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Load Testing and Numerical Modelling of Quebec Bridges
Essais de charge et modéles numériques de ponts au Québec
Belastungsversuche und ihre numerische Modellierung bei Brücken Quebecs

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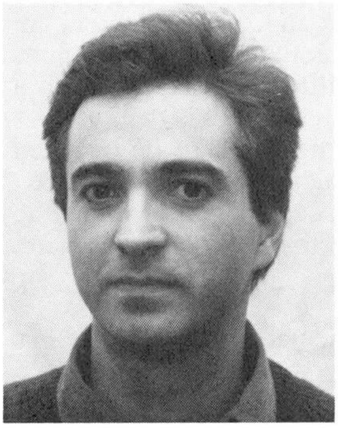

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SUMMARY

This paper presents some of the results concerning the structural evaluation of the Quebec bridges. The approach consists here of two steps. The first one is an experimental step based on the extensive use of a mobile laboratory for static and dynamic measures. The second step is based on the construction and calibration of a three dimensional finite element model. The results which are also compared to standard simplified evaluations show some of the advantages of this kind of more sophisticated evaluations.

RÉSUMÉ

Cet article présente les résultats de l'évaluation de certaines structures de ponts du Québec. L'approche comporte d'abord un volet expérimental effectué à l'aide d'un laboratoire mobile pour les mesures statiques et dynamiques. Puis, un modèle numérique complet en trois dimensions est construit puis calibré sur les résultats expérimentaux. Les résultats obtenus démontrent certains avantages de ce type d'évaluations poussées par rapport aux évaluations standards simplifiées.

ZUSAMMENFASSUNG

Der vorliegende Artikel beschreibt die Bewertungsergebnisse bestimmter Brückentragwerke von Quebec. Dabei handelt es sich zunächst um ein Experiment, das mit Hilfe eines mobilen Labors im Hinblick auf die Erfassung von Statik und Dynamik durchgeführt wurde. Danach wurde ein komplettes dreidimensionales numerisches Modell erstellt, das auf die Ergebnisse des Experiments zugeschnitten ist. Die so erzielten Ergebnisse zeigen deutlich die Vorteile dieser Bewertungsmethode gegenüber einfachen Standardberechnungen.

1. INTRODUCTION

Aging of structures and public budget cuts require more than ever before the exploration of the remaining structural capacity. This is especially true, in many countries, in the field of bridge structures since a major number of these bridges have been build many decades ago. Since then, usual deterioration and fatigue as well as important increase of traffic and loading intensity have been observed. Many aspects concerning the remaining bridge capacity are still unknown. For this reason, the Quebec government has initiated an extensive program of experimental in situ measurements of bridges showing real or potential structural problems.

The paper presents the experimental results as well as the calibration of a related detailed tridimensional finite element model for recent specific bridge cases. The experimental part has been performed here by an efficient mobile laboratory for which strain gauges and accelerometers are available. A special processing of strain data makes it possible to obtain directly the stress resultants (axial force, moments) instead of local stresses. These values are then directly comparable to a simple or more complex finite element model to be calibrated. This calibrated model is then used as the most realistic model for the more complex or more intensive loading cases to be checked according to actual design codes [13]. Both static effects as well as dynamic behavior can be taken into account. interesting results for a first bridge have already shown the scientific as well as the economic benefit of such an approach [1]. New results for two other deck slab steel truss bridges are now available and will be presented in this paper.

The first objective of the paper is to expose the methodology of the experimental approach as well as the calibration of the finite element model for the two bridges. It also shows how much valuable information can be readily available in such a combined approach. Finally, the paper also shows how standard evaluations of existing bridges may sometimes be inadequate in predicting the exact remaining capacity. In fact, in situ measurements and refined modelling can yield in a cost reduction of expected repairs. It also improves the engineer confidence in the making of repair decisions.

2. STANDARD BRIDGE EVALUATION

The standard evaluation practice of old bridges is usually based on methods which are similar to those used for new bridges. Standard linear two dimension displacement models (rigidity method) are normally selected. A distributed factor is used to consider the transverse non symmetrical distribution of the live load. The evaluation is based mainly on the results of an inspection of the bridge which can show structural problems such as rust, cracks and fatigue in members. The assumptions of this analysis are mainly the same as in the original design. One of the main differences lies however in the intensity of the live loads to apply. In fact, truck weights have severely increased for the past decades. That is why, a few years ago, a standard truck specifically developed for the Quebec traffic, the QS660 truck, has been adopted for design or analysis of new and old bridges. This truck has a total weight of 660 kN with a 60 kN, 240 kN, 200 kN, 160 kN distribution from front to end of the truck and with related distances of 4 m, 6 m and 6m respectively and a width of 1.8 m. For instance, for old bridges, like the two bridges presented in this paper, the design live loads were two to three times less than those of the QS660 truck.

For this reason, these live loads are now often critical for the structure. Moreover, the live loads must naturally be increased to keep into account the dynamic effects. This dynamic allowance factor can be related to many factors as shown in the literature [10]. In the Canadian Bridge Code [13], however, the rate of this factor is only related here to the first flexural frequency. A maximum value of 40% increase will be applied for a frequency lying between 2.5 and 4.5 Hz. Otherwise, this value will be at least 20%. For steel truss bridges, this factor is often at its maximum value.

If the evaluation shows an acceptable resistance to the loadings in the Norm, it is considered as being over. If the bridge shows too much weakness, it might be replaced, repaired or limited to lighter vehicles. Sometimes, however, the results of the analysis lies somewhere in a "grey zone". This means that the analyst has a good presumption that the standard evaluation is probably too conservative due to approximations such as the two dimension modelling or as neglecting the rigidity of some components. Two more check steps are then available as described in the next two sections.

3. EXPERIMENTAL EVALUATION

The Ministry of transportation of Quebec acquired, in 1990, a mobile laboratory specifically dedicated to the testing of bridges. Actually, this mobile laboratory has two independent data acquisition systems. One for static tests that includes 60 channels for strain gauges and 20 for other sensors such as LVDT or full-bridge gauges. The dynamic test system embodies 12 channels for accelerometers and 8 channels for strain gauges. As many as 8 pressure tubes can be used to evaluate the total time of the forced vibrations and the averaged speed of the vehicles over the bridge. Up to 50000 samples per second can be recorded from each instruments. The laboratory also includes a micro computer, a color plotter and two printers. A 5000 watts diesel generator is the main power supply of the vehicle. Test trucks used to load and excite the bridge are standard 10 wheel trucks and weight around 260 kN (250 kN for Armagh and 265 kN for Sainte-Marie). Because of the Canadian weather, all tests are performed from late spring to mid fall. It is impossible to give here all the details of the experimental results so that it has been decided to present specific results for each of the two bridges. The Armagh bridge has been selected to present the static approach while the the Sainte-Marie bridge will show the dynamic results (see the corresponding subsections 6.1 and 6.2).

4. FINITE ELEMENT EVALUATION

Once the in-situ tests have been performed and the experimental values postprocessed, a more refined numerical model is build. A commercial program GIFTS (CASA) has been used here on a IBM PC 386. Models are kept moderately large with about 1000 degrees of freedom. The model is kept linear and truss, beam, spring and quadrilateral shell elements are used. A diagonal mass matrix has been selected for computing the eigenmodes via the subspace iteration algorithm. The basic assumption here is that experimental results make the best information available about the structure in spite of normal experimental inaccuracies. Since a first finite element model never fits exactly experimental values, this one has to be calibrated by an iterative process in order to get closer to the experimental data. Once the model has been locally calibrated (local forces in specific member) or more globally calibrated (via dynamic modal values), this one is believed to be adequate in all its tested and non tested members.

There is no standard recipe or standard steps for a successful calibration. The choice of the components to be calibrated depends on many factors like the kind of structures (steel or concrete, for instance) and the results of the in-situ inspection (actual state of some components). For the kind of bridges which are studied in this paper two main factors were selected. The longitudinal restraints of the supports have been modified to take into account the partial restraint of the theoretically free supports. Spring elements have been tested to model this partial restraint. Some artificial beam elements have also been used to link the deck to the trusses in order to modulate the participation of this deck. This appeared to be the most important step. To a lower extent, we also checked the influence of the Young modulus of concrete. A slight dynamic calibration of the mass of the deck (pavement thickness) has also been tested. Finally, once the model has been calibrated, we assume a linear behavior of the structure.Loadings according to the Canadian Code are then performed with strictly localized QS660 trucks and maxima efforts are extracted. These efforts are considered to be the "real" efforts that would appear in the structure in its actual state.

5. DESCRIPTION OF THE TWO BRIDGES

5.1 The Armagh Bridge

The Armagh bridge is a deck slab steel truss bridge which is located on the small river near Saint-Cajetan d'Armagh (Fig. 1). It was build in 1931 and is 51.2m long and 7.3m large. This average size structure has three spans which are 11.9m, 27.4m, 11.9m long with the central one with steel trusses. The thickness of the concrete slab is 0.178m and is 0.065m for the pavement and no specific composite action is present (in theory only). For this structure, steel and concrete Young modulus are 200000 MPa and 250 Mpa. The total number of elements for the standard 2-dimensional truss model is 33. Practically all structural components have been discretized for the 3-dimensional model. These include trusses, stringers, floor beams, the bracings and the concrete slab (Fig. 3). The total number of elements of all kinds is 415 while the total number of degrees of freedom is 976.

5.2 The Sainte-Marie Bridge

The Sainte-Marie bridge is also a deck slab steel truss bridge which is located on the rather large "Chaudière" river in the Sainte-Marie Town (Fig. 2). It is an important bridge in the area. It was build in 1918 and is 130m long and 6.6m large. This structure has two 65m identical spans. Only one steel truss span has been studied. The thickness of the concrete slab is 0.178m and is 0.065m for the pavement and no specific composite action is present in theory. The total number of elements for the standard 2-dimensional truss model is 54. Practically all components have been discretized for the 3-dimensional model that is trusses, stringers, floor beams, the bracings and the slab (Fig. 4). The total number of elements is 523 while the total number of degrees of freedom is 1246.

6. RESULTS

6.1 The Armagh Bridge

As mentioned previously, the Armagh bridge has been specifically selected here to present the static results. Six steel members have been instrumented with a total of 40 strain gauges. These members are those identified with numbers on Fig. 1. Each member had at least 3 or 4 gauges in order to determine all internal efforts developed in the member (axial force N; bending moments M_x and M_y; warping moment B). Considering that the recorded deformation is the sum of the deformation produced by each effort, we have [15]:

$$\varepsilon_i = \frac{N}{EA} + \frac{M_x}{EI_x} y_i + \frac{M_y}{EI_y} x_i + \frac{B}{EI_\omega} \omega_i \qquad (1)$$

where E is the Young modulus, A is the cross-section area, I_x and I_y are the moment of inertia with respect to the x and y axis, x_i, y_i and w_i are the x, y and sectorial coordinates of the strain gauge. Having a similar equation for each strain gauge i of a member, this sets up a system of equations easy to resolve for the four unknown N, M_x, M_y and M_ω.

The tests took place on August 1991, during which 8 load paths from A to H (from one sidewalk to the other) have been followed along the bridge. For each path, 12 positions of the 27 tonne 2 axle truck have been occupied (see Fig 1). The truck positions were selected as a function of the second row of wheels being placed directly over each vertical member of the truss. Since a simultaneous calibration of all tested members at any truck position was not possible in practice, the direction of the calibration was dictated by the values of a very few selected members for selected truck positions. Once these values had a satisfactory convergence towards the experimental results, graphics of the axial force in all members as a function of truck positions were plotted. Two examples of these results are shown in Fig. 5 and 6. The Fig. 5 shows the best results while the Fig. 6 shows the worst one which are actually rather good too. The precision of all other results lie between these two cases. Moreover, some dynamic data have been obtained from a few gauges and one accelerometer. The FFT gave us the first flexural frequency which value was 4.10 Hz. As a final check, we computed this value for the calibrated model and obtained 3.83 Hz which is quite satisfactory.

The calibrated model was then ready for its maximum loading according to the Canadian Code. The maximum forces induced by two QS660 trucks (one at 100% and one at 70%) were computed. A significant reduction of 55% to 170% in the load effects was observed compared to the standard analysis for the horizontal top and bottom chords, reducing by almost the half the number of these members to be replaced. Diagonal members improved slightly but appeared to remain weak and will have to be replaced. The vertical members at both extremities of the steel truss appeared to be more loaded in the refined model (by 34%). This shows an interesting example in which the standard approach is here less conservative and then less secure than its more refined counterpart.

6.2 The Sainte-Marie Bridge

The dynamic tests of the Ste-Marie bridge were performed in April 1992. For these tests, 8 low frequency accelerometers have been used, 3 on both sides of the deck for vertical movements measured and 2 for horizontal movements at the abutment and at mid-span, as shown in Fig. 2. One pressure tube was placed at each end of the span. A total of 10 runs have been made at different speeds along the center of the roadway or on the right lane side. The maximum speed was limited to 40 km/h because of the very steep hill at the entry of the bridge.

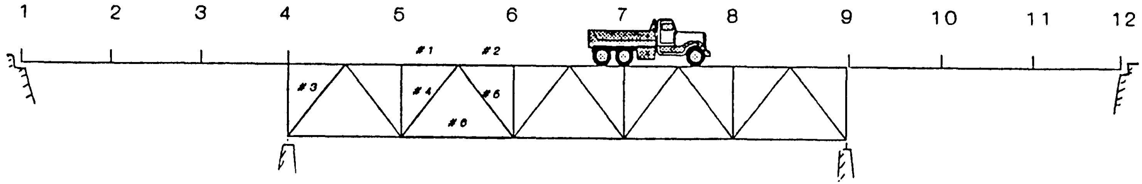

Fig. 1 Armagh Bridge

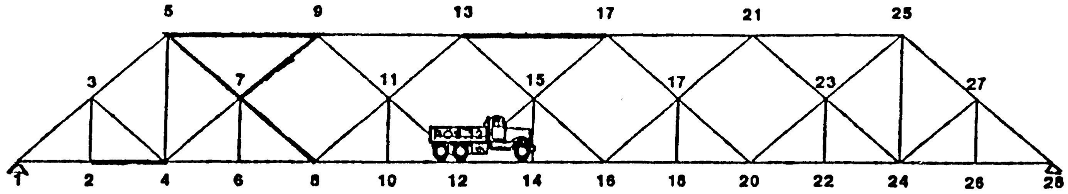

Fig. 2 Sainte-Marie Bridge

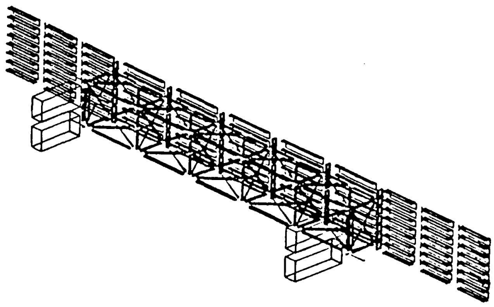

Fig. 3 Numerical Armagh Bridge model

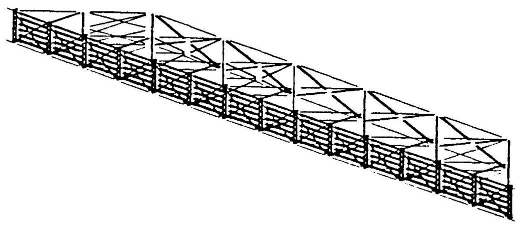

Fig. 4 Numerical Sainte-Marie Bridge model

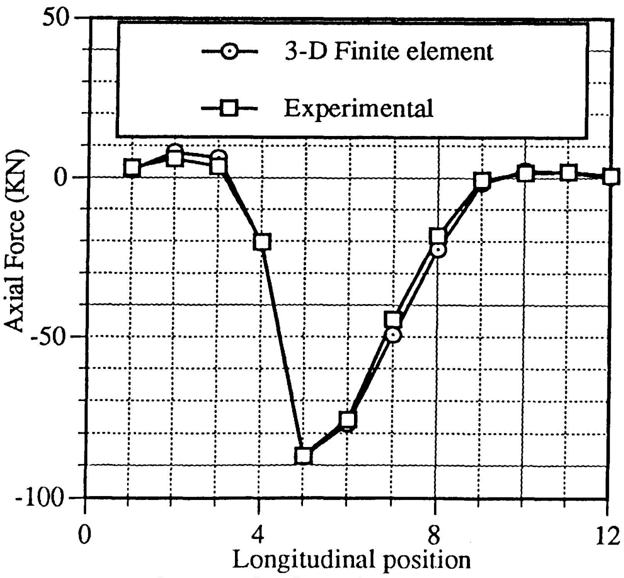

Fig. 5 Axial force in member # 3

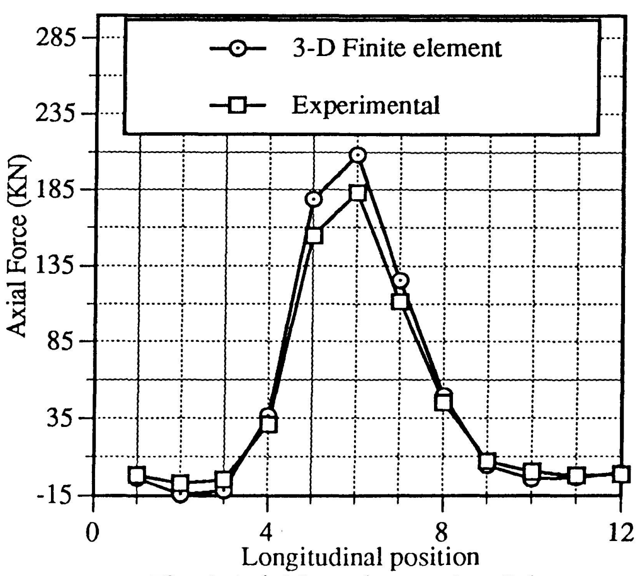

Fig. 6 Axial force in member # 6

In order to evaluate the frequencies of vibration and the corresponding mode shapes included in the response of a bridge under traffic loading, the response of one accelerometer placed away from eventual nodal points is considered as the excitation in the modal analysis. The five other accelerometers measured the outputs. This approach is preferred since the excitation force, which is the variation of the load intensity with time, is not measured.

The computation of the FFT of these 6 signals displays how the energy is distributed in the frequency domain. High peaks usually correspond to the bridge frequencies of vibration. However, during the forced vibration, the frequencies caused by the bridge-vehicle interaction may be included in the spectra [2]. To avoid this potential confusion, the coherence between the excitation signal and the outputs was also computed. The coherence indicates the degree of linearity between the input and the output [10], [6], [7]. Since the amplitude and phase of all points follow a fixed relationship when the bridge vibrates in one of its modes, the coherence should be over 0,9 at mode frequencies of the bridge, and low at other frequencies such as the frequencies of the bridge-vehicle interaction [12]. One may also compute the FFT of each signal for each runs and take the average FFT in the subsequent calculations. This may eliminate noise and some irrelevant peaks. To identify experimental mode shapes, the amplitude of the imaginary part of the transfer function FRF and the phase of the same function give the relative displacements and phases of other accelerometers relative to the reference accelerometer at all frequencies [10]. The procedure described above has been followed for the 8 dynamic runs and the results for accelerometer 2 are presented in Fig. 7. The experimental operating mode shapes are also presented in Fig. 8. One should note a lack of symmetry of three mode shapes, probably due to the short duration of each free vibration records.

The static calibration technique was exactly the same as for the Armagh bridge. Then, a final tuning of model was imposed in order to get closer to the first flexural frequency. A slight modification of the mass of the slab (for pavement) improved the values of the four eigenvalues. These numerical eigenmodes are shown in Fig. 8. A good correlation is observed between the experimental and the numerical values. The model showed however that the experience failed to give some higher flexural modes with lower values than the fourth one but this was due to the small number of accelerometers.

7. CONCLUSIONS

7.1 Summary of the results

In this paper, we presented practical results for two old bridges. The paper showed how experimental results from a mobile laboratory can help to build a well calibrated numerical model. Once calibrated, this model can then give realistic structural values in any part of the structures. The postprocessing of the experimental dynamic data can give the eigenvalues as well as the eigenmodes which can be compared to the numerical model. These values show how standard evaluations are sometimes inadequate. They are often too conservative and, in this case, the combined experimental-numerical approach can save money. They are sometimes (more rarely) not enough conservative and in this case are less secure. Consequently, this combined experimental-numerical approach is certainly going to become a must for the analysis of complex or doubtful aging structures. Final decisions could then be made with a high degree of confidence, allowing engineers to make the proper economic and safety decisions.

7.2 Future considerations

It should be noted that since complete tests for a bridge may request between one to two weeks, the testing program is usually limited to about 10 bridges per season. This kind of program should remain a current practice for the next few years. After this, rapid testing (one or two days per bridge) based only on dynamical excitations might be sufficient for typical or simple bridges. Calibration of the finite element method would then be based only on the dynamic tests. This would avoid the very time consuming strain gauges set up and would partially reduce the weather limitations of these gauges. It would also require the systematic use of the modal synthesis techniques. For the moment, our limited number of accelerometers could still be a problem for more complex structures. For some evaluations based on precise investigations of stress components such as the axial force of certain members, strain gauges will remain an essential tool.

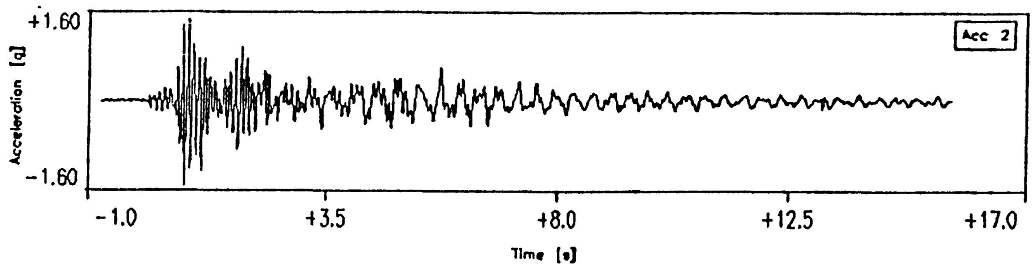

Fig. 7 Accelerations of accelerometer 2

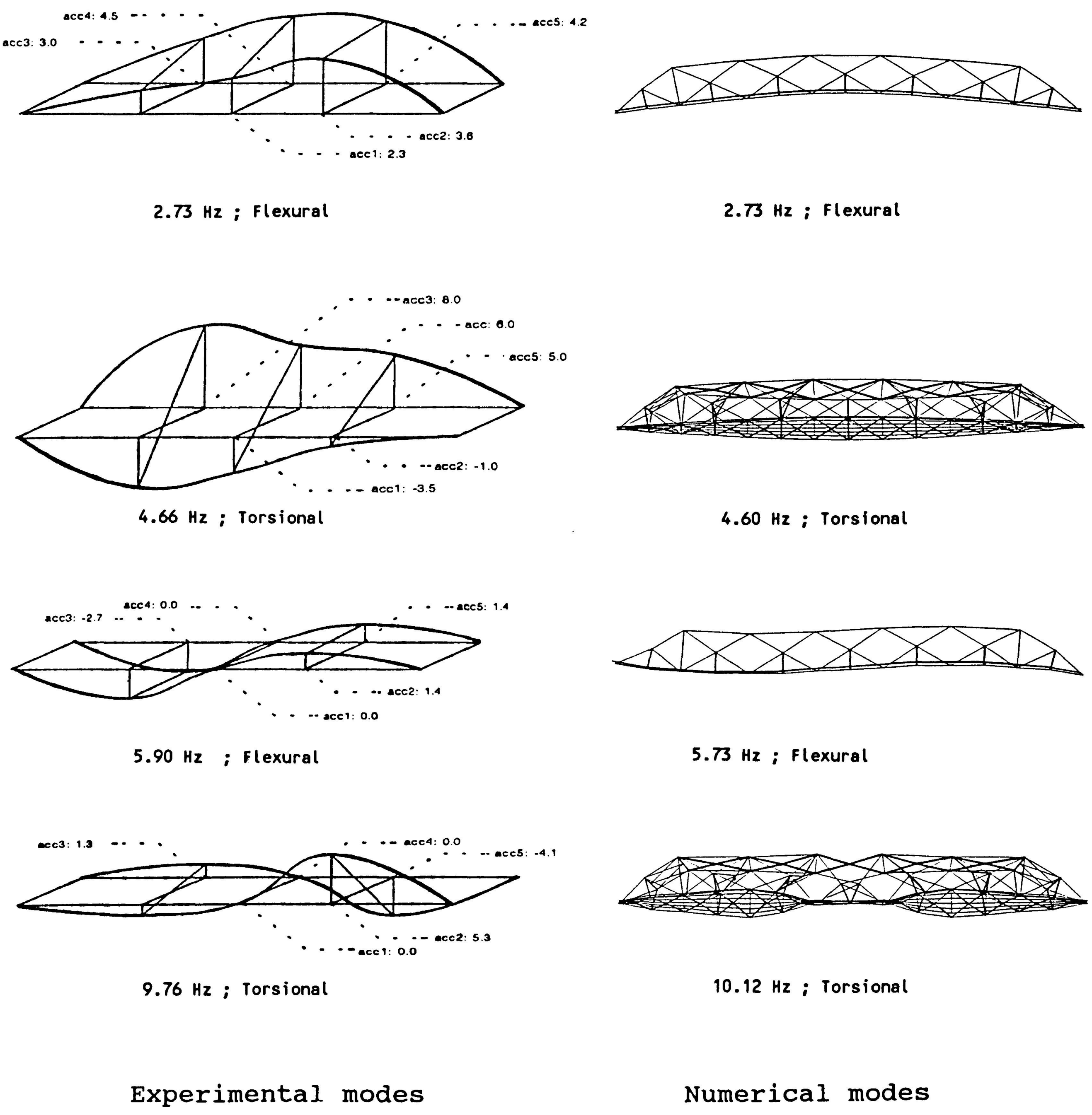

Fig. 8 Experimental and numerical vibration modes

On a more general basis, many aspects concerning old bridge structures are still to be clarified, however. This is why an important scientific collaboration has been settled in order to improve our knowledge of some complex experimental aspects of different kinds of old bridge structures as well as some specific bridge components. This has already led to many interesting results such as the experimental determination of the dynamic amplification factor [6],[7],[10], the ultimate capacity of noncomposite concrete slab on steel girder bridge [8] or the in-situ study of a prestressed bridge [11]. This collaboration has also led, much in the same way, to many improvements and developments of the "domestic" finite element tools to be used soon for bridge modelling. Fundamental algorithmic aspects for dynamic analysis [3] and new thick/thin shell elements for concrete structures [5],[14] have been developed as well as non linear analysis of noncomposite effects in slab [9]. A finite element program written in C (called CLE) is under development for direct bridge-structure interactions [2]. All these developments will be necessary to yield in a better understanding of the behavior of the increasing number of old bridge structures.

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