Reaction modulus of soils (sand and mud) for both diameters of piles

Soil	**Modulus K_f - 1 800 mm pile (MPa)**	**Modulus K_f - 1 500 mm pile (MPa)**
Sand	31,47	28,76
Mud	6,84	6,41

2.4. Effective rigidities

The Cerema handbook [5] advises, for existing bridges seismic diagnosis, to evaluate dynamic properties of reinforced concrete elements on the basis of cracked stiffness, considering that this level of damage is most likely to be reached with a significant earthquake event. Effective rigidities of piles and piers are assessed using the second method proposed in Annex C of Eurocode 8-2 [4], according to the following process:

1. A first seismic calculation is performed, considering gross inertia of piles (or piers);
2. Stresses in uncracked sections of piles (or piers) are evaluated;
3. Sections with stresses lower than -1.5*f_{ctm} (where f_{ctm} is the mean tensile strength) are considered cracked. A uniform height of cracked sections is kept for all piles (or piers) of a support;
4. Cracked inertia is evaluated with Eq. (2):

$$E_c.I_{cr} = \frac{1.2M_{Ed}}{\Phi_{Ed}} \tag{2}$$

where I_{cr} is cracked inertia, E_c is the Young modulus of concrete, M_{Ed} and Φ_{Ed} correspond to the attained point on the moment-curvature law of the reinforced concrete section under design seismic action (*i.e.* the maximum bending moment over the considered cracked zone and associated curvature). Cracked inertia is then multiplied by 1.2, in order to take into account average rigidity of the element in the whole cracked area (stiffening effect of the uncracked distance between two adjacent cracks), and applied over the whole considered cracked zone.

5. Cracked inertia in both directions (longitudinal and transversal) are integrated in the model. A new seismic calculation is performed with updated mechanic characteristics;
6. An iterative calculation (steps 4 and 5) is performed until convergence (variation of cracked inertia lower than 5%).

The whole methodology is achieved in both directions for all piles, then for all piers. Table 5 and Table 6 summarize results obtained with the presented methodology, respectively in the longitudinal and transversal directions: cracked height is expressed as a range percentage of total height of the considered structural element (pile or pier), and inertia is expressed as a range percentage of gross inertia.

Table 5 – Range of cracked height and effective rigidities of piles and piers in the longitudinal direction

Elements		**Soft foundations assumption**		**Rigid foundations assumption**	
		Height (%)	**Inertia (%)**	**Height (%)**	**Inertia (%)**
Piles of cable-stayed structure	Head	19 - 24	44 - 48	0	100
	Base	19 - 28	42 - 55	0	100
Main piers of cable-stayed structure	Head	0	100	0	100
	Base	0	100	0	100
Abutment-piers of cable-stayed structure	Head	0	100	0	100
	Base	0	100	50	24 - 46
Piles of South approach	Head	12 - 50	45	0	100
	Base	12 - 50	45	0	100
Piers of South approach	Head	0 - 30	30 - 100	0 - 25	15 - 100
	Base	0 - 20	30 - 100	0 - 25	15 – 100

Table 6 – Range of cracked height and effective rigidities of piles and piers in the transversal direction

Elements		**Soft foundations assumption**		**Rigid foundations assumption**	
		Height (%)	**Inertia (%)**	**Height (%)**	**Inertia (%)**
Piles of cable-stayed structure	Head	19 - 24	44 - 52	0	100
	Base	19 - 28	40 - 58	0	100
Main piers of cable-stayed structure	Head	0	100	0	100
	Base	0	100	0	100
Abutment piers of cable-stayed structure	Head	0	100	50	46 - 49
	Base	75	52 - 59	50	45 - 69
Piles of South approach	Head	12 - 50	45	0	100
	Base	12 - 50	45	0	100
Piers of South approach	Head	0 - 30	30 - 100	0	100
	Base	0 - 20	30 - 100	0	100

Seismic justifications are then carried out with efforts obtained with the model including effective rigidities of beam elements.

2.5. Seismic spectrum

According to the new French seismic zoning, Saint-Nazaire Bridge is located in zone 3, equivalent to moderate seismic hazard. The reference peak ground acceleration on type A ground (*i.e.*, rock) is 1.1 m/s^2, for a new bridge of normal importance. This value corresponds to maximum ground acceleration for 475 years return period. It must be weighted by the importance factor γ_I, the ground coefficient S, the topographic amplification τ and regarding the limit state considered. The structure is ranked of major socio-economic significance: the importance factor $\gamma_I$is 1.4. Considering the very poor quality of foundation soil, the safety assumption of an E ground type is considered in this study. The value of the ground coefficient S is 1.8. However, the Saint-Nazaire bridge location has no topographic distinctive characteristic, then $\tau = 1$. We consider the limit state of Significant Damage (SD); no reduction factor is applied to the design acceleration. In view of these elements, the reference peak ground acceleration considered in this study is:

$$a_{ref-SD} = \gamma_I * S * \tau * a_{ref} = 1.4 * 1.8 * 1 * 1.1 = 2.77 \text{ m/s}^2 \quad (3)$$

The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum. The elastic response spectrum $S_e(T)$ is defined by the following expressions:

$$0 \le T \le T_B : S_e(T) = a_{ref-SD} * \left[1 + \frac{T}{T_B} * (2.5 * \eta - 1)\right] \quad (4)$$

$$T_B \le T \le T_C : S_e(T) = a_{ref-SD} * \eta * 2.5 \quad (5)$$

$$T_C \le T \le T_D : S_e(T) = a_{ref-SD} * \eta * 2.5 * \left[\frac{T_C}{T}\right] \quad (6)$$

$$T_D \le T : S_e(T) = a_{ref-SD} * \eta * 2.5 * \left[\frac{T_C * T_D}{T^2}\right] \quad (7)$$

where T is the vibration period of the system; T_B is the lower limit of the period of the constant spectral acceleration branch; T_C is the upper limit of the period of the constant spectral acceleration branch; T_D is the value defining the beginning of the constant displacement response range of the spectrum and η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping. Values of T_B, T_C and T_D depend on the seismic zone and the ground type. In this study, their values are, respectively, 0.08, 0.45 and 1.25. The horizontal and vertical elastic response spectra are presented in Fig. 4.

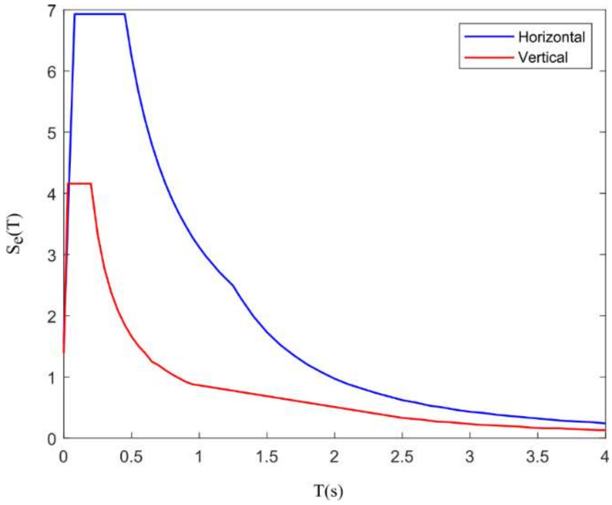

Fig. 4 - Horizontal and vertical spectra

3. Modal analysis

For each structure, the main cable-stayed metal structure and the South concrete approach viaduct, modal analysis is performed with the range approach presented in section 2.3. As intuitively expected, eigenfrequencies obtained are higher for the "rigid foundations" assumption. Additional eigenfrequencies related to foundation piles modes are also observed in the "soft foundations" assumption. Table *7* shows, for the 10 first modes of the main structure, the modal type, the eigenfrequency and the cumulated modal masses in the three directions for the two hypothesis of foundation flexibility. In Table *7*, BAL stands for a balancing mode, LONG for a longitudinal mode, BEND for a bending mode and FOUND for a pile mode.

Table 7 - Eigenfrequencies and cumulated modal masses in the three directions of the 10 first modes of the main cable-stayed metal structure

	Soft foundations assumption					Rigid foundations assumption				
N°	Type	f (Hz)	M_x (%)	M_y (%)	M_z (%)	Type	f (Hz)	M_x (%)	M_y (%)	M_z (%)
1	BAL 1	0.256	0.00	1.28	0.00	BAL 1	0.262	0.00	3.15	0.00
2	LONG	0.262	39.41	1.28	0.00	LONG	0.286	20.01	3.15	0.00
3	BEND 1	0.329	39.41	1.28	2.51	BEND 1	0.329	20.01	3.15	2.44
4	BAL 2	0.415	39.41	6.73	2.51	BEND 2	0.505	20.09	3.15	2.44
5	BAL 3	0.452	39.41	22.57	2.51	BAL 2	0.639	20.09	3.31	2.44
6	BEND 2	0.481	41.28	22.57	2.52	BAL 3	0.701	20.09	12.51	2.44
7	FOUND	0.482	41.28	49.29	2.52	BEND 3	0.741	20.09	12.51	4.21
8	BEND 3	0.496	53.64	49.29	2.52	BEND 4	0.844	24.21	12.51	4.21
9	FOUND	0.501	53.64	75.91	2.52	BAL 4	0.851	24.21	12.52	4.21
10	BEND 4	0.514	64.39	75.91	2.52	BEND 5	0.902	24.47	12.52	4.71

Eigenfrequencies can be compared to those obtained experimentally in 1999 and summarized in [7]: 10 accelerometers were positioned on the main deck, 3 on the South mast and 14 on various cable-stays to identify modal characteristics of the cable-stays under ambient vibrations and wind loading. Analysis of recordings of the longest cable-stay allowed to identify the first three bending modes frequencies of the main structure: 0.323 Hz, 0.476 Hz and 0.732 Hz. These values are consistent with the values obtained with the "rigid foundations" assumption: 0.329 Hz, 0.505 Hz and 0.741 Hz, which correspond, respectively, to a difference of 1.8%, 5.7% and 1.2%. This assumption is the closest to the actual behavior of the structure in a service state. Eigenfrequencies obtained with the model are also consistent with those obtained in previous dynamic studies conducted [8, 9]. Modal shapes of the first three bending modes of the main cable-stayed structure are presented in Fig. 5.

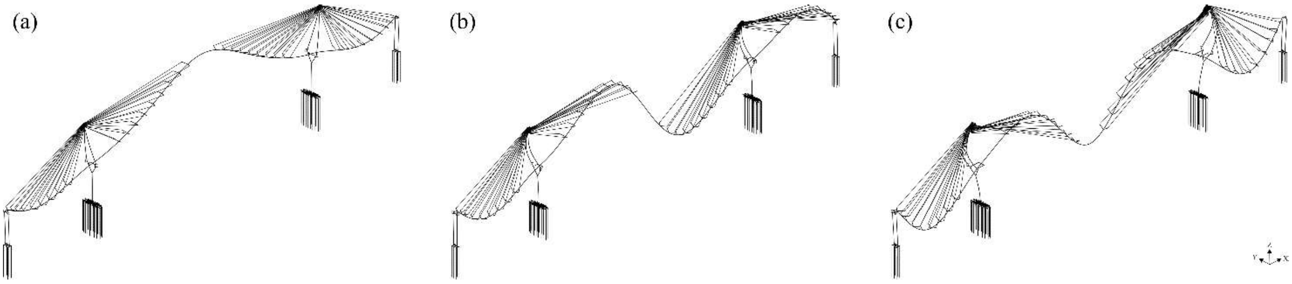

Fig. 5 – Modal shapes of first three bending modes of the main cable-stayed spans, with eigenfrequencies of 0.329 Hz, 0.505 Hz and 0.741 Hz

A percentage of modal masses above 90% of the total vibrating mass of the structure (in every direction) is mandatory to perform the seismic analysis. This leads to consider 350 modes for the main structure for both assumptions and, 400 and 600 modes, respectively for the "soft" and "rigid" assumptions, for the South approach spans.

4. Seismic analysis results

The effects of the seismic action are evaluated through a multimodal analysis using the Eurocode 8 horizontal and vertical response spectra, presented in Section 2.5. Due to the high percentage of vibrating modal mass requested (about 90% for the vertical direction and more than 95% for the two horizontal directions), it was not deemed necessary to weight the efforts calculated by PCP to account for residual modes contribution. In each direction, the efforts obtained are the result of a quadratic combination of the contribution of each mode. As prescribed in Eurocode 8 Part 1 [10], seismic effects for the three principal directions (Longitudinal, Transversal and Vertical) are combined considering a preponderant direction and two concomitant directions, leading to the following combinations:

$$E_1 = E_L \pm 0.3E_T \pm 0.3E_V \tag{8}$$

$$E_2 = E_T \pm 0.3E_L \pm 0.3E_V \tag{9}$$

$$E_3 = E_V \pm 0.3E_L \pm 0.3E_T \tag{10}$$

Global seismic effects A_{Ed} correspond to the envelope of the combinations presented in Eqs. (8), (9) and (10). The seismic combination presented in Eq. (11) is considered for the verification of structural parts.

$$E_d = G_k + P_k + A_{Ed} + \psi_{21}Q_{1k} \tag{11}$$

where « + » means « combined to », G_k refers to permanent charges with its characteristic values, P_k is the characteristic value of the prestressing action, A_{Ed} is the seismic action, Q_{1k} is the nominal value of traffic loads and ψ_{21} refers to the combination coefficient corresponding to quasi-permanent charges due to traffic supposed simultaneous to seismic action. The concomitance between traffic loads and seismic action should only be considered for structures withstanding heavy traffic (urban structures with an intense traffic or railway bridges with a huge traffic). Then, the coefficient ψ_{21} is null for the Saint-Nazaire Bridge. For the main structure and the South approach, the following elements are verified, following prescriptions of Eurocode 8: bearings, decks, piers and foundation piles. Specific parts of the main structure are also verified, such as the prestressed bars located at the abutment piers of the cable-stayed structure and the cable-stays.

For elastomeric bearings, Eurocode 8 prescribes that maximal distortion is limited to 2 when subjected to horizontal efforts. Under the “soft foundations” assumption, bearings of piers S3 to S28 are verified, apart from S26. Overruns of 15% to 50% are observed for bearings of piers S26 and S29 to S32. Under the “rigid foundations” assumption, similar overruns are observed for bearings of piers S19 to S32. The European norm NF EN 15129 [11] allows bearings distortion to reach 2.5. Based on this increased limit, bearings of piers S26, S29, S30 and S32 remains not verified for the “soft” assumption. For the “rigid” assumption, S23, S26 and S27 overruns are still greater than 15%, up to 23%. A proportional calculation indicates that bearings of the South approach are able to resist to a seismic level corresponding to 65% of the reference level for a new structure. Concerning the bearings of the main span, located at the mast, very small overruns are observed, and only in the “rigid” assumption.

A relative study is carried out for the decks. The efforts obtained with the seismic combination are compared to the efforts from the fundamental ULS combination. Considering the flexibility of the cable-stayed span structure, the efforts of the seismic combination are well below the efforts of the fundamental ULS combination. For the access spans decks, more rigid, we observe a lower margin but the efforts are still below the efforts obtained with a fundamental ULS combination. For both structures, we note small differences for the efforts in the decks between the two assumptions.

Bending and shear verifications of the piers are conducted in accordance with the prescriptions of Eurocode 2 [3]. Unlike decks, the foundation stiffness has a strong influence on the efforts in the piers. While all the piers are justified for the soft assumption, nearly no piers are justified in longitudinal axis bending (axis parallel to the traffic) with the rigid assumption. Piers resist to a 100% seismic reference level for the “soft foundations” assumption. This percentage of seismic reference level decreases to 50% for longitudinal axis bending for the piers for the “rigid foundations” assumption. Regarding shear, for structures requiring design shear reinforcement, Eurocode 2 indicates the shear resistance V_{Rd} is the smallest value of:

$$\begin{cases} V_{Rd,s} = \dfrac{A_{sw}}{s.z.f_{ywd}.cot\,\theta} \\ V_{Rd,max} = \dfrac{\alpha_{cw}.b_w.z.v_1.f_{cd}}{cot\,\theta + \tan\theta} \end{cases} \tag{12}$$

where A_{sw} is the cross-sectional area of the shear reinforcement, s is the spacing of the stirrups, f_{ywd} is the design yield strength of the shear reinforcement, z is the level arm of internal forces, θ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force ($1 \leq \cot\theta \leq 2.5$), v_1 is a strength reduction factor for concrete cracked in shear and α_{cw} is a coefficient taking account of the state of the stress in the compression chord. Considering Eq. (12), with the “soft foundations” assumption, piers S27 to S31 are not verified in the longitudinal direction, and S16, S18 to S31 are not verified in the transversal direction. For S28 to S31, the applied shear force exceeds from 50% to 120% the shear resistance. For the “rigid foundations” assumption, only S10 to S14 are justified at shear in the longitudinal direction, and only S4 is justified in the transversal direction. In the longitudinal direction, the shear resistance overruns for the longest piers (S3 to S18) are under 35% and are up to 120% for the shortest piers (S19 to S31). For the transversal direction, the shear force increases inversely proportional to the piers length and the overrun range from 5% to 10% for longest piers (S3-S5) to 170% to 200% for the shortest ones (S27-S31).

In the current standard, when members require design shear reinforcement, the shear resistance of concrete is neglected. Reflections are under way for the future standards of Eurocode for the concrete participation in shear resistance. The Cerema handbook [5] proposes an approach, adapted from [12] where the shear resistance at significant damages limit state can be evaluated with the following equation:

$$V_{RSDLs} = 0.75(V_c + V_s + V_p) \tag{13}$$

where V_c is the contribution of concrete to shear resistance, function of the displacement ductility μ_d:

$$V_c = k\sqrt{\frac{f_c}{\gamma_c CF}}0.8A_c - F_{conc} \tag{14}$$

where k = 0.3 for $\mu_d < 2$ then linearly decreasing to 0.1 for $\mu_d = 4$ then to 0.05 for $\mu_d = 7$; A_c is the confined concrete section; CF is the confidence coefficient ($CF = 1$ in this study); F_{conc} is the concomitant shear force under the effect of the seismic component in the direction perpendicular to the direction studied (calculated using a simplified method based on an acceleration corresponding to 0.3 x 0.7 x γ_{age} x a_{ref-EL}, with $\gamma_{age} = 1$); V_s is the contribution of shear reinforcement bars, calculated with Eq. (12) considering an angle θ of 30°; V_p is the contribution of the compression strut between the point of application of the normal force at the head of the pier and the compressed zone at the bottom, as illustrated in Fig. 6, and evaluated with:

$$V_p = N.tan\frac{D-x}{2H} \tag{15}$$

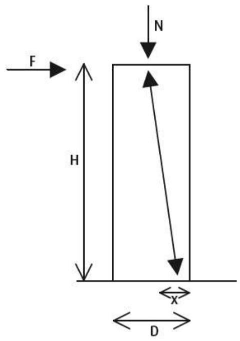

Fig. 6 - Contribution of the compression strut in the shear resistance

Based on this approach, for the “soft foundations” assumption, piers can resist a 100% seismic reference level for new structure compared with the level of 45% with the current Eurocode 2 approach. In the “rigid foundations” assumption, they can resist to a level of 50% whereas, with the current Eurocode 2 approach, shorter piers cannot resist, even to small seismic reference levels. The abutment piers, particular because of their connection to the main deck, are able to resist to a 100% seismic reference level (only 70% with the Eurocode 2 approach). Concerning the foundation piles, they are verified to a 100% seismic reference level, according to the French norm NF P94-262 [6].

Cerema handbook [5] prescribes that second order effects must be considered on slender elements when second order bending moments exceed 5% of first order bending moments. For the sake of brevity, detailed results of second order effects are not be presented in this paper. However, they did not question the previous conclusions. For the structural parts specific to the main span, *i.e.* the prestressed bars located at the abutment piers and the cable stays, verifications indicate that these parts are able to resist to the seismic combination. Finally, an analysis of displacements is carried out to estimate the risks of pounding: (i) between the main span and the approach spans at the abutment piers and, (ii) between grouped approach spans at expansion joints. This study shows that the risk cannot be excluded. Especially, risks of impacts at the abutment piers of the main structure would result in an increase of horizontal forces in the prestressed bars, a particularly vulnerable point of the structure [13]. Yet, the evaluation of efforts induced in structural parts by seismic pounding requires an extensive analysis that will need to be detailed in a future work.

5. Conclusions

Saint-Nazaire Bridge, composed of two prestressed concrete approach viaducts and a main cable-stayed metal structure, is currently the second longest bridge of France, and is ranked of major socio-economic significance. Recent evolutions of French seismic zoning have located the structure in zone 3, making the evaluation of its seismic performance essential. This paper describes the construction of a three-dimensional linear elastic beam model to assess the seismic performance of the main cable-stayed structure and the South approach through multimodal spectral analysis. An iterative methodology is proposed and applied to evaluate effective rigidities of cracked concrete piers and foundation piles. Because of the uncertainty regarding the residual thickness of pile sheathes, soil rigidities and liquefaction, a range calculation is performed based on two assumptions. The modal analysis conducted has shown that eigenfrequencies were consistent with previous studies and instrumental results. The multimodal response spectrum analysis showed that the structure is able to resist to a seismic level of 65% of the reference seismic level for new structures, apart from some piers in the rigid foundation assumption that resist to 50% of the reference level. However, current French seismic regulations do not require existing structures to be verified for the same level of seismic action, as new structures would. Cerema recommends establishing the level of reinforcement, if necessary, based on the best cost/performance compromise and advocates a minimum value of 2/3 of the maximum acceleration used for the design of a new structure. As illustrated in this paper, this limit of 65% is reached for elastomeric bearings of shorter piers of the South approach and overtaken for the same piers regarding shear in the transversal direction. An optimization of the structure's supports condition, like the replacement of bearings on the most rigid piers, the implementation of additional transverse stoppers or abutment dampers, could allow a better efforts repartition in the structure. Prestressed bars located at the abutment piers should be object of attention considering these parts are vital elements for the structure. Instrumentation of these elements over a significant period of time would ensure their proper functioning in service. In addition, securing these connecting elements would make it possible to reduce the overall vulnerability of the structure, including under seismic hazard.

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