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EXPERIMENTAL METHODS USED IN SYSTEM IDENTIFICATION OF CIVIL ENGINEERING STRUCTURES

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ABSTRACT

To experimentally identify the dynamic characteristics of a structure, also referred to as system identification, two methods are available: Forced Vibration Testing (FVT) and Ambient Vibration Testing (AVT). The basic ideas of these methods are shortly presented. The main part of the paper deals with practical problems which are to be overcome when performing such a system identification test. To achieve reliable, good quality results, a number of parameters have to be optimized when planning an experimental system identification investigation. Among others, such parameters may be the means and location of excitation, the density of the measurement point grid, the sampling rate and the length of the time window. As an illustration, several examples of good and worse tests on bridges and other civil engineering structures are discussed.

1. FORCED VIBRATION TESTING

1.1. Basics

With Forced Vibration Testing (FVT) the structure to be identified is artificially excited with a forcing function in a point i and its response $y_k(t)$ to this excitation is measured together with the forcing signal $x_i(t)$ (Fig. 1). Transformation of these time signals into the frequency domain and calculation of all Frequency Response Functions (FRF's) H_{ik} between the response and the forcing function time signals yields the Frequency Response Matrix, also referred to as Transfer Matrix, $H(i\omega)$ (Figs. 2 and 3).

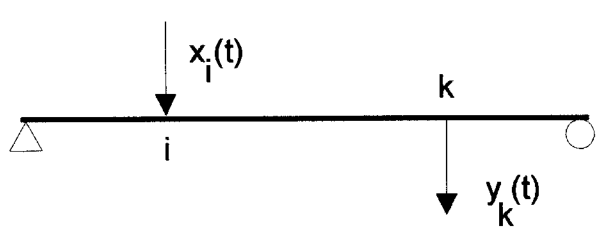


Fig. 1: Forced Vibration Testing Scheme.

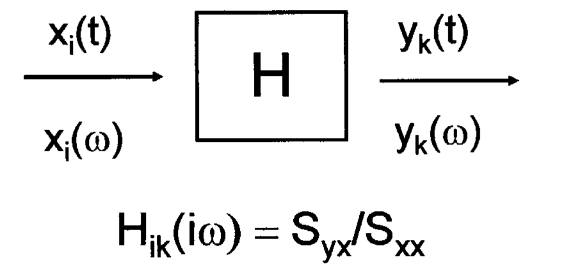


Fig. 2: Calculation of the Frequency Response Function H_{ik}.

$$H(i\omega) = \begin{matrix} H_{11} & \bullet & \bullet & \bullet & \bullet \\ \bullet & \bullet & \bullet & \bullet & \bullet \\ \bullet & \bullet & H_{ik} & \bullet & \bullet \\ \bullet & \bullet & \bullet & \bullet & \bullet \\ \bullet & \bullet & \bullet & \bullet & H_{nn} \end{matrix}$$

Fig 3: The Frequency Response Matrix.

For linear systems, the Frequency Response Matrix is diagonal. This means that it suffices to either determine one row or one column of this matrix (Fig. 3). The choice is to either keep the excitation point constant and rove the response points over the structure or vice versa. Because it is not so easy to move the exciters used in civil engineering investigations, the first method is used here. In mechanical engineering, where the structures to be tested are comparatively smaller and easy to excite, e.g. with a hammer, the latter of the procedures mentioned is more common.

The Frequency Response Matrix contains all the information necessary to determine the dynamic natural properties of the structure under investigation (natural frequencies and the associated mode shapes and damping coefficients). Dedicated software packages are available on the market to extract these modal parameters from the results of a Forced Vibration Test.

1.2. Excitation

Generally speaking, the means of excitation has to be chosen such as to

- excite all natural frequencies of interest,
- be significantly larger in effect than any other “unwanted” excitation
  (because: the processing procedures are based on the assumption that the measured, artificial excitation is the only source of excitation during the tests).

Broad-band vibration generators excite all natural vibrations of the structure in this frequency band at the same time. Examples are impulse hammers and servo-hydraulic or electro-dynamic shakers generating random or swept-sine type forces. Narrow-band vibration generators excite one specific frequency at a time. Mechanical devices using counter-rotating masses can be mentioned here. Of course, hydraulic or electric shakers can also be used as narrow-band exciters.

Broad-band exciters are very time effective, but they have to have (relatively) more energy disposable than narrow-band exciters. These devices distribute their energy on many frequencies at a time. Using a narrow-band exciter is very time consuming, but such a device concentrates all the energy available into a specific frequency.

To excite civil engineering structures, hydraulic and electric shakers are better suited than hammers. Compared with mechanical structures, the fundamental natural frequency of a civil engineering structure is low. The average value, e.g. for some 200 highway bridges in Switzerland is $f \approx 3$ Hz [1]. The frequency resolution to be achieved with an FVT investigation has hence to be high, let's say $\Delta f \approx 0.01$ Hz. This resolution is directly related to the length of a time window to be transformed into the frequency domain: $\Delta f = 1/T$. The length of this window has hence to be at least $T = 100$ s in this case. And: The quality of the FRF's determined also depends on the number of averages which can be performed when transforming the time data into the frequency domain. Something like 10 is a good value here. Considering an overlap of 50%, at least 500 s of stationary structural response has then to be generated to determine a reasonably well averaged FRF. This is not possible with using a hammer.

The shaker systems used in the examples discussed later are presented in Figures 4 to 9.

Fig 4: Servo-hydraulic shaker producing a 5 kN vertical force amplitude for 2.3 < f < 100 Hz (left). 500 kg mass are fixed to the piston rod of a cylinder with a ±50 mm stroke. The 40 l/min (280 bar) hydraulic power pack and associated air-cooler used to drive this cylinder are shown above.

Fig 5: Servo-hydraulic shaker producing a 20 kN vertical force amplitude for 1.8 < f < 100 Hz (left). 500 kg mass are fixed to the piston rod of a cylinder with a ±125 mm stroke. The 80 l/min (280 bar) hydraulic power pack and associated air-cooler used to drive this cylinder are shown above.

The Diesel power generators used to make the shaker systems self-contained units when performing FVT's somewhere out in the field are shown in the Figures 7 and 8 for the shakers shown in the Figures 4 and 5/6 respectively. Note that the shakers are sitting on load cells to directly measure the dynamic force put into the structure. Due to space restrictions, the electronics necessary to drive the shakers can not be shown here.

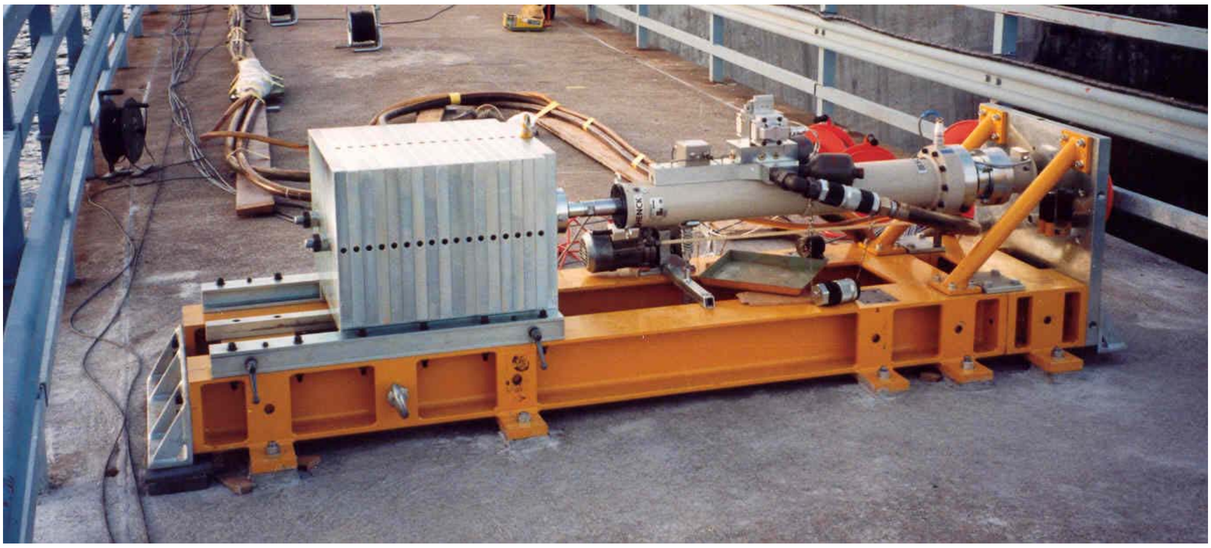


Fig. 6: Servo-hydraulic shaker producing a 32 kN horizontal force amplitude for $1.5 < f < 100$ Hz. 1'000 kg mass are fixed to the piston rod of a cylinder with a ±125 mm stroke. The 80 l/min (280 bar) hydraulic power pack used to drive this cylinder is shown in Figure 5.

Fig. 7: 60 kW Diesel power generator, driving the "small" shaker (Fig.4)

Fig. 8: 120 kW Diesel power generator, driving the "large" shakers shown in Figures 5 and 6.

Fig 9: Electro-dynamic shaker, producing a 0.45 kN force amplitude for $5 < f < 50$ Hz. 30 kg moving mass, ±80 mm stroke. Power supply: 220 V, 1.2 kW, no cooling required.

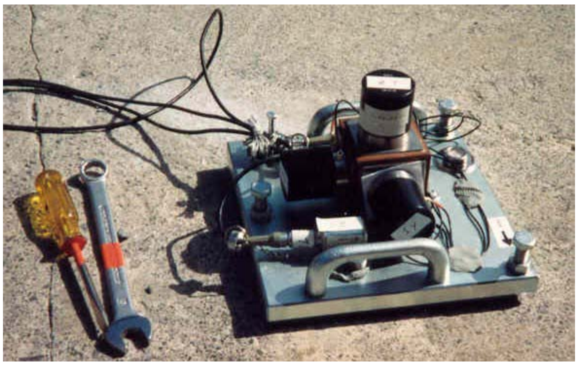


Fig. 10: 3D acceleration response measurement point (B&K 8306, 10 V/g).

1.3. Response

The type of sensor chosen for the response measurement has to fit the requirements concerning sensitivity and frequency range. Also because they are much easier to apply and rove over a structure, accelerometers are the best choice in most cases. Measuring displacement in many points is a very cumbersome task for civil engineering structures. Velocity transducers are well suited for structures exhibiting a fundamental natural frequency $f > 4.5$ Hz. Most civil engineering structures exhibit lower frequencies. Therefore, highly sensitive accelerometers are mainly chosen to investigate such structures (10 V/g). Piezoelectric sensors are suited for structures with a fundamental natural frequency $f > 1$ Hz. For structures exhibiting lower frequencies, sensors of the force balance type have to be used.

As a next point, the measurement direction(s) and the measurement point grid density have to be chosen. The basic rule here is: Information on the mode shapes is available in measured points and directions only. This choice can be made in a much more reliable way when based on the results of a preliminary Finite Element analysis of the structure. In most of the cases discussed later, this FE analysis was anyway the first step of the procedure, because the major goal of the experimental system identification was to update the preliminary FE model based on the experimental results. This updated FE model could subsequently be used as a basis to identify problem solutions performing parameter studies.

It can be seen from the examples discussed later that the number of measurement points can be as high as 200 to 300. The number of degrees-of-freedom to be measured is even higher in cases where it is necessary to measure in two or three directions per point. It is therefore standard practice to simultaneously use a limited number of sensors and to rove this set of sensors over the structure until the measurement point grid is completed. A test is therefore separated into several setups. As the forcing signal is always measured too, there is no problem for the processing software to subsequently glue together the information gathered from the different setups.

2. AMBIENT VIBRATION TESTING

2.1. Basics

No artificial exciter is used with Ambient Vibration Testing (AVT). The response of the structure to ambient excitation is measured instead. With civil engineering structures, ambient excitation can be wind, traffic or seismic micro-tremors. The more broad-band the ambient excitation, the better the results. Otherwise, there is some risk that not all natural frequencies of the structure are excited.

Generally speaking: The information resulting from the force input signal $x_i(t)$ with FVT investigations is replaced with the information resulting from the response signal $y_R(t)$ measured in a reference point R (Fig. 11).

The first software package to extract modal parameters from AVT investigations has been developed by a civil engineer in the early nineties of the last century. Today, there are several packages on the market making use of the frequency domain procedures shown schematically in Fig. 12. One of them offers more sophisticated methods like FDD (Frequency Domain Decomposition) and EFDD (Enhanced FDD), the latter also including estimation of damping values. These methods have been protected by a US patent recently (www.svibs.com).

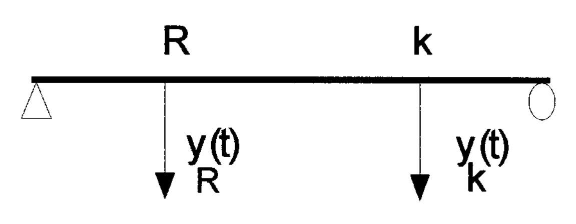


Fig. 11: Ambient Vibration Testing Scheme; R is a reference point, k is a roving point.

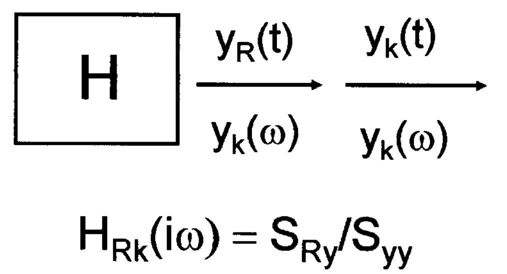


Fig. 12: Calculation of the cross relationship between the reference point R and roving response point k signals.

However, the most recent signal processing tools are not based on an analysis in the frequency domain as shown in the figure above. Stochastic Subspace Identification (SSI) is a method working completely in the time domain. Basically, a multi-order model is looked for which synthesizes the measured time signals in a optimum way. This method has especially been developed for AVT investigations.

Concerning response measurement requirements, the same basic rules apply as for FVT investigations. It is wise to use more than one reference point unless the structure to be tested is very simple. If response measurements are three-dimensional, at least one 3D-point has to be chosen as a reference. The risk of the reference point sitting in the node of a mode can be reduced significantly by choosing more than one reference point. As a rule of thumb, the length of the time windows acquired should be 1'000 to 2'000 times the period of the structure's fundamental natural vibration.

3. FORCED VERSUS AMBIENT TESTING

The main advantage of FVT is the fact that this method provides "scaled" results. Because the input force is measured, information on the mass and stiffness matrices of the structure is gathered. This allows automated updating of FE models. Model updating using the results of an AVT investigation is possible with manual techniques only.

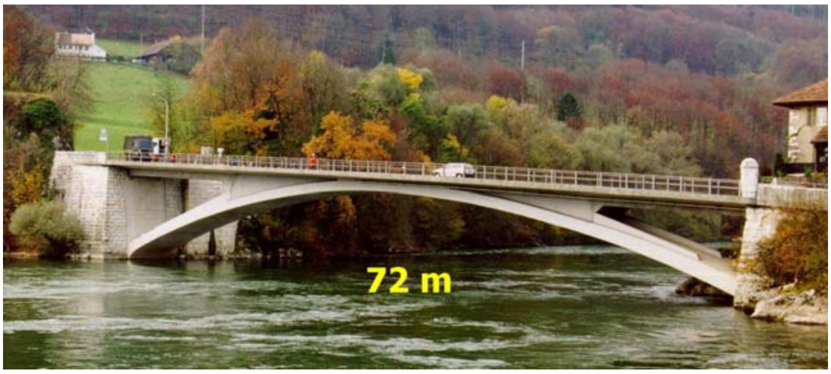
The main advantage of AVT is the fact that no artificial excitation is necessary. This makes such tests comparatively cheap. In addition, AVT investigations can be performed without embarrassing the normal user. This fact is very important for highway bridges.

Ambient excitation is of the so-called multiple-input type. Wind, traffic and micro-tremors are acting on many points of a structure at the same time. In the contrary, a forced vibration is usually of the single-input type. For small structures, this difference is not important. The example "Westend Bridge" is presented to illustrate the limits of a single-input FVT investigation. For large and complex structures, AVT has hence an advantage on the excitation side.

Ambient excitation being non-controllable usually results in a lack of stationarity. This may lead to problems due to the non-linearity of the structure (no civil engineering structure behaves in a really linear way). In case of the excitation amplitude being significantly different for each of the setups, a certain scatter in the results may occur. This is not the case for FVT where the structural vibrations induced can be kept stationary.

4. EXAMPLES OF FVT-INVESTIGATIONS

4.1. A "short" bridge: Bridge on the Aare River at Aarburg



This single span arch bridge crosses the Aare River at Aarburg [2]. The completely clamped-in arch is a reinforced concrete structure from 1912 (Robert Maillart) whereas the bridge deck has been rebuilt in 1968 as a pre-stressed structure. The bridge deck is horizontally free at both abutments. This results in the structure being simple (one span) and exhibiting simple and clearly defined boundary conditions. Therefore, the results of the single-input FVT investigation were of very good quality. The shaker (Fig. 4) was located on the downstream curb 10 m from the abutment. The forcing signal was of the random type with an upper band limit of f_c = 36 Hz. The measurement point grid consisted of 105 and 41 3D-acceleration measurement points (Fig. 10) on the bridge deck and the arch respectively. One 3D-point was always located in the driving point (underneath the shaker). Another five 3D-points were roved over the structure. Fig. 13 gives an idea of the resulting grid density. The sampling rate was s = 80 Hz, the length of a time window T = 51.2 s, the number of averages was 8, the total net testing time per cycle was 7 minutes, the total test took four nights between 10 pm and 5 am.

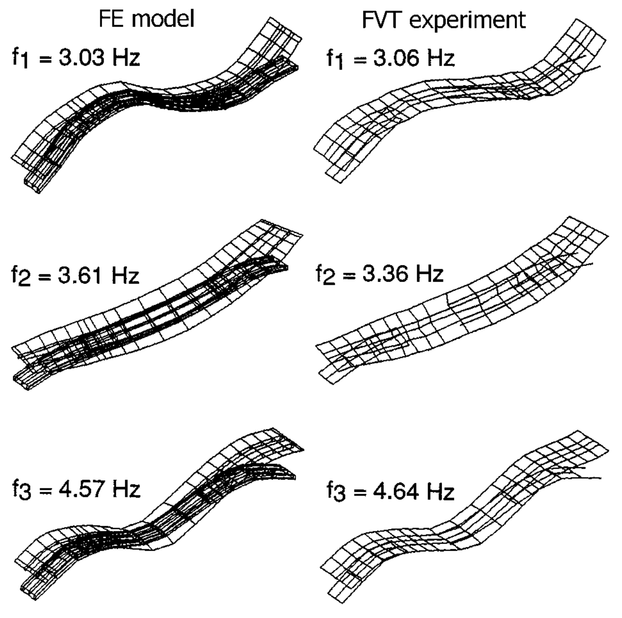


Fig. 13: Frequency and shape of the first three bridge modes; FVT results to the right, updated FE model to the left.

Modal Correlation : (MAC > 0.750)   Set_1: a1   Set_2: e1

	Set_1 mode	Set_1 Hz	Set_2 mode	Set_2 Hz	diff Hz	diff %	MAC
1	1	3.029	1	3.062	0.033	1.1	0.909
2	2	3.611	2	3.361	-0.250	-6.9	0.929
3	3	4.572	3	4.640	0.068	1.5	0.933
4	4	5.817	4	5.714	-0.103	-1.8	0.936
5	5	6.773	5	7.665	0.892	13.2	0.901
6	6	7.922	6	8.793	0.871	11.0	0.753
7	7	8.071	7	9.019	0.949	11.8	0.893

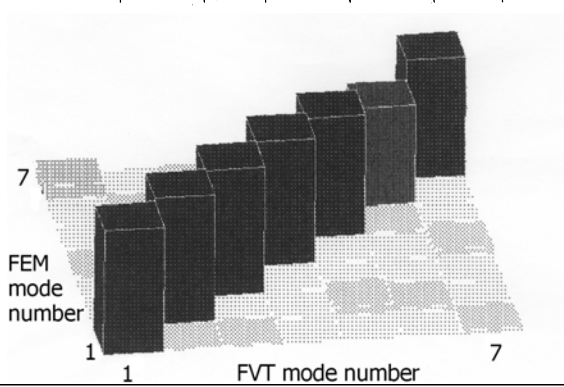


Fig. 14: MAC matrix (FVT versus FE model) for the first seven bridge modes. (scaling of the vertical axis: 0 to 1.)

4.2. A “long” bridge: Westend-Bridge Berlin

This eight-span continuous beam bridge is part of the heavily trafficked Berlin city belt [3]. The relatively short single columns are all clamp-ed into a flat foundation. With the exception of one column in the middle of the bridge, which is also clamped into the bridge deck, the columns are pinned at their top end. There are several problems with this bridge from the point of view of a single-input FVT investigation. The bridge is too long and has too many spans. Especially bad is the short 31-m-span in the middle of the bridge which cuts the structure into two dynamically almost "decoupled" parts (at least what vertical bending modes is concerned).

The reason for this somehow strange span arrangement is the fact that the spans in the left-hand part of the photo (and in the right-hand part of the drawing shown in Fig. 15) are crossing several railway lines. The columns had to respect the position of the rail tracks. It was not possible to install a second shaker in this part of the bridge because access to the area underneath these spans was, a) not allowed and, b) not possible with vehicles. Due to the heavy traffic on the bridge, the time window for the tests was 10 pm to 4 am only. Outside of this window, nothing could be left on the bridge. There was hardly enough time to install/remove one shaker at position #75 (Fig. 15) with the infrastructure (hydraulic power pack, power generator) being located just underneath this point on the ground.

The shaker (Fig. 4 in 1993 and Fig. 5 in 1995) was located in point #75 (Fig. 15). The forcing signal was of the random type with an upper band limit of $f_c = 20$ Hz. The measurement point grid consisted of 215 and 32 3D-acceleration measurement points on the bridge deck and the columns respectively (Fig. 15). One 3D-point was always located in the driving point (underneath the shaker). Another five 3D-points were roved over the structure. The sampling rate was $s = 64$ Hz, the length of a time window $T = 16$ s, the number of averages was 12, the total net testing time per cycle was about 4 minutes, the total test took four nights.

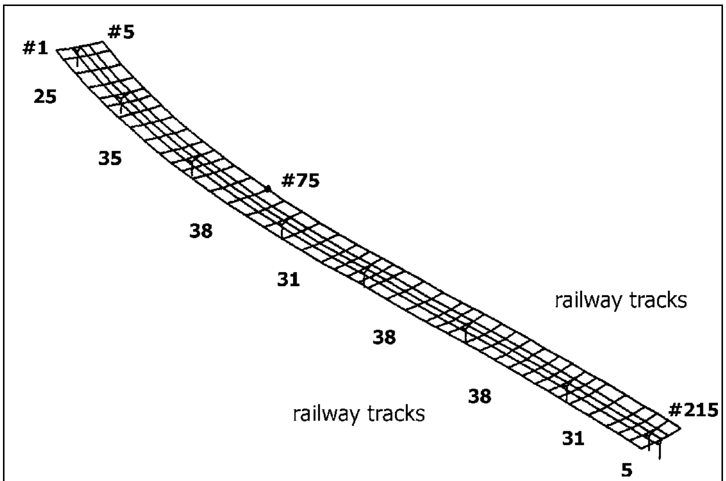


Fig. 15: Measurement point grid for the Westend-Bridge. #75 is the driving point (shaker position); the length of the individual spans is indicated in m.

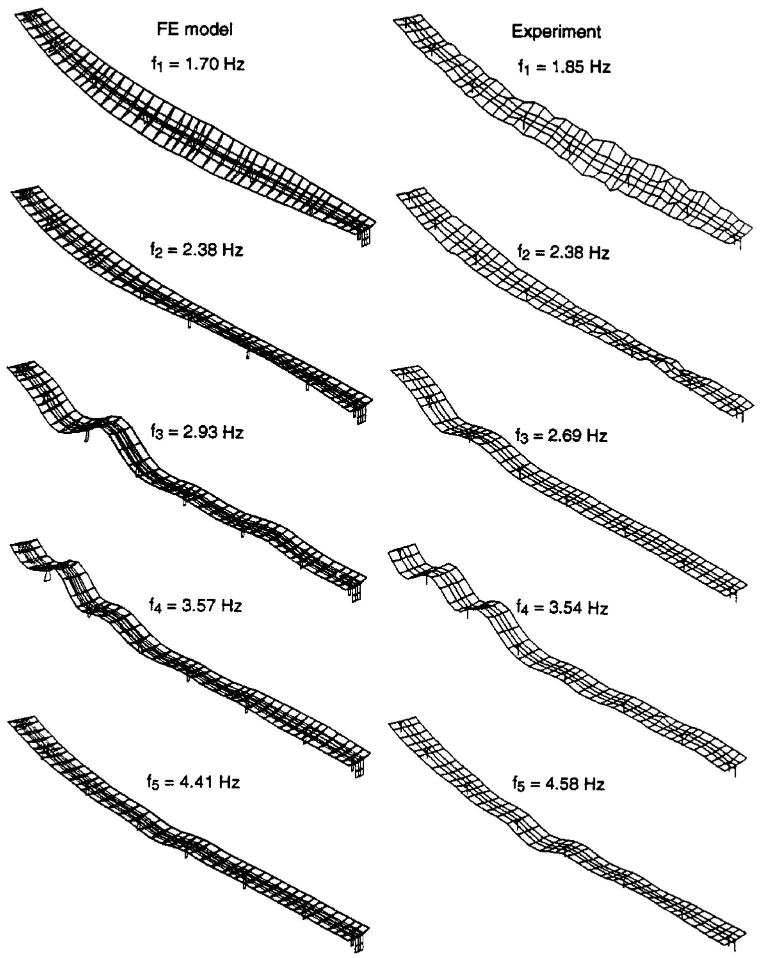


Fig. 16:

Frequencies and shapes of the first five (out of nine [3]) modes of the Westend Bridge as derived from the FVT investigation and an updated FE-model. (MAC for the modes shown is in the MAC = 0.74…0.83 range.)

It can be seen that:

a) the frequency of f_1 is too low to properly be excited,

b) the second torsional mode is not properly excited at the right-hand side of the bridge,

c) the bending modes are well excited in the area of the shaker but not at the right-hand side of the bridge,

d) the natural vibration dictated by the 31 m middle span is properly excited (f_5).

4.3. A dam: Norsjö Dam Sweden

Norsjö Dam is a cylindrically shaped reinforced concrete structure with a length at the crest of 170 m and a maximum height of roughly 46 m [4]. The radius of curvature of the dam upstream face is r = 110 m. The dam width is 2.5 m at the crest and 5.5 m at the bottom (Fig. 17).

The results of an FVT investigation of this dam using the shaker shown in Figure 6 were of extraordinary good quality. Considering the results of a preliminary FE analysis, the shaker was located some 60 m from the abutment at the first spillway wall (left-hand side on the photo). The forcing signal was of the random type with a band-limit at f_c = 20 Hz. The tests could only be performed when the powerhouse was out of operation.

The measurement point grid consisted of 270 3D-acceleration measurement points distributed over the dam crest and eight further levels between crest and foundation (it can be seen from Figure 17 that the whole downstream face of the dam is accessible), spillway walls and powerhouse. Four 3D-points were roved over the structure. The sampling rate was s = 100 Hz, the length of a time window T = 41 s, the number of averages was 8, the total testing time per cycle was about 6 minutes, the total test took two days (of a weekend) and four nights.

Twelve modes with f = 3.0…13.5 Hz could be identified. Updating of an FE model was possible with very good results [4].

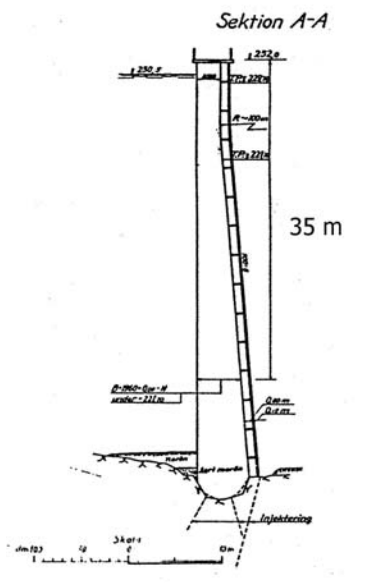


Fig. 17: Dam cross section.

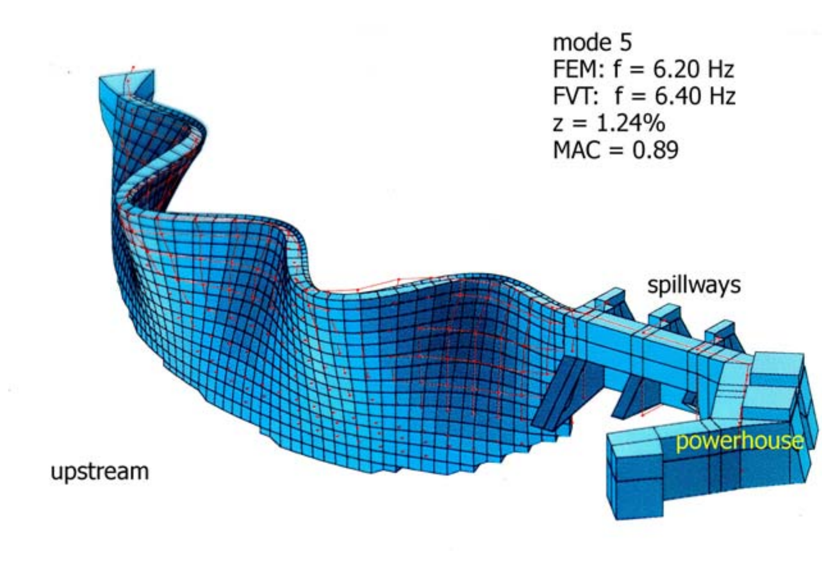


Fig. 18: Results for mode No. 5 (FEM and FVT shapes overlaid!)

4.4. An office building: Unique Airport Zurich

A 26 x 9 m office building was built into an existing 8 m high hall. This office building consisted of 4 m high columns leaving open the "ground floor" and of a "first floor" light-weight structure on top of this containing the offices. The intermediate steel floor consists of main girders every 8.6 m in both directions and intermediate girders. People working in these offices complained about disturbing floor vibrations.

The results of an FVT investigation of this building using the shaker shown in Figure 9 were of good quality. The shaker was placed 1 m out of mid-span of one of the main cross girders [5]. The forcing signal was of the random type with a band-limit at f_c = 20 Hz.

The measurement point grid consisted of 72 (vertical) 1D-acceleration measurement points distributed over the steel slab (Fig. 19 illustrates the resulting grid density). Fixation to the steel girders was done from the bottom side of the slab. In every setup, the forcing signal and 8 vertical accelerations were measured. The sampling rate was s = 60 Hz, the length of a time window T = 34 s, the number of averages was 6, the total net testing time per cycle was about 4 minutes, the total test took a couple of hours during one night.

Three modes with f = 4.3…5.8 Hz could be identified (this is of course much too low for an office floor). Updating of an FE model was quite difficult, because the system boundaries were not clearly defined (influence of the secondary walls and the roof?). However, after some attempts, MAC values in range MAC = 0.77…0.81 could be achieved. The updated FE model was used to evaluate rehabilitation measures.

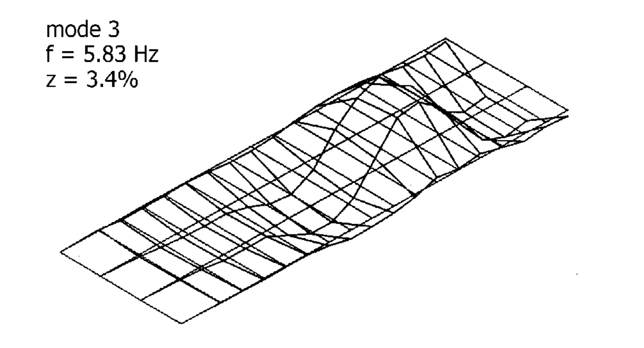


Fig. 19: FVT results for mode 3.

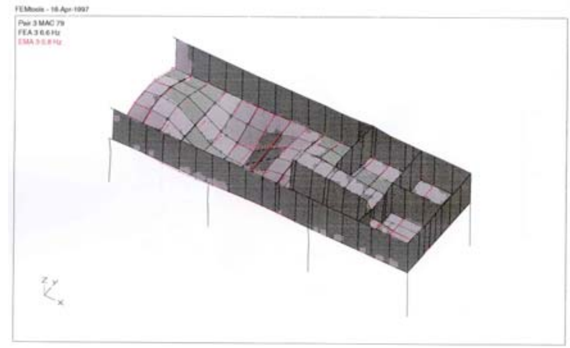


Fig. 20: FEM and FVT shapes overlaid!

5. EXAMPLES OF AVT-INVESTIGATIONS

5.1. A "long" bridge: Ganter Bridge

This two-lane highway bridge with a total length of 678 m has eight spans with a length between 35 m and 174 m [6], [7]. The height of the tallest pylon is 172 m.

Figure 21 shows the measurement point grid chosen for an AVT investigation. Including 192 points, this covered the bridge deck on its entire length plus the two tallest piers.

At the bridge deck, the upstream sensors were of the 3D-, the downstream sensors of the vertical 1D-force balance type. As traffic remained open during the tests the accelerometers were placed inside the box girder. 3D sensors were fixed to the piers through professional climbers rappelling from the pier top (see photo). The two reference points chosen were equipped with a 3D and a 1D sensor respectively. Three 3D and three 1D sensors were roved in pairs along the structure.

The big challenge of this test was to organize the cable management. The cable length available was 300 m per 3D and 1D sensor and the bridge deck was not accessible for any piece of equipment (no curbs). It was therefore necessary to break down the test into four phases. Phase 1: The measurement center was located close to pier 2 on the ground (Station 1 in Fig. 21). Measurement of the bridge deck between the north abutment and the reference points. Phase 2: The measurement center was located between piers 4 and 5 (Station 2). Measurement of the bridge deck between the south abutment and the reference points. Phases 3 and 4: Measurement of piers 4 and 3 with the measurement center being located at stations 2 and 1 respectively.

The sampling rate chosen was s = 20 Hz, the length of a time window 53 minutes for the bridge deck and 27 minutes for the piers. The test took ten working days.

A total of 25 modes could be identified in the frequency band f = 0.40…3.88 Hz [6], [7]. AVT proved to be a very good method to identify the dynamic parameters of such a large structure exhibiting very low natural frequencies.

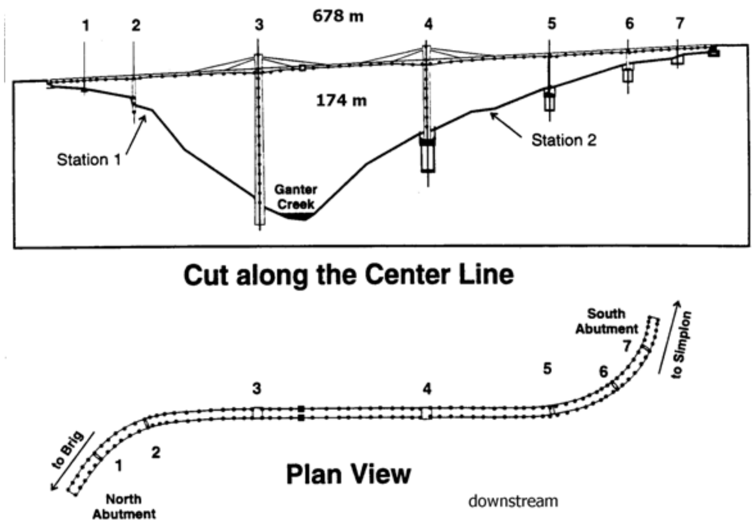


Fig. 21: Measurement point grid. Reference points: two black squares in span 3-4.

Mode	Description of the Mode Shape	Frequency (Hz)
1	1st lateral	0.40
2	2nd lateral with vertical bending	0.67
3	1st vertical bending	0.74
4	3rd lateral mode	0.96
5	2nd vertical bending	1.05
6	4th lateral	1.43
7	3rd vertical bending	1.61
8	5th lateral	1.82

Fig. 22: Frequencies and shape of modes No. 1 to 8.

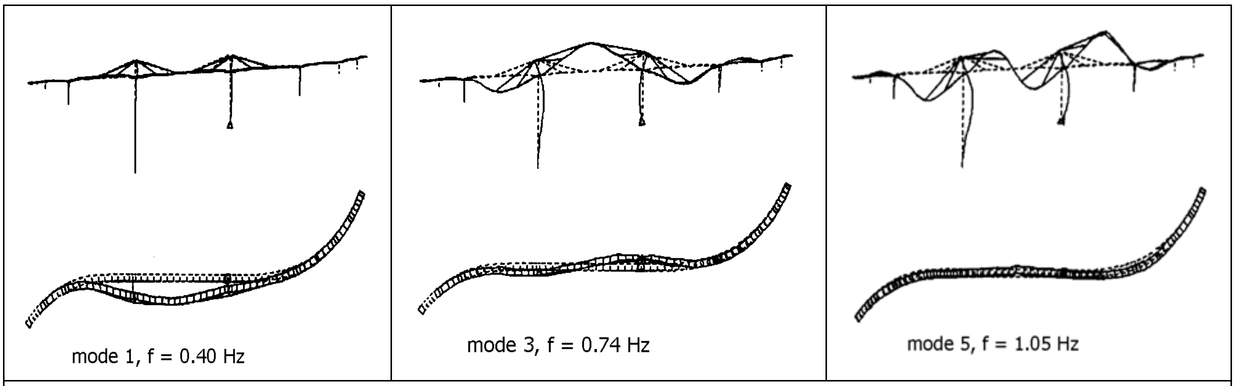


Fig. 23: Shapes of the Modes No. 1, 3 and 5 (elevation looking upstream and plan view).

5.2. A "short" bridge: Regensdorf Bridge Zurich

The Regensdorf Bridge is a 30 m long and 14 m wide, skew steel-concrete composite structure [7]. It consists of six riveted main girders and a concrete deck.

The bridge undersight being inaccessible (railway lines), the challenge of the AVT investigation was to organize the measurements without disturbing the (heavy down-town Zurich) traffic too much. The existence of sidewalks is very helpful in such a case.

Figure 24 shows the bridge geometry and the measurement point grid. The two reference points are indicated with solid squares. Seven sensors were roved over the bridge deck. In a first phase, the curbs and the bridge centerline were measured without any interference to the traffic. Based on the intermediate (on-line) results it was decided to measure two additional lines. For this, the traffic flow was restricted to the remaining part of the bridge deck.

The sampling rate was s = 80 Hz, the length of a time window 7 minutes. The test was accomplished in one day.

A total of 7 modes could be identified in the frequency band f = 4.80…18.55 Hz [6]. Extraction of the mode shapes was based on vertically oriented measurements only, although some of the sensors were of the 3D-type.

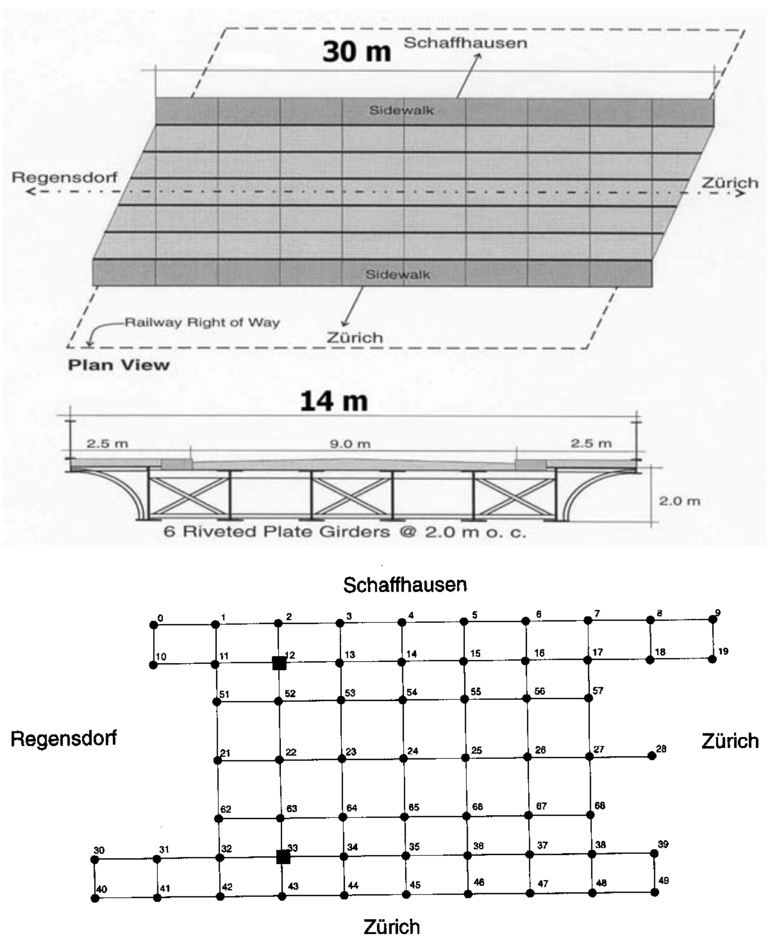


Fig. 24: Geometry and measurement point grid.

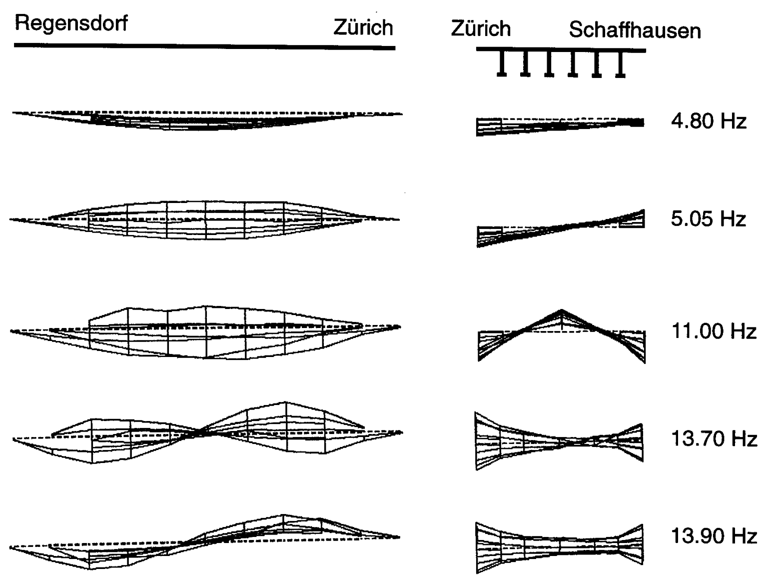


Fig. 25: Frequency and shape of the first five modes.

5.3. A cable-stayed bridge: Rhein Bridge Schaffhausen

The Rhein Bridge Schaffhausen is a curved cable-stayed highway bridge with a main span of 126 m, an end span of 26 m and a deck width of about 20 m [6]. The inclined pylon has a height of 51 m. Six cable pairs are supporting the main span and there are eight pairs of back stay cables.

Figure 26 shows the measurement point grid used for the AVT investigation to identify the dynamic parameters of the bridge. This covered 138 points on the deck and pylon. 3D reference sensors were located on top of the pylon and at the main span quarter point. At the latter point a second 1D reference sensor was placed at the other side of the cross section. Measurements on the pylon and at the bridge centerline were made with 3D sensors.

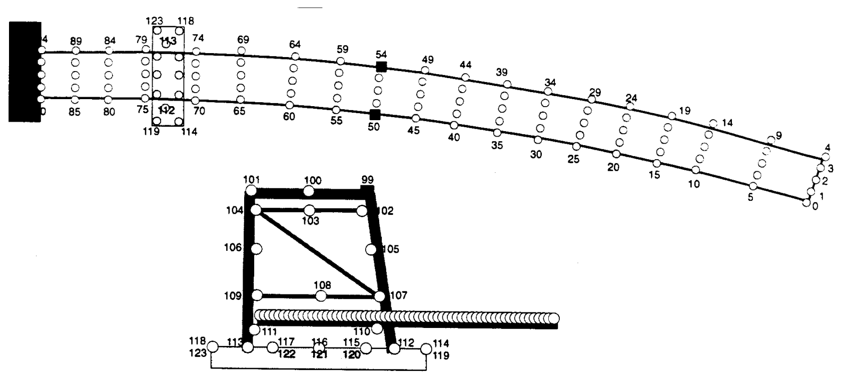


Fig. 26: Measurement point grid. Reference points are indicated with black solid squares.

Two 3D and three 1D-sensors were roved over the bridge. In addition to the measurement points shown in Figure 26, all main span stay cables and four back stays were instrumented with a 3D sensor 1.5 m above the bridge deck. The tests happened a couple of weeks before the bridge was opened to traffic. Therefore, problems with interference with traffic or construction work did not occur.

The sampling rate was s = 40 Hz, the length of a time window 15 minutes. The test was accomplished in two and a half days.

A total of 59 modes of the main span cables with a frequency in the range f = 1.3…9.7 Hz and 16 modes of the back stays with f between f = 2.0 Hz and f = 8.7 Hz were identified first. This allowed excluding these modes from looking for a structural mode (Fig. 27).

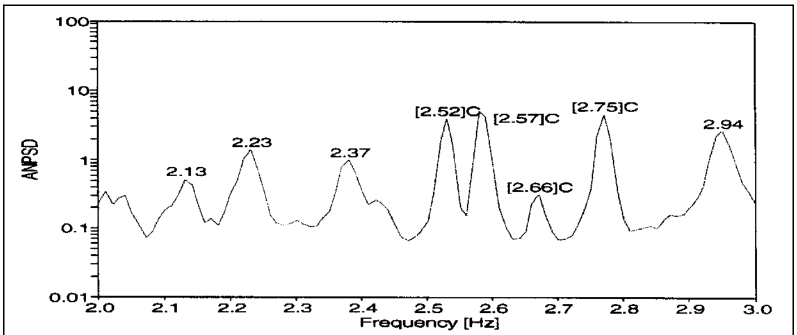


Fig. 27: Detail of the spectrum of the vertical movements (ANPSD). Known cable modes are indicated with "C".

As a consequence, identification of 23 structural modes was facilitated significantly [6].

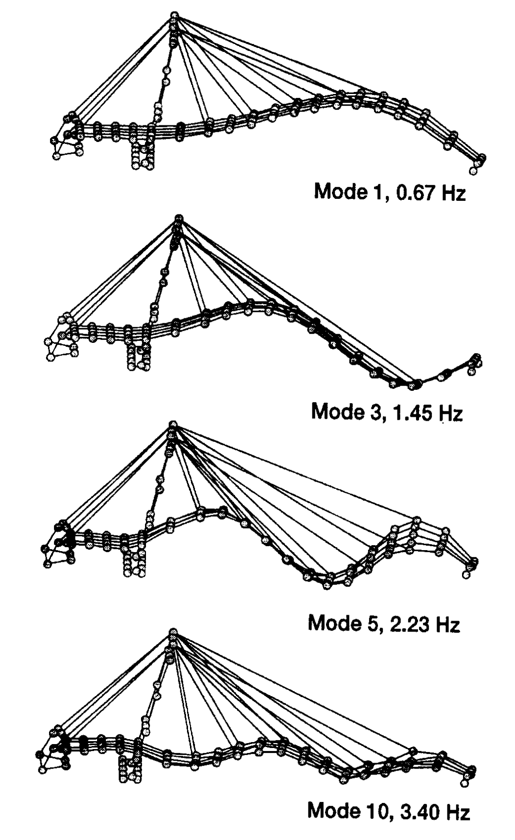


Fig. 28: Some bending modes

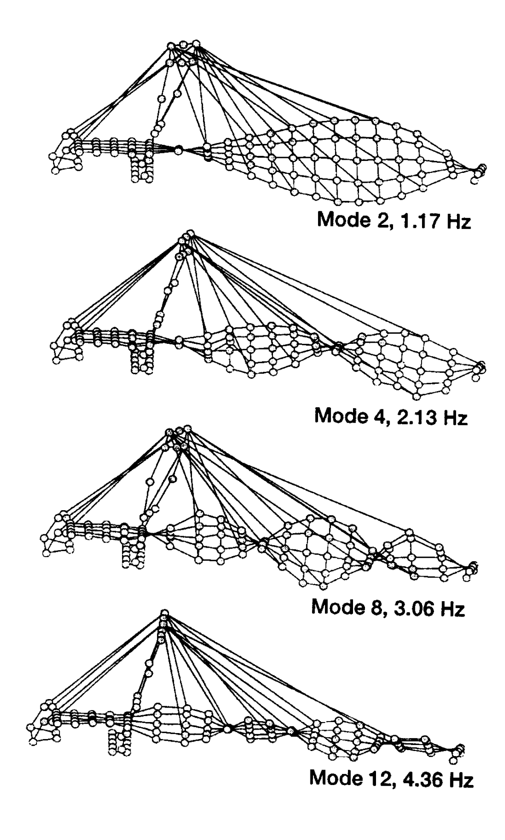


Fig. 29: Some torsional modes

5.4. A bell tower: St. Peter and Paul, Zurich

The tower of St. Peter and Paul Church is a 52 m high stone structure built in 1895. The measurement point grid used for an AVT investigation consisted of three horizontal degrees of freedom (accelerations) measured on the levels 2 to 6 as shown in Fig. 30. Three sensors remained as reference points at level 6 while three further sensors were roved over the levels 2 to 5 in four setups.

To define the instrumentation necessary to determine the mode shapes of a structure like a tower or a tall building, several assumptions can be made: a) the vertical components can be neglected, b) the rectangular shape of the structure's cross section for a certain level above ground remains unchanged, and, c) the movements of the structure in the horizontal plane are small. It then suffices to measure three of the eight possible degrees of freedom of a rectangular cross section: x and y in one corner and y in a neighboring corner (Fig. 30). Advanced software packages allow to determine the remaining DOF's using so-called "slave node equations".

The sampling rate was s = 25 Hz, the length of a time window 30 minutes. The test was accomplished in one afternoon.

Nine modes with frequencies f = 2.30…8.9 Hz could be identified.

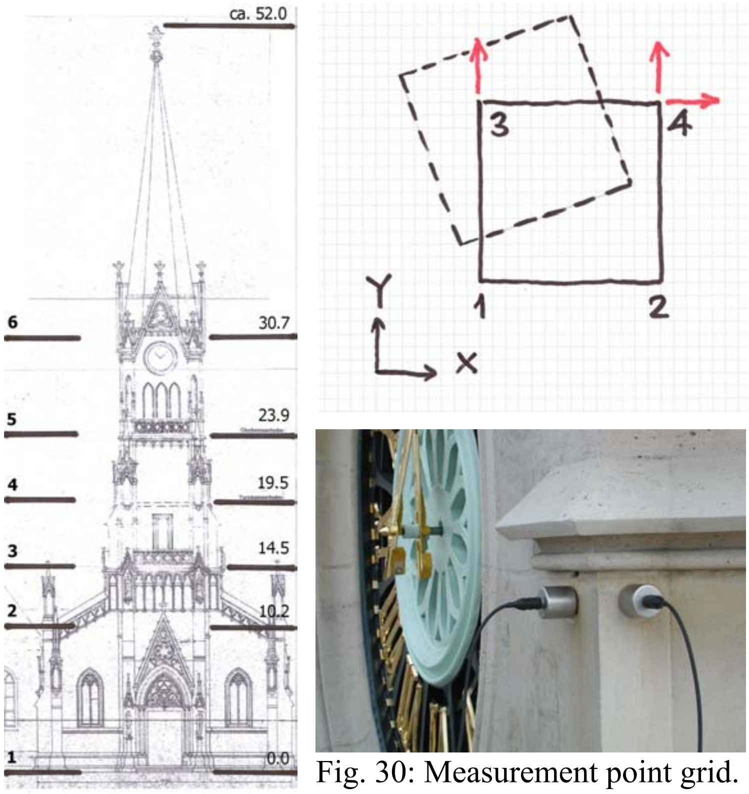


Fig. 30: Measurement point grid. Three sensors per measurement level.

Fig. 31: The measurement center located at level 5.

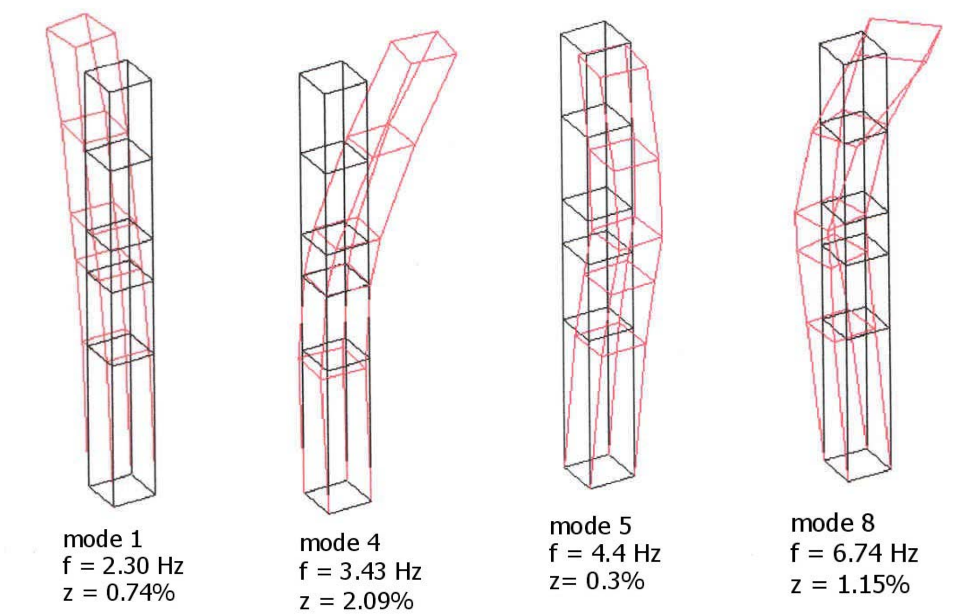


Fig. 32:

Frequency, shape and damping coefficient of the modes No.1, 4, 5 and 8.

5.5. An office building floor: Mainau Zurich

This office building is a five-story structure consisting of five concrete slabs supported on some core walls and several façade columns. The maximum dimension is about 7 m (the diagonal between the corner columns and the core wall corner. One of these slabs (the second floor) had interior walls neither in the story below nor in the story above the slab (Fig. 33). People working at this floor complained about excessive floor vibrations.

To identify the dynamic parameters of such a structure, experience has shown that it is a good idea to artificially increase the level of structural vibrations during the "AVT" investigation. Moving on the floor and throwing a 5 kg medical ball from a height of roughly 1 m in irregular intervals of one to four seconds has proven to be a very efficient means of excitation for concrete floors with dimensions of this size (Fig. 34). The advantages of this procedure are three-fold: a) the vibration level induced in this way is definitely larger than any "noise" vibration induced by any "dynamic" piece of equipment in the building (including the vibrations induced by the ball thrower's walking), b) the impulses generated by the ball (obviously; according to experience) have an optimum duration and the frequency band of interest is excited very nicely, and, c) the risk of the excitation sitting in a node of a structural natural vibration is zero.

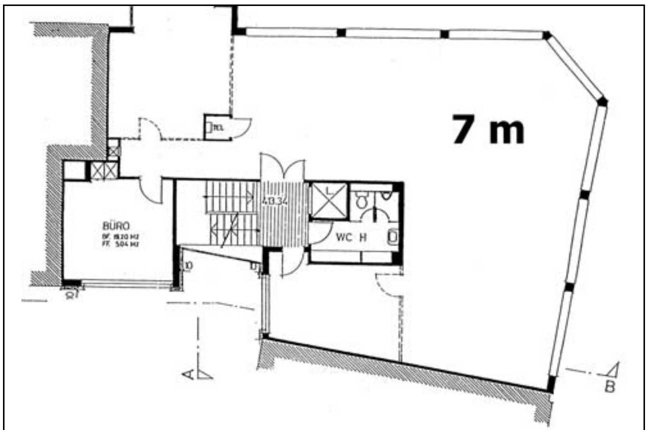


Fig. 33: Plan view of the second floor.

The measurement point grid consisted of three vertical reference sensors and 35 roving measurement points. These were covered with five roving sensors in seven setups. The resulting density of the measurement point grid can be evaluated from the mode shapes shown in Figure 35.

The sampling rate was s = 100 Hz and the length of the time windows 5 minutes. Three floors of this building were tested in this way during one weekend.

Eight modes could be identified for the second floor slab (f = 7.36…26.3 Hz). The fundamental frequency for the first and third floors was higher than 11 Hz.

Fig. 34: Accelerometer and medical ball. The sensors were placed on the concrete without fixation but after removal of the carpet.

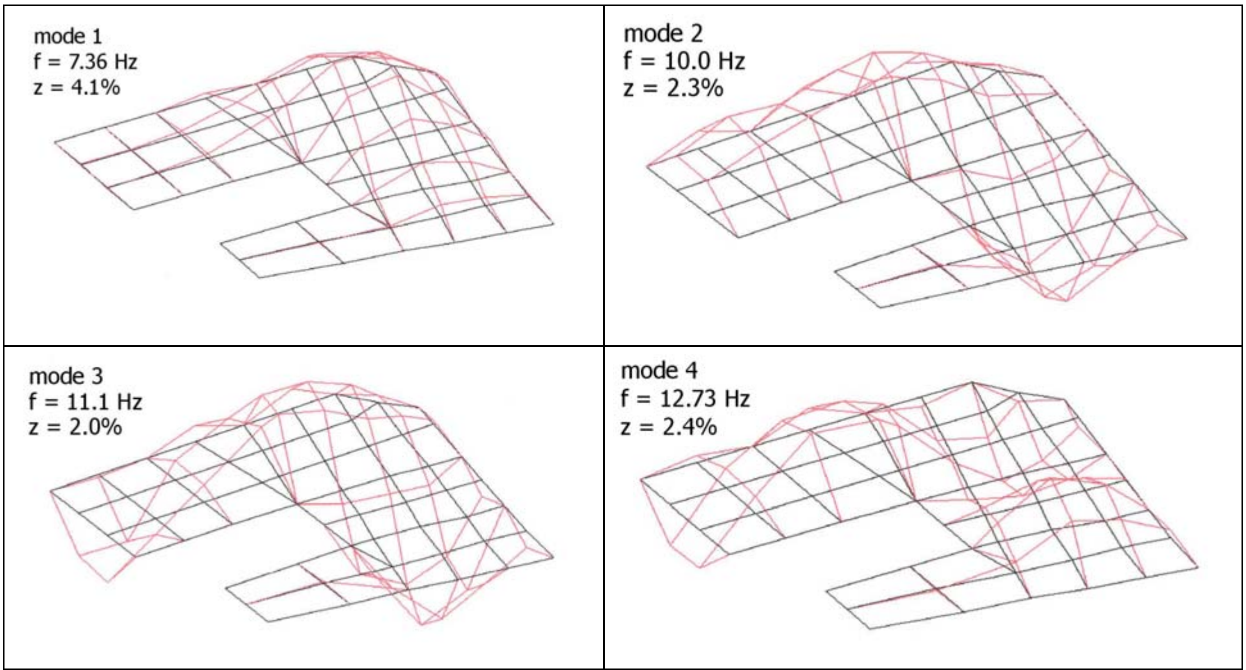


Fig. 35: Frequency, mode shape and damping for the four first modes of the second floor slab.

REFERENCES

[1] R. Cantieni, Dynamic Load Tests on Highway Bridges in Switzerland - 60 Years Experience of EMPA. EMPA Report No. 211, (1983).

[2] R. Cantieni, Y. Deger, S. Pietrzko, Modal Analysis of an Arch Bridge: Experiment, Finite Element Analysis and Link, Proc. 12th International Modal Analysis Conference (IMAC), January 31 - February 3, 1994, Honolulu, Hawaii, (1994) 425-432.

[3] Y. Deger, R. Cantieni, S.J. Pietrzko, W. Rücker, R. Rohrmann, Modal Analysis of a Highway Bridge: Experiment, Finite Element Analysis and Link, Proc. 13th International Modal Analysis Conference (IMAC), Nashville, Tennessee, 13-16 February 1995, Vol. 2, (1995) 1141-1149.

[4] R. Cantieni, Assessing a Dam's Structural Properties Using Forced Vibration Testing, Proc. IABSE International Conference on Safety, Risk and Reliability - Trends in Engineering, Malta, March 21-23 (2001).

[5] R. Cantieni, S.J. Pietrzko, Y. Deger, Modal Investigation of an Office Building Floor, Proc. 16th International Modal Analysis Conference (IMAC), Santa Barbara, California, 2-5 February 1998, Vol. 2, (1998) 1172-1178.

[6] A.J. Felber, R. Cantieni, Introduction of a new Ambient Vibration System - Description of the System and Seven Bridge Tests, EMPA Report No. 156'521, Duebendorf (1996).

[7] A.J. Felber, R. Cantieni, Advances in Ambient Vibration Testing: Ganter Bridge, Switzerland, Structural Engineering International (6), Number 3, (1996) 187-190.