Dynamic Modeling of Bridges: Observations from Field Testing

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Different bridge types and design configurations used in bridge engineering result in largely different dynamic mechanisms of response. Therefore, one of the most important problems to resolve about their theoretical dynamic modeling is to choose the most adequate and simple model for a particular bridge. The work carried out during the dynamic testing of several bridges is reviewed. The objective was to check the feasibility and accuracy of various numerical and analytical models assumed for the dynamic behavior of various bridge types (box-girder, slab bridges, and cable-stayed). Experimental field test data were used to derive the dynamic properties of the bridges. The most appropriate analytical models and dynamic properties of the elements used in the discretization mesh were derived for each bridge type on the basis of the correlation between the theoretical and experimental results. Taking these results into account, a set of practical recommendations for the dynamic modeling of different bridge types is proposed.

In recent years there has been increased interest in studying the dynamic performance of bridges, even for those not subjected to important or high dynamic loads such as wind or earthquakes. At the same time, more reliable electronic equipment and measuring devices for recording dynamic field data have been more widely available. Therefore, the number of dynamic tests performed in bridges is constantly increasing. In Europe, dynamic tests are often performed in addition to the mandatory static tests to verify the acceptability of a bridge before opening it to traffic. Moreover, a dynamic loading test can also be used during the service life of the bridge to monitor its performance. The design of the field test and the analysis of the dynamic test results require a reliable dynamic theoretical model. The reliability and robustness of such a model in describing the dynamic behavior of the bridge are important for the following reasons:

1. During the preparation phase of the dynamic test, the results of the analytical model will help choose the best instrumentation for and the location of the bridge;
2. During the dynamic test, the model will be a tool to check the reliability and adequacy of the recorded data; and
3. During the analysis of the experimental data, a comparison between experimental and theoretical results can provide a warning about inadequate performance of the bridge.

Therefore, careful attention should be paid to the development of theoretical models for the dynamic analysis of bridges. Two factors must be kept in mind:

1. Dealing with a particular structure with an important flexural work and a specific geometry (such as a plane structure with loads orthogonal to this plane) it will be possible to achieve good results with specific and simplified models.
2. On the other hand, the highly different bridge typologies, both longitudinal and transversal, with different dynamic mechanisms of response will not allow the development of general models or modeling criteria valid for all bridge types.

Moreover, the bridge designer always has to perform a static analysis, therefore a static model is always available. For this reason it is of interest to derive dynamic models that require little additional effort than that required for the static analysis. Bearing all these factors in mind, this paper compares the results of theoretical dynamic analyses and dynamic testing of various bridge types. The comparison of the results leads to a set of conclusions and practical recommendations for the dynamic modeling of bridges using simple finite element models (as the grillage formed by beam elements) or theoretical expressions (Rayleigh's method).

FEASIBILITY AND ACCURACY OF SIMPLE DYNAMIC MODELS FOR BRIDGE ANALYSIS

The accuracy of the dynamic modeling of different bridges is checked via a comparison between the results of analytical models and field tests. The comparison is performed as a function of bridge type comparing only the vibration frequencies. This is because in most of the cases presented the complete mode shapes were not obtained by testing and only the relative amplitudes of displacements or accelerations in the points instrumented were used to identify the experimental frequency with the corresponding vibration mode. In all cases, the finite element method (FEM) dynamic model is based on beam elements forming a frame or grillage with flexural and torsional dynamic behavior represented in respective mass and stiffness matrixes (*1*). The calculation of theoretical natural frequencies and mode shapes from the mass and stiffness matrixes of the global structure is based on the subspace iteration method (*2*).

Box-Girder Prestressed Concrete Bridges

To avoid distortion of the cross section, box-girder prestressed concrete bridges are not constructed with transverse bracings or diaphragms besides those located at the supports. This results in more freedom in selecting the location of the nodes of the mesh.

Alfonso X Bridge

This was a seven-span continuous prestressed box-girder bridge. The deck was supported on each pier and abutment by two pot bear-

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ings [i.e., an elastomeric pad confined in a metallic cylindrical body (*3*)]. The bridge was demolished in May 1985 for urban development. Before demolition, several static and dynamic tests were performed, including a test to failure (*4*). The dynamic test consisted in the placement of eight displacement transducers and two accelerometers in different spans of the bridge (midpoint and quarter-point). The bridge was dynamically excited by passages of a single, two-axle truck with a gross weight of 136 kN at various speeds from 2.8 to 14 m/sec, and quick releasing of a concrete cube (gross weight of 120 kN) previously loaded in the middle of the sixth span, obtaining a free-damped vibration of the bridge. The direction of the motion was vertical.

The analysis was carried out using a one-dimensional dynamic beam model (continuous beam) with 44 nodes and 10 elements per span. This finite element dynamic discretization using one-dimensional elements was chosen to be equal to the static model. The natural frequencies obtained by using the above model are compared with experimental ones in Table 1. In addition, the frequency of the first mode obtained using Rayleigh's method (*1*) was calculated, the magnitude of the self-weight being used as the applied load in the appropriate direction to obtain a deflection profile similar to the first vibration mode, and the previous discretization of the static model was also used. This assumption required almost no additional work because the static model was already defined. The result was f_1 = 1.77 Hz. This result is good (4 percent error), considering the simplicity of the applied method. As indicated in Table 1, the agreement with the FEM models is good for the lowest frequencies. The maximum error is 11 percent in Vibration Mode 5.

Diagonal Viaduct

The diagonal viaduct is a three-span (39-, 49.10-, and 39-m) continuous prestressed box-girder bridge. The cross section is 1.964 m deep, and the deck width is 10.95 m. The bridge is simply supported in bending and fixed in torsion on each support (Figure 1). In this case the instrumentation consisted of four displacement transducers and four accelerometers located in the bridge, as indicated in Figure 1. The vibration of the bridge was forced by

1. Passages of 1 two-axle truck (gross weight, 140 kN) at speeds from 2.7 to 22.2 m/sec over the undisturbed pavement;
2. Passages of the same truck over an artificial obstacle (the standardized RILEM plank) placed in the central section of the center span; and
3. Passages of 2 two-axle trucks with various relative positions and velocities to simulate controlled real traffic conditions.

TABLE 1. Natural Frequencies in Hertz in the Alfonso X Bridge

Mode	Theoretical	Experimental
f_1(Bending)	1.71	1.70
f_2(Bending)	3.05	2.97
f_3(Bending)	3.99	3.80
f_4(Bending)	4.99	4.50
f_5(Bending)	5.83	5.20

In all cases the direction of the recorded motion was in the vertical plane. For the dynamic analysis, a grillage model was used. The bridge was modeled as one longitudinal beam (spine model) that accounted for the overall mass, central moment of inertia, and flexural and torsion modulus of the overall cross section. The modification of the original statical model consisted of adding only the inertial characteristics (mass and rotational mass). Ten elements per span were used for the longitudinal beam. In addition, transverse elements are used to link the bearings and the longitudinal spine. The mechanical properties of the transverse beams were based on the properties of the diaphragms over the piers and the abutments. The theoretical results and those derived from the dynamic test performed are shown in Table 2. The first natural frequencies in bending and torsion are predicted almost exactly. Even for higher modes the error is less than 6 percent.

Box-Girder Composite Bridges

The bridges are located on Barcelona's littoral ring road over Highway A-19. These are 4 two-span bridges. The four bridges have the same longitudinal configuration with span lengths of 20.55 and 44.55 m. A typical cross section is shown in Figure 2. The steel box girder has a different thickness that ranged from 10 to 15 mm in the bottom flange and from 10 to 25 mm in the webs. To reduce the box distortion, a diaphragm is placed every 4.05 m. In addition, transverse stiffeners are placed at 1.35 m spacings. A total length of 16.20 m of the bottom flange centered on the pier is filled with concrete to better resist the negative bending. Two elastomeric bearing pads are used on the pier and abutments. For the dynamic test of this bridge displacement and acceleration transducers were also used. Two displacement transducers were placed close to the abutment section to check the possibility of uplifting associated with the important difference between span lengths. The instrumentation was disposed to measure the motion in the vertical direction. The excitation was achieved via the existing real traffic on the bridge.

Analogously to the diagonal viaduct, the dynamic model consisted of a simple FEM forming a grillage (therefore the finite elements are two-node beam elements) with a unique longitudinal element representing the overall cross section. A total of 38 beam elements and 39 nodes are used. The use of a one-dimensional model is possible thank to the presence of the bracings, which help avoid distortion of box shape. A node of the grillage was placed at the location of every transverse diaphragm to account for concentrated mass increments. The transverse elements over the piers and abutments linking the support nodes with the longitudinal spine were assumed as a rigid link in bending. This would accurately simulate the clamping action for torsion at the supports. Because of the unbalanced span lengths, the bridge is anchored in one abutment by means of high-strength bolts. Therefore, vertical springs were placed in the model at the nodes in this abutment. The properties of the springs are deduced from the total cross-sectional area of the bolts and corresponding modulus of elasticity.

Theoretical and experimental results are presented in Table 3. The experimental frequencies in torsion of the long span were not deduced because no instrumentation was placed to this end in the dynamic field test. Despite its simplicity, the models give results accurate enough for the lower modes in both bending and torsion.

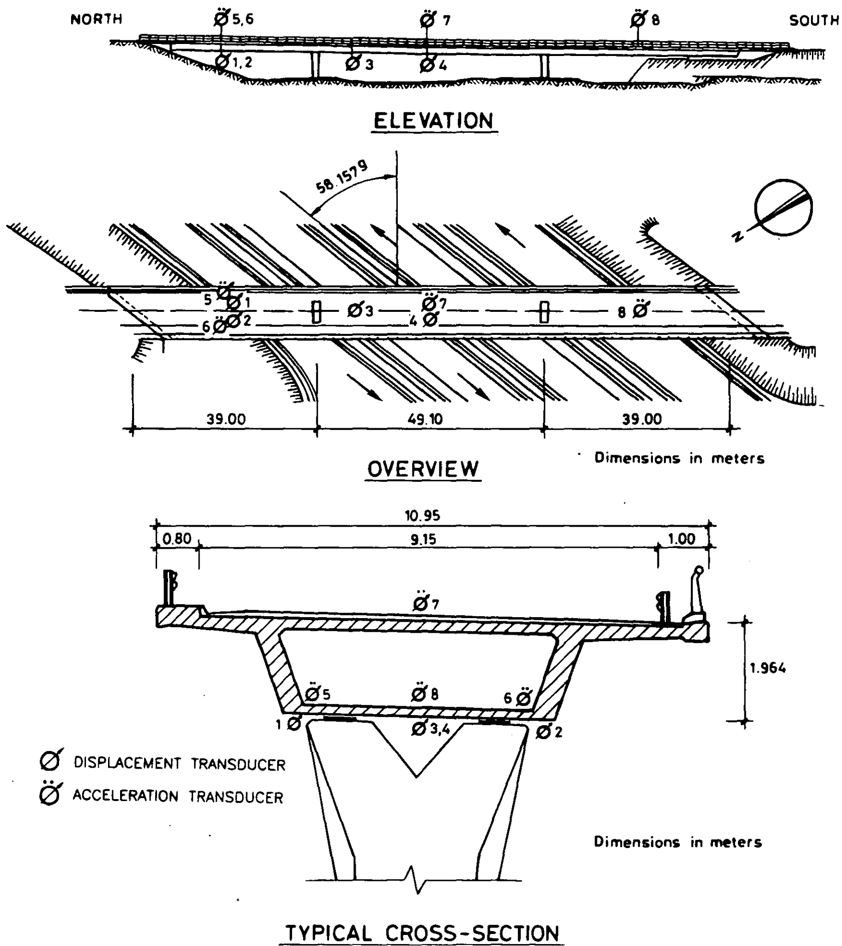


FIGURE 1. Description of diagonal viaduct.

Concrete Slab Bridges

Three prestressed concrete slab bridges with the following different geometric properties are presented:

1. Ratio of width to span,
2. Skewness, and
3. Horizontal curvature.

The comparisons between the results of these three bridges would provide a review of the model requirements to accurately evaluate the natural frequencies in torsion when the structural response is clearly two-dimensional (2-D) (slab) rather than one-dimensional (1-D) (beam).

Bridge OF-56.1 on Highway A-7

Bridge OF-56.1 is a prestressed concrete slab with four spans (11.5, 17.05, 17.05, and 11.05 m.). The deck is 6.5 m wide with circular voids of 45 cm in diameter (Figure 3). The bridge is straight, and two elastomeric bearings are placed at the piers and abutments.

The dynamic tests consisted in the placement of two accelerometers in the center of one of the longest spans and close to the sidewalks. The third accelerometer was placed in the center of the section located in the quarter-span of the contiguous shorter span. The vibration of the bridge was obtained using a two-axle truck with a total weight of 100 kN crossing the bridge at different speeds along one of the lanes, resulting in an eccentric (torsionally) excitation. Only the motion in the vertical direction was recorded and analyzed.

Because of the small width of the deck compared with its span length, a one-dimensional model (beam behavior) was used that would lead to good results. These were then compared with the results of a 2-D grillage model. The comparison is used to make recommendations for modeling wider, skewed, or curved slabs using 2-D grillage models (more than one longitudinal element in the cross section). Two dimensional grillage models are widely used by bridge designers in the static analysis of slab bridges. The calculation of properties (torsion modulus, bending stiffness) of the longitudinal and transverse elements of the grillage is very well documented for the analysis of static behavior of slab bridges (*5–8*). However, the main difficulty when a dynamic analysis is required that uses a grillage model is in deducing the inertial prop-

TABLE 2. Natural frequencies in Hertz in the Diagonal Viaduct

Mode	Theoretical	Experimental
f_1(Bending)	2.25	2.25
f_2(Bending)	3.53	3.48
f_3(Bending)	4.46	4.21
f_4(Torsion)	8.41	8.38

erties (mass and rotational mass) of longitudinal and transverse elements. To answer this question, the following dynamic models are defined:

1. A grillage model of the complete bridge with only one longitudinal member (1-D) and transverse members only in the sections over piers and abutments to adequately model the conditions at the supports.
2. A 2-D grillage of the main span only, with 5 longitudinal and 18 transversal fibers or ribs, resulting in 90 nodes and 157 elements. The following possibilities were also investigated: (2A) perfect clamping in bending at the supports and (2B) perfect hinge in bending at the supports. For each of the 2A and 2B assumptions, three cases were analyzed:
   a. Rotational mass (I_p) of longitudinal elements evaluated relative to the centroid of the global cross section,
   b. I_p of interior longitudinal elements evaluated with respect to own centroid and I_p of exterior elements relative to the edge grillage beam, and
   c. The same as Item 2 for interior elements and exterior elements with respect to the centroid of global cross section (Figure 4).

The mass and rotational mass of transverse elements in the grillage were always assumed to be 0.

3. Rayleigh's method is used to evaluate the torsional frequency using a 2-D static grillage. The load pattern consists of a set of concentrated loads in each of the edge nodes with the same magnitude and opposite direction, depending on the edge, to achieve a deformation similar to the torsional mode shape.

For the sake of comparison, the torsional frequency (f_T) was evaluated by means of the theoretical equation deduced for a beam model with both ends fixed in torsion and with uniform inertial and structural properties along the longitudinal axis (*9*) (Model 4):

$$f_T = \frac{1}{2L}\sqrt{\frac{GJ}{I_p}} \tag{1}$$

where G = transversal elasticity modulus,
J = torsional stiffness, and
L = span length under consideration.

The results are shown in Table 4. In this test, only the four lowest natural frequencies could be obtained (three in bending and one in torsion). The following observations can be made:

1. Model 1 gives very good results for bending modes,
2. In the case of this narrow slab, Model 1 gives also good results for torsion modes, but not as good as those of the bending modes,
3. For Model 2, the frequencies in bending are between those of Case 2A (ends fixed) and 2B (ends hinged).
4. The best results for the frequency in torsion are obtained when rotational inertia of the longitudinal members is calculated using Criterion c.
5. Rayleigh's method, which gives good results in bending, gives very bad results in torsion for a 2-D grillage model.
6. Despite the variations in mass and stiffness along the length of the bridge (because of absence of voids over piers and abutments) the theoretical expression (*1*) (Model 4) gives results similar to those of Model 1.

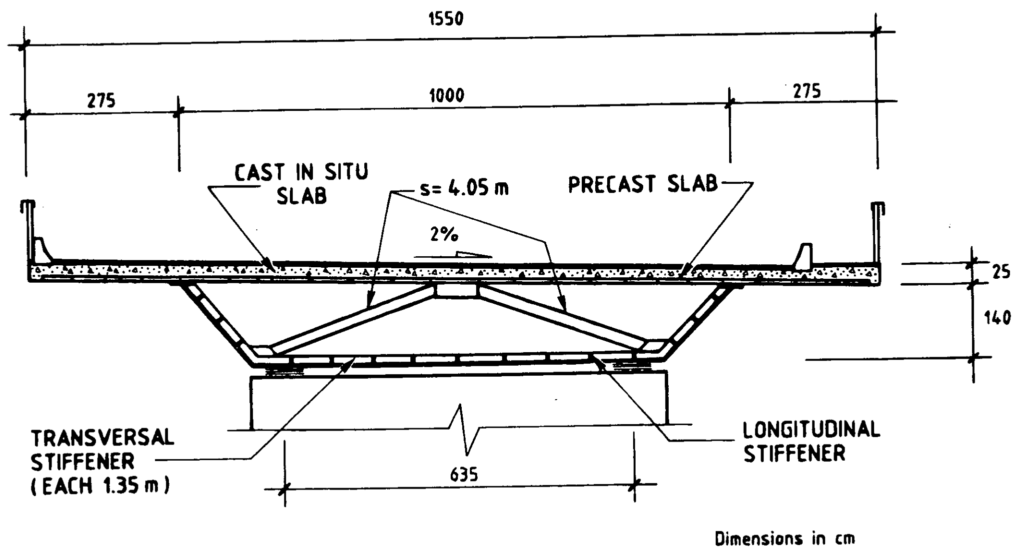


FIGURE 2. Cross section of bridges over Highway A-19.

TABLE 3. Natural Frequencies in Hertz in the littoral Ring-Road Bridges

Mode	Theoretical	Experimental
f_1(Bending)	1.73	1.70
f_2(Tor.long span)	2.24	-
f_3(Bending)	3.60	3.20
f_4(Tor.long span)	4.48	-
f_5(Tor.short span)	4.51	4.48
f_6(Bending)	6.75	6.90
f_7(Tor.long span)	6.85	-
f_8(Tor.long span)	9.05	-
f_9(Tor.short span)	9.10	10.30

(-) Not recorded

Bridge OF. 49-1 on Highway A-7

Bridge OF. 49-1 is a prestressed concrete slab with four spans (12.053, 17.649, 17.649, and 12.627 m). The deck width is 10.5 m with circular voids of 50 cm in diameter (Figure 5). The bridge is curved with a radius of 200 m. It is highly skewed at piers and abutments ($\alpha = 67.33^g$). As in the case of Bridge OF 56-1, three accelerometers were used, two of them placed at the halfway point of one of the longest spans and the other halfway across the shortest span to measure the motion in the vertical direction. The excitation of the bridge was achieved in the same way as that of Bridge OF 56-1. In this case, two models were used:

1. Grillage with only one longitudinal fiber where the mass and stiffness properties of the cross section are concentrated, resulting in 59 nodes and 58 elements. (The longitudinal fiber is divided into 12 beam elements per span to take into account the zone without circular voids over supports.)
2. Grillage model of the longest span with 9 longitudinal beams and 23 transverse beams, resulting in 207 nodes and 382 elements. Assigning structural and mass properties to the various elements was done while bearing in mind the conclusions derived from the slab bridge. Also the possibilities of perfect clamping and perfect hinging at both ends were investigated.

Table 5 shows the theoretical and experimental results obtained. As can be seen, even in this case of a wide, curved, and skewed slab bridge, the one-dimensional model gives good results for both bending and torsion frequencies. The frequency in torsion of Model 2 is also reasonably good.

Bridge OF.50-2 on Highway A-7

Bridge OF.50-2 is a prestressed concrete slab with four spans (17.014, 21.328, 26.555, and 21.328 m) and a deck width of 10 m. The cross section has four circular voids 0.85 cm in diameter. Two elastomeric bearing pads are placed in the abutments and only a single bearing is used at the circular piers. In this way, the slab is not restrained against torsion over the piers. This bridge presented an important sulfate-attack reaction in the concrete, and therefore extensive static and dynamic tests to check its structural performance were carried out. A total of 7 displacement transducers and 4 one-axial accelerometers and one triaxial accelerometer were used. In this case the directions of motion of interest were vertical and longitudinal and transverse in the horizontal plane. A total of 42 truck passages were performed with the following different configurations:

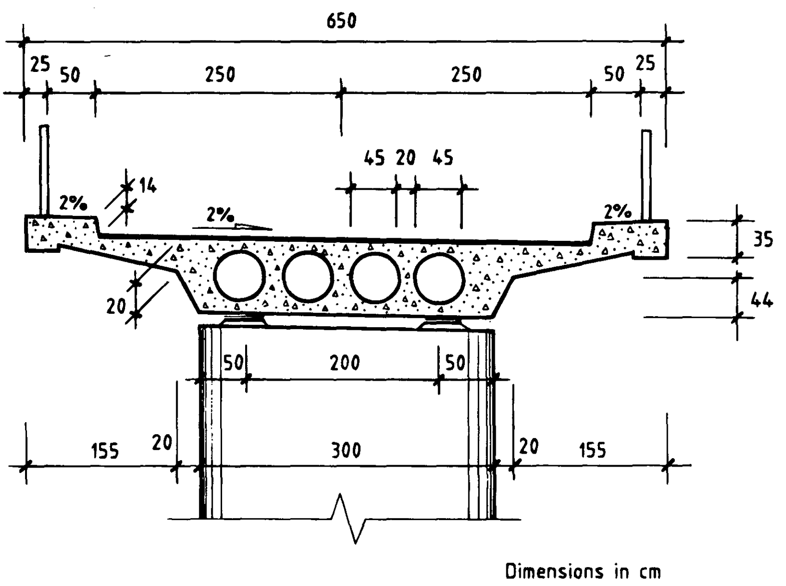
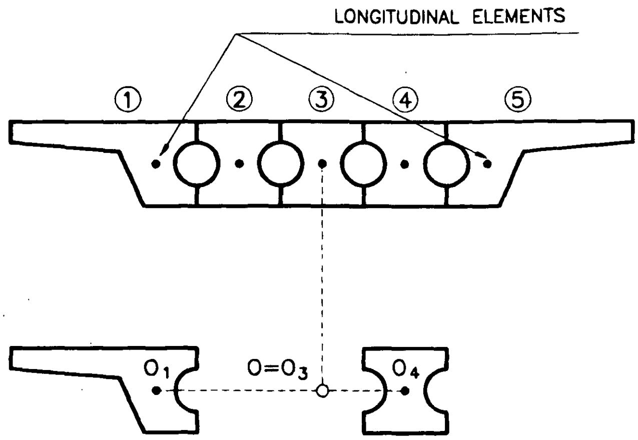


FIGURE 3. Cross section of Bridge OF-56.1.



O = Centroid of overall cross section

O_i = Position of longitudinal beam element

$$I_p = I_0^1 + \sum_{i=2}^{n-1} I_{O_i}^i + I_0^n$$

FIGURE 4. Calculation of rotational mass of longitudinal members in a 2-D grillage.

1. In Phase 1, a single truck with a gross weight of 260 kN crossed the bridge at different velocities and centered with respect to the longitudinal axis of the deck.
2. Phase 2 was similar to Phase 1, but in this case the truck ran eccentrically, resulting in vibrations mainly controlled by the torsion characteristics of the deck.
3. Phase 3 consisted in the simulation of real traffic by crossing two trucks at the same time with different distances and relative velocities between them.
4. The last part of the test was a braking test in which a truck was suddenly stopped over the deck after reaching its maximum speed. In this case the triaxial accelerometer recorded the accelerations excited in the horizontal plane.

As in Model 1 of the previously discussed slab bridges, the model used for the dynamic analysis is the grillage with one longitudinal member (spine model) and transverse elements at the abutments, resulting in 45 nodes and 44 elements (Figure 6). Table 6 shows the correlation between the theoretical and experimental natural frequencies. The agreement in bending modes is good, even for the higher modes. The first torsional frequency presents an error of 1.5 percent, and the second presents an error of 9.6 percent.

TABLE 4. Theoretical Frequencies in Hertz Derived from Different Dynamic Models and Comparison with Experimental Model in Bridge OF-56.1

	Theor.									
		(2A)			(2B)					
Mode	(1)	a	b	c	a	b	c	(3)	(4)	Exp.
f_1(Bending)	6.04	11.29	11.29	11.29	4.98	4.98	4.98			6.09
f_2(Bending)	8.66	31.04	31.04	31.04	19.9	19.9	19.9			8.22
f_3(Bending)	13.5									13.4
f_4(Tor. long)	13.9	14.03	22.25	14.18	11.8	20.1	11.9	27.7	13.84	12.6
f_5(Bending)	14.4									-
f_6(Tor. short)	20.5								20.55	-
f_7(Tor.long)	28.2	29.30	48.10	29.55	25.1	44.3	25.3			-
f_8(Trans.Bend.)		39.31			39.2					-

(-) Not recorded

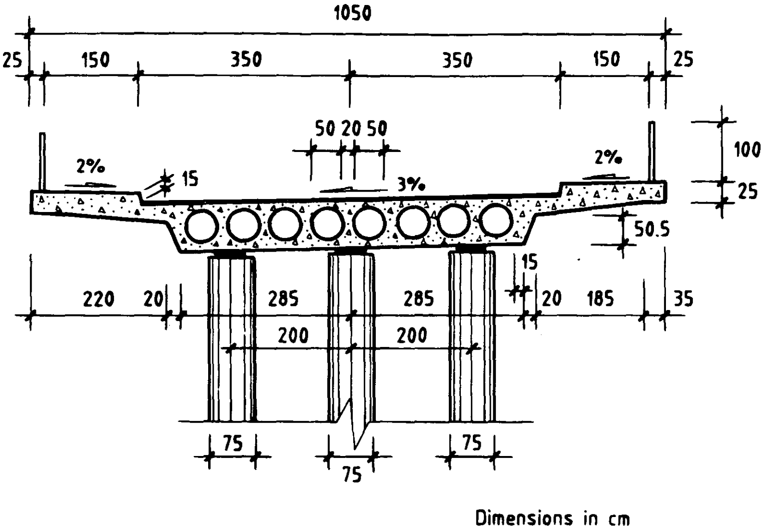


FIGURE 5. Cross section of Bridge OF-49.1.

Cable-Stayed Bridges

The Alamillo Bridge, located in Seville (Spain) over the Guadalquivir River, is a 200-m-span cable-stayed bridge with an inclined tower 130 m high and 13 pairs of cable stays. The deck width (3 × 2 lane of traffic + pedestrian lane) is 32 m, and the main dimensions of the cross section are shown in Figure 7. To compare the dynamic parameters of the constructed bridge with those of the model tested in a wind tunnel, a set of dynamic tests was decided with the following characteristics:

1. Location of a triaxial accelerometer at the top of the pylon to record motions in the three possible directions, and three uniaxial accelerometers to measure vertical accelerations on the deck. One displacement transducer was also installed close to the center span.
2. Excitation was achieved by means of passages of 2 two-axle trucks of 200 kN through the bridge deck at different speeds. Passages were made with and without a standardized obstacle in the pavement. Also the trucks ran symmetrically (one in each traffic direction) in some passages and eccentrically (the two trucks side by side in the same traffic direction) in the rest.

TABLE 5. Theoretical Frequencies in Hertz Derived from Different Dynamic Models and Comparison with Experimental Model in Bridge OF-49.1

	Theor.			
Mode	(1)	(2A)	(2B)	Exp.
f_1(Bending)	5.91	10.31	4.97	5.86
f_2(Bending)	6.39			6.10
f_3(Torsion)	7.47	9.57	8.21	7.60
f_4(Bending)	8.66			8.39
f_5(Torsion)	9.60			9.25
f_6(Trans. B.)		14.13	14.04	-
f_7(Bending)	17.0		16.68	-
f_8(Trans. B.)	17.3	17.50		-

(-) Not recorded

The dynamic model of the bridge was constructed with the following assumptions:

1. One-dimensional beam elements are used for the tower and deck, accounting for geometrical nonlinearity.
2. The bridge geometry is discretized as a plane (2-D model) with only one plane of cables and 3-D motions permitted.
3. The nodes were placed at the points where concentrated mass or inertia mass are present and where cables join the tower or deck.
4. A large number of elements is used to correctly account for the geometric variability. The model has a total of 81 nodes and 63 elements in the deck, 17 in tower and 13 in cables (total of 93 elements).
5. The elastic modulus of the cables was evaluated using the Ernst theory (*9*).

The bridge discretization is presented in Figure 7. Table 7 presents the experimental and theoretical natural frequencies, assuming (nonlinear) or not (linear) the geometric nonlinearity. As shown, neglecting the geometric nonlinearity did not significantly change the results. The experimental frequencies in the transverse direction of the deck were not measured. The observed good agreement validates the simplified theoretical model used. Rayleigh's method was used to evaluate the frequency of the first flexural mode by assuming a concentrated load in the deck to achieve a deflected shape similar to that of the vibration mode. The result was f_B = 0.33 Hz, similar to the measured value. Using Equation 1 for evaluation of torsion in the deck, the result was f_T = 1.03 Hz, which is in good agreement with the experimental value in spite of impor-

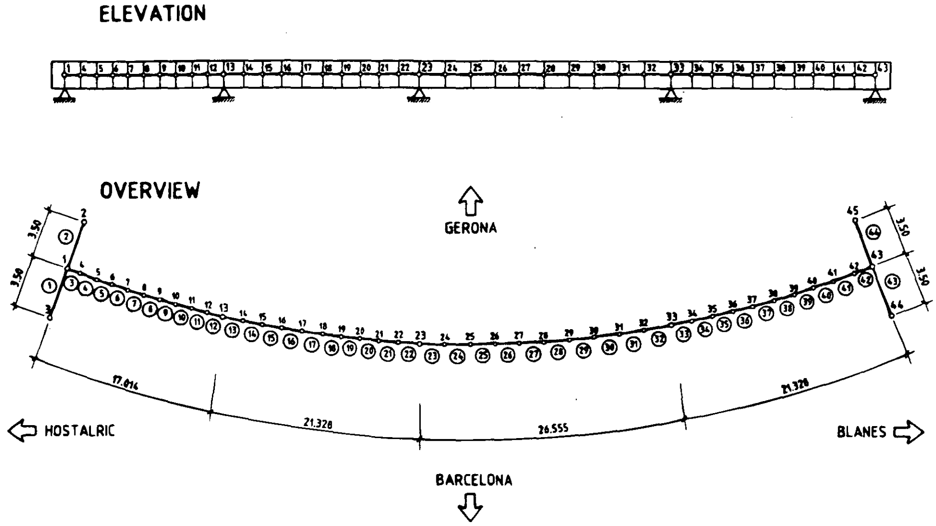


FIGURE 6. Discretization of Bridge OF-50.2.

TABLE 6. Natural Frequencies in Hertz in Bridge OF-50.2

Mode	Theoretical	Experimental
f_1(Torsion)	2.57	2.61
f_2(Bending)	3.98	3.98
f_3(Torsion)	5.00	4.52
f_4(Bending)	5.96	6.09
f_5(Bending)	7.11	7.00
f_6(Torsion)	7.49	-
f_7(Bending)	9.75	9.56

(-) Not recorded

tant variations in the geometry and the structural properties along the length of the deck. This is believed to be because of the lack of interaction between the torsion in the deck and the tower through the cables.

CONCLUSIONS AND RECOMMENDATIONS

The application of very simple dynamic models to the evaluation of the dynamic parameters of bridges is discussed. The observations made indicate that the grillage model, which is widely used for static analysis, can also be applied for the dynamic analysis with few additional calculations. The following recommendations are made concerning the dynamic models presented:

1. When the bridge has a clear 1-D behavior (beam) as in the case of box-girder bridges with one cell and slab bridges with width/span ratio less than 0.6, a model with only one longitudinal fiber reflecting the properties of the overall cross section produces accurate results if the following requirements are adhered to:

   a. A node is placed where important concentrated masses or inertia masses are present.
   b. A sufficiently large number of beam elements is used to divide the longitudinal fiber (see Figure 6) and to properly account for the geometric, inertial, or structural variations along the bridge. If the bridge does not present important variations it is proposed that a minimum of 10 elements per span be used.
   c. In addition to the data necessary for the static analysis, the mass and rotational mass per unit-length are required.

2. When the bridge has a 2-D response (such as slab bridges with a width/span ratio greater than 0.6 or precast girder bridges with upper slab) a grillage with several longitudinal beams is necessary. In this case, also the same static element properties of the static analysis are used. The mass should be located on the longitudinal elements. In addition, the rotational mass should be calculated for interior longitudinal elements with respect to their own centroid and for the exterior elements with respect to the centroid of the global cross section.

3. Rayleigh's method gives accurate results for frequency in bending but inaccurate results in the calculation of the frequency of torsion when using a grillage model with more than one longitudinal fiber with a load pattern to obtain a deflected shape that is similar to the first torsional mode (see Model 3 of Bridge OF-56.1). This is because in the model all the longitudinal members dissipate energy as a result of the torsional rotation and that rotation does not correspond to the real bridge deformation.

4. The analytical equation (Equation 1) gives accurate torsion frequencies when the bridge has more than one bearing device over piers or abutments (ends fixed to torsion), even if small variations of mass or stiffness are present along the longitudinal axis of the bridge.

5. In cable-stayed bridges the geometric nonlinearity can be neglected in the dynamic model of the complete bridge (see Table 7). Also the 1-D model gives good results even for wide bridges.

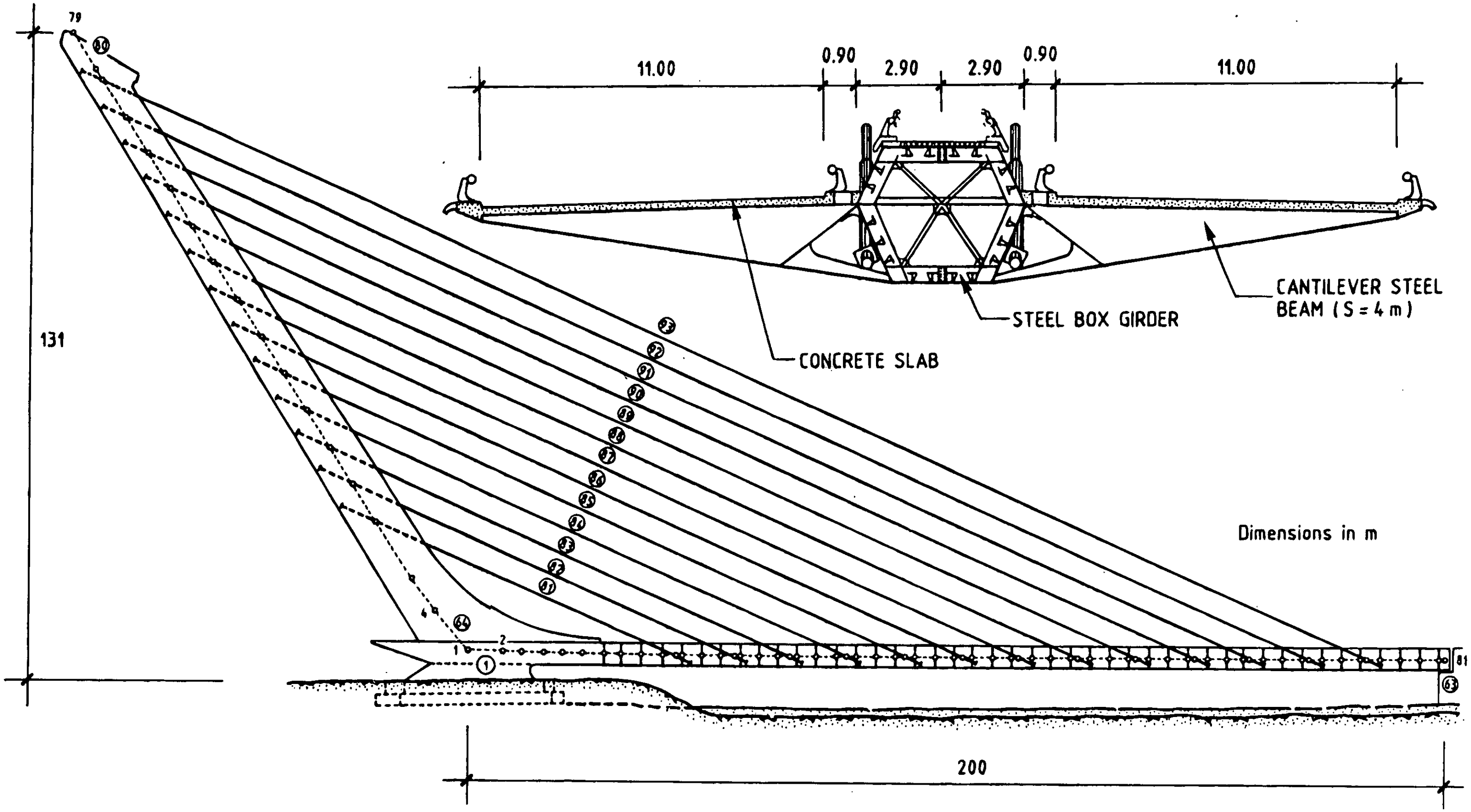


FIGURE 7. Cross section and FEM discretization of Alamillo Bridge.

TABLE 7. Natural Frequencies in Hertz of Alamillo Cable-Stayed Bridge

Mode	Theoretical		
	Linear	Non Linear	Experimental
f_1(Trans. pylon 1)	0.296	0.292	0.30
f_2(Long. pylon+deck 1)	0.375	0.373	0.40
f_3(Long. pylon+deck 2)	0.613	0.610	0.66
f_4(Trans. deck 1)	1.087	1.088	-
f_5(Long. pylon+deck 3)	1.202	1.191	1.205
f_6(Torsion deck 1)	1.219	1.235	1.155
f_7(Trans. pylon 2)	1.587	1.583	1.537
f_8(Long. pylon+deck 4)	2.190	2.196	2.155
f_9(Torsion deck 2)	2.301	2.298	2.295
f_{10}(Long. pylon+deck 5)	2.350	2.312	2.78
f_{11}(Trans. deck 2)	3.251	3.244	-

(-) Not recorded

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