

MODAL INVESTIGATION OF AN OFFICE BUILDING FLOOR

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Abstract

Procedures and results of an experimental and analytical Modal investigation performed on an 8.6 x 25.8 m office building floor are presented. This floor is part of a lightweight steel construction added inside of an existing 8 m high large steel frame building at an intermediate level of roughly 4 m. Complaints of people working in the new office of unacceptable floor vibrations induced by other people walking from their desk to the copying or fax machine were the reason for this investigation.

To identify the problem a preliminary test was performed. The floor vibrations were measured with a minimum instrumentation, i.e. two accelerometers, and data processing was performed on site with a two-channel frequency analyzer. One main result was that the maximum acceleration amplitude was $a \approx 0.25 \dots 0.30 \text{ m/s}^2$ and hence roughly ten times higher than acceptable as e.g. indicated in [1]. Furthermore, the lowest two natural frequencies of the floor were identified to $f_1 \approx 4.15 \text{ Hz}$ and $f_2 \approx 5.90 \text{ Hz}$. This meant that the floor was excited by the second and third harmonics of a walking person's step frequency, $f \approx 2 \text{ Hz}$ [2].

The main investigation firstly included a Forced Vibration Modal Test. Excitation was provided by an electrodynamic vibration generator. This induced random-type vertical forces in a point chosen on the floor. The dynamic response (acceleration) was measured in the vertical direction in a total of 72 points. Processing of the measured signals yielded three natural frequencies in the range $f = 4.29 \dots 5.83 \text{ Hz}$ as well as the corresponding mode shapes and damping.

Secondly, a Finite Element model of the system was created. Usually, this is done prior to the experimental investigations. Assuming a certain minimum accuracy of the FE-model this can be used when planning the tests, i.e. when choosing the measurement point grid and the driving point location based on the calculated mode shapes. This was however not possible in the case under discussion here. The dynamic behavior of the structure proved to be influenced not only through the steel floor properties but also through the walls and roof. Because of a lack of knowledge concerning the walls' material properties and system stiffness a priori modeling of these structural parts was not possible before the tests with sufficient accuracy. This was hence done after the tests and the model was then updated on the basis of the test results. Model updating yielded good correlation for the three modes identified with MAC-values $\text{MAC} = 77 \dots 81\%$.

Based on this updated Finite Element Model a number of rehabilitation measures were checked concerning their effectiveness. The goal was to increase the fundamental natural floor frequency to $f > 9 \text{ Hz}$ [2]. As a result, addition of at least six columns or, alternatively, of a strong longitudinal girder plus three columns was suggested. Strengthening of the floor cross beams was found to be not sufficiently effective. Placing of tuned mass dampers was found to be not suitable because three different floor vibrations would have been subject of damping measures. The final problem solution is still to be chosen.

Commissioned by the owner of the building, EMPA performed an experimental and analytical System Identification on a steel floor having recently been added to the building as an office area in 1997. Complaints of people working in the new office of unacceptable floor vibrations induced by other people walking from their desk to the copying or fax machine were the reason for this investigation. After a preliminary test having been performed to identify the problem, the floor was subject of a Forced Vibration Modal Test. Subsequently, a Finite Element model of the new building was created. This Finite Element model was then updated based on the Forced Vibration Test results and used to assess the effectiveness of rehabilitation measures.

2 THE STRUCTURE

The several years old main building is a quite large steel frame structure with a mesh width of 8.6 m. A new office building was added in a corner of this building partly making use of the existing columns and main beams. Using the original axes denomination the new building covered the area between the longitudinal axes K and L and the transverse axes 10, 11, 12 and 13 (Fig. 1). Complicating the situation, the four original columns in the axis K had a free length of roughly 8 m whereas the free length of the columns in the axis L was separated into twice 4 m through an intermediate beam along axis L. This difference is due to the fact that axis K is at the interior of the building whereas axis L defines one of the building's façades. Most of the beams shown in Figure 1 were added to the existing structure: The longitudinal beam in axis K, two UNP 320, fixed to the interior columns at a height of 4 m, main cross beams HEA 260 in the axes 10 to 13 and in the third points between these and an additional mesh of secondary beams HEA 120 in the cross and longitudinal directions. No columns were added which kept the free span to 8.6 m in the transverse (cross) and the longitudinal directions.

The floor construction placed on top of these steel beams consisted of a corrugated steel sheet SP 80/0.88, an additional 5 mm steel plate, probably some further layer of concrete or gypsum of unknown thickness and a thin hard-felt carpet.

"Exterior" walls were mounted along the axes K (with windows) and 10 (no windows). In the other two office exterior axes L and 13, the existing walls were kept. Figures 2 and 3 give an impression of the new office structure as seen from the inside of the main building and a view of the new steel floor undersight.

The floor area was not completely free of interior walls. Such walls were concentrated in the square K/L/10/11 (see Figure 11, Paragraph 6). In the remaining area some twenty people are working using computer workstations.

3 THE PROBLEM

Every time one of the people working in the office area walked to the copying or fax machine, floor vibrations were induced which made some of the quite large computer screens shake quite considerably. This shaking was especially obvious in the mid-span region of the square K/L/11/13. The investigation described here was instigated as a consequence of the corresponding complaints.



Fig. 2 View of the new office building "exterior wall" in axis K. To the far left the old wall in axis 13 can be seen, in the middle and to the right the two old columns in the axes 12 and 11.



Fig. 3 View of the new steel floor underside. Cross beams from the left to the right, the corrugated steel sheet on top of them.

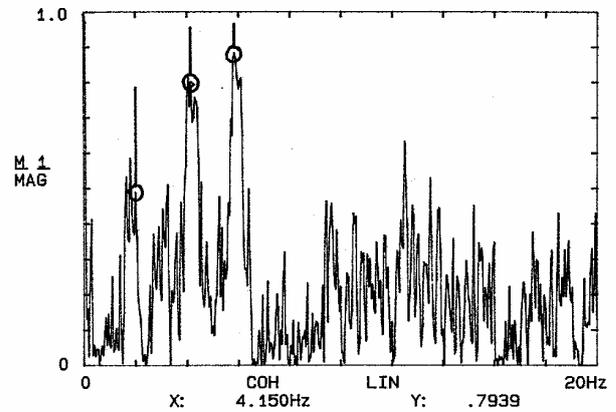
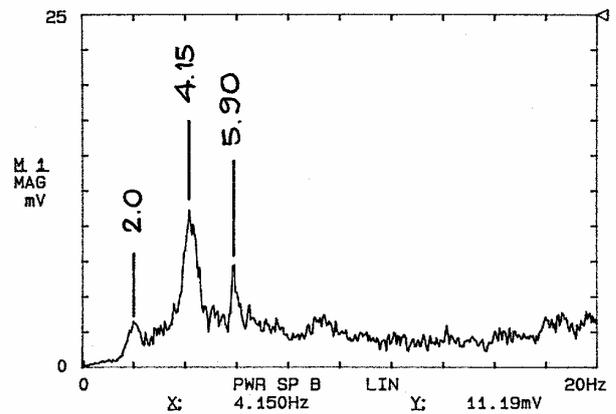
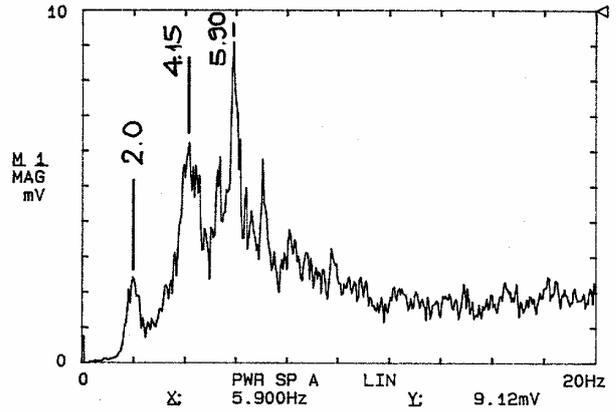


Fig. 4 Results of the preliminary tests: Top two figures: Frequency spectra as measured in the points A and B (see Fig. 1). The Coherence Function clearly shows a quite low value for the forced vibration at $f \approx 2$ Hz and higher values for the two floor natural vibrations.

4 PRELIMINARY TEST

To identify the problem, a preliminary tests using a minimum equipment was performed. Two accelerometers were placed in the critical region (Fig. 1). Considering the floor construction being quite stiff, the accelerometers were fixed to steel plates of roughly 10 kg mass equipped with three pinned bearings and placed on the carpet. The floor vibrations were then measured during a couple of hours of normal office operation. This kind of system excitation was primarily chosen to determine the actual vibrations' intensity and frequency content. It was however clear, that besides the peaks representing the floor's natural frequencies, a peak would show up in the frequency spectra measured at the forcing frequency, $f \approx 2$ Hz [2]. The results of on-line signal processing however showed a clear separation between the different peaks (Fig. 4). It was hence not necessary to perform additional tests with e.g. heel drop excitation. These would have yielded floor natural frequencies only.

On-line signal processing was performed using a two-channel frequency analyzer. Figure 4 shows two spectra and the corresponding Coherence Function.

The results of the preliminary tests were:

- The maximum floor vertical acceleration amplitude was $a = 0.25 \dots 0.3 \text{ m/s}^2$, which is roughly ten times larger than usually rated as acceptable [1],
- The first two natural floor frequencies are $f_1 \approx 4.15 \text{ Hz}$ and $f_2 \approx 5.90 \text{ Hz}$.

This means that the floor was excited through the second and third harmonics of the forces produced by a walking person [2].

To get into a position to suggest effective rehabilitation measures, a full Forced Vibration Modal Test and subsequent Finite Element model updating were subsequently performed.

5 MAIN FORCED VIBRATION MODAL TESTS

These tests took place on a working day between 5 pm and 10 pm, i.e. after the last man working in this office had left the site. The only problem to be solved later-on was to convince the room cleaning woman showing up at 8 pm to take a day off.

5.1 The Excitation

An electrodynamic vibration generator Type APS Electro-Seis Model 400 was used to excite the floor in the vertical direction. With a 30.6 kg moving mass, a stroke of 158 mm and a power consumption of 1.2 kW the maximum force amplitude of this device is $F = 445 \text{ N}$ for frequencies in the range $f = 2.2 \dots 12.0 \text{ Hz}$. The shaker is fixed to a load-measuring platform developed by EMPA (Fig. 5). As the shaker is quite heavy compared to the dynamic force produced no firm connection with the structure to be excited is necessary. The choice of the driving point was quite straightforward: Somewhere in the region of the dynamically most active floor area (Fig. 1). As the two or three lowest floor modes were of concern only, no attempts were undertaken to place the shaker clearly outside of the mid-span region. This was also forced through the office being quite densely populated with furniture leaving open but a small corridor in the mid-span region.

The forcing function was chosen as a random signal limited to the $f = 0 \dots 20 \text{ Hz}$ frequency band. This was produced by the software package used for signal acquisition and processing.

5.2 Response Measurements

Vertical floor vibrations induced by the shaker operation were measured as accelerations in 72 points. A measurement unit consisted of a piezo-type accelerometer (sensitivity: 10 V/g, resolution: 10^{-6} m/s^2) mounted on a supporting aluminum plate. In contrary to the preliminary tests, the

accelerometers were now fixed to the floor underside, i.e. directly to the steel beams (Fig. 6). As mentioned above, the furniture population in the office was quite dense. In addition, with the measurement point grid being closely connected to the steel beams (Fig. 7), this was the easier and better solution. Ladders were however required to reach the measurement points. As drilling of holes into steel beams is not very popular with the owners, the supporting aluminum plates were rigidly clamped to the beam's flanges with the help of screw-clamps (Fig. 6). Eight such measurement units were simultaneously in operation. This helped to keep the total testing time at a reasonable level.

Subsequent to the measurements in vertical direction, measurements in the two horizontal directions were performed in points No. 52 and 28 in the transverse and in point No. 36 in the longitudinal direction.



Fig. 5 The electrodynamic vibration generator.

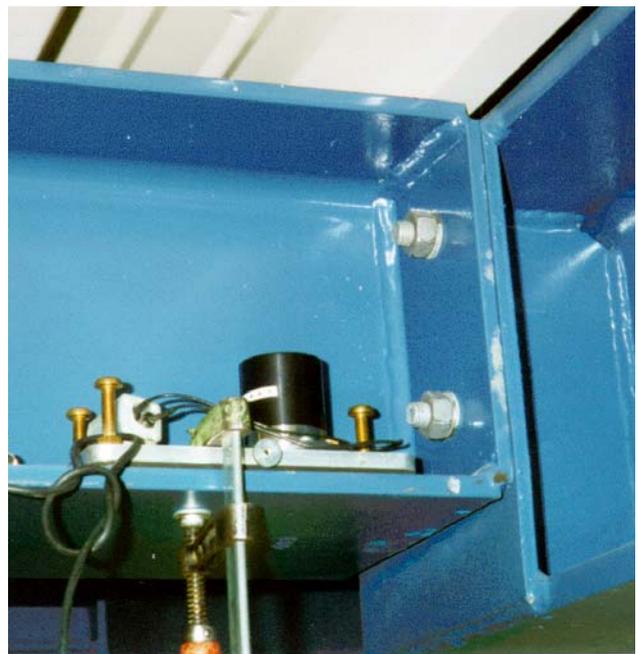


Fig. 6 An accelerometer fixed to a cross beam flange.

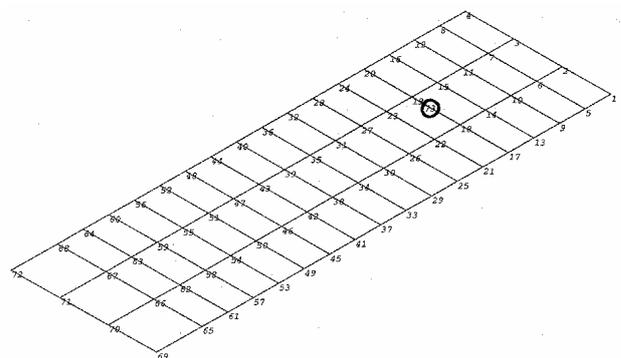


Fig. 7 The measurement point grid (axis 10 to the left). The driving point No. 73 is encircled.

5.3 Signal Acquisition and -processing

An EMPA-developed amplifier provided the signal necessary to drive the accelerometers and amplified the incoming signals. Subsequent signal conditioning (anti-alias filtering, after-filtering amplification), 16-bit-digitization and signal acquisition was performed with the help of a DIFA/SCADAS front-end controlled by a HP 725/100 computer and using CADA-X.

The continuous band-limited random-type shaker lead signal covering 100% of the time window was generated using the DIFA/SCADAS front-end. The signals from eight response measurement points plus the driving point force signal were acquired simultaneously, Fourier transformed and recalculated to obtain a leakage-free estimate of the frequency response functions.

5.4 Results of Modal Parameter Estimation

Modal parameters of the floor were estimated in two steps using CADA-X. Pole values (damped natural frequencies, damping ratios) and modal participation factors were calculated using the Least Square Complex Exponential Algorithm for the single input case. A helpful method for the evaluation of the number of physical modes is the Complex Modal Indicator Function calculated from all measured FRF's (Frequency Response Functions) as given in Figure 8. The maxima of this function indicate the natural frequencies of the floor. The three modes of the floor, as indicated by the Complex Modal Indicator Function significantly contribute to the dynamic response of the floor in the selected frequency range $f = 1 \dots 15$ Hz. This was confirmed by an exact analysis based on non-linear weighted residues of the driving point FRF in this frequency range.

The natural frequencies and damping ratios were extracted from the measured FRF's. The accuracy of the estimated natural frequencies is within 3% and the accuracy of the estimated critical damping ratios within 6%. The quality of the estimated modal parameters was investigated by correlating the measured and synthesized FRF's on the basis of estimated modal parameters. The correlation coefficient between these functions exceeds 96%. The values of the natural frequencies and corresponding damping coefficients given in percent of critical are presented in Table 1 and Figure 9.

Table 1 Natural frequencies, damping and mode shapes of the floor investigated.

Mode Nr.	Frequency f [Hz]	Damping ζ [%]	Mode Characteristics
1	4.29	2.76	Vertical bending no nodes in the longitudinal direction
2	4.83	2.74	Vertical bending; one node in the longitudinal direction
3	5.83	3.41	Vertical bending; two nodes in the longitudinal direction

For all modes, the relative immobility of the floor in the area 10/11/K/L where interior walls exist is obvious. It becomes also clear from the mode shapes that the cross beam in axis 13 does not act as a stiff support but shows large relative movements.

The measurements performed in the two horizontal directions yielded that these are comparatively small and do not influence the floor vertical bending behavior.

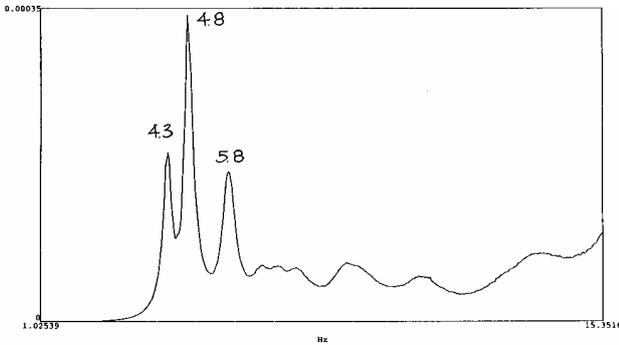
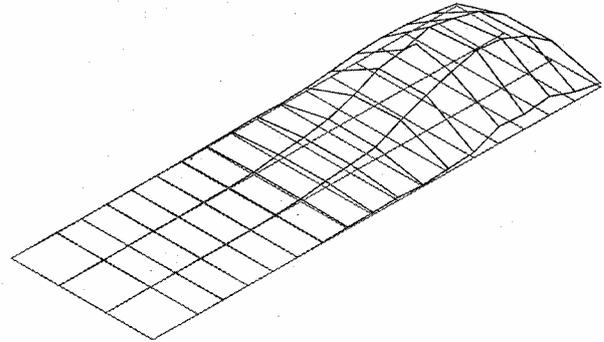
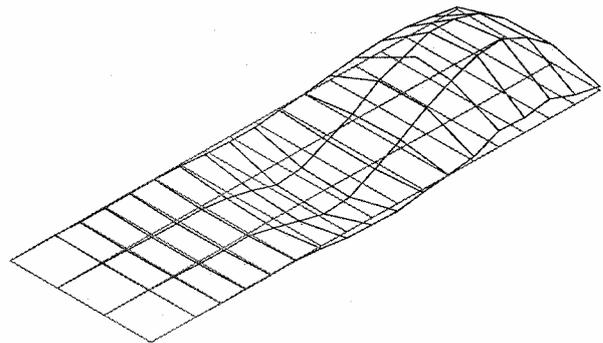


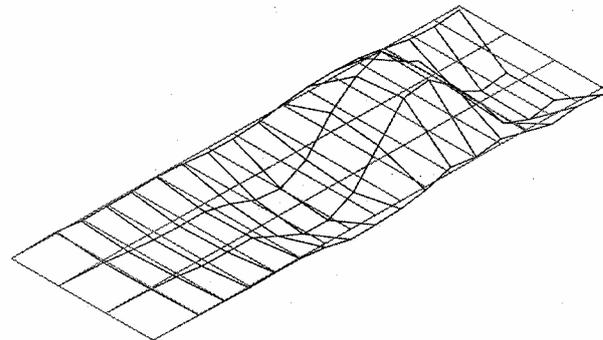
Fig. 8 The Complex Modal Indicator Function.



$$f_1 = 4.29 \text{ Hz}, \zeta_1 = 2.76\%$$



$$f_2 = 4.83 \text{ Hz}, \zeta_2 = 2.74\%$$



$$f_3 = 5.83 \text{ Hz}, \zeta_3 = 3.41\%$$

Fig. 9 Modes No. 1 to 3 as determined experimentally.

6 FINITE ELEMENT MODELING AND UPDATING

Contrary to the usual procedure no preliminary FE model was generated in advance. The first model generated with the MARC software package included all the main structural components, i.e. the steel columns and steel girders, using beam elements of type 52 (Fig. 10). The cross sectional beam parameters were taken from corresponding tables. The bottom ends of the columns were assumed as rigidly fixed. The natural frequencies obtained with this quite coarse model lay far outside the acceptable range. On the other hand some unrealistic modes were found. It became clear that the simulation quality achieved was too poor and that a finer modeling was indispensable. One of the possible improvements was to include the exterior as well as the interior walls and the roof into the model. In addition, a more detailed modeling of the boundary conditions was advisable.

To represent the interior and exterior walls, plate elements of Type 75 were easily added to the model. However, a special problem had to be solved to model the roof, which consists basically of steel girders and cover plates. Inclusion of these elements as such yielded a number of low-frequency natural vibration modes of the roof structure which were of no interest at all. A successful way to properly simulate the roof was deleting the roof construction from the model and replacing its effects through springs attached to the upper edge nodes of the plates representing the interior and exterior walls. After some trial steps the stiffness coefficients of these springs were set to $k = 5 \cdot 10^6$ N/m in both horizontal directions.

The final FE model consisted of a total of 595 elements (242 Type 75 plate and 353 Type 52 beam elements, Fig. 11). The Young's moduli were chosen as follows:

- Steel 200,000 N/mm²
- Floor topping 200 N/mm²
- Exterior walls 1,000 N/mm²
- Interior walls 5,000 N/mm²

This FE model enabled to correlate all three significant modes measured by experimental modal analysis. The correlation delivered FEA/EMA-eigenfrequency differences of up to 15 % and MAC (Modal Assurance Criterion) values within the range 60...75 %.

To achieve a better agreement between the simulation model and reality, potential parameter modifications were investigated as a next step. Since the detailed construction of the floor topping and the walls was not exactly known, the corresponding material densities were the main parameters to be optimized using the *FEMtools* software package. The sensitivity analyses performed for several parameter selections proved this estimation to be true. It is worthy to mention that the office floor stationary live load represented e.g. by furniture certainly influences the overall floor mass. This was considered only very coarsely in the previous modeling, and this should be taken into account by these density variations. During the system mass optimisation steps, automatic changes in density of up to 20% were permitted in groups of plate elements.

The approach to improve the correlation quality was not always straightforward. Once again, the "Model Updating" was primarily an engineering task (definition of the system and its boundaries). The subsequently applied automatic updating routines could bring limited remedies only, played therefore rather a secondary role and were stopped after a reasonable agreement between experiment and FE-analysis, $MAC \approx 80\%$, was achieved.

The results of the model updating process can be taken from Table 2 and Figures 12 and 13. These were judged to be good enough to make use of the model to assess the effectiveness of rehabilitation measures.

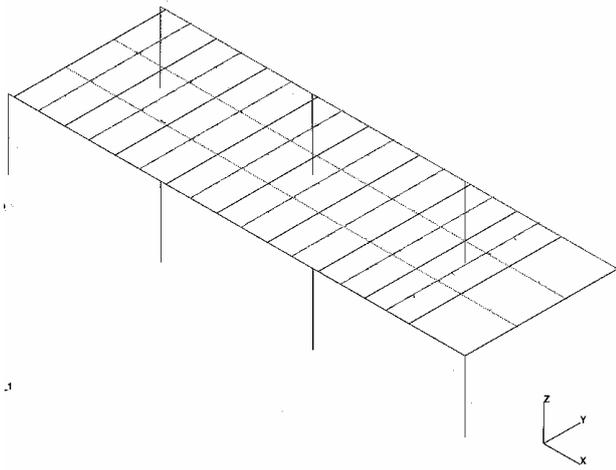


Fig. 10 The first version of the Finite Element Model consisting of steel elements only (axis 10 to the right).

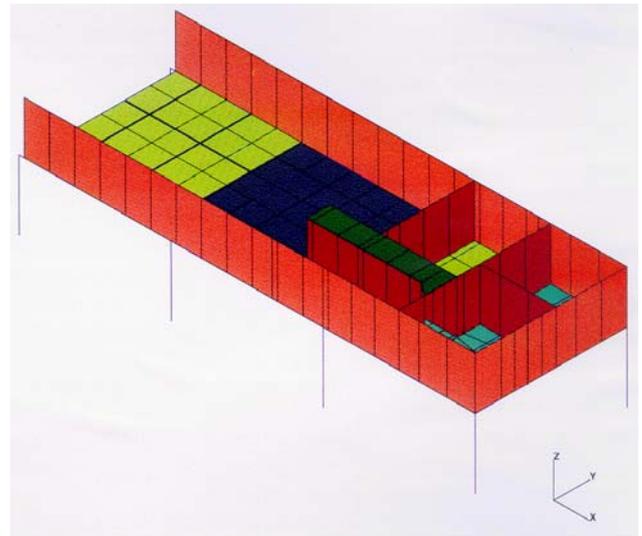
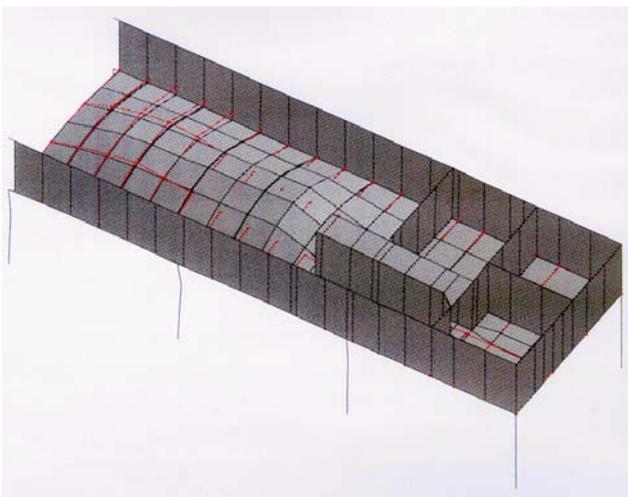


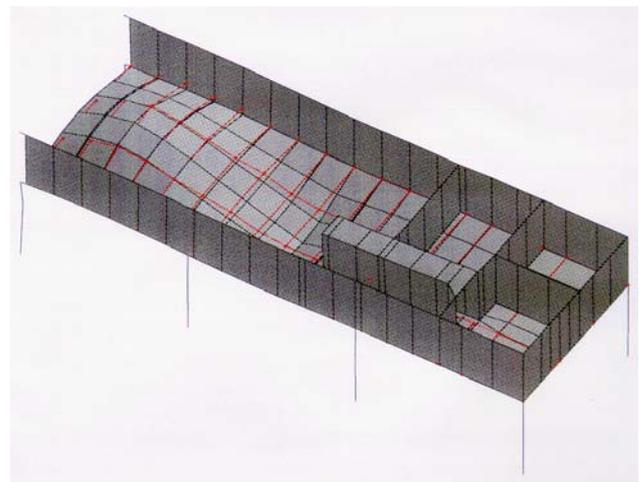
Fig. 11 The updated Finite Element model covering the steel construction as well as exterior and interior walls (axis 10 to the right).

Table 2 Comparison of the shapes of the lowest three floor modes as determined experimentally (EMA) and analytically (FEA).

Mode pair	EMA Nr.	Freq. [Hz]	FEA Nr.	Freq. [Hz]	MAC [%]
1	1	4.29	1	4.3	81
2	2	4.83	2	4.7	77
3	3	5.83	3	6.6	79

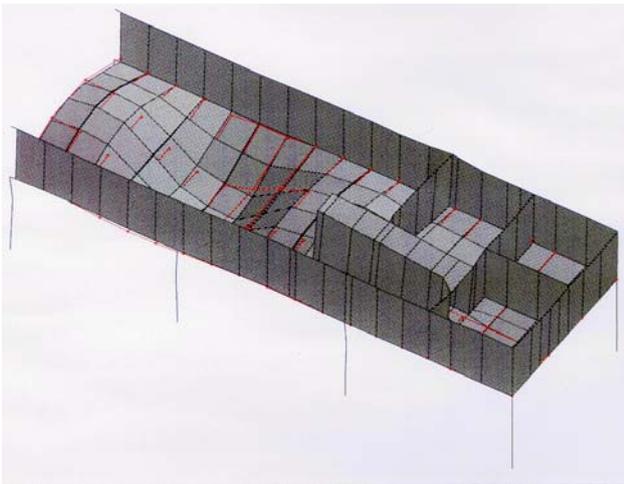


Mode 1, MAC = 81%



Mode 2, MAC = 77%

Fig. 12a Modes Nos. 1 and 2 as determined experimentally and analytically (axis 10 to the right).



Mode 3, MAC = 79%

Fig. 12b Mode No.3 as determined experimentally and analytically.

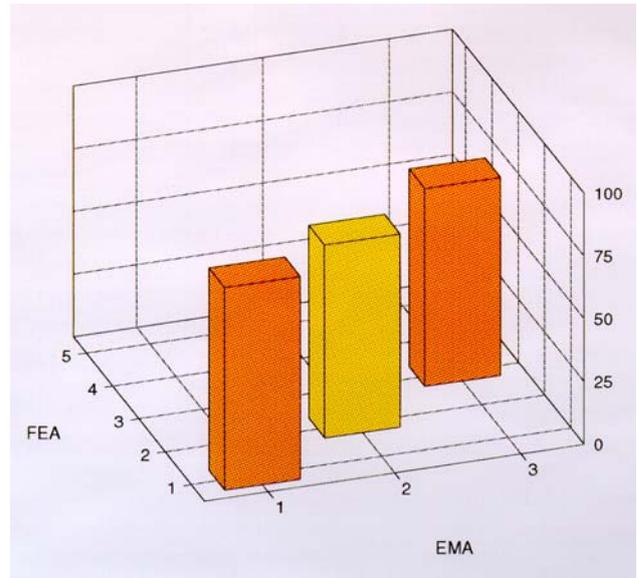


Fig. 13 The MAC-matrix.

7 REHABILITATION MEASURES / EVALUATION USING FE ANALYSES

As the vibration problem at hand could not be solved by influencing the forcing processes two possible solutions remained:

- Stiffening of the structure such as to increase its fundamental natural frequency to $f > 9$ Hz,
- Placing of passive tuned mass dampers.

Placing of tuned mass dampers was soon rejected due to the following reasons: a) It would have been necessary to install three dampers tuned to each of the natural floor frequencies being in the critical range, b) the effectiveness of a damper is the better the lower the damping of the vibration to be suppressed is (the optimum value $\zeta = 1\%$ is exceeded by all floor natural vibrations), c) such dampers require maintenance and are not noise-free when operating.

The structural changes and their effect on the fundamental floor natural frequency as determined using the updated FE model are given in Table 3. Three of the solutions are promising.

Table 3 The rehabilitation measures investigated and their effect on the floor fundamental natural frequency.

Rehabilitation measure	Fundamental floor frequency f [Hz]
Addition of 7 intermediate columns (Fig. 14)	13.04
Addition of 6 intermediate columns; effectiveness depending on which one of the 7 columns is omitted	7.08...11.21
Addition of 4 intermediate columns; omitting every second of the 7 columns	7.09
Increasing the cross beam stiffness by a factor of 4	7.56
Addition of a stiff longitudinal beam plus three intermediate columns (axes 10, 11 and 12)	11.4

Addition of an interior wall in longitudinal direction between the axes 13 and 12	5.0
Addition of two interior walls in cross direction between the axes 13 and 12	5.8
Addition of two interior walls in cross direction plus stiffening of the cross beam in axis 13	6.5

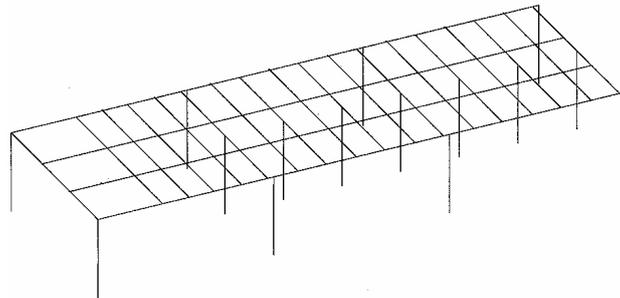


Fig. 14 Rehabilitation through adding of seven columns (compare with Fig. 10, but axis 10 now to the left).

8 DISCUSSION

Excessive vibrations of floors loaded by walking people are inasmuch simple problems to solve as the forcing function is quite well known. The first task is to determine the floor natural frequencies f_i . For lightweight structures with f_i being in the range of the first three harmonics of the forcing function there is then usually no other solution than to stiffen the floor until its natural frequency is $f > 9$ Hz. The most effective but not always practical way to reach this goal is addition of columns. Stiffening of the floor itself is cumbersome because any stiffening measure also increases the floor mass which is contraproductive.

Considering the means necessary to get enough insight into the problem to be able to solve it: In the case presented in this paper the low-cost version using two accelerometers and a 2-channel analyzer would not have been appropriate. A complete modal test covering the whole structure yielded the information necessary to generate and update a Finite Element Model of the quite complex structure. This model then served as a reliable basis to study the effectiveness of possible structural modifications. Such a test can be performed in a couple of hours and is definitely worth the money spent. The optimisation of the Finite element model and its use for parametric studies opens new possibilities for efficient evaluation of potential rehabilitation measures. These additional investigations are possible in a reliable manner using the software tools available today with practically acceptable effort and increase the proper use of test results substantially. The combined use of experimental and analytical approaches as described above is successfully applicable for any similar cases.

9 REFERENCES

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