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Mechanical vibration — Evaluation of measurement results from dynamic tests and investigations on bridges

Vibrations mécaniques — Évaluation des résultats de mesures relatives aux essais dynamiques et aux investigations sur les ponts

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

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ISO 18649 was prepared by Technical Committee ISO/TC 108, *Mechanical vibration and shock*, Subcommittee SC 2, *Measurement and evaluation of mechanical vibration and shock as applied to machines, vehicles and structures*.

INTERNATIONAL STANDARD **ISO 18649:2004(E)**

Mechanical vibration — Evaluation of measurement results from dynamic tests and investigations on bridges

1 Scope

This International Standard provides methodology for the evaluation of results from dynamic tests and investigations on bridges and viaducts. It complements the procedure for conducting the tests as given in ISO 14963 and considers

- the objectives of the dynamic tests,
- the techniques for data analysis and system identification,
- the modelling of the bridge, and
- evaluation of the measured data.

NOTE 1 The evaluation may seek to define all of the dynamic characteristics of each mode of vibration examined, i.e. frequency, stiffness, mode shape and damping, and their non-linear variation with amplitude of motion. These can supply information on the dynamic characteristics of a structure for comparison with those assumed in design, or as a basis for condition monitoring or system identification.

The dynamic tests considered in this International Standard do not replace static tests.

This International Standard gives guidance on the assessment of measurements carried out over the life cycle of the bridge. The stages of the life cycle that are considered are

a) during construction and prior to commissioning,

b) during commissioning trials,

c) during specified periods throughout the life of the bridge, and

d) immediately prior to decommissioning the bridge.

This International Standard is applicable to road, rail and pedestrian bridges and viaducts (both during construction and operation) and also to other works, provided that they justify its application. The application of this International Standard to special structures (cable-stayed or suspension bridges) requires specific tests that take into account the particular characteristics of the work.

NOTE 2 Throughout this International Standard, “bridges and viaducts” are called “bridges”. The term “viaduct” is used only when it is necessary to distinguish between these.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 2041, *Vibration and shock — Vocabulary*

ISO 14963, *Mechanical vibration and shock — Guidelines for dynamic tests and investigations on bridges and viaducts*

ISO 14964, *Mechanical vibration and shock — Vibration of stationary structures — Specific requirements for quality management in measurement and evaluation of vibration*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 2041 and the following apply.

3.1
buildability
property of a structure that enables construction to proceed in a safe, timely and economic fashion

NOTE The buildability of bridges may require construction to proceed in a strong wind, so wind effects on vibration may need to be monitored.

3.2
environmental compatibility
environmental impact on a new bridge, which may need to be evaluated, involving wind effects, air noise and ground vibration

3.3
serviceability
limit state beyond which a structure no longer satisfies the operating requirements such that it is no longer fit for purpose

3.4
monitoring
programme of measurements, usually over a period of time, whereby changes in an appropriate parameter may be interpreted as indicating a change in the state of the structure

NOTE It is important to establish a benchmark and allow for changes attributable to cyclic environmental factors such as diurnal or seasonal changes of temperature and humidity.

3.5
running safety
property whereby traffic crossing a bridge at an appropriate speed is not deleteriously affected in maintaining direction or stability

3.6
riding quality
property whereby occupants of vehicles crossing a bridge at appropriate speed are not exposed to such levels of vibration as to adversely affect their comfort

4 Vibration measurement

4.1 General considerations

The guidelines for vibration measurements as given in ISO 14963 shall be observed and the quality requirements for these measurements as given in ISO 14964 shall be fulfilled. Measurements may be carried out on bridges under construction and in commissioning and on bridges in service.

4.2 Monitoring of a bridge during construction and for commissioning

4.2.1 Objectives of vibration monitoring

Figures 1 and 2 illustrate the relationships between the various stages involved in vibration monitoring.

The objectives of vibration monitoring shall be specified as follows:

a) evaluation of the accuracy and buildability of construction;

b) evaluation of structural performance during construction and upon completion;

c) assessment of the safety of the bridge during construction and upon completion;

d) evaluation of serviceability upon completion;

e) evaluation of environmental compatibility;

f) determination of the initial characteristics of vibration for maintenance and for calibration of the numerical model of the bridge in service;

g) feedback to structural design.

Uncertainty of results in each process of measurement and evaluation cannot be avoided and there is a possibility to include uncertainty as shown in Figure 1. Therefore, reduction and qualification of measurement uncertainty and error are needed in the process.

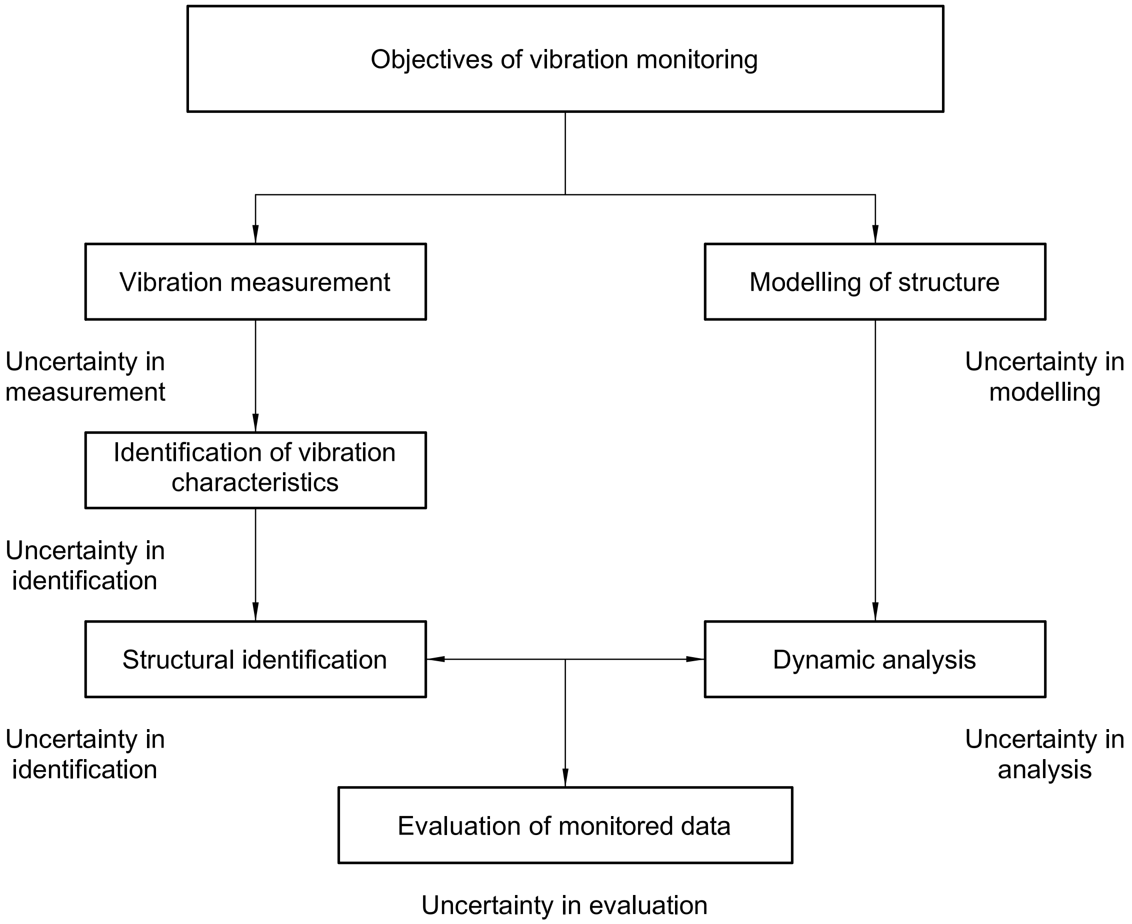

Figure 1 — Flowchart of vibration monitoring of a bridge

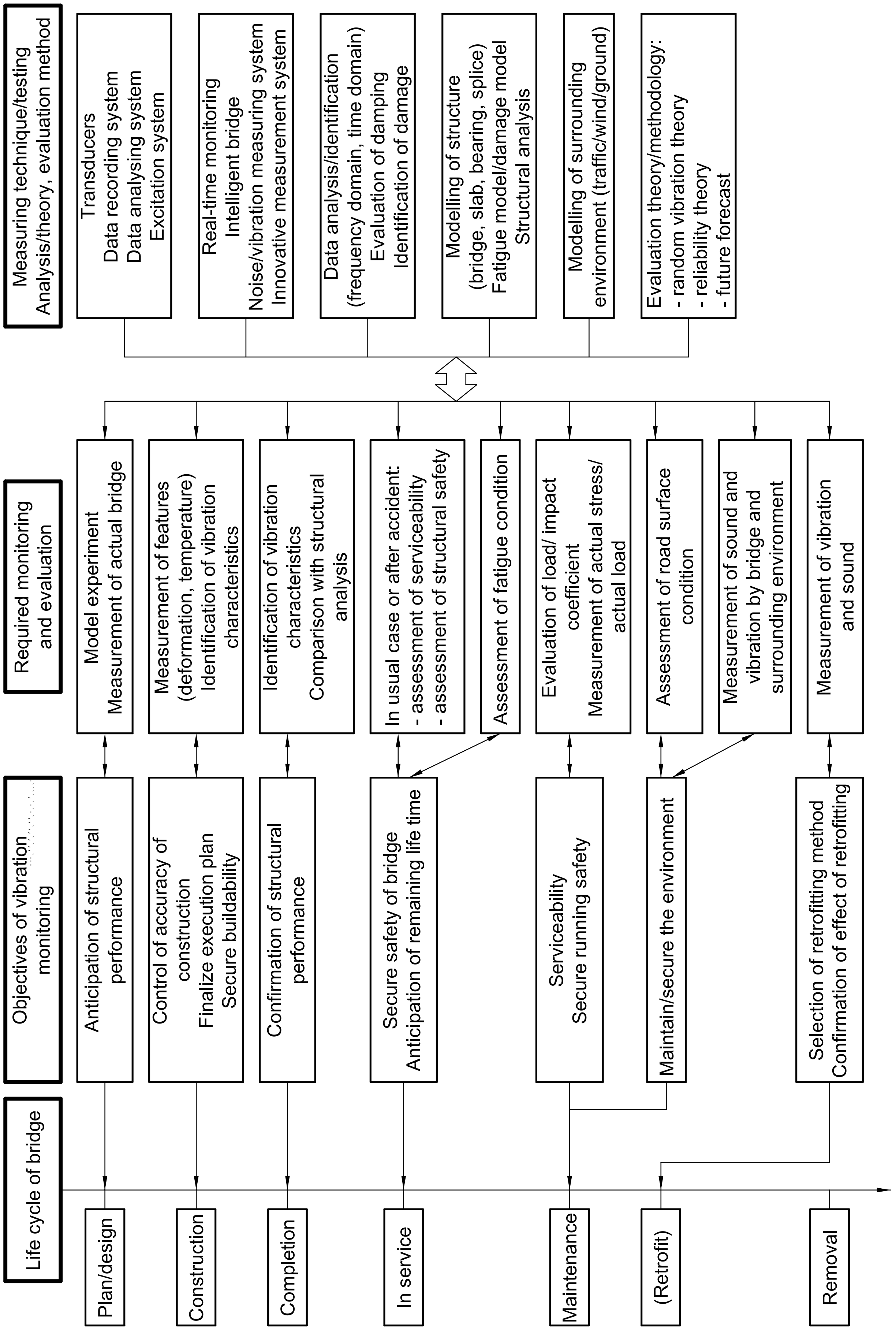

Figure 2 — Overview of vibration monitoring of a bridge

4.2.2 Evaluation of construction management

4.2.2.1 General

Vibration measurements on bridges may be conducted during construction. For example, vibration tests on cables of cable-stayed bridges or suspension bridges are used to control the tension of the cables. In order to control the profile of the bridge under construction, measurement of the vibration of cables is required. Dynamic measurements may also provide an indication of when high vibration levels will have an adverse effect on construction.

4.2.2.2 Evaluation of cable tension

Dynamic characteristics are greatly influenced by the support conditions. Cable tension of a cable-stayed or suspension bridge is one of the main parameters for construction management. Vibration of cables is easily measured for the determination of the natural frequency of transverse vibration. This depends upon cable tension and is given by a well-known equation. In this case, the numerical model will need to consider bending rigidity and the end support of the cables.

4.2.2.3 Evaluation of buildability of construction

Vibration measurements can provide the required information to determine when construction work is either unsafe or the quality control is likely to be adversely affected. If the bridge vibration and wind and earthquake excitation are continuously monitored, the decision can be made when an allowable limit is exceeded.

4.2.3 Characteristics for the evaluation of structural performance

4.2.3.1 General

The natural frequency, damping and dynamic response of the structure and the surrounding area and sound propagation from/through structure are measurable characteristics which can be used for the evaluation of structural performance.

4.2.3.2 Natural frequencies and mode shapes

The natural frequency and its mode shape are easy parameters to measure. The support conditions and the temperature of the structure are major factors influencing natural vibration; hence they should be monitored before and after construction. Geometrical non-linearity of flexible bridges and material non-linearity of superstructures on the substructure are aspects that should be considered. These aspects are as follows:

- natural frequencies;
- modal shapes;
- movements of shoe and boundary conditions of structures;
- geometrical non-linearity effects of the structure;
- material non-linearity of the ground;
- effects of isolator and vibration control devices;
- effects of temperature.

NOTE Isolator and control devices to reduce vibration can also introduce non-linearities.

4.2.3.3 Damping

The damping coefficient, or logarithmic damping ratio, can also be measured. The measurement of damped free vibration produced by stopping the forced vibration provides a direct measurement of damping characteristics, at least for the fundamental mode. Amplitude and temperature dependencies are important factors for damping measurement. It may be necessary to consider the effects of support condition and isolation devices. When damping characteristics are required for large-amplitude motion, forced vibration tests that generate high-amplitude vibration are appropriate. Evaluation for strong earthquakes or wind may require damping values for large-amplitude motion.

Elements affecting the damping characteristics of bridges are as follows:

- aerodynamic and hydrodynamic effects;
- connections and joints;
- bearings and shoes;
- pavement (rheology of materials);
- effects of substructures;
- effects of the foundation.

The amplitude dependencies of frequencies and damping characteristics of bridges require careful analysis of the data. Different damping characteristics will be provided by different structural types and in different locations, so the overall damping effect is the integral of these elements.

4.2.3.4 Characteristics of the dynamic responses of a structure with the surrounding media

The measurement of dynamic response may involve strain, acceleration, velocity or displacement. It is also important to consider boundary conditions. The results from ambient vibration tests or impact tests may not be appropriate for some dynamic response evaluations because of the small amplitude of the loading. Using forced-excitation tests, resonance response curves can provide data for larger amplitude motion. Accurate analysis of ambient vibration for small amplitudes may be suitable for the structural health monitoring of bridges. Tests using moving vehicles can give the dynamic response related to the speed and pattern of the vehicles. Fatigue analysis requires dynamic response as a stress range histogram. The points to be considered are as follows:

- accuracy of the ambient vibration analysis;
- impact test for the dynamic property of the surrounding media;
- effects of water or tidal flow;
- excitation method.

4.2.3.5 Sound radiation around/through the structure

Microphones placed on the surrounding ground can detect the sound radiation from bridges due to moving vehicles. Characteristics of sound propagation are used to evaluate the environmental effects on the surrounding area. Parameters to be measured are as follows:

- sound level;
- sound frequency;
- traffic density;

- traffic speed;
- types of vehicles;
- impulsive effects;
- roughness of road/track surface;
- ground stiffness and its interaction with the substructure.

4.2.4 Assessment of safety during construction and upon completion

4.2.4.1 Confirmation of design for earthquake performance

Vibration monitoring is needed for safe construction in a highly seismic area. Depending on the data, engineers can assess the risk during construction, and this may influence the construction. Data on vibration under severe loading conditions are important. The assessment is based on

- characteristics of natural vibration and its damping,
- dynamic response characteristics,
- reinforcement of the structure,
- isolation system on the bridge, and
- diagnosis of structural health after a disaster.

In the design process for earthquake performance, the numerical model for dynamic response should be constructed by a combination of the total/part of the superstructure used in static design and the substructure including the basement and surrounding ground. These data should be utilized in the evaluation analysis.

Measurement of the natural vibration of the substructure after its construction and the non-linear vibration properties of the ground should be taken into account. Evaluation of the damping characteristics is accomplished by comparison of the measured data with the assumed values used in the design process. Support condition and amplitude dependency should also be taken into account. The effect of temporary structures and the pavement on vibration properties should also be considered.

4.2.4.2 Confirmation of design for wind

The dynamic response for wind can be measured and compared with assumed values. Assumed values may be obtained through experiments in a wind tunnel as a part of structural design process. Measured data can include the effects of the velocity and direction of the wind and its amplitude dependency. After analysing all these effects, damping devices may be considered.

4.2.4.3 Confirmation of fatigue design

Fatigue design considers the dynamic stress range of members and the number of cycles encountered. In this case, the stress range is given by the sum of the static stress and the coupling effect with a moving vehicle. Monitored data for actual stress should be compared with the assumed values used in fatigue design. The dynamic amplification factor to amplify the static stress range is used and it depends on the road/track profile and travelling pattern of the traffic load. The coupling effects with vehicles are needed to monitor the structural health of the bridge. Non-stationary vibration due to irregular undulation of the surface of the track and road can be important.

4.2.5 Serviceability of a completed bridge

The vibration perception of pedestrians, vibration effects on moving vehicles, and the comfort of passengers are part of potential serviceability problems. Vibration monitoring is undertaken to evaluate these effects and the design should be checked and necessary measures should be considered.

In the evaluation of the vibration perception of pedestrians, the amplitude of the dynamic response as well as the frequency of vibration should be considered. In the evaluation of the effects on moving vehicles and the comfort of the passengers, the amplitude of the dynamic response on the floors and wheel axles of the vehicle should be taken into account.

In the comparison between measured data and numerical results from modelling of the moving loads, coupling vibration effects should be taken into account. Bridge vibration due to moving vehicles and the comfort of passengers are also problems that should be considered for serviceability.

4.2.6 Evaluation of environmental compatibility of a completed bridge

Environmental vibration, noise and change of wind direction should be taken into account in the evaluation of environmental compatibility. Monitored data are used to analyse these effects and are compared with the dynamic characteristics of the structure. Necessary modifications may be required depending on the results. Numerical simulation of the propagation of ground vibration and sound radiation may be used to identify the level of those effects.

4.2.7 Determination of the initial vibration characteristics of a completed bridge

Long-term monitoring will start after construction and the initial values of vibration characteristics are required to monitor changes in parameters due to deterioration or damage. The effects of deterioration or damage on the vibration characteristics are generally small, so an effective method to extract the required information about damage should be used. Local excitation and the application of beating phenomenon due to those small differences of modal parameters are useful methods.

4.2.8 Feedback to overall performance

Data given through the above-mentioned evaluation should be fed back to design engineers to apply to future designs. Classification of the data is also helpful when the data are used in the future for the design of all types of bridges.

4.3 Monitoring of a bridge in service

The objectives of vibration monitoring of a bridge in service are

- evaluation of the travelling load,
- evaluation of the structural performance,
- evaluation of wind effects and hydrodynamics,
- assessment of safety,
- assessment of serviceability, and
- assessment of environmental compatibility.

Normal and emergency monitoring of bridge vibration are used depending on the maintenance management of the bridge. Detailed analysis to identify the damage and defects is needed. Traffic conditions and the roughness of road and rail surfaces, and wind and hydrodynamic effects will have a significant impact on the fatigue stress. Dynamic effects should be monitored through measurement.

5 Data analysis and method of structural identification

5.1 General

Structural identification aims to provide a good correlation between the numerical model of the bridge and experimental measurements. From the model, which may be based on finite elements, the modal parameters are identified, primarily the modal frequencies and associated mode shapes. The same parameters can be identified by experiments, which should also determine the damping value for each mode. The identification methods may adopt either time-domain or frequency-domain procedures depending on the problem.

5.2 Data analysis and domain

Data analysis may be conducted in the time domain or the frequency domain, or both domains. Depending on the problem, the engineer should decide which to use. In Clause A.1 the relationship between the time and frequency domains is shown.

In the analysis of vibration data, statistical distributions of stress, velocity, acceleration and displacement are obtained. The distributions of stress and displacement are used for evaluation of the condition of bridges.

5.3 Digitizing

Data obtained from experiments are usually digitized from the analog signal using an analog-to-digital (A/D) converter. The selection of the sampling frequency or time step for the digitization procedure is very important, and care should be taken in order to retain the required accuracy from the analog data. The following main problems should be considered.

a) **Error in A/D transformation**

The sampling frequency of the A/D converter should be carefully specified and a resolution consistent with the objective of the targeted natural frequency is recommended.

b) **Misreading of peak values**

Both in the time domain and the frequency domain, there is a possibility that the true peak values may not be identified during digitization. Hence the shape of the transfer function as well as peak values should be considered in the identification of frequency and damping when using the half-power bandwidth method.

c) **Resolution of FFT**

In Fourier transformation of the digitized data, it is impossible to get higher frequencies than those specified by the time interval of digitized data, Δt. This limiting frequency is called the Nyquist frequency (see [15]) and is expressed as $f_{\mathrm{N}} = 1/(2\Delta t)$. The frequency resolution Δf is given by the inverse of the total time T as $\Delta f = 1/T$.

5.4 Identification of vibration characteristics in the time domain

5.4.1 General

The natural frequency, modal shape and damping coefficient should be identified when determining the vibration characteristics of a system. It is recommended that the damping coefficient be identified in the time domain. If the non-linearity and amplitude dependency are significant, analysis should be performed in the time domain.

In the time domain, ideally one mode should be considered at a time, which may require the data to be extracted from the measured data by filtering. There are situations where closely spaced frequencies exist, depending upon the type of structure and the constitution of members. Here, it will be difficult to extract data for one mode and the identification method for closely spaced modes system shall be used.

5.4.2 Extraction of single natural frequency component

Measured data generally include many vibration modes and it is difficult to identify vibration characteristics accurately in the time domain. Ideally single-mode data should be extracted from the measured data (see Clause B.1). The frequency may be identified by the following:

- transformation of the measured data to the frequency domain;
- filtering to extract single-mode data;
- inverse transformation from the frequency domain to the time domain.

Low-pass, high-pass and band-pass methods may be used for filtering data, depending on the circumstances.

5.4.3 Natural frequency

Identification of the natural frequency of a single mode from the time history of the signal may consider the time period between

- the peak responses, and
- zero-crossings.

If the bridge has isolation devices or the support has amplitude dependency, the natural frequency will usually vary with the amplitude of motion. In this case, extracted single-mode data should be inverse transformed to the time domain to compare these vibration amplitudes with measured data.

5.4.4 Natural frequency mode

If the damping is small and the system is considered as a proportionally damped system, the relative displacement vector of the natural frequency mode is constant and not dependent on time. In this case, the frequency mode should be obtained by plotting the relative amplitude as normalized values. If the bridge has large damping devices, the vibration characteristics will exhibit non-proportional damping. In this case, the measured mode has a phase shift and the modal shape changes even in one period of vibration. It is difficult to identify this in the time domain.

5.4.5 Damping

The damping coefficient ζ is identified by the natural vibration data of a single mode in the time domain (see Figure 3). The logarithmic decrement δ is obtained by

$$\delta = \ln \frac{x_{i-1}}{x_i} \tag{1}$$

and the damping coefficient is given by

$$\zeta = \frac{\delta}{2\pi} \tag{2}$$

Measured damping coefficients can vary depending on the effect of transient vibration. Hence averaging over the different parts of waves, or piecewise waves of different amplitude, should be used in the identification. The curve-fitting method adopting a non-linear least-squares approach should be used for identification of the damping coefficient (see [15]).

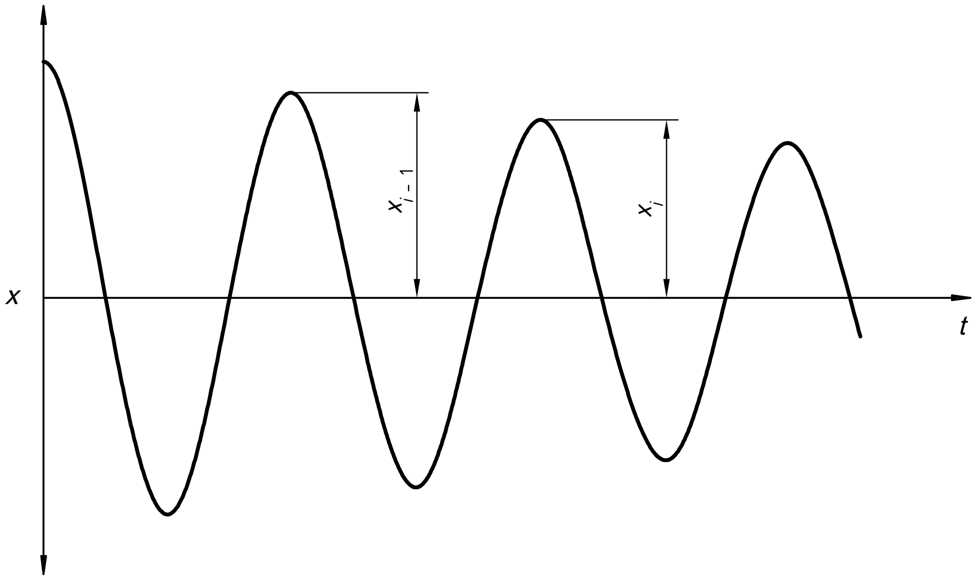

Figure 3 — Identification of the damping coefficient in the time domain

5.4.6 Identification of vibration characteristics with closely spaced modes

In large-scale structures with a variety of structural types, there can be modes with closely spaced frequencies.

EXAMPLES

a) Suspended slab bridge: the first symmetrical mode is close to the first anti-symmetrical mode depending upon the sag-span ratio.

b) Cable-stayed bridge: in multicable-type cable-stayed bridges, closely spaced frequencies often occur between the total vibration system and the cable system.

c) Bridge which has a tuned mass damper: here the vibration beating phenomenon is seen and it is difficult to identify the vibration characteristics by the above method.

The methods recommended for these cases are as follows:

- estimation by using beating wave data as the resultant of two single modes;
- extraction of the separate frequencies by superposing the beating data with a weighted function;
- the curve fitting method;
- the EK-WGI method (extended Kalman filter).

5.4.7 Random decrement technique (RD method)

With natural or ambient vibration, the expected value of the random excitation force should be considered to be white noise. In order that the superposition of certain numbers of wave data yield a meaningful component of a natural frequency, care should be taken with regard to amplitude dependency and the band-pass filter method used.

5.5 Identification of vibration characteristics in the frequency domain

There are some advantages when frequency domain analysis is used.

- The frequencies are clearly seen in the transfer function and power spectrum.
- Modal characteristics can be identified using modal analysis.
- Multiple transfer functions are obtained as the impulse response function to relate input to output effect.
- Statistical analysis is easily applied by assuming a stationary process.
- Non-stationary (impulsive) vibration effects need to be carefully considered to extract these parameters.

However, there are disadvantages.

- Analysis is impossible in the case of non-linear systems and time-dependent systems.
- Depending on the problems, accuracy may be decreased.

Non-stationary spectrum analysis, such as wavelet analysis, may avoid some of these disadvantages. Calculation of the transfer function from measured data is shown in Clause A.2.

5.6 Structural identification and inverse analysis

In the theory of system identification, the structural identification method is applied and a mathematical model is identified to relate the excitation (input) to the vibration response (output) (see Figure 4). The accuracy of the mathematical model is evaluated using the object function of error between the mathematical model and the real structure. The characteristic matrices are defined as the coefficient matrices of the governing equations of motion with multidegrees of freedom as mass, damping and stiffness matrices. Modal parameters are identified by using complex eigenvalue analysis on these governing equations (see Clauses A.3 and B.2).

For higher-frequency modes, many measurement points are needed to measure the mode shape. The application of structural identification and inverse analysis can be used to identify the vibration characteristics of the data from a limited number of measurement points. Stochastic error function analysis to evaluate the accuracy of identified parameters is also useful.

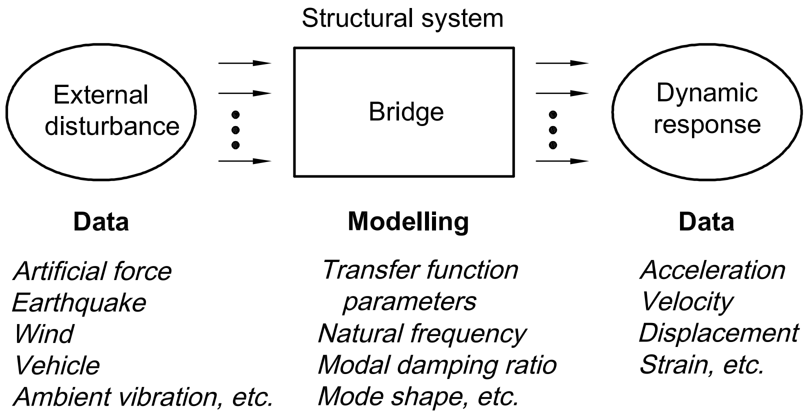

Figure 4 — Relationship between input and output of a structural system

6 Modelling bridges and their surrounding environment

6.1 Modelling bridge structures

A bridge structure is a three-dimensional structure that has beam, plate, cable, pillars, foundation and other elements. To model those geometrical structures, the structural and material characteristics of the members should be taken into account. Generally, it is recommended that the appropriately detailed model be used in the analysis. Depending on the problem to be modelled, simplified and effective beam or plate theory may be applicable. In dynamic analysis by the finite-element method (FEM), there are several methods to model the mass of a structure, such as lumped or consistent mass models.

Bridge types are classified as

- girder bridge,
- truss bridge,
- framed bridge,
- arch bridge,
- suspended slab bridge,
- suspension bridge, and
- cable-stayed bridge.

These bridges are generally supported on several piers and abutments to transfer forces to the ground. If the width of the bridge is small compared to its longitudinal length, the bridge structure is modelled using plane or space frame elements in dynamic analysis. Axial, bending and torsional rigidity of those frame elements are taken into account in the numerical analysis. Recommendations for modelling the different types of bridge structures are as follows.

a) **Girder bridges**

These consist of main girders with open cross section or closed cross section (box), cross beams (floor beams) to connect the main girder and slab. If the width of the bridge is small compared to its span, the bridge may be modelled by beam elements considering bending vibration. In the case of curved girder bridges, the coupling effect between bending and torsional vibration should be included.

b) **Truss bridges**

These consist of spatial frame elements, and the connections of elements are considered as pin (hinge) joints to take only axial deformation of members into account. Spatial truss bridges usually have many members, so that a reduction or simplification of the number of degrees of freedom in the numerical modelling should be considered. The effective beam theory, replacing spatial structure with simple beam elements, is a useful technique. Another simplification is to use the model of a spatial mass system. The stiffness of one panel of a spatial truss is modelled using effective box elements, and the mass of the panel is taken into account by concentrating the mass at the centre of the box element.

c) **Framed bridges**

The axial rigidity of the members of framed bridges is not negligible. Out-of-plane and torsional deformation should be taken into account in three-dimensional analysis of dynamic response. If a framed bridge is supported on weak ground, the effect of its support conditions should be considered in the numerical modelling. The support conditions may contribute to damping, so modification of the ground or surrounding environment is important.

d) **Arch bridges**

In the modelling of arch bridges, polygonal approximation by using straight elements is useful. The effect of the connection between the arch (including lower chord) and the vertical stiffening member is considered as a pin (hinge) or rigid joint depending on the problem. The geometrical non-linearity of arch members is not negligible in some cases; the stresses and displacements are then different from the results obtained using the small deformation theory. If the span of the arch is small, the small-deformation theory is applicable.

e) **Suspended slab bridges**

These have thin slabs and cables, and large deformations can be encountered. Hence wind-induced vibration and the serviceability for pedestrians may need to be considered. If the sag-to-span ratio f/l is large, the numerical model should include these appropriate coupling effects. Three-dimensional modelling is recommended to include the torsional modes of vibration.

f) **Suspension bridges**

The vibration behaviour of suspension bridges is considered in two systems depending on the problem to be solved. One of the vibration systems consists of the combination of cables, stiffening girders and vertical hangers; the another is the complete vibration system including the tower, pier, foundation and its surrounding environment. The first is mainly used for performance response analysis in wind load and dynamic response analysis under traffic load; the second is used mainly for performance during earthquakes. The cable and structural elements of stiffening girders have different vibration characteristics. Then the dynamic behaviour is complicated and the system has coupling effects between vertical and torsional modes. Finite-deformation theory is normally used to model these suspension bridges.

g) **Cable-stayed bridges**

These consist of towers, inclined cables, stiffening girders and substructures. The elements have different vibration characteristics and coupling vibration occurs. Sometimes coupled vibration between the cables and the girder is significant. High-frequency modes have strong coupling effects between them. Three-dimensional modelling is recommended. For the performance assessment of wind resistance, simplified modelling may be used.

6.2 Modelling of traffic loads

6.2.1 Modelling of vehicles

In the analysis of coupling vibration between the bridge and vehicles, there are several methods of vehicle modelling. Depending on the problem to be solved, one-degree-of-freedom (1-DOF), two-degrees-of-freedom (2-DOF) and multi-degrees-of-freedom (M-DOF) models are used for vehicle modelling. To fit measured results with the numerical simulation, vehicle modelling is important. If the coupling effect is taken into account, the effect of a continuous series of vehicles should not be neglected. If the effect of continuous traffic is considered, the dynamic amplification effect will be reduced compared to the results for a one-vehicle model in numerical simulation. The distribution of distances between vehicles is also a factor that should be considered.

For the modelling of a railway vehicle, a multi-body vibration model comprising a car body, truck and wheelset connected with springs and dampers should be used. The bogie-type train and articulated-type train should be distinguished in modelling. The number of vehicles in a train, the length of the train and the wheelbase are factors that should be considered in the calculation of the dynamic amplitude of the bridge response.

6.2.2 Surface roughness

The effect of the surface roughness of the track is not negligible when considering the coupling effect of the vibration between the bridge, track surface and vehicle. ISO 8608 may be used to evaluate the coupling effect.

6.3 Modelling of human walking and its dynamic effect

Walking models and the dynamic load on the bridge are used to evaluate the vibration serviceability of pedestrian bridges. Several methods for modelling the walking load are shown in Annex C.

The horizontal vibration mode of a pedestrian bridge of the suspension type is accelerated by the coupling effect between (half the) step frequency and the natural frequency of the horizontal transverse mode of the bridge. In this case, the lateral load model is useful to evaluate these coupling effects (see [13], [14] and [22]).

6.4 Wind load

Wind load for a bridge is taken into account in static and dynamic behaviour as follows.

a) Static:

- static deformation by static air pressure;
- unstable phenomenon due to static air pressure (divergence, horizontal buckling).

b) Dynamic:

- vortex flow vibration;
- galloping vibration and flutter vibration.

The vibration due to fluctuating air pressure or non-stationary varying air pressure depends on the characteristics of natural wind. If the effects of wind are to be considered in the evaluation, the variation characteristics of wind velocity should be taken into account as follows:

- standard deviation of the wind action;
- power spectrum density of the wind velocity;
- spatial scale of the random variation;
- spatial correlation of the wind velocity;
- vertical angle of the wind,
- frequency distribution of the velocity.

6.5 Modelling of the ground for viaduct vibration

In analysis of the coupling vibration between a viaduct and the surrounding ground, ground modelling should consider the ground characteristics, which may include stratification, embedded structures and other systems. But modelling of the assumed semi-infinite uniform elastic half-space of the three-dimensional coordinate is normally used for simplification.

7 Evaluation of monitored data and its application

7.1 Evaluation method and evaluation criteria

7.1.1 Classification of evaluation

Measured data are analysed in order to yield information on bridge performance, and quantitative or qualitative decisions are made based on this information. The evaluation of monitoring data is undertaken to study safety, integrity diagnosis, serviceability and the environmental effect of structures. In the evaluation process, the reliability and propriety of the structure should be clarified, so that standards can be established as a basis for evaluation. Generally, reasonable or standardized values or limit-state values of evaluation parameters are used as an evaluation standard.

As the standard values for evaluation will be different for different situations, they should be divided into categories. In some cases, evaluation will be undertaken for a combination of categories. If the evaluation is qualitative and not quantitative, comparable evaluation should be undertaken in the manner described for the following four categories.

Category 1: when values of limit states exist. The basis of the evaluation is to check whether the monitored values exceed the limit state.

Category 2: when defined values or defined ranges of condition exist. The basis of the evaluation is to compare the monitored data with those defined values or ranges.

Category 3: when estimated values for healthy condition or similar condition of the structure exist. If the evaluation values for limit states or defined states are not clear, the values which are estimated numerically for the case of healthy condition or similar condition are useful. In this case, the basis of the evaluation is to check whether the monitored data exceed those estimated values or whether they are within the range of estimated values with an acceptable error.

Category 4: when monitored data for previous condition or numerical values for similar conditions exist. If previously monitored data or data for an assumed condition are available, the measured data can be compared with those data to evaluate how much the structure improved or deteriorated.

7.1.2 Factors to be considered in evaluation

The following factors should be considered in evaluation:

a) measurement uncertainty in the monitoring data;

b) variability of the structural characteristics to be evaluated;

c) discrepancy between the measured (monitored) state and the evaluated state when standard values are calculated as evaluation parameters. The differences between actual and standard conditions should be taken into account. As an example, it is recommended that the error between the measured and estimated natural frequency of a typical mode should not be over a given percentage.

7.2 Evaluation during construction

For the evaluation of bridge vibration during construction, limit values of vibration are set by the authorities. Amplitude, velocity, damping and frequency of vibration are major indicators. The construction manager should utilize the monitoring data to decide how to manage the construction.

Damping devices may be needed if the vibration exceeds the limit. Assessment of these devices should be undertaken by evaluating the measured data.

7.3 Evaluation of structural safety in service

7.3.1 Evaluation of damage

Generally, if damage increases on a structure, the lower natural frequencies decrease and damping coefficients increase. As these changes are small, it is difficult to identify the damage accurately. Measurements on that part of the structure where the probability of damage is high may help to detect local damage through analysis of the dynamic response.

As the damage propagation within bridges is generally slow, depending on the external conditions, monitoring of vibration characteristics over the long term is helpful for the evaluation of structural health. The general tendency for the change of structural characteristics as damage propagates can be evaluated.

7.3.2 Confirmation of modification effects

When a bridge is modified or repaired and its vibration monitored, evaluation of the modifications can be identified through a comparison of the results before and after rehabilitation. Two cases shall be distinguished, as follows.

Case 1: The reinforcement is undertaken to reduce the bridge vibration itself. Measurements will enable evaluation of the reduction in the vibration.

Case 2: Measurements are undertaken to evaluate the indirect effects of reinforcement by detecting the change of vibration characteristics on parts of the structure.

7.3.3 Safety evaluation of a damaged bridge

In an emergency situation after an earthquake, fire, strong wind or flood, it is necessary for some bridges to be reinstated for rescue work. In this case, the remaining capacity of damaged bridges must be evaluated quickly. After the repair of damage, measurements should be undertaken to confirm the effectiveness of the repair by evaluation of the measured results.

7.3.4 Evaluation of fatigue condition

Statistical distribution of stress for bridge elements is obtained through vibration measurement in the form of a vibration-time history. The rain flow method is used to obtain the stress range distribution and its frequency of occurrence. The evaluation of fatigue condition is undertaken mainly by using Miner’s law:

$$D = \sum_{i} n_i / N_i \tag{3}$$

where

n_i is the actual repeated cycle;

N_i is the limit number to failure for each stress range $\left(i = 1, 2, ..., k\right)$; N_i is generally obtained in the laboratory.

In the analysis of monitored data, statistical distributions of stress, velocity, acceleration and displacement are obtained. The distributions of stress and displacement are used for the condition evaluation of bridges.

7.3.5 Running safety of trains on rails

The safety criterion is determined by the risk of derailment of a moving train. The risk of derailment is principally evaluated using the forces acting between wheels and rails. The reduction of wheel load, the magnitude of lateral thrust of the wheel, and the ratio of lateral thrust and wheel load (called the derailment coefficient) are frequently used for evaluation. For the prevention of ballasted track instability, the bridge deck acceleration is also used.

7.4 Evaluation of serviceability

7.4.1 Serviceability evaluation of a highway bridge

The sensitivity criterion for the human body on a walkway of a highway bridge has not been agreed internationally, but a general criterion on the human sensitivity exists (see ISO 2631-1). Depending on the problem, acceleration or velocity is measured and evaluated through comparison with the criterion.

7.4.2 Serviceability evaluation of a railway bridge

The serviceability on a railway bridge is primarily evaluated by the riding quality of passengers (see ISO 2631-4). The riding quality is usually rated using the magnitude of acceleration measured on the floor in car bodies. The vibrations induced by the deflection and/or bridge end angular rotations are usually transitional, so the peak value of the acceleration is suitable for evaluation.

7.4.3 Serviceability evaluation of a pedestrian bridge

The frequency and acceleration or velocity of measured response are the parameters that should be evaluated for the serviceability assessment of pedestrian bridges. The sensitivity criterion for the human body exists as an allowable level of vibration (see ISO 2631-1).

7.5 Evaluation of environmental vibration

Ground vibration and infrasound due to a viaduct should be evaluated through measured data. The frequency and magnitude of the measured data should be compared with the criteria for the comfort and performance of the human body subject to vibration (see ISO 2631-1).

Annex A
(informative)

Data analysis in time and frequency domains

A.1 Expression of response in time and frequency domains

A linear system forced by an external excitation is expressed by using the condition variable as

$$\dot{\boldsymbol{x}} = \boldsymbol{A}\boldsymbol{x} + \boldsymbol{B}\boldsymbol{f} \tag{A.1}$$

where

$\boldsymbol{x}$, $\dot{\boldsymbol{x}}$ are a condition variable vector and its time derivative;

$\boldsymbol{f}$ is the external excitation vector;

$\boldsymbol{A}$, $\boldsymbol{B}$ are coefficient matrices.

The equation of motion of the nth degree of freedom for excitation is given by

$$\boldsymbol{M}\ddot{\boldsymbol{y}} + \boldsymbol{C}\dot{\boldsymbol{y}} + \boldsymbol{K}\boldsymbol{y} = -\boldsymbol{M}\boldsymbol{r}\ddot{x}_g \tag{A.2}$$

where

$\boldsymbol{y}$ is the displacement vector of the response;

$\boldsymbol{M}$ is the mass matrix;

$\boldsymbol{C}$ is the damping matrix;

$\boldsymbol{K}$ is the stiffness matrix;

$\boldsymbol{r}$ is the vector of the influence coefficient;

$\ddot{x}_g$ is the acceleration of excitation.

If Equation (A.2) is replaced using the relationships

$$\boldsymbol{x} = \begin{bmatrix} \dot{\boldsymbol{y}} \\ \boldsymbol{y} \end{bmatrix}, \quad \boldsymbol{A} = \begin{bmatrix} -\boldsymbol{M}^{-1}\boldsymbol{C} & -\boldsymbol{M}^{-1}\boldsymbol{K} \\ \boldsymbol{I} & \mathbf{0} \end{bmatrix}, \quad \boldsymbol{B} = \begin{bmatrix} -\boldsymbol{r} \\ \mathbf{0} \end{bmatrix}, \quad \boldsymbol{f} = \begin{bmatrix} \mathbf{1} \\ \mathbf{1} \end{bmatrix} \ddot{x}_g$$

where $\boldsymbol{I}$ is an indentity matrix and $\mathbf{1}$ is a column vector of ones, Equation (A.2) is reduced to Equation (A.1).

The impulse response of $\boldsymbol{x}$ for Equation (A.1) due to an impulse input $\boldsymbol{f} = \mathbf{1}\delta(t - t_0)$ for Equation (A.1) at $t = t_0$ is given as

$$\xi\left(t - t_0\right) = \mathrm{e}^{\boldsymbol{A}\left(t - t_0\right)}\boldsymbol{B}H\left(t - t_0\right) \tag{A.3}$$

where

δ is the Dirac delta function;

H is the Heaviside step function.

The impulse response due to $\boldsymbol{f}$ is given by

$$\boldsymbol{x}(t) = \int_{-\infty}^{\infty} \xi(t - t_0)\, \boldsymbol{f}(t_0)\, \mathrm{d}t_0 \tag{A.4}$$

$$= \int_{-\infty}^{\infty} \xi(\tau)\, \boldsymbol{f}(t - \tau)\, \mathrm{d}\tau$$

It is necessary to use the Duhamel integral to obtain the total response in the time domain. In order to transform the relationship between the response and excitation to the frequency domain, Fourier transform on Equation (A.4) should be used as

$$\int_{-\infty}^{\infty} \boldsymbol{x}(t)\, \mathrm{e}^{-\mathrm{i}\omega t} \mathrm{d}t = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \xi(\tau)\, \boldsymbol{f}(t - \tau)\, \mathrm{e}^{-\mathrm{i}\omega t} \mathrm{d}\tau \mathrm{d}t \tag{A.5}$$

$$= \int_{-\infty}^{\infty} \mathrm{e}^{\boldsymbol{A}(\tau)} \boldsymbol{B} H(\tau)\, \mathrm{e}^{-\mathrm{i}\omega\tau} \mathrm{d}\tau \int_{-\infty}^{\infty} \boldsymbol{f}(t - \tau)\, \mathrm{e}^{-\mathrm{i}\omega\tau} \mathrm{d}(t - \tau)$$

$$\boldsymbol{X}(\omega) = \boldsymbol{H}(\omega)\, \boldsymbol{F}(\omega)$$

where $\boldsymbol{H}(\omega)$ is the Fourier transform of the impulse response function, and it is called the transfer function or frequency response function.

By this procedure in the frequency domain, multiplication of functions may be used instead of the Duhamel integral for the time domain as, for example, for the power spectrum $\boldsymbol{S}$

$$\boldsymbol{S}_{xx}(\omega) = \boldsymbol{H}(\omega)\, \boldsymbol{H}(-\omega)\, \boldsymbol{S}_{ff}(\omega) \tag{A.6}$$

In Equations (A.5) and (A.6), $\boldsymbol{X}$ and $\boldsymbol{F}$ are vectors and $\boldsymbol{H}$ is a matrix for the multiple-input multiple-output (MIMO) system. If it is a single-input single-output (SISO) system, they are scalar functions

$$X(\omega) = H(\omega)\, F(\omega) \tag{A.7}$$

For power and cross spectra, they are

$$S_{xx}(\omega) = H(\omega)\, H(-\omega)\, S_{ff}(\omega) = \left|H(\omega)\right|^2 S_{ff}(\omega) \tag{A.8}$$

$$S_{fx}(\omega) = H(\omega)\, S_{ff}(\omega) \tag{A.9}$$

$$S_{xx}(\omega) = H(\omega)\, S_{xf}(\omega) \tag{A.10}$$

In general, the vibration response $\boldsymbol{x}$ is measured, and $\boldsymbol{f}$ may also be measured depending on the problem. Comparison of the measured spectrum with assumed values in design and the calculation of parameter identification should be carried out to evaluate the structural condition.

A.2 Calculation of the transfer function from measured data

The transfer function $H(\omega)$ is given by using Equation (A.7) as

$$H(\omega)=X(\omega)/F(\omega) \tag{A.11}$$

Attention should be paid to errors which are included in measured data. If the measured data X^* have the error $N_1(\omega)$, i.e.

$$X^*(\omega)=X(\omega)+N_1(\omega) \tag{A.12}$$

then applying Equation (A.7) yields

$$H^*(\omega)=X^*(\omega)/F(\omega)=X(\omega)/F(\omega)+N_1(\omega)/F(\omega) \tag{A.13}$$

Therefore, the error appears directly in the transfer function if the ratio of the Fourier spectra is simply taken. On the other hand, if the cross spectrum with F on Equation (A.12) is taken

$$S_{fx^*}(\omega)=S_{fx}(\omega)+S_{fn_1}(\omega) \tag{A.14}$$

When there is no correlation between input and noise, $S_{fn}(\omega)=0$ and the transfer function $S_{fx^*}(\omega)=S_{fx}(\omega)$ is given without any effect of error as

$$H_1(\omega)=S_{fx}(\omega)/S_{ff}(\omega) \tag{A.15}$$

Similarly, if the input measurement F^* has a noise effect and if it has no correlation with the output, the transfer function is

$$H_2(\omega)=S_{xx}(\omega)/S_{xf}(\omega) \tag{A.16}$$

Also, if there is no error in the measured data, Equations (A.11), (A.15) and (A.16) are the same:

$$\frac{H_1(\omega)}{H_2(\omega)}=\frac{S_{fx}(\omega)S_{xf}(\omega)}{S_{xx}(\omega)S_{ff}(\omega)}=\gamma_{xf}^2 \tag{A.17}$$

This is called coherence and it takes 0 to 1 by the relationship between internal product and magnitude. If there is no noise, the estimation H_1 and H_2 take the same value and the coherence is 1. So coherence is used as a certainty index of measured data. Generally, H_1 is used as an estimation of the transfer function.

It is recommended to take the ratio of cross spectra and not to take the ratio between the Fourier spectra when the transfer function is to be obtained from measured data.

A.3 Calculation of transfer function by analysis

The equation of motion of the nth degree of freedom is

$$\boldsymbol{M}\ddot{\boldsymbol{x}}+\boldsymbol{C}\dot{\boldsymbol{x}}+\boldsymbol{K}\boldsymbol{x}=\boldsymbol{f} \tag{A.18}$$

Applying Fourier transform to both sides of the equation:

$$\left[-\omega^2\boldsymbol{M}+\mathrm{i}\omega\boldsymbol{C}+\boldsymbol{K}\right]\boldsymbol{X}(\omega)=\boldsymbol{F}(\omega) \tag{A.19}$$

The transfer function between the displacement response and an external excitation is given as

$$H(\omega) = \left[-\omega^2 \boldsymbol{M} + \mathrm{i}\omega \boldsymbol{C} + \boldsymbol{K}\right]^{-1} \tag{A.20}$$

A direct estimation of the transfer function is possible by calculating the above inverse matrix. But the frequency of each mode and its contribution by direct calculation are unclear. It is therefore recommended to obtain the transfer function in modal space. The formulation of the transfer function of a non-proportionally damped system is explained as an example. If the bridge has damping devices or isolators, the structure is considered a non-proportionally damped system. Then it is necessary to use analysis of non-proportionally damped systems as the identification of the total system of the bridge.

By transformation of the variables of Equation (A.18), a symmetrical ordinary differential equation is obtained:

$$\boldsymbol{A}\dot{\boldsymbol{z}} + \boldsymbol{B}\boldsymbol{z} = \boldsymbol{g} \tag{A.21}$$

where

$$\boldsymbol{A} = \begin{bmatrix} \boldsymbol{C} & \boldsymbol{M} \\ \boldsymbol{M} & \boldsymbol{0} \end{bmatrix}, \ \boldsymbol{B} = \begin{bmatrix} \boldsymbol{K} & \boldsymbol{0} \\ \boldsymbol{0} & -\boldsymbol{M} \end{bmatrix}, \ \boldsymbol{z} = \begin{Bmatrix} \boldsymbol{x} \\ \dot{\boldsymbol{x}} \end{Bmatrix}, \ \boldsymbol{g} = \begin{Bmatrix} \boldsymbol{f} \\ \boldsymbol{0} \end{Bmatrix}$$

The solution of the eigenvalue calculation of the left-hand side of Equation (A.21) is given by complex eigenvalues and eigenvectors of the $2N$th degree, and these are the combination of conjugate values of the Nth degree. The eigenvalues may be ordered $\lambda_j = S_j$ for $j = 1, 2, ..., N$, and $\lambda_{j+N} = S_j^*$. The eigenvectors are given as

$$\boldsymbol{V}_j = \begin{Bmatrix} \phi_j \\ S_j \phi_j \end{Bmatrix}, \ \boldsymbol{V}_{j+N} = \begin{Bmatrix} \phi_j^* \\ S_j \phi_j^* \end{Bmatrix} \tag{A.22}$$

where

$$\boldsymbol{\varphi}_j = \left\{\phi_{1j}, \phi_{2j}, \cdots, \phi_{Nj}\right\}^T$$

In this calculation, the meaningful number of eigenvalues and modal shapes is N for S_j and ϕ_j for an Nth degree system. Frequency and modal damping are given by

$$\omega_j = \mathrm{Im}\left(S_j\right) \tag{A.23}$$

$$\zeta_j = -\frac{-\mathrm{Re}\left(S_j\right)}{\varpi_j} \tag{A.24}$$

By normalizing the eigenvectors $\boldsymbol{V}_j$ for the matrix $\boldsymbol{A}$, the jk element of Equation (A.20) is given as

$$H_{jk}(\omega) = \sum_{r=1}^{N} \left(\frac{\phi_{jr}\phi_{kr}}{\mathrm{i}\omega - S_r} + \frac{\phi_{jr}^*\phi_{kr}^*}{\mathrm{i}\omega - S_r^*} \right) \tag{A.25}$$

If the system is considered a proportionally damped system, the mode shapes are real numbers and

$$H_{jk}(\omega) = \sum_{r=1}^{N} \frac{2\phi_{jr}\phi_{kr}\left(\mathrm{i}\omega + \zeta_r \omega_r\right)}{-\omega^2 + 2\mathrm{i}\zeta_r \omega_r \omega + \omega_r^2} \tag{A.26}$$

Annex B
(informative)

Identification of vibration characteristics

B.1 Single-mode method

By assuming that the coupling effect between the different modes is small, and considering that the system is the sum of single-degree-of-freedom systems, frequencies and damping coefficients are obtained for each mode. The curve-fitting method is useful to identify the peak of the transfer function by fitting those values of the 1-DOF systems. If the system has closely spaced frequencies, identification should be carried out in a multi-degree-of-freedom (M-DOF) system.

It is recommended to use the half-power method applied to the peak of the transfer function, and the curve-fitting method applied to the wide range of the transfer function.

B.2 Least-squares method

In order to identify the natural frequency, damping and mode shapes in the frequency domain, identification of these parameters from the transfer function is carried out to fit with the measured transfer function.

To fit the measured transfer function $H_{\mathrm{m}}(\omega)$ with the assumed transfer function of a non-proportionally damped system $H_{\mathrm{c}}(\omega)$ for both real and imaginary parts, the following evaluation function

$$E = \int_{\omega_1}^{\omega_2} \left[\mathrm{Re}\left(H_{\mathrm{m}} - H_{\mathrm{c}}\right)^2 + \mathrm{Im}\left(H_{\mathrm{m}} - H_{\mathrm{c}}\right)^2 \right] W(\omega)\, \mathrm{d}\omega \qquad \text{(B.1)}$$

is minimized by changing the assumed values of frequency and damping. The range ω_1 to ω_2 is the frequency range for identification, and $W(\omega)$ is a weighting function which accounts for the increase in values due to band-pass filtering within the frequency range.

Annex C
(informative)

Modelling of walking load

Taking the effect of both right and left steps of walking into account, the walking load $f(t)$ is defined (see [7]) as

$$f(t) = \frac{P}{M_1} \phi(0{,}9 f_1) \sin \omega_1 t \tag{C.1}$$

where

P is the amplitude of the pulsating force;

M_1 is the generalized mass for the fundamental mode;

ϕ is the normalized fundamental mode shape;

0,9 is the width of step, in metres;

f_1 is the lowest natural frequency of the bridge;

ω_1 is the angular frequency of the bridge, in radians per second;

t is the time.

The walking load $f(t)$ has been defined (see [24]) as a half-sinusoidal function by neglecting the negative load of one step as

$$f(t) = F \sin \frac{\pi t}{T_\mathrm{C}} \tag{C.2}$$

where

F is the weight of the walking body, in newtons;

T_C is the contact time of one step;

t is the time.

The walking load $f(t)$ has been defined (see [17]) by a half-cosine function also for a positive load

$$f(t) = \alpha W \cos 2\pi f t \tag{C.3}$$

where

α is the amplification factor of the load;

W is the weight of the body, in newtons;

f is the walking frequency.

These equations represent time variation of foot pressure on the bridge. Taking the speed and location into account, walking load is considered as a moving load along the bridge.

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