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"Vibration measurements and their assessment concerning the effect on a masonry chimney"

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Contents

Abstract (italiano)

Questa tesi si occupa della misurazione delle vibrazioni di un fumaiolo industriale nella piccola cittadina di Bretten durante i lavori di demolizione degli edifici circostanti. Il monitoraggio di eventi simili svolge sempre un ruolo importante nella protezione della popolazione e degli edifici stessi. Esistono diversi tipi di oscillazioni che dipendono dalla fonte di vibrazioni: possono derivare da traffico stradale e ferroviario, macchine e dalle persone stesse.

Per alcuni edifici, le previsioni di vibrazione devono essere determinate per prendere misure precauzionali. Questi possono essere eseguiti alla fonte, nel percorso di propagazione e nel sito di immissione sull'edificio. Alcuni autori hanno creato diversi modelli con diversi gradi di difficoltà per determinare gli effetti delle oscillazioni il più preciso possibile. Questi modelli presentano però un elevato coefficiente di sicurezza, perché presentano diverse incertezze nella determinazione di diversi parametri.

La norma tecnica tedesca DIN 4150 si occupa della valutazione delle vibrazioni sulle persone e sugli edifici. I valori di riferimento, che dipendono dalla frequenza dell'effetto, indicati dalla DIN 4150-3 sono dei limiti in cui è sicuro che non si verificheranno danni sugli edifici, nel caso in cui non vengono superati. Se questi vengono comunque oltrepassati, non può essere garantito, che non nascono delle fessure e che portano al cedimento degli stessi. L'esempio di Bretten dimostra tuttavia che ciò non sempre avviene.

Abstract (english)

This master thesis deals with the vibration measurement of a chimney in Bretten (Germany) during the demolition work of the surrounding buildings. The monitoring of these events always plays a major role in the protection of the population and the buildings themselves. There are different types of oscillations that depend on the vibration source: they can originate from road and rail traffic, machines and people themselves.

For some buildings, vibration prognoses must be determined in order to take precautionary measures. These can be carried out at the source, the propagation path and the immission site. Different authors have produced different models with different degrees of difficulty in order to determine the effects of the shocks as accurately as possible. These models have a high safety coefficient, because they have several inaccuracies.

The German DIN 4150 deals with the assessment of the vibrations on people and buildings. The stated frequency - dependent reference values of DIN 4150 - 3 are values

where one can be sure that no damage will occur. If this is exceeded, this cannot be guaranteed, so that cracks may form. Therefor the chimney in Bretten is a good example.

Abstrakt (deutsch)

Diese Masterarbeit befasst sich mit der Erschütterungsmessung eines Schornsteines in Bretten während der Abbrucharbeiten der umliegenden Gebäude. Das Monitoring dieser Ereignisse spielt immer eine größere Rolle, um die Bevölkerung und die Gebäude selber zu schützen. Es gibt unterschiedliche Arten von Schwingungen, die von der Erschütterungsquelle abhängen: sie können vom Straßen- und Schienenverkehr, von Maschinen und von Menschen selber stammen.

Für manche Bauten müssen Erschütterungsprognosen durchgeführt werden, um rechtzeitig Minderungsmaßnahmen ergreifen zu können. Diese können an der Quelle, am Ausbreitungspfad und am Imissionsort durchgeführt werden. Verschiedene Autoren haben unterschiedliche Modelle mit unterschiedlichem Schwierigkeitsgraden erstellt, um die Effekte der Schwingungen zu bestimmen. Diese Modelle haben einen hohen Sicherheitskoeffizienten, da sie mehrere Ungenauigkeiten beinhalten.

Die deutsche DIN 4150 befasst sich mit der Beurteilung der Schwingungen an Menschen und Gebäuden. Die angegebenen frequenzabhängigen Anhaltswerte der DIN 4150 – 3 sind Werte, die sicherstellen, dass bei ihrer Einhaltung keine Schäden entstehen. Beim Überschreiten dieser Anhaltswerte müssen aber nicht zwangsweise Schäden z.B. Risse entstehen. Dies zeigt auch das Beispiel in Bretten

1. Introduction

Nowadays dynamic loads play an important role in assessing the load bearing capacity of each construction, especially in vibration-sensitive structures, i.e. those with low mass and high slenderness. This is a big problem because the technology is so far that light, material-saving buildings can be build. In addition, today's society is demanding more and more environmental protection, i.e. greater protection against sounds and vibrations. For this reason, a pure static design isn’t sufficient to have a good construction and dynamic processes have to be considered.

In the course of time, several norms were defined for the definition, measurement and evaluation of vibrations on people and buildings. The German DIN 45669 of September 2010 deals with the measurement of vibrations. It consists of two parts: the first part is more about the vibration measuring devices and their requirements; in the second part the measurement method itself is explained. The German DIN 1311 of February 2000, on the other hand, describes the oscillating systems. It consists of five parts:

1. Description of the basic concepts
2. Vibrating systems with a degree of freedom
3. Vibrating systems with finitely many degrees of freedom
4. Linear continuum, waves
5. Vibrations of non-linear, self-excited and parameter-excited systems

As a result of the increasing population, the building density and the demand for services are both growing. It is therefore often inevitable that buildings are situated in locations, that are subject to vibration. These places can be near the railway tracks, above the underground, near industrial facilities and road traffic. All these can be sources of vibration, where dynamic energy has been introduced into the soil, in the form of soil waves. The geometrical boundary conditions, such as embankments and incisions, have a high influence on the propagation of vibrations. In a model, one must also know the type of vehicles and the machines, as well as the condition of the superstructure and the substrate.

The DIN 4150 norm has thus been developed. It consists of three parts: the first part deals with the preliminary determination of vibration variables. In the second part, the effects on people in buildings are described and the third part deals with the effects on the buildings.

The second part of the norm deals with the limitations of the vibrations, in order to avoid for humans feeling uncomfortable. Because of certain shocks, the internal organs of the human being can come into resonance with it, which means that their response is decisively amplified. This causes to a person insecurity, and moreover the need to run away from the structure because, among other things, they are afraid that the structure could break down. However, in order to prevent damage to a structural system, special requirements of DIN 4150-3 must be observed. These were determined by older measurements. If they aren't exceeded, then there is a high probability that the building has no damage. If it still has it, there are often other reasons behind it. If the indications are exceeded, the integrity or stability can no longer be guaranteed.

Even during various construction methods of civil engineering and earthworks, vibrations can be introduced into the ground. These spread into the subsoil and are transferred to nearby buildings. This can also lead to nuisances of the inhabitants and to the interference of the building fabric. For this reason, a vibration prognosis must be carried out before every use of vibration-intensive construction methods. In critical cases, vibration measurements should be provided.

Demolition work can cause short-term vibrations due to falling components. For many decades they have been carried out without any appreciable attention. This fact resulted, among other things, from the lack of knowledge and underestimation of this work. The number and importance of the safety regulations to be observed during the demolition are no less than in the construction of buildings. Often, the duration of the demolition is significantly lower, which means that the safety regulations must be implemented quickly and in a complex manner. In these cases, one is often concerned with materials which have a partial lower load-bearing capacity than they have been used because they have been damaged in time, e.g. the reinforcement can be weakened by corrosion attacks. The demolition industry and its technology have developed considerably in recent years in order to take into account the needs of the inhabitants and the surrounding buildings. The demolition technology is nowadays very highly technical and greatly increased the qualification requirements for the demolition specialists to guarantee this.

In the literature, several authors have dealt with the subject of vibration, and several models with different degrees of difficulty have been created for the transmission path and the source. While in the static calculations the material stress with the strains and stresses predominantly takes centre stage, the structural dynamics verification is more about the oscillation paths, oscillation velocities and accelerations. The problem is that the propagation path of the waves passes through several different materials. This would require knowing the material properties of all the substances involved, in order to create as accurate a calculation model as possible. As someone can guess, this can lead to elaborate models in the presence of several materials and soil layers. This is based on complex integral equations. Their analytical solution is only possible for a few simple problems. It was only through the introduction of computing systems that these integrals could be solved numerically and with boundary value problems and initial problems.

Not only the vibration prognosis was very important, but also the measurement of the oscillation plays an important role. As a result of technological progress, measuring instruments have become more and more accurate and reliable. Previously, mechanical measuring instruments with analogue data transmission were used. Today, however, measuring devices are used, where the measured data are passed on to a computer by means of electrical signals. This can convert these electrical signals into usable values. In addition, it is now possible to query the data via wireless network, so an expert witness isn't forced to drive to the spot and to take these values, but he can call them directly from his own office. In addition, the measuring instruments should be connected with an acoustic sound so that the occupants and workers can be alerted as soon as the indications according to DIN 4150 are exceeded.

This work deals, among other things, with the demolition work of an old industrial building near an old chimney, which is to be preserved. In this case, measuring instruments were used which alerted the demolition worker as soon as the indications were exceeded. The Materials Testing and Research Institute MPA Karlsruhe of the Karlsruhe Institute of Technology (KIT), was asked to develop and to use a new system for this project to storing events separately. The author has written a software program with National Instruments software DIAdem, which places different files as soon as a shock occurs (chapter 5). Furthermore, in the same chapter typical measuring devices are described which measure the oscillation distance, the speed or the acceleration of the waves. However, before starting a measurement, you need to know what a vibration is. Chapter 2 defines, among other things, the different vibration modes and the dynamic parameters of the ground. Chapter 3 shows different sources of vibration, and in Chapter 4 the wave propagation and transmission to the building. In addition, the soil-building interaction is briefly described here. Chapter 6 shows the DIN 4150-3, which is necessary to calculate the indications for the alarm triggering. Subsequently, the chimney is described in detail in chapter 7 and the evaluation is carried out.

2. Principles for describing the vibrations

This chapter describes the basic principles and parameters in order to be able to handle the vibrations more precisely.

2.1. The oscillation

A vibration is a time, repetitive fluctuation from a state variable of a system. That means it is a deviation from an average value. There are different types of vibrations:

- Harmonic oscillation (Fig. 2.1)
- Periodic vibration (Fig. 2.2)
- Transient oscillation (Fig. 2.3)

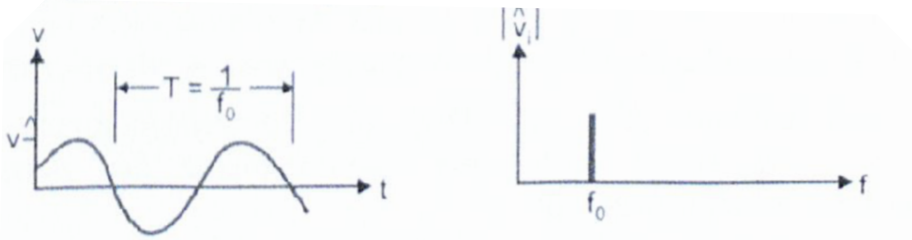


Fig. 2.1: harmonic oscillation and related amplitude spectrum

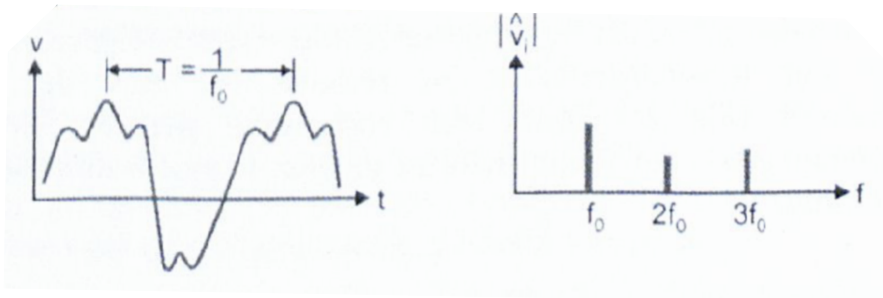


Fig. 2.2: periodic vibration and related amplitude spectrum

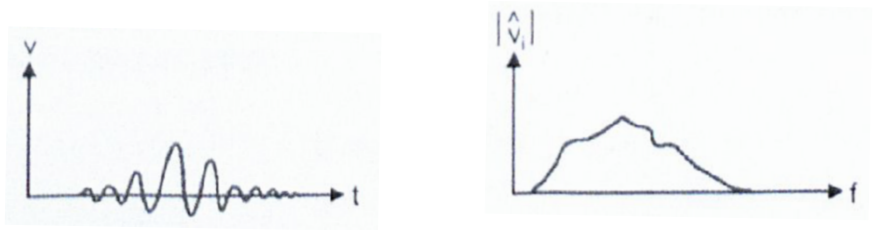


Fig. 2.3: transient oscillation and related amplitude spectrum

The simplest form of vibration is the harmonic oscillation which is described with the amplitude u (in this case the path), the angular frequency ω and the phase shift angle φ and it is a function of the time t:

$$u(t) = u \cdot sin(\omega t + \varphi)$$

The duration of a vibration is given by the period $T = 2\pi / \omega$ and from this the frequency f can be determined as the reciprocal of the period. The vibration velocity v(t) and the oscillation acceleration a(t) are important parameters to describe a harmonic oscillation. They can be calculated as follows:

$$v(t) = 2\pi \cdot f \cdot u(t)$$

$$a(t) = 2\pi \cdot f \cdot v(t)$$

The oscillation speed doesn't correspond with the propagation speed v_A. The latter depends, inter alia, on the properties of the medium being passed through. If this velocity has been determined, the wave length of the oscillation λ can be calculated like:

$$\lambda = \frac{v_A}{f}$$

The effective real vibrations that can be recorded in buildings are either periodic or transient and nonperiodic oscillations, which can be seen as the result and superposition of individual vibrations. The vibration intensity in buildings depends strongly on the frequency of the stimulating ground vibrations. Therefore, an amplitude spectrum is often represented as a diagram which reports information on the frequency content of a vibration. As can be seen in Fig. 2.1, Fig. 2.2 and Fig. 2.3, a harmonic oscillation results in a line spectrum, a continuous spectrum being calculated in a nonperiodic oscillation.

If a static system such as a building or a construction part is excited to vibrate, the response of this system depends on the frequency of the excitation. Depending on the characteristics of the system, an amplitude reduction or amplitude increase can be achieved.

This process is described with a wave transfer factor $k^{ü}$ which is the ratio between the amplitude of the oscillations of the system under consideration and the exciting vibrations:

$$k^{ü} = \frac{v_{Sis}}{v_{exc}}$$

These factors are system- and frequency-dependent.

Taking as an example a muted one-mass oscillator with harmonic excitation, there are three parameters that influence this system:

- the mass m
- the stiffness value k
- the damping factor ζ

These parameters can be used to determine the eigenfrequency f_{Eigen}:

$$f_{Eigen} = \frac{1}{2\pi}\sqrt{\frac{k}{m}\sqrt{1-\zeta^2}}$$

An amplitude increase is achieved by excitation with the eigenfrequency. The wave transfer factor depends on the damping factor ζ and its maximum value is obtained in the case of resonance with the following formula:

$$k^{ü}_{max} = \frac{1}{2\zeta\sqrt{1-\zeta^2}}$$

During the transmission of vibrations from the ground to the building foundation, it is possible to set $\zeta \geq 0{,}25$ for unconsolidated rocks, from which one obtains $k^{ü}_{max} \leq 2$. On the other hand, in the case of the transmission from vertical components to a reinforced concrete floor, the damping factor is between $0.02 \leq \zeta \leq 0.05$. This results in a wave transfer factor of $10 \leq k^{ü}_{max} \leq 25$. In the case of resonance, therefore, there are strong amplitudes increases.

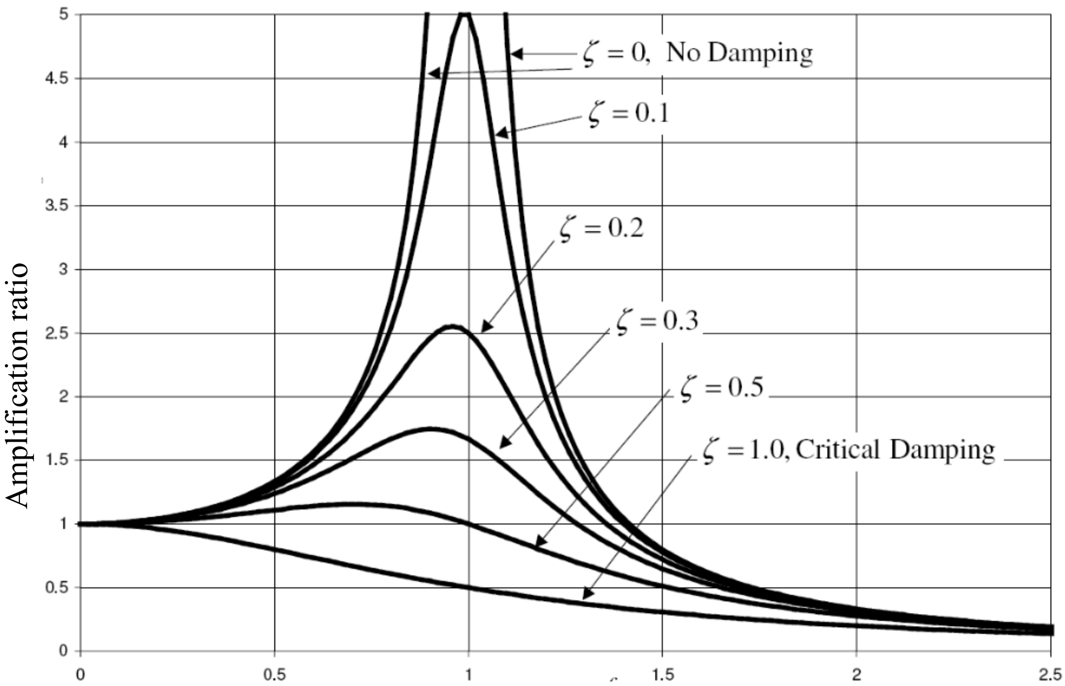


Fig. 2.4: transmission behaviour of a damped one-mass oscillator

2.2. The ground parameter

In addition to the damping factor ζ, further soil parameters are necessary to describe the wave propagation in the ground: the dynamic shear modulus G, the transverse contraction number ν and the dynamic elastic modulus E. The stress-strain behavior of the soil can be illustrated by a dynamic shear stress. In the stress-strain diagram, a hysteresis loop is thus formed. A measure for the attenuation is the dissipated energy per cycle of this loop, whereby its surface content is

$$\frac{1}{2}\tau_{max}\gamma_{max}$$

The shear module is the secant module of the maximum shear stress. It shrinks with increasing of the shear stress or with increasing of the shearing strain. This is illustrated in Fig. 2.5

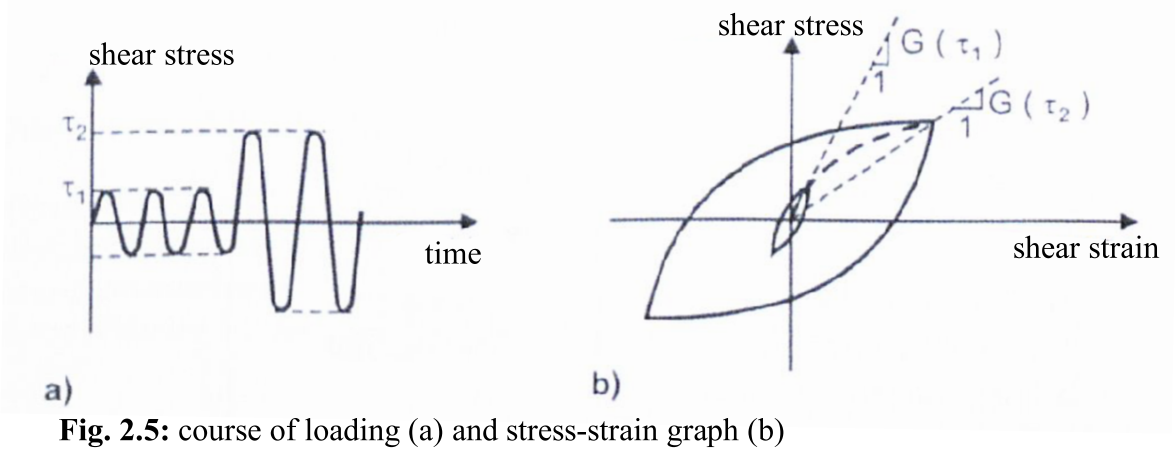


Fig. 2.5: course of loading (a) and stress-strain graph (b)

As can be seen, the maximum shear modulus G_{max} is obtained with very small shear distortions. The distortion is e.g. in the case of shocks caused by ramming systems or vibration compressions under 10^{-3}%. It means that the maximum shear modulus G_{max} can be used in the models. This modulus is determined for non-cohesive soils as a function of the bulk density D_r and the mean effective ground tension σ'_m:

$$G_{max} = K(D_r)\sqrt{\sigma'_m}$$

For cohesive soils, the shear modulus is given as a function of the undrained shear strength c_u: the values of G_{max} are between 1.500 and 2.500 c_u.

Table 2.1a and Table 2.1b lists the dynamic shear modulus of different soil types.

Soil type	**G_{max} [MN/m²]**
Non-cohesive soil	
Sand, looser	50 – 120
Sand, medium density	70 – 170
Gravel, sandy, dense	100 – 300
Cohesive soil	
Clay, soft to stiff	20 – 50
Clay, semi-hard to hard	80 – 300

Table 2.1a: reference value for dynamic shear modulus

Soil type	**G_{max} [MN/m^2]**
Rock	
Rock, layered, brittle	1.000 – 5.000
Rock, massive	4.000 – 20.000

Table 2.1b: reference value for dynamic shear modulus

The transverse contraction number ν for dynamic effects can generally be equated with the number of static effects. Their influence on the vibration propagation processes is comparatively low. Reference values are listed in Table 2.2:

Soil type	**ν [-]**
Sand and gravel	0,25 – 0,35
silt	0,35 – 0,45
Clay	0,45 – 0,49

Table 2.2: reference value for transverse contraction number

While the shear modulus decreases with the increment of the shearing stress and shear strain, the damping factor ζ increases. In general, the value ζ for cohesive soils is somewhat greater than the value ζ for non-compliant soils. If, moreover, it is below a limit which corresponds to the limit value for the maximum shear module, the damping factor ζ remains constant.[1]

2.3. *The Fourier-Transformation*

Periodic and non-periodic oscillations can be regarded as an infinite sum of harmonic oscillations. This sum can be represented by means of a Fourier series in the first case or Fourier integral in the second case. If, for example, a Fourier series is taken, the following decomposition can be carried out for the representation of a periodic vibration with the period T:

$$x(t) = a_0 + \sum_{n=1}^{\infty} [C_n \cdot \cos(n\omega t) + S_n \cdot \sin(n\omega t)] = a_0 + \sum_{n=1}^{\infty} a_n \cdot \cos(n\omega t - \alpha_n)$$

thereby is:

$$a_0 = \frac{1}{T} \int_0^T x(t) dt$$

[1] M. Achmus, J. Kaiser, F. tom Wörden; Bauwerkserschütterungen durch Tiefbauarbeiten – Grundlagen, Messergebnisse, Prognosen; p. 1 - 9

$$C_n = \frac{2}{T}\int_0^T x(t) \cdot \cos(n\omega t)\, dt$$

$$S_n = \frac{2}{T}\int_0^T x(t) \cdot \sin(n\omega t)\, dt$$

$$\tan\alpha_n = \frac{S_n}{C_n}$$

$$a_n = \sqrt{C_n^2 + S_n^2}$$

The individual harmonic oscillations each have the circular frequency $n\omega$, that means, they are an integer multiple of the fundamental frequency ω. a_0 is a constant value, which plays only a subordinate role in the building dynamics. It describes, among other things, the static, time-independent part of the process, or in other words, it is that burden by the mass which causes the initial deviation. a_n is the amplitude of the individual harmonic oscillations. For each harmonic, the frequency f_n can also be determined as follows:

$$f_n = \frac{n\omega}{2\pi}$$

If one knows the amplitude and frequency of the individual partial oscillations, then a discrete amplitude spectrum can be created, where the x-axis represents the frequency and the y-axis the amplitude. From this, the frequency with the highest amplitude is determined, because it characterizes the total vibration.

If, on the other hand, non-periodic oscillations such as earthquakes or winds are examined, a Fourier integral is used instead of a Fourier series. The corresponding formula are as follows:

$$x(t) = \frac{1}{2\pi}\int_0^\infty [C(\omega) \cdot \cos(\omega t) + S(\omega) \cdot \sin(\omega t)]d\omega$$

With

$$C(\omega) = 2\int_{-\infty}^{\infty} x(t) \cdot \cos(\omega t)\, dt$$

$$S(\omega) = 2\int_{-\infty}^{\infty} x(t) \cdot \sin(\omega t)\, dt$$

In this case, the amplitude spectrum is continuous, and the amplitude can be determined as follows:

$$a(\omega) = \sqrt{C(\omega)^2 + S(\omega)^2}$$

After the dimension of the spectrum is different from the dimension of x(t), one often speaks of an amplitude density spectrum. The core statements always remain the same: a frequency with a large amplitude density characterizes the oscillation. With this, the natural frequencies of components can be determined with the aid of a vibration measurement since these have a very marked amplitude. [2]

It should be noted that different types of Fourier-transformations exist. For example, there are the Discrete Fourier Transformation (DFT). It forms a time-discrete finite signal, which is periodically continued, to a discrete periodic frequency spectrum. It is a special form of a signal, which is defined only to certain, as a general rule at equidistant times. It is obtained from a time-continuous signal where a value is taken from this time-continuous sequence at specific points in time. Mathematically speaking, a time-discrete signal can be described as a sequence x[n] of real numbers with $n \in \mathbb{N}$. The index n represents, among other things, the time variable normalized to the sampling rate.

The Discrete Fourier Transformation processes a sequence of numbers $a = (a_0, \ldots, a_{N-1})$ by representing them as values $a_k = A(z_k)$ of a polynomial with complex coefficients:

$$A(z) = \frac{1}{N}(a_0 z_0 + a_1 z_1 + \ldots + a_{N-1} z_{N-1})$$

where the arguments $z_0, z_1, \ldots, z_{N-1}$ are N evenly distributed points on the unit circle of the complex number plane:

$$z_k = e^{\frac{2\pi i}{N}k} = \cos\left(\frac{2\pi}{N}k\right) + i\sin\left(\frac{2\pi}{N}k\right)$$

The polynomial A(z) can then be represented as a time-continuous periodic function by linking it to a function z(t) uniformly rotating around the unit circle:

$$z_k(t) = e^{k \cdot 2\pi i \frac{t-t_0}{T}} = \cos\left(2k\pi\frac{t-t_0}{T}\right) + i\sin\left(2k\pi\frac{t-t_0}{T}\right)$$

The period in this case is T/k and the corresponding frequency k/T.

This numerical transformation can often take a long time. In order to accelerate them, the Fast Fourier Transformation (FFT) can be used. Here, previously calculated intermediate results are reused, thereby arithmetic computing operations are saved. There are different methods of the FFT, which bear the name of their developers. The best known method is by James Cooley and John W. Tukey. This algorithm is a typical divide-and-conquer method, where the problem is recursively decomposed into smaller and simpler sub-problems until it can be solved. The solutions to the overall problem can then be defined from these partial solutions. In order to be able to use this method, the sampling points must be a power of two. This shouldn't be a serious limitation in the measurement

[2] Prof. Dr. Ing. Lothar Stempniewski, Dipl. Ing. Björn Haag; Baudynamik-Praxis, Mit zahlreichen Anwendungsbeispielen; p. 2-5

technology as it can be freely chosen within this framework. In these cases, one often speaks of a Radix-2-FFT.

The Cooley and Tukey algorithm decomposes the calculation of a DFT of size 2n into two computations of a DFT of size n: one with the even and the other with the odd indices. After the transformation, the two partial results must again be compiled into a Fourier transformation of the size 2n. This calculation of a half-length DFT requires only a quarter of the complex multiplications and additions of the original DFT. This leads to a significant time-saving.

In this case, the Discrete Fourier Transformation of the vector $(a_0, \dots, a_{2n-1})$ with a dimension of 2n can be represented as follows:

$$f_m = \sum_{k=0}^{2n-1} a_k e^{\frac{2\pi i}{2n} mk} \qquad m = 0, 1, \dots, 2n-1$$

From this, the entries with even $(a'_k = a_{2k})$ and odd $(a''_k = a_{2k+1})$ indices are listed separately, where $k = 0, 1, \dots, n-1$. The corresponding DFT are marked with f'_m and f''_m. Both have the dimension n. Thus the series described above can be represented as follows:

$$f_m = \sum_{k=0}^{n-1} a_{2k} e^{\frac{2\pi i}{2n} m(2k)} + \sum_{k=0}^{n-1} a_{2k+1} e^{\frac{2\pi i}{2n} m(2k+1)}$$

$$f_m = \sum_{k=0}^{n-1} a'_k e^{\frac{2\pi i}{n} mk} + e^{\frac{\pi i}{n} m} \sum_{k=0}^{n-1} a''_k e^{\frac{2\pi i}{n} mk}$$

In order to accelerate this process even more, Radix-4 and Radix-8 algorithm have been introduced. The main difference is the representation of the different number of data points to be processed with a power of 4 and 8 respectively. The processing structure remains unchanged. Only the butterfly graph is different, because each element has four or eight data paths instead of two that must be linked to one another. This illustration is the graphical representation of the transformation from the basic function of the Fourier transformation to the FFT. By this speed advantage, the Radix-4 algorithm has 25% and the Radix-8 algorithm has 40% less multiplication. [3]

[3] Apurba Das; Signal Conditioning – An Introduction to continuous Wave Communication and Signal Processsing; p. 159 - 216

3. The vibration source

Vibrations can arise for various reasons. In all construction tasks of civil engineering, e.g. during drilling, pressing of sheet piles or during excavation, surface waves are spread in the ground. However, these operations cause only minor shocks. The situation is different when it is planned to perform dynamic activities on the ground, such as the ramming and vibrating of different types of piles, or stomping and vibrating methods for surface and depth compaction.[4]

But not only construction work can lead to vibrations, also rail traffic can. It can be critical in cities, or in all other cases, where the route is close to buildings, or when it is under a structure, as can be done on subway routes. In this case there are generated space waves instead of surface waves.[5] The vibrations generated from the passage of trains cause not only disturbances to the nearby structures, but also annoyance to the inhabitants alongside. For that reason the vibration assessment is important for the correct structural functioning and for environmental reasons.[6]

The standard DIN 4150-1 also distinguishes different sources of vibration:

- Suggestions from individual events: these include all those events which are separated in time in such a way that their effects cannot come together. Their occurrence can often be fixed in time or at least influenced. In addition, measures can be taken to determine their strength. Such individual events generally don't lead to pronounced resonances of buildings or their components. The standard describes the blasting and falling masses for this type of source.
- Suggestions from building activity: these include all those methods which cause vibrations during construction work. The vibration are often characterized as pulse-shaped or stationary point sources.
- Suggestions from traffic: this includes those shocks that come from rail and road traffic. In rare case, these excitations can only be influenced by organizational measures. Frequently they are accompanied by further emissions of other species, which leads to the assessment of the overall effect.
- Suggestions from machine operation: These stimuli are characterized, among other things, by the role and design of the machines. The excitations can be periodic or also shock-like. The emission behaviour is strongly influenced by the type of installation: the machines can be located on building floors, individual foundations or on foundation slabs. The basic frequency consists of a broadband spectrum. This is determined by the rotational speed of the machine

[4] M. Achmus, J. Kaiser, F. tom Wörden; Bauwerkserschütterungen durch Tiefbauarbeiten – Grundlagen, Messergebnisse, Prognosen; p.19
[5] Verein deutscher Ingenieure; Erschütterungen – Ursache und Minderung von Störungen oder Schäden; p.71
[6] N. Chouw, G. Schmid; Erschütterungsausbreitung und Erschütterungsreduzierung; p.151

drive or by the cycle times of the moving machine components. Furthermore, as the movement of the machines is more harmonious, the spectrum will be narrower. At the same time, the shorter the duration of the shock is, the frequency band will be wider.[7]

3.1. Compaction equipment

Compaction equipment includes vibratory rammer, vibratory plates, and vibratory rollers.

3.1.1. Vibratory rammer

The vibratory rammer consists of a spring system which stores and delivers energy generated by the drive motor so that the plate reaches its highest speed when impacting the ground. By a subsequent tension of the springs in the opposite direction the device lifts up to a maximum of 80 mm. Through the spring system, the tamper runs forward, so you only have to guide it. Such a device can weigh from 25 kg to 200 kg and perform 500 to 800 beats per minute. This results in a striking and vibrating effect. Since these machines have a favourable ratio between power and dead weight, they are mainly used for small compacting work in confined areas. The energy input into the ground, which is responsible for the propagation of vibrations, can in this case be approximated as a product of dead weight and falling height.

3.1.2. Vibratory plates

Vibratory plates or agitator plates can weigh significantly more: between 100 kg and 750 kg. These devices have a larger contact area, which means that the surface pressure on the ground is less. However, by rotating shafts with imbalances, these plates are vibrated with an excitation frequency of 50-100 Hz, and centrifugal forces are produced which are greater than the dead weight. On the basis of these forces, the plate rises several millimeters and also moves forward and backward. The degree of lift-off and the resulting vertical vibration amplitude depends, inter alia, on the equipment and soil properties and on the device settings.

The maximum oscillation displacement amplitude A_0 can be calculated approximately from the centrifugal force F_0, the operating frequency ω_B and the oscillating device mass G_{dyn}:

$$A_0 = \frac{F_0 / \omega_B^2}{G_{dyn}}$$

[7] DIN 4150 – 1: 2001 – 06; Vibrations in buildings – Part 1: Prediction of vibration parameters; p. 8 – 13

From this, the maximum energy absorbed in the ground can be determined as:

$$E_{max} = \pi F_0 A_0 = \pi \frac{F_0^2}{\omega_B^2 G_{dyn}}$$

3.1.3. Vibratory rollers

Pulled and self-propelled vibratory rollers have eccentric shafts inside the bandages, which produce centrifugal forces as a function of the rotational frequency. The transmission of the vibrations to the device is avoided by rubber buffering. The vibration emission depends, among other things, on the device weight, the maximum centrifugal force, the operating frequency (typically about 50 Hz) and the maximum oscillation displacement amplitude. The vibration intensity can be set as oscillation energy per period E_0, which can be determined as the ratio of the motor power W_W and the operating frequency f_W. Depending on the type of roll and the operating condition, different components of the motor power are required. In addition, the greater the denser and stiffer the ground, the greater the amount of energy emitted in the form of elastic waves.

The ground speed at the distance r_W from the vibrating roller can be projected as follows:

$$v = 100 \frac{\sqrt{E_0}}{r_w}$$

with

$$E_0 = \frac{W_W}{f_W}$$

For tandem rollers, the twofold energy is introduced, which thus causes an increase of the vibration speed for $\sqrt{2}$ – times.

3.2. Piling implement

Piling equipment include vibrators and driving rams.

3.2.1. Vibrators

The vibrators or shakers, which ram the pile element like sheet piles, with the aid of vertical vibrations into the ground, generate stationary vibrations, such as Fig. 3.1, which is derived from DIN 4150-1: 2001-06, shows.

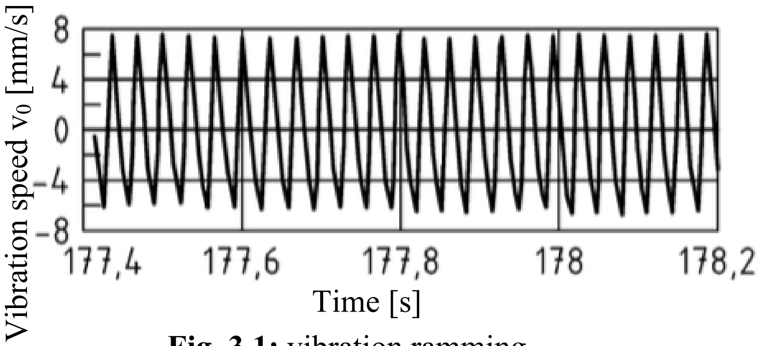


Fig. 3.1: vibration ramming

The pile is connected to the vibrator, which consists of pairs of unbalanced mass, by means of a hydraulic collet. These unbalanced mass revolve in opposite directions to one another at the same rotation speed. In doing so, the horizontal components of the centrifugal force are cancelled and the vertical are added, which means that only vertical vibrations are produced. Due to the vibrations, the soil grains are vibrated, which results in a considerable reduction of the shear strength for not too close non-cohesive soils. In addition, each individual vibration is a ram stroke. Its energy depends on the centrifugal force F_0 and the oscillation displacement amplitude A_0. For these reasons, the pile can penetrate the ground even under a lower static load. The centrifugal force is the product of the weight of the unbalanced masses G_U, the lever arm referring to the rotational axis r_U and the square of the angular frequency ω_U:

$$F_U = G_U \cdot r_U \cdot \omega_U^2$$

The oscillation displacement amplitude A_0 depends on the masses involved in the oscillation, such as the unbalances masses G_u, the vibrator G_{Vib}, the pile G_{Rg} and the ground G_B:

$$A_0 = \frac{G_U \cdot r_U}{G_{Vib} + G_{Rg} + G_B}$$

The maximum energy introduced into the ground during a swing period can be determined as follows:

$$E_{max} = \pi \cdot F_U \cdot A_0$$

Only part of this energy is transformed into vibration waves. The greater the resistance of the soil, the more energy is emitted.

High working frequencies are more favourable than lower ones, since the resonance range of projectiles floor is generally avoided. It can be problematic in the case of the occurrence of ram resistors, since here the operating frequency can drop to the ceiling resonance range. As a result, a stronger energy can be emitted, which produces even greater vibrations even at longer distances.

The DIN 4150-1: 2001-06 specifies the following formula for this kind of rams in order to calculate the oscillation speed v_1 at a distance R from the excitation point:

$$v_1 = v_0 \cdot R^{-0,5} \cdot e^{-\alpha \cdot R}$$

with α = abatement coefficient.

In a bilogarithmic diagram, this formula represents a curve as shown in Fig. 3.2.

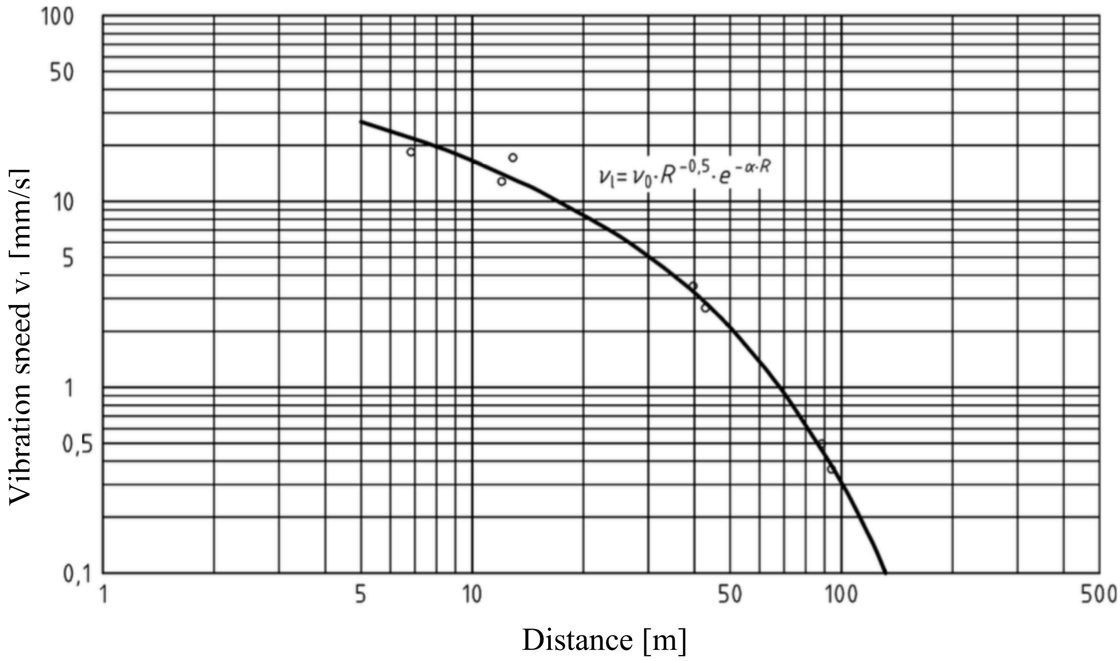


Fig. 3.2: vibration propagation: maximum oscillation speeds

3.2.2. Impact hammers

Impact hammers and striking hammers can drive the pile into the ground by means of a falling weight. This can cause periodic excitations of the soil. It can often be assumed that a pile impact has already subsided before the next one begins. These devices include diesel hammers, drop hammers, hydraulic hammers and high-impact hammers.

In the cylinder of a diesel hammer, flapping pistons fall from a height of about three meters, which compress the air. At the same time, diesel fuel is injected onto the cylinder bottom, which is ignited by the compressed air. This explosion causes a blow both on the pile material and on the piston, which is again shot upwards. The impact energy consists of two components: the fall energy, which is the product of the weight of the mass and the height of the fall, and a portion of the explosive energy. With this machine, stroke rates between 30 and 50 beats per minute can be achieved.

In the case of a drop hammer, an impact weight is simply pulled with a rope up to a height of approximately 1,2 meters and then dropped onto the pile material. The piston can weigh up to 16 t and can bounce up to 40 times per minute on piles.

Devices such as a double acting hydraulic hammer have a closed cylinder next to the piston. The piston is lifted hydraulically and released in a second moment. The dropping is additionally accelerated with compressed air or hydraulic fluid, which act from above. With such machines, maximum energies of up to 3000 kNm and stroke rates of 50 to 60 beats per minute can be achieved.

High-impact hammers have lighter pistons that drop from a height of 0,5 m. Compressed air or steam can be used to accelerate the fall. Due to the low drop height and the low weight, lower energies of around 30 kNm can be achieved. However, the number of hits is significantly higher: it can be up to 600 beats per minute.

The grounding speed due to driving rams and also vibration rams can be predicted using the following equation:

$$v\left[\frac{mm}{s}\right] = K\frac{\sqrt{E^{\alpha}}}{r^{\beta}}$$

where r [m] is the distance from the shock spring to the point under consideration, K is a proportionality factor, and E [kNm] is the impact and vibration energy. The parameters K, α and β have different values depending on the author. "Attewell and Farmer" recommended to use the values K = 24, $\alpha = \beta = 0{,}95$ to calculate the probable grounding speed. On the other hand, the values K = 47,4, $\alpha = \beta = 1{,}0$ should be assumed to be on the safe side.[8]

The DIN 4150-1: 2001-06, on the other hand, gives a different formula for determining the oscillation speed v_1 at a distance R:

$$v_1 = v_0 \cdot R^{-1{,}0} \cdot e^{-\alpha \cdot R}$$

This is a similar formula to that of a vibration ram. In this case, the exponent of the distance R increases from 0,5 to 1.

In a bilogarithmic diagram, this formula also represents a curve as shown in Fig. 3.3.

[8] M. Achmus, J. Kaiser, F. tom Wörden; Bauwerkserschütterungen durch Tiefbauarbeiten – Grundlagen, Messergebnisse, Prognosen; p.19 - 29

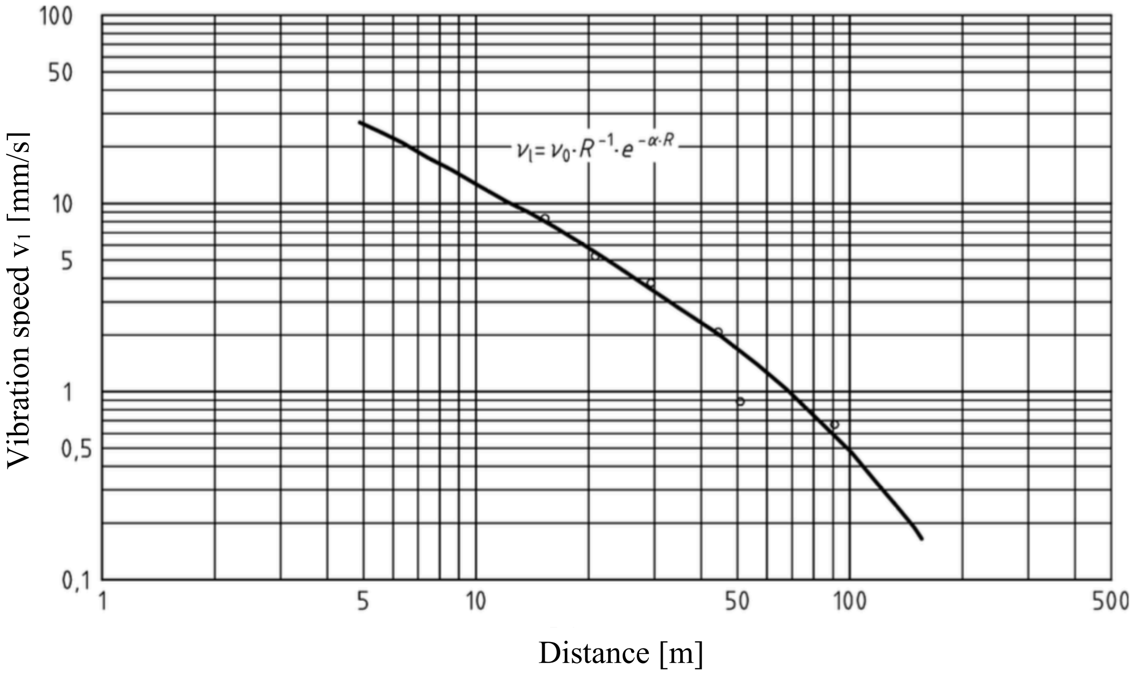


Fig. 3.3: vibration propagation: maximum oscillation speeds

3.3. Subway tunnels

Moving subway trains emit vibrations during their journey, which rise above the tunnel and the ground to a building. Due to the narrow inner-city areas, a more accurate prediction of the vibrations by means of models is becoming more and more important. These vibrations are caused by dynamic loads such as:

- the driving conditions
- the variations in the contact spring stiffness and the rigidity of the superstructure
- the position modification and ripple of the rails
- the non-circular wheels
- the route taken
- the natural vibrations of vehicle parts
- the shocks and gaps on switches
- uneven welds

The first four points are the most important excitation depending on the frequency and size of the generated dynamic forces. The driving conditions include the curve progression and the starting and braking states, which can be determined with elementary models of the dynamics. The stiffness changes due to discrete support of the rails on the sleepers. The excitation frequencies, due to this variation, depend, inter alia, on the driving speed, on the interfering wave length and on the threshold distance. The determination of this frequency is important, because it affects the mass participation of the coupled system in the dynamic process. Investigations have shown that the dynamic loads increase with the underground rigidity. In addition, this force rises with increasing speed or frequency until it exceeds the wheel-road-underground eigenfrequency. In this case, the stimulating forces remain almost constant.[9]

Due to its stiffness, the soil isn't only a propagation medium, but also affects the behaviour of the tunnel itself. The amplitude of the exciting waves decreases with the depth, thereby the building vibration reduces with the profoundity of the foundation. However, in these cases, the waves that enter and reflect on the basement walls must be taken into account, since these can increase the stimulation of the structure. In addition, the relative distance between the tunnel and the house above or adjacent to it plays an important role. If, for example, the building is located directly above the tunnel, a certain coupling between them and the floor must be taken into account.

Investigations have shown that the tunnel ceiling has an increased eigenvibration behaviour. Its first natural frequency lies in a region where the excitation from the vehicle is very strong. However, these effects can be disregarded in the case of low-lying tunnels, since with increasing the profoundity this resonance frequency decreases considerably. For this reason, only that one, that is near the surface, is relevant and the position of this frequency is affected by the ground strength.[10]

The DIN 4150 - 1: 2001 - 06 shows in plant A.6 the dependence of the maximum oscillation speed on the distance from the outer wall of the tunnel using a graph. Fig. 3.4 shows that this dependency is approximately linear in a bilogarithmic diagram.

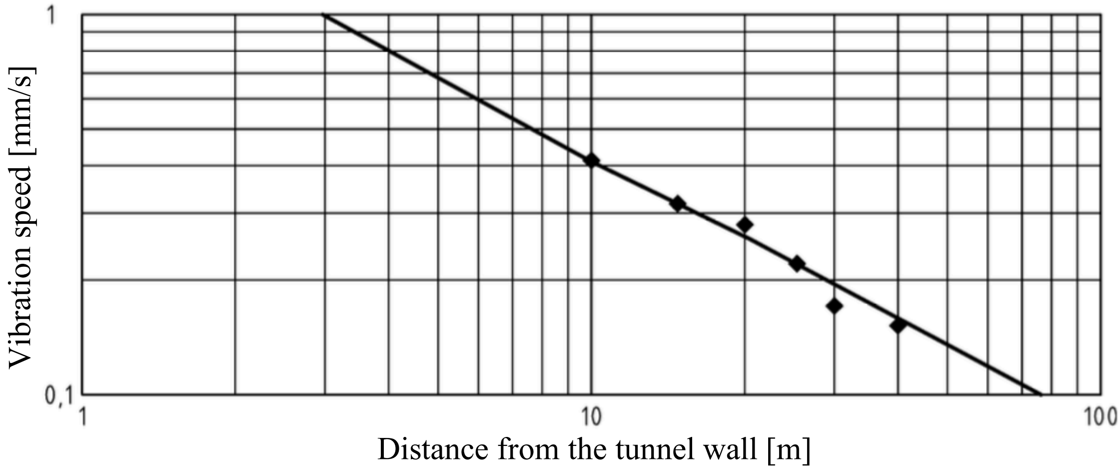


Fig. 3.4: maximum oscillation speed

3.4. Rail traffic

The above-ground rail transport also leads to shocks, which are particularly detrimental to the wellbeing of people in terraced buildings. Rarely, there are some damage to structures near the route. The reasons for the formation of these vibrations can be found

[9] Nawawi Chouw; Günther Schmid; Erschütterungsausbreitung und Erschütterungsreduzierung; p. 59 - 63
[10] Verein deutscher Ingenieure; VDI – Schwingungstagung ,94; Erschütterungen – Ursachen und Minderung von Störungen oder Schäden; p. 71 – 75 and p. 89 – 90

in chapter 3.3. The strength and the frequency range of these excitations are basically influenced by the following points:

- Constructive design of the superstructure: softer bearing and larger superstructure are shake-resistant
- Size of the rail profile: larger profiles bend less between two thresholds, thus the excitation is lower
- Driving speed: the higher the speed, the stronger the vibration
- Wheel set distances and vehicle mass: larger masses lead to higher vibrations

According to the Federal Pollution Control Act of Germany, harmful environmental impacts must be avoided or kept as low as possible according to the state of the art in the upgrading and new construction of traffic routes. For this reason, vibration prognoses are often conduct. The forecast model of the igi Niedermeyer Institute can be mentioned here. It describes the propagation of the waves from the railway track to the floors of a nearby building. It is derived from a relatively simple spectral propagation equation whose terms are evaluated by statistical measurements derived from other places. The shocks are caused by excitation mechanisms between the wheel and the rail. The wagons, the existing superstructure and the subsoil influence the vibration. For this reason, the following points are implemented in the forecast model:

- Route guidance in the embankment or on the cutting slope
- Type of superstructure: solid track or crushed stone
- Anti-vibration measures

The vibrations propagate on the surface and are transferred to the floors over the foundations and walls. Generally, the maximum oscillation is in the centre. According to the model, the immission level to be determined on the ceiling L_{Im} is calculated from the emission level L_{Em}, which is determined eight meters from the track centre, minus a number of different losses:

- ΔL_{Bod}: level decrease in the ground during the propagation from the emission location to a point on the top ground surface in about one meter distance in front of the building. It is composed of the geometrical attenuation ΔL_{geo}, which is a radiation attenuation and a material attenuation ΔL_{mat}, which is a dissipative attenuation.
  The geometrical attenuation depends on the calibration parameter p:
  $$\Delta L_{geo} = 10 \cdot p \cdot log\left({}^{r}/_{r_0}\right)$$
  This depends, in turn, on the nature of the source of vibration (punctiform or linear, mobile or immovable) and on terrain morphology (layer construction, groundwater and terrain kink).
  The material damping is determined by the distance between the track and the emission location (generally 8m), the number of the axes n and the abatement coefficient α. The last one is influenced by the surface wave speed c_R and by the damping factor ζ.

$$\Delta L_{mat} = 20 \cdot log\left({a_0(r,\alpha)}/{a_0(8m,\alpha)} \right)$$

with $a_0(r,\alpha) = \frac{1}{n}\sum e^{-\alpha r_i(r)}$ $and\ \alpha = 2\pi \cdot f \cdot \zeta / c_R$

- ΔL_{Geb}: Level decrease and increase during the transition from the upper edge of the terrain to the foundations and walls on the floor. As a rule, you have a reduction in the speed of vibration when moving from the soil to the foundations, but during the forwarding in the walls, there is again an increase until the floors.
- ΔL_{Sch}: Level reduction due to possible vibration intervention. Measures include the bearing of buildings on elastic elements or floor slits between the railroad track and the building to interrupt the vibrations. These slots are stabilized by introducing gas-filled mats.

The emission site is recorded eight meters from the track centre in order to still typical near-field phenomena of this source of vibration since these are very difficult to compute. Among these are the mixture of different types of waves in the area of the source and the different construction of the route.

The emission level L_{Em} has been calculated under standard conditions by means of regression analyses, which is the function of the various types of brake systems and the relevant vehicle speeds. Any deviations must be taken into account when determining it:

- Vibration effects of trains traveling on other tracks than those close the building must be calculated in the model with the corresponding distances.
- If the track is in an embankment or a cutting slope, the level must be reduced in the ratio of the cutting slope depth or the height of the embankment and the shear wave velocity.
- If, instead of the ballast structure, a ballastless track is installed, minor vibrations are emitted as the ballastless track has a higher stiffness.[11]

3.5. Demolition works

When components are dismantled, the effects of vibration must be anticipated, which must be estimated before the start of construction, in order to avoid foreseeable damage to a structure. This is achieved by recognizing the planned construction phases and local conditions, such as the relative distance and depth of the buildings. In addition, documentation such as a ground assessment and technical details of the construction equipment are useful to provide a more accurate forecast. The expected vibrations can be determined according to DIN 4150. In general, demolition of foundations and components in contact with the ground always leads to greater influence.

During demolition, equipment such as rock chisel and hydraulic pliers or explosives are used, which lead to vibrations.

[11] Nawawi Chouw; Günther Schmid; Erschütterungsausbreitung und Erschütterungsreduzierung; p. 79 - 93

In the event of an explosion, the vibrations occur only once, while they occur over a prolonged period in the case of mechanical demolition, but in the first case they are relatively high. The lower the coupling to the ground, the lower are the vibrations. Their value, however, also depends on the arrangement, amount and position of the explosive charges. The sprinklers are often made up of several spatial and temporally separated single explosions. Each individual charge leads to a certain excitation, which is superimposed and causes a total shock. Below the earth's surface, as in foundations, retaining walls and basement walls, their effects are significantly higher than above the earth's surface, such as walls, columns and ceilings. In both cases, the maximum expected oscillation velocity $v_{max,exp}$ [mm/s] depends on the ratio of the charge quantity per time step L_{quant} in kg and the distance R_{expl} in m:

Above the earth's surface: $$v_{max,expl} = 100 \cdot \frac{L_{quant}^{2/3}}{R_{expl}}$$

Below the earth's surface: $$v_{max,expl} = 250 \cdot \frac{L_{quant}^{2/3}}{R_{expl}}$$

That is, the larger the distance between the detonation and the building, the lower the oscillation will be. This is also illustrated by an example in Annex A of DIN 4150-1: 2001-06. Fig. 3.5 shows the vibration propagation as a function of the distance. It can be seen that an explosion results in a short-term vibration that takes less than one second.

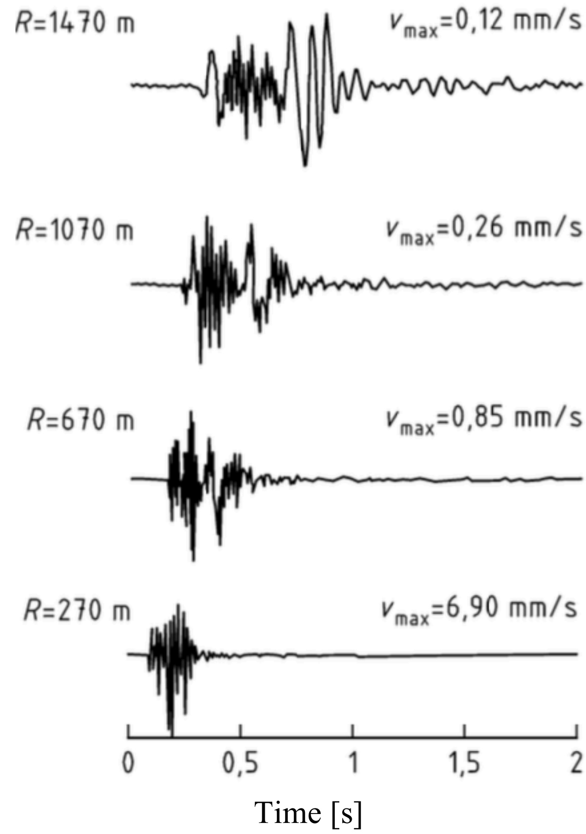


Fig. 3.5: time course of vibration propagation of explosions

Even during the use of wrecking balls for the dismantling of components, short-term vibrations occur. The greatest vibrations are obtained, if these balls are used to destroy foundations or components in contact with the ground. They are significantly smaller in a horizontal impact. In this case, the size of the oscillation depends on the stiffness of the component.[12]

The influence of rock chisels depends primarily on the impact energy, which in principle is lower than in the case of the driving ram, as well as on the site of application. The greater the distance between the site and the foundation is, the lower the vibration in the surrounding area, since a large mass of the rest of the building must be stimulated. On the basis of measurements, it was observed that individual impacts and post-shrinkage are noticeable close to the application point. On the other hand, uniform sinusoidal oscillations occur in larger distances.

However, not only the demolition process, but also falling building rubble, moving trucks and excavators bring shocks. In order to keep traffic shake as low as possible, the road surface must remain free of rubble, earth and other small parts.[13]

Among other things, the DIN 4150 - 1: 2001 - 06 can be used to determine short-term shocks due to debris crashes on the basis of the following factors:

- Height of fall
- Mass of the falling object
- Stiffness of the soil
- Type of soil
- Soil layer
- Impact surface

The maximum oscillation speed v_{max} can be calculated approximately as a function of the distance R and the drop energy $E_h = G \cdot h$:

$$v_{max} = k \frac{\sqrt{E_h}}{R^m}$$

The coefficients k and m are determined empirically, with respect to the subsoil, the droping behaviour and the distance range.

Also in this case, the guideline provides an example of a collision of a cracked reinforced concrete chimney. In Fig. 3.6, it can be seen clearly that the vibration due to falling components last longer than during explosions. For this reason, the explosions are often seen as measures for the reduction of vibration.[14]

[12] Deutscher Abbruchverband e.V.; Abbrucharbeiten – Grundlagen, Planung, Durchführung; p. 47 - 52
[13] Report from Draheim Ingenieure; Abbruch eines Luftschutzbunkers an der Rütscher Straße in Aachen – Erschütterungsprognose für den Abbruch und Konzept zum Erschütterungsschutz
[14] DIN 4150 – 1: 2001 – 06; Vibrations in buildings – Part 1: Prediction of vibration parameters; p. 9 – 10

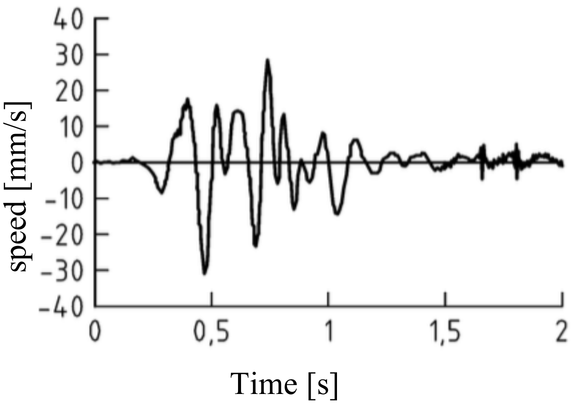


Fig. 3.6: time course of vibration propagation of falling building rubble

4. Wave propagation and transmission to the building

To describe the wave propagation in the ground, the soil dynamics must be used. The ground has three functions:

- It works like a spring or a damper at the source (emission)
- It is the transmission path from source to receiver (transmission)
- It is a spring or a damper at the receiver (immission)

Recent measurements have shown that the transmission of vibrations take place in the ground, in water and also in air. The latter can lead to perceptible vibrations in buildings if resonance is present. The propagation of the vibrations in the ground takes place in form of waves. If this process isn't disturbed by obstacles, such as different structures, then this propagation area is called "free field". The vibrational variables in this zone, such as the maximum oscillation speed, are the basis for determining the values in a building.

4.1. The wave propagation

An infinitely extended continuum filled with soil mass and bounded by a plane is defined as a half-space. This has a stress-free surface. In a homogeneous half-space, the waves are called "free-field vibrations". If the soil mass is affected by inhomogeneity, such as foundations or basement walls, new secondary waves are created here, which overlap with the primary free-field vibrations. For this reason, a foundation acts as receiver of the free-field wave and at the same time source of the secondary waves. Therefore, during the vibration measurement in its vicinity this resulting wave and not the free-field oscillation is measured. In the following, it is assumed that the half-space is homogeneous and isotropic, as well as a linear-elastic constitutive law.

As a result of temporal changes of deformations or forces that occur in an infinitely extended, homogeneous medium, is the formation of continuous waves. In a continuous oscillation, the phase position changes at a point in time from cross section to cross section, which leads to a certain phase velocity. Only this position moves and not the material. As a result, waves are often referred to as energy flows. The propagation speed depends on the type of oscillation and the characteristics of the medium.

If the medium is a solid, then the waves are referred to as a structure-borne noise or as an agitation. These are perceptible by feeling. Fluids are referred to as liquid sound, and in gases from airborne sound. Both are perceptible by hearing.

Periodic excitations produce stationary waves (Fig. 4.1). Here is specified the phase velocity c, which indicates the propagation velocity of a particular oscillation state. In the case of non-periodic excitations transient waves (Fig. 4.2) are produced. This is referred to as the wave velocity c, which determines the propagation velocity of a distinctive point of the oscillation. By measuring these quantities, the dynamic stiffness module and the shear module can be determined.

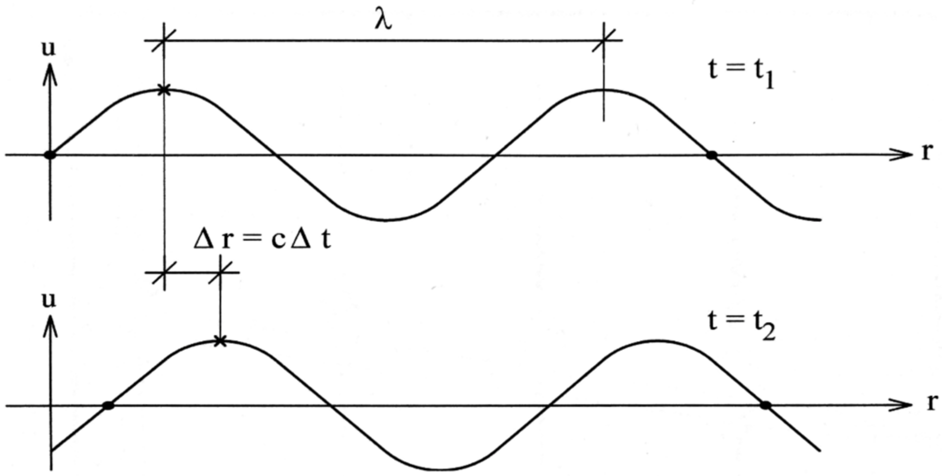


Fig. 4.1: phase velocity of stationary waves

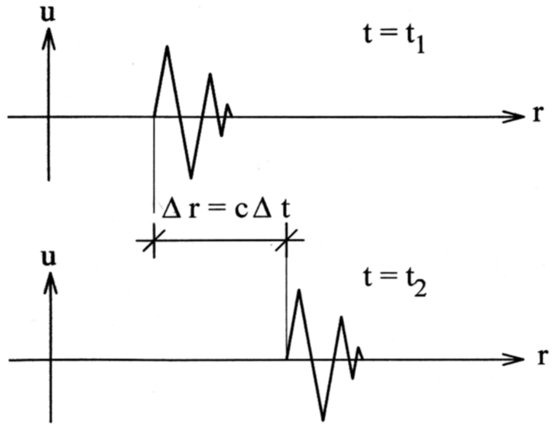


Fig. 4.2: waves velocity of transient oscillations

4.1.1. Types of waves

Basically, two wave types are defined in a linear-elastic, homogeneous, isotropic half-space: the body waves (Fig. 4.3) and the Rayleigh waves (Fig. 4.4). The first are distinguished in longitudinal and transversal waves and can spread throughout the half-space. The Rayleigh waves propagate only parallel to the surface of it.[15]

[15] Helmut Kramer; Angewandte Baudynamik, Grundlagen und Praxisbeispiele; p. 247 – 254

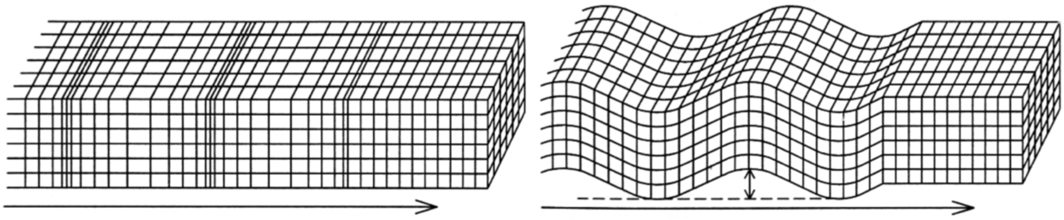


Fig. 4.3: body waves: left → longitudinal wave, right → transversal wave

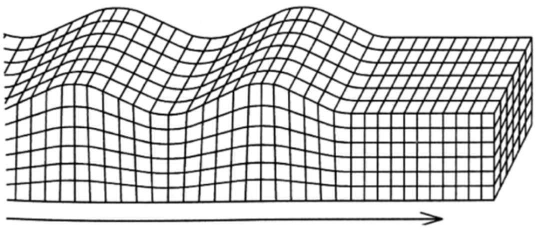


Fig. 4.4: Rayleigh wave

Longitudinal waves are also referred to as compression waves or P-waves. Such a wave is obtained when no rotation of an infinitesimal small particle occurs during the propagation of the displacement state. The centre of gravity of this particle doesn't move in the two mutually orthogonal directions. A displacement component is present in the third direction, namely in the direction of propagation. In this type of wave, volume and shape changes occur. The propagation rate is calculated as follows:

$$c_p = \sqrt{\frac{E_{s,d}}{\rho}}$$

With $E_{s,d}$ the dynamic stiffness module at hindered lateral expansion and ρ the density of the soil including the water in the pores.

Transversal waves are also referred to as shear waves or S-waves. In this case a propagation of a volumetric faithful displacement state takes place. That means, there is only a pure shear deformation and the volume doesn't change. Thereof the propagation velocity from the dynamic equilibrium[16] of the displacements in one direction is defined like:

$$c_s = \sqrt{\frac{G_d}{\rho}}$$

where G_d defines the dynamic shear module.

[16] In „Wolgang Haupt; Bodendynamik, Grundlagen und Anwendung" p. 56 – 58 is the description of the dynamic equilibrium

The dynamic shear module, which increases in depth in the natural soil, can be written as a function of the Poisson's ratio and the dynamic stiffness modulus:

$$G_d = \frac{1-2\nu}{2(1-\nu)} E_{s,d}$$

The values of the Poisson's ratio can be found in the table 2.2 and of the dynamic shear module in the table 2.1. In chapter 2.2, the dependency on the shear distortion can also be taken.[17]

If two elastic, homogeneous bodies lie adjacent to each other along a surface, additional boundary conditions must be fulfilled in dynamic equilibrium. The extreme case is the boundary surface on the free earth surface. Here stresses can't form by displacements in the half-space. Lord Rayleight recorded mathematically for the first time the Rayleigh waves. They are surface waves whose propagation velocity c_r is given in relation to the longitudinal or transversal wave:

$$c_r \approx 0{,}9c_s \qquad c_r \approx 0{,}5c_p$$

If the body waves dominate in the near field[18], then the surface waves play an important role in the far field[19]. The oscillation amplitude of a Rayleigh wave contains a vertical $\bar{v}_r$ and a horizontal $\bar{u}_r$ component. This results in vertical and horizontal excitations on a structure. In Fig. 4.5, its course is illustrated as a function of the depth.

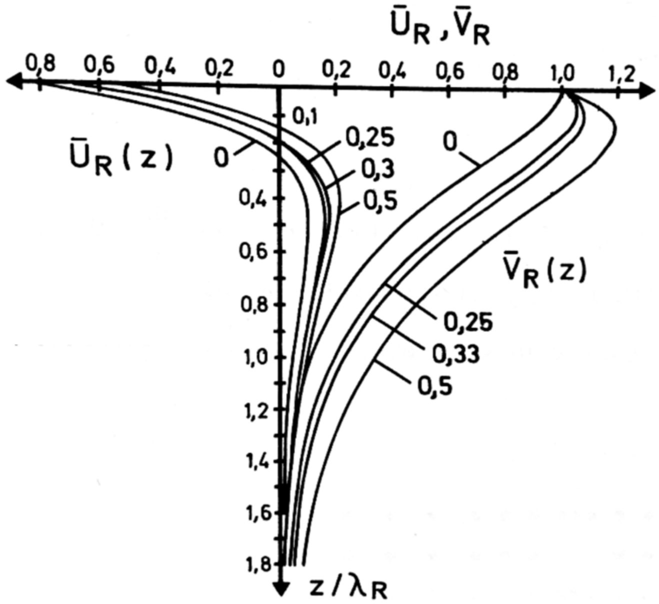


Fig. 4.5: amplitude course of the Rayleigh wave for different Poisson numbers ν

[17] Wolgang Haupt; Bodendynamik, Grundlagen und Anwendung; p. 55 - 65
[18] Near field: the distance r between object and source is less than $\lambda_r = c_r/f$
[19] Far field: the distance r between object and source is greater than $\lambda_r = c_r/f$

As can be seen in Fig. 4.5, the amplitude of both components decreases with the depth z. The deep action of a Rayleigh wave is limited in $z \leq \lambda_r$. To a depth of z = 0,3 ÷ 0,5 λ_r, the shear module can be determined with the propagation speed c_r of the Rayleigh wave. If a soil equivalent spring or a soil equivalent damper is to determined, then the shear module must be calculated at the depth of z = r_0 (= replacement radius[20]) for translation and z = 0,5r_0 for rotation below the foundation base.

The propagation velocity c_p, which shouldn't to be confused with the particle velocity, is in gases and fluids different again. A few suggested values form them are:

- Air: c_p = 332 m/s
- Water: c_p = 1485 m/s
- Concrete: c_p = 3500 m/s

4.1.2. The energy transport

In the propagation of mechanical waves, no mass or matter is transported, but only energy. According to the energy conservation rate, the sum of kinetic E_{kin} and deformation energy E_{def} remains constant:

$$E_{\mathrm{kin}} + E_{def} = \mathrm{const.}$$

The amplitude of a one-dimensional elastic harmonic wave on the half-space surface due to a line source is given as a function of the time t and the distance r from the wave source:

$$u(t, r) = \hat{u}(r) \cdot \sin\left(\Omega \mathrm{t} - \frac{2\pi}{\lambda} r\right)$$

where λ defines the wavelength and Ω the excitation angular frequency is.

If $\Omega \mathrm{t} = \frac{2\pi}{\lambda} r$, than the amplitude is u = 0. That mean, the deformation energy E_{def} is equal to 0. This corresponds to the zero crossing of a harmonic oscillation.

Considering an infinitesimal small mass particle of a body with the mass dm and volume dV through which a one-dimensional harmonic wave passes, then it generates a certain kinetic energy:

$$dE = \frac{1}{2} dm \cdot \hat{v}^2 = \frac{1}{2} dV \cdot \rho \cdot \hat{v}^2$$

[20] r_0 is the radius of the circular foundations. For rectangular foundations, with edge length 2a and 2b, it represents an equivalent radius. It is calculated according to the situation as follows:

- Translation: $\mathrm{r_{0x}} = \mathrm{r_{0y}} = \mathrm{r_{0z}} = \sqrt{4\mathrm{ab}/\pi}$
- Tilting around x: $r_{0\varphi x} = \sqrt[4]{16ab^3/3\pi}$
- Tilting around y: $r_{0\varphi y} = \sqrt[4]{16a^3b/3\pi}$
- Torsion: $r_{0\varphi z} = \sqrt[4]{8ab(a^2 + b^2)/3\pi}$

The oscillation speed of this mass particle at the position r and with $\Omega t = \frac{2\pi}{\lambda} r$ is:

$$\hat{v} = \frac{du(t, r = const.)}{dt} = \hat{u} \cdot \Omega \cdot \cos\left(\Omega t - \frac{2\pi}{\lambda} r\right) = \hat{u} \cdot \Omega$$

From this the energy density w can be determined:

$$w = \frac{dE}{dV} = \frac{1}{2} \cdot \rho \cdot \hat{v}^2 = \frac{1}{2} \cdot \rho \cdot (\hat{u} \cdot \Omega)^2$$

On the other hand, the energy transport per unit area is defined as follows:

$$\frac{dE}{dV} = \frac{dE}{dA \cdot dr} = w \rightarrow \frac{dE}{dA} = w \cdot dr = \frac{1}{2} \cdot \rho \cdot (\hat{u} \cdot \Omega)^2 \cdot c \cdot dt$$

In the case of a punctiform wave source at the earth's surface, the waves in the half-space propagate hemispherical. The energy decreases with increasing of the sphere area.

4.1.3. Propagation of vibrations

Oscillations propagate in the surrounding in the form of free-field vibrations and can also be felt at great distances when they stimulate buildings to resonance vibrations. They can be reduced or strengthened when they are moving from the ground to the building. Therefore, propagation laws of oscillations are defined only for free-field vibrations. Without the right knowledge of the building structure and soil conditions, it isn't possible to determine its effect on the building. Therefore, the knowledge accumulated on a building cannot be transferred to others.

In the case of stationary sources, such as industrial machines, quarries and traffic routes, the propagation laws can be determined by vibration measurements. Demolition blasting's aren't stationary and often take place in close, developed areas. This leads to uncertainties in the definition of these laws. Piling work has shown that in these cases the emitted velocity amplitude is almost independent of the soil layers. The pile material cuts the ground like a knife. Significantly higher speeds are only measured at layer boundaries, since here secondary waves are reflected which are superimposed with the primary waves. After penetration into the layer they take off again. This facilitates the transferability of measurement results to other cases, where pile materials are used.

At layer boundaries, where the material properties change abruptly, the wave propagation leads to reflections and refraction, which have different effects:

- In the presence of a vertical layer boundary which depth is at least as low as the wavelength of the surface wave, the energy and therefore also the amplitude decrease during the transition.
- The reflection on the horizontal ground layers can lead to an undesirable reduction of the radiation damping.

- The course of the soil layer can be followed with the aid of the refraction seismics. [21]

In principle, the continuity conditions of the stresses and displacements must be fulfilled at the interfaces. For this reason, waves are reflected, refracted, and new waves are created. The type of the new oscillation exactly corresponds to the other type of the incoming wave. That means, if a P-wave hits the boundary, additional S-waves are generated on it. Through the relationships represented in the optics, the propagation directions of the reflected and refracted waves can be determined.

The amplitude of these waves depends on the angle of incidence, the ratios of the wave resistances, the Poisson numbers and the densities of the materials, which are involved. However, the last two mentioned ratios are of subordinate importance, since these vary only slightly in the natural soil. The essential parameter is the ratio of the waves' speed as these change considerably.

In this context, the critical angle is also defined. This is the angle of incidence in which the broken wave propagates parallel to the boundary surface. This has an important significance in a refraction-seismic examination method. If the body-waves impinge perpendicularly to the layer boundary, the respective other body-wave types don't arise here, as in the case of an oblique incidence. If, in addition, the boundary of a free surface or an absolutely rigid edge is present, then the incident wave is completely reflected.

When waves are propagated, it must be taken into account that the amplitude is more strongly reduced, when the soil material is cyclically deformed. Because of this, mechanical energy is converted into thermal energy and is than dissipated. In these cases, one speaks of a damping. This causes an increase in the wave velocity in the ground and plays a more important role in the far-field than in the near-field.

4.1.4. Wave propagation in the inhomogeneous half-space

The above described relationships can be applied only in the case of homogeneous, isotropic soil. In reality, the existing site have never these characteristic. Often, the ground consists of layers that are parallel or inclined to the surface. Its thicknesses remain constant or can also vary. At layer boundaries, soil properties, such as the shear modulus and the Poisson number, can change abruptly. The density changes, however, only in a small interval. Often, the transition isn't clear, but there is a transition area whose thickness is in the decimetre range. It is important to note that layers can be completely different from a technical-structural point of view, but are similar in dynamic respects.

As already mentioned, the reflection and refraction of the body waves occurs at precise layer boundaries. However, if the transition from one stratum to another is continuous or with a constant change in the soil properties within it, then a continuous change in the propagation direction takes place. The rock can often be very complicated to grasp as one

[21] Helmut Kramer; Angewandte Baudynamik, Grundlagen und Praxisbeispiele; p. 254 – 262

or more joints, which have a different degree of separation and which are filled with different filling materials, strongly influence the spreading. Basically, the wave velocity in the pure rock material is higher than the total composition.

Storage density, grain curves and soil materials may vary from place to place. These lead to local fluctuations in the shear modulus and thus to local inhomogeneity, which can occur practically everywhere in the soil. However, these aren't essential from a technical-structural point of view. They work as an additional material damping, after a kind of dispersion of the waves is formed. The effect is greater, the greater the local inhomogeneity ranges compared to the wavelength and the greater the deviations in the dynamic properties. Grounds which have neither different layers nor local inhomogeneity are homogeneous with regard to the propagation of waves. The oscillation speed rises with increasing of the static pressure, that means with increasing depth, since the superposition of the soil growths.

The water has no shear strength but increases slightly the density. For this reason, the shear waves are hardly affected by the ground water in the pores. The ground water between the soil grains, on the other hand, is stiff against volume changes. Therefore, the compression waves are strongly influenced, since they can spread particularly well here. The compression wave velocity in pure water is approximately 1440 m/s and in the ground water it can be higher or lower depending on the pore proportion. Rayleigh waves cause shear deformations. Therefore, the ground water doesn't affect its spread. This implies that the water plays an important role only close to the source at the earth's surface, since the compression wave is additionally reflected by the water level. [22]

4.2. Soil-building interaction

Homogeneous, infinitely extended soils, like infinitely extended plates, have no natural frequency. It is only through the interplay between soil and foundation that a vibrating system is created. The ground represents with its spring stiffness and damping the complex stiffness[23], while the base is the mass of the vibration system. The complex

[22] Wolgang Haupt; Bodendynamik, Grundlagen und Anwendung; p. 72 – 93

[23] The Voigt-Kelvin model is considered for the definition of the complex stiffness. This is a combination of a linear-elastic spring (with the stiffness k) with a linear viscous damper (with a damping c). The real approach for a harmonic force $(\hat{F}\sin\Omega t)$ is:

$$c\dot{u}(t) + ku(t) = \hat{F}\sin\Omega t$$

The real solutions are:

$$u(t) = \hat{u}\sin(\Omega t - \alpha)$$
$$\dot{u}(t) = \hat{u}\Omega\cos(\Omega t - \alpha)$$

with Ω the excitation angular frequency and α the caster angle. If the solutions are used in the approach, one obtains:

$$c\cdot\hat{u}\Omega\cos(\Omega t - \alpha) + k\cdot\hat{u}\sin(\Omega t - \alpha) = \hat{F}\sin\Omega t$$

If $\Omega t = \pi/2$, then one obtains:

$$c\cdot\hat{u}\Omega\sin\alpha + k\cdot\hat{u}\cos\alpha = \hat{F} \Rightarrow \frac{\hat{F}}{\hat{u}} = c\Omega\sin\alpha + k\cos\alpha = \sqrt{k^2 + (c\Omega)^2}$$

The term $\sqrt{k^2 + (c\Omega)^2}$ is defined as the dynamic stiffness.
Consider the complex notation and its solutions:

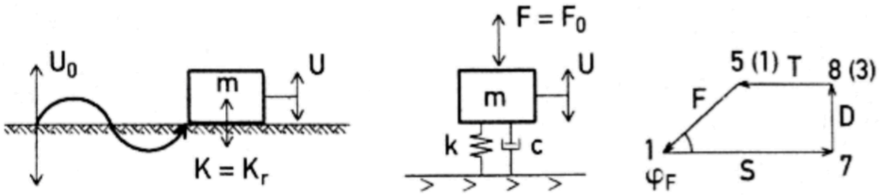
stiffness of the ground is determined for a massless, rigid plate, the basic dimensions of which correspond to the foundation one. In a second step, the soil is replaced by discrete springs and dampers.

4.2.1. Indirect stimulation of the foundations

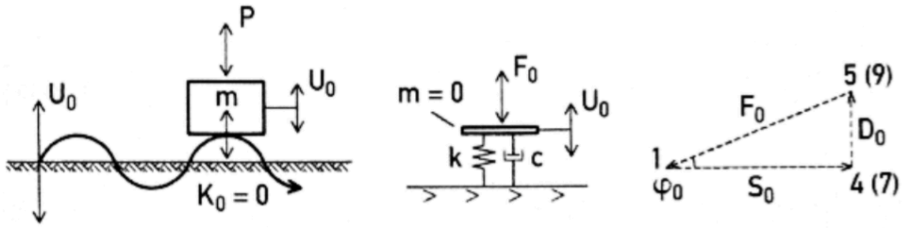
The foundation is loaded with a fictitious force $P = m\ddot{u}_0$ so that there is no tension between it and the free-field vibrancy. As a result, the free-field oscillation passes through the base undisturbed. In a second moment, it is loaded with the opposite fictitious force $P = -m\ddot{u}_0$, which produces the relative displacements $\hat{u}_r$ between foundation and ground. This secondary displacement propagates in the vicinity of the plate and superimposes itself with the primary free-field vibrations $\hat{u}_0$. The vectorial sum of $\hat{u}_r$ and $\hat{u}_0$ describes the absolute displacement amplitude $\hat{u}$ of an indirectly excited foundation. (Fig. 4.6)

$$\begin{cases} c\dot{u}(t) + ku(t) = \hat{F}e^{i\Omega t} \\ u(t) = \hat{u}e^{i\Omega t} \\ \dot{u}(t) = \hat{u}\mathrm{i}\Omega e^{i\Omega t} \end{cases} \Rightarrow c\hat{u}\mathrm{i}\Omega + k\hat{u} = \hat{F} = \hat{u}\cdot(c\mathrm{i}\Omega + k)$$

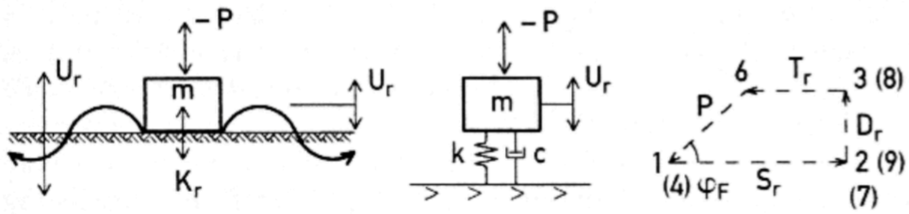
The size $(c\mathrm{i}\Omega + k)$ is known as the complex stiffness.



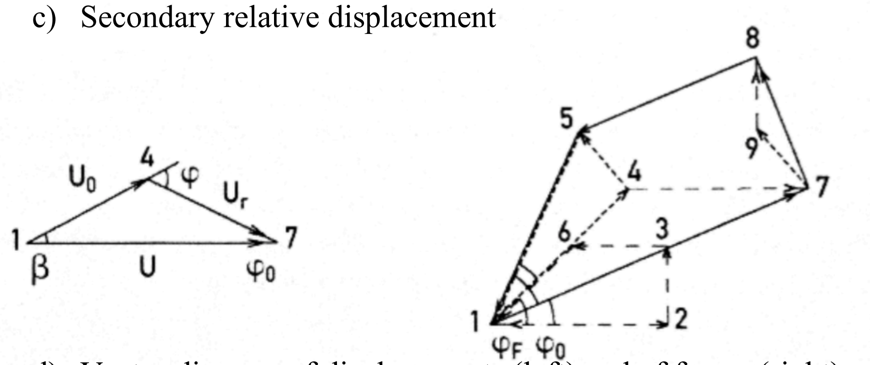
a) Indirect excited foundation



b) Primary free field oscillation



c) Secondary relative displacement



d) Vector diagram of displacements (left) and of forces (right)

Fig. 4.6: Subsoil as half-space and spring-damper system

If we first examine a one-mass oscillator, which is excited indirectly, then the harmonic displacement-amplitude acts at the base point:

$$u_0(t) = \hat{u}_0 \cos(\Omega t - \alpha)$$

$\hat{u}_0$ can generally be measured on the foundation. In this model, which is shown schematically in Fig. 4.7, the absolute displacement amplitude $\hat{u}_F$ of the oscillating mass m must also be defined.

The deformation process of the system can be divided into two phases to define the relative displacements $\hat{u}_r$ between the vibrating mass and the base point. First, the displacement of the mass $\hat{u}_F$ is fixed and only the spring is moved about $u_0(t)$. In a second moment $u_0(t)$ is fixed and the mass m is lengthened about $\hat{u}_F$.

The relative displacement is thus:

$$l_0 + u_0(t) = l_1 + u_F(t) \;\Rightarrow\; u_r(t) = \Delta l = l_0 - l_1 \;\Rightarrow\; u_r(t) = u_F(t) - u_0(t)$$

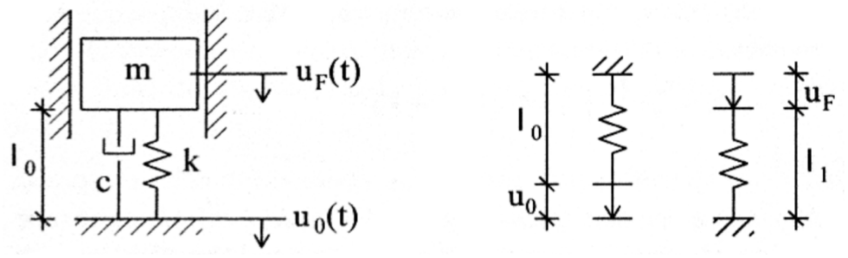


Fig. 4.7: One-mass oscillator with indirect excitation

From the dynamic equilibrium the inhomogeneous differential equation for this system can be written as follow:

$$m\ddot{u}_F(t) + c\dot{u}_r(t) + ku_r(t) = 0$$

$$m\ddot{u}_F(t) + c\big(\dot{u}_F(t) - \dot{u}_0(t)\big) + k\big(u_F(t) - u_0(t)\big) = 0$$

$$m\ddot{u}_F(t) + c\dot{u}_F(t) + ku_F(t) = c\dot{u}_0(t) + ku_0(t)$$

Their result is composed from the general, for the homogeneous differential equation, and particulate solution. The first term is neglected, because the damping of this part is small. For the latter solution the harmonic approach for the displacement of the mass is established as follow:

$$u_F(t) = \hat{u}_F \cos(\Omega t - \alpha)$$

$$\dot{u}_F(t) = -\hat{u}_F \Omega \sin(\Omega t - \alpha)$$

$$\ddot{u}_F(t) = -\hat{u}_F \Omega^2 \cos(\Omega t - \alpha)$$

Assuming for t = 0 and considering the trigonometric relations, the following equation is obtained:

$$-m\hat{u}_F\Omega^2 \cos\alpha + c\hat{u}_F\Omega \sin\alpha + k\hat{u}_F \cos\alpha = c\hat{u}_0\Omega \sin\alpha + k\hat{u}_0 \cos\alpha$$

$$-\hat{F}_T \cos\alpha + \hat{F}_D \sin\alpha + \hat{F}_R \cos\alpha = \hat{F}_2 \sin\alpha + \hat{F}_1 \cos\alpha$$

Graphically (Fig. 4.8) $\hat{F}_T$ and $\hat{F}_R$ are parallel to each other, but they have the opposite direction, and $\hat{F}_D$ acts perpendicular to them.

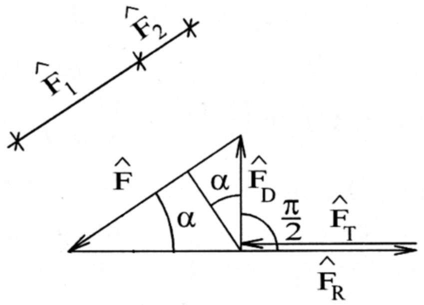


Fig. 4.8: Vector diagram of dynamic forces

By the theorem of Pythagoras one obtains:

$$\hat{u}_F^2[(k-m\Omega^2)^2+(c\Omega)^2]^2 = \hat{u}_0^2[k^2+(c\Omega)^2]^{\;2}$$

$$\hat{u}_F = \hat{u}_0\sqrt{\frac{k^2+(c\Omega)^2}{(k-m\Omega^2)^2+(c\Omega)^2}}$$

If we introduce the quantities $\omega = \sqrt{k/m}$, $\eta = \Omega/\omega$, $c_{krit} = 2\sqrt{km}$ and $D = c/c_{krit}$, [24] then the magnification function for the indirect excitation, which is shown in Fig. 4.9, can be written as:

$$V_3 = \frac{\hat{u}_F}{\hat{u}_0} = \sqrt{\frac{1+\left(\frac{c\Omega}{k}\right)^2}{\left(1-\frac{m\Omega^2}{k}\right)^2+\left(\frac{c\Omega}{k}\right)^2}} = \sqrt{\frac{1+\left(\frac{2D\sqrt{km}\Omega}{k}\right)^2}{\left(1-\frac{\Omega^2}{\omega^2}\right)^2+\left(\frac{2D\sqrt{km}\Omega}{k}\right)^2}}$$

$$V_3 = \frac{\hat{u}_F}{\hat{u}_0} = \sqrt{\frac{1+\left(\frac{2D\Omega}{\omega}\right)^2}{\left(1-\frac{\Omega^2}{\omega^2}\right)^2+\left(\frac{2D\Omega}{\omega}\right)^2}} = \sqrt{\frac{1+(2D\eta)^2}{(1-\eta^2)^2+(2D\eta)^2}}$$

[24] c_{krit} is the critical damping, which is the smallest damping of a free oscillation. D is the degree of damping

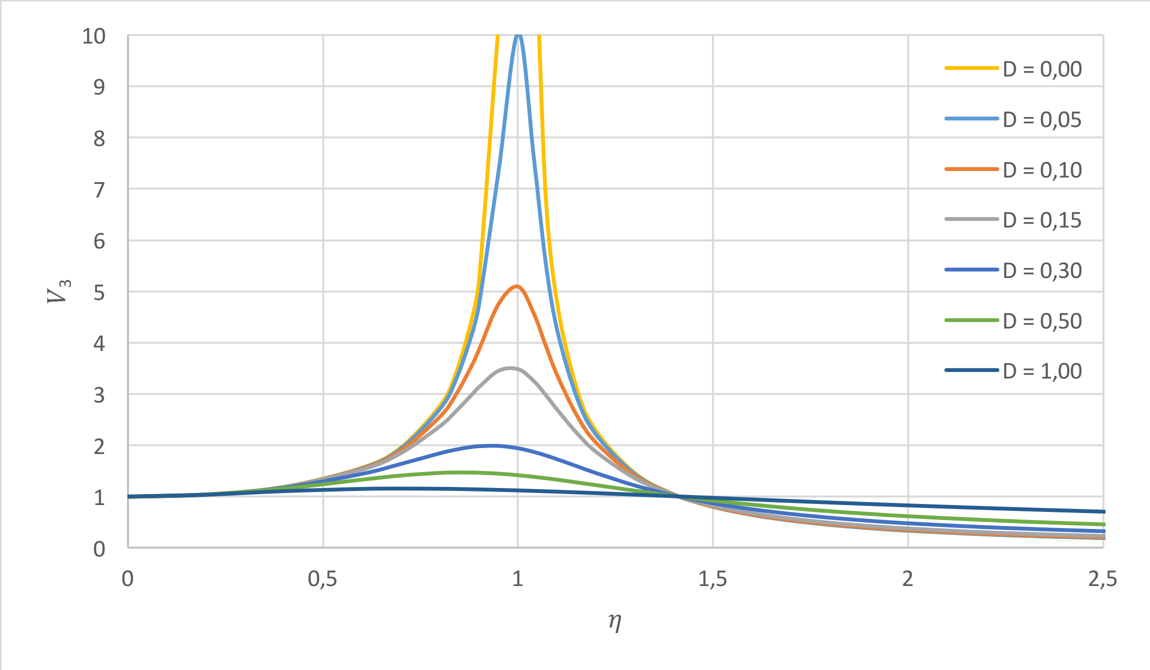


Fig. 4.9: magnification function for the indirect excitation

From the vector diagram of the displacement (Fig. 4.6 d)) and with the aid of the law of sines, the magnification factor can also be determined as follows:

$$V = \frac{\hat{u}}{\hat{u}_0} = \frac{\sin\varphi}{\sin(\varphi - \beta)}$$

On the other hand, from the vector diagram of the forces (Fig 4.6 d)) on the foundation there is $\varphi = \varphi_F$ and $\varphi_F - \beta = \varphi_0$. These two new quantities can be determined by the trigonometry:

$$\varphi_F = arctan\left(\frac{D}{S-T}\right) = arctan\left(\frac{c\Omega}{k - m\Omega^2}\right)$$

$$\varphi_0 = arctan\left(\frac{D_0}{S_0}\right) = arctan\left(\frac{c\Omega}{k}\right)$$

If $V = V_3$ then follows $\hat{u} = \hat{u}_F$.

In the case of indirect excitation by ground wave of foundations of slender towers, attention must be paid to the tilting vibration as soon as the resonance oscillation of the tower is reached. This is particularly important for bases whose dimensions aren't small compared to the wavelength of the surface wave. [25]

[25] Helmut Kramer; Angewandte Baudynamik, Grundlagen und Praxisbeispiele; p. 145 – 154 & p. 164 – 168 & p. 263 - 267

4.3. Other building site models

So far, only linear-elastic, homogeneous half spaces have been investigated, which are loaded on their interface with a circular or rectangular surface load. The real soil around the foundation doesn't always have the ideal characteristics of the model, which leads to this model being modified, or even to select a new one.

4.3.1. The embeddings

Foundations are usually not built on the ground surface but embedded in the soil. By this embedding, the springing and the damping are increased. The base of the foundation can still rest on a homogeneous elastic half-space. On the side, however, there is an elastic layer whose parameters can differ with those of the half-space. In order to simplify the analytical calculation, it is assumed that the reaction in the base is the same as that on the surface. For the lateral reactions, an elastic layer is assumed which rests unconnected on the lower half-space. That means, the lateral layer moves independently of the support. For this reason, this method can also be applied to ground which aren't homogeneous. [26]

4.3.2. The heterogeneity

The elastic properties of the floor under the foundation vary depending on the position, since the static stresses and the dynamic strain amplitudes, which influence these elastic parameters, change. An exact vibration analysis of such soils is very difficult, so the model of the homogeneous half-space is maintained and an equivalent elastic constant is determined. Frequently, the required elastic values are determined at a depth of half a foundation width when the translation is examined and, in the case of rotation, they are calculated at a depth of a quarter of the foundation width, similar to a homogeneous half-space.

A further method for determining the equivalent elastic shear modulus is the determination of the influence line as a function of the depth. The substrate is then subdivided into n layers, and for each layer, the reciprocal shear module and the associated section on the influence line are calculated and multiplied with each other. If the products are summed up, the reciprocal, equivalent shear module is obtained. However, this method provides too favourable damping, which have been shown in reality by experiments. For this reason, the radiation damping should be set to zero in these cases, and only with the dissipation attenuation should be calculated. [27]

[26] In "Wolgang Haupt; Bodendynamik, Grundlagen und Anwendung" formulas for the stiffness and damping's are listed on p. 163 - 164

[27] Wolgang Haupt; Bodendynamik, Grundlagen und Anwendung; p. 161 - 171

4.4. *Shock-induced sinkings*

Ground waves lead in loose sands and silt to compaction. Therefore, they change some parameters of the materials and cause also sinkings. This occurs particularly in frequently repeated dynamic effects. In road construction this is used to improve the soil. However, for other sources of vibration this may cause unpleasant damage on or adjacent to a building foundation without preannouncement.

In the immediate vicinity of the source loosely stored non-cohesive soil will be very strongly compressed. However, regions bordering this zone can be loosened. These can migrate through subsequent rearrangements up to the areas close to the foundation. Small loosened areas aren’t the problem, since they are bridged by stress redistribution in the ground, without cracks being visible in the structure. If this zone increases continuously, a sudden collapse of the arch and shear resistance occurs, which is resulting in settling cracks.

Even water-saturated, uniform, loosely storage, non-cohesive soils, which are cyclically loaded, can lead to problems. As the pore water pressures rise, the effective tensions in the ground drop until the shear strength of the soil is lost. This results in soil liquefaction. The described problem is mainly caused by earthquakes, since the effect can be very strong. In the case of the shocks treated here, it is therefore seldom that this phenomenon occurs.

According to DIN 4150-3 (Fig. 4.10), it is necessary to specify the distances between the vibration source and the building's lower edge to avoid vibration-induced sinkings. However, studies have shown that these distances are underestimated at low frequencies. The norm specifies that this distance must be selected so that an angle of 30° from the vertical line is present. In the presence of groundwater, it must be at least 45°.[28]

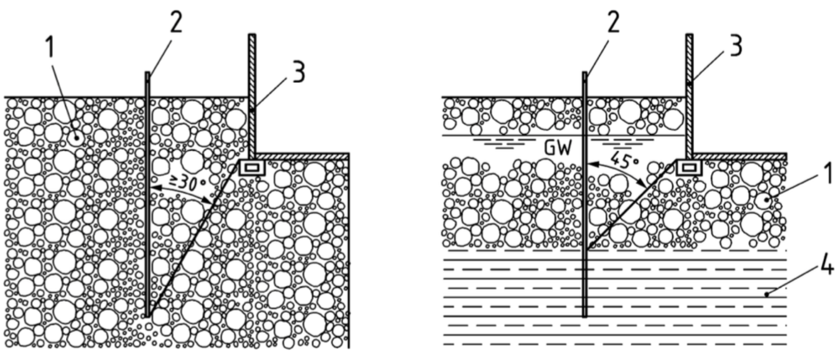


Fig. 4.10: Description of the distances: (1) sand, gravel, (2) pile wall, (3) building, (4) clay, silt, (GW) ground water

[28] Helmut Kramer; Angewandte Baudynamik, Grundlagen und Praxisbeispiele; p. 270 – 272

5. Measurement of vibrations

Vibrations are completely described by amplitude, frequency and zero phase angle. If an oscillation generator carries out forced vibrations, the excitation forces must also be determined. Thus, the vibration measurement technique deals with the measurement of paths, velocities, accelerations, forces, frequencies and phase angles. Newer sensors transform the mechanical measured values into a proportional electrical quantity, and then they are amplified and displayed with the help of electrical instruments. Three are the essential advantages of an electrical measurement as opposed to a mechanically acting device:

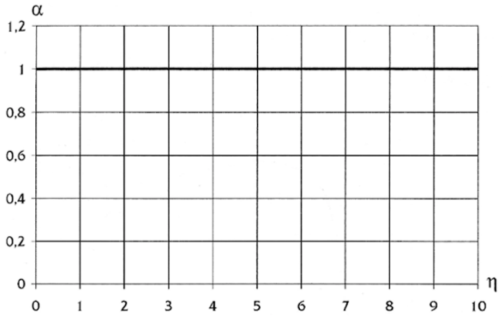
- The measuring instrument can be kept lightweight and small
- The use of an electronic amplifier increases the sensitivity
- Several quantities, remote measurements and registrations can easily be carried out[29]

Important parameters of a vibration sensor are the amplitude and the phase frequency response, which indicates the phase difference between the input signal and the output signal. Both sizes are frequency-dependent. The mechanical input signal can be determined as a ratio of the output signal and the amplitude frequency response. During the simultaneous measurements of oscillations with different frequency contents, the phase frequency response is used to bring the input signals in phase. The vibration sensors can also be classified according to the basic mechanical principle:

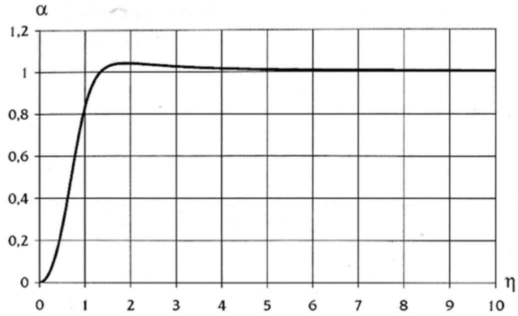
- Relative measurement: determination of the movement between two points. Constant amplitude and phase frequency response → the variable to be measured is reproduced independently of the frequency distortion-free. (Fig. 5.1 a) and Fig. 5.2 a))
- Force measurement: Measurement of the inertial force exerted on its mass from a harmonic oscillator over its spring. Up to half of the resonance frequency, the amplitude and phase frequency response are almost constant. At resonance, there is the highest amplitude frequency response of almost 17, with subsequent reduction of the value. The phase frequency response is 90° at the resonance and then increases further. (Fig. 5.1 c) and Fig. 5.2 c))
- Absolute measurement: Determination of the absolute movement of a point. The amplitude frequency response starts at zero, increases until the frequency ratio is 2, wherefrom it keeps almost constant to 1. The phase frequency response goes from zero to 90 ° in resonance to 180 ° (Fig. 5.1 b) and Fig. 5.2 b)).[30]

[29] Dr. Ing. Erhard Hübner; Technische Schwingungslehre in ihren Grundzügen; p. 273
[30] Helmut Kramer; Angewandte Baudynamik, Grundlagen und Praxisbeispiele; p. 300 – 304

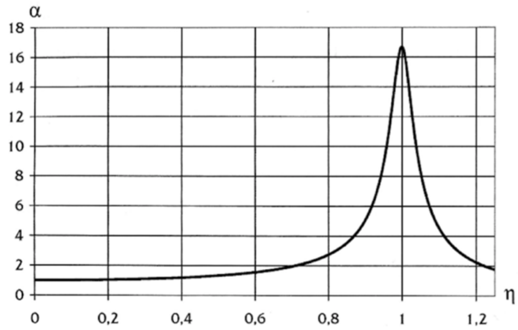
In this work, only measurement methods or devices are described which are used to determine the paths, the speeds and the acceleration.



a) relative measurement

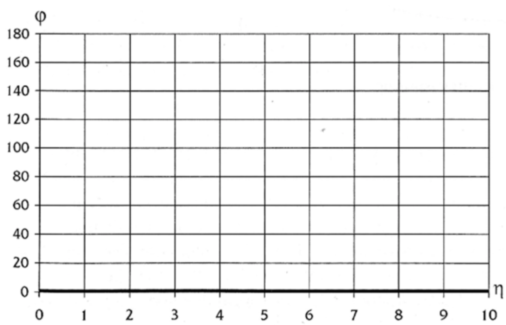


b) absolute measurement

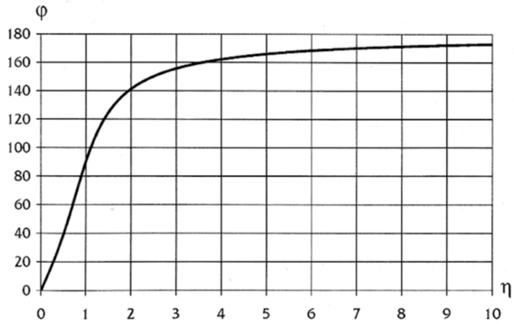


c) force measurement

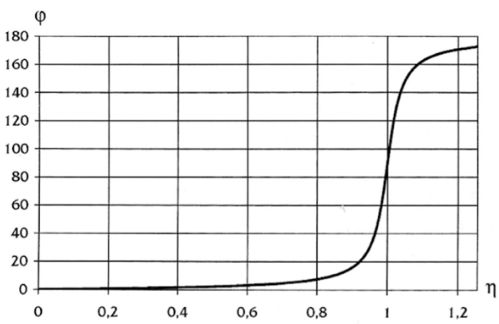
Fig. 5.1: amplitude frequency response α in dependency oft he frequency ratio η



a) relative measurement



b) absolute measurement



c) Force measurement

Fig. 5.2: phase frequency response φ in dependency oft he frequency ratio η

5.1. Path measurements

A mechanically acting distance measuring device consists of a stylus. It moves with the object to be measured and transmits through a lever system which acts as an amplifier to a writing system that is moving at a constant speed. (Fig. 5.3)

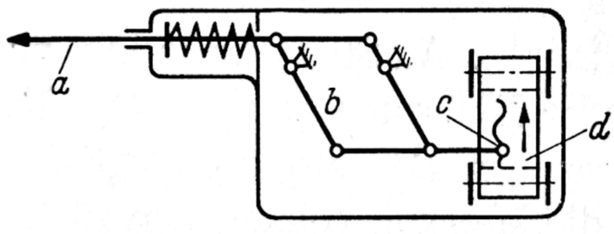


Fig. 5.3: odometer with mechanical transmission elements; a = stylus, b = transmission lever, c = pen, d = movable waxed paper

There are two methods for the electrical procedure: the inductive and the capacitive measuring method:

An inductive measuring device consists in turn of a stylus. This is connected to a dip anchor, a ferromagnetic material such as soft iron. This anchor is located in a magnetic field of two windings forming the branches of an alternating current bridge. The bridge is fed with alternating voltage of high frequency and is balanced in such a way that in the zero point of the dip anchor no voltage is applied in the bridge diagonal AB. If the anchor moves, the inductance of the two windings changes, and an alternating voltage occurs from the frequency of the applied voltage, which is amplified and displayed on an instrument. The electric charge resulting from this is proportional to the displacement. Fig. 5.4 schematically illustrates such an inductive displacement sensor.

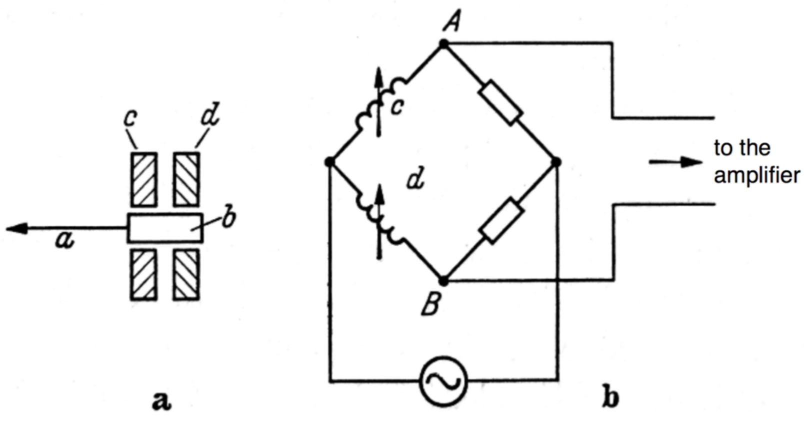


Fig. 5.4: inductive Odometer; a) inductive encoder, a = stylus, b = dip anchor, c = d = electrical winding; b) electrical sensor bridge

Basically, the capacitive measuring method is similar to the inductive method. Capacitors are used instead of the inductors. The plate capacitors move relative to each other, which leads to a change in the alternating current resistance. Such methods are used when vibration movements of rotating parts are to be measured without contact.

5.2. Speed measurements

A speed-sensing measurement system consists of a stylus, which is provided with a winding. This is located in a plunger coil, a permanent magnet. By moving the stylus, voltages are induced in the winding. According to Maxwell's law of electricity, these voltages are proportional to the speed of relative motions between the winding and the magnetic field. Thus, these tensions are again amplified and can be indicated by an instrument.

The speed is the change over time of the displacement, so one can also use distance measuring devices to determine it. The measured values obtained from such a sensor are subjected to a running differentiation. This differentiation can be implemented with the help of electrical circuits.

The disadvantage of this system is that only relative measurements can be carried out. If these systems are used to determine absolute values, then a fixed reference system must be determined, which isn’t always available. A simple solution is a sprung coupling of an auxiliary mass of suitable size to the housing of the sensor. The auxiliary mass is a spring-excited oscillator. The housing is placed on the measurement object and the auxiliary mass is excited by this housing to vibrate. A measuring system is arranged between the auxiliary mass and the housing. The sensor thus doesn’t evaluate the vibration quantity to be measured, but the relative value v_c between the auxiliary mass and the housing. It can be shown that the ratio between the relative value v_c and the oscillation value v_A to be measured depends on the frequency ratio η:

$$\frac{v_c}{v_A} = \frac{1}{1 - \frac{1}{\eta^2}}$$

The frequency ratio is the ratio of the measuring frequency Ω and the characteristic frequency f_0 of the measuring device. This characteristic frequency is dependent on the stiffness k of the spring which couples the auxiliary mass to the housing and the weight m of the mass:

$$f_0 = \frac{1}{2\pi}\sqrt{\frac{k}{m}} = \frac{\omega_0}{2\pi}$$

This is illustrated graphically in Fig. 5.5.

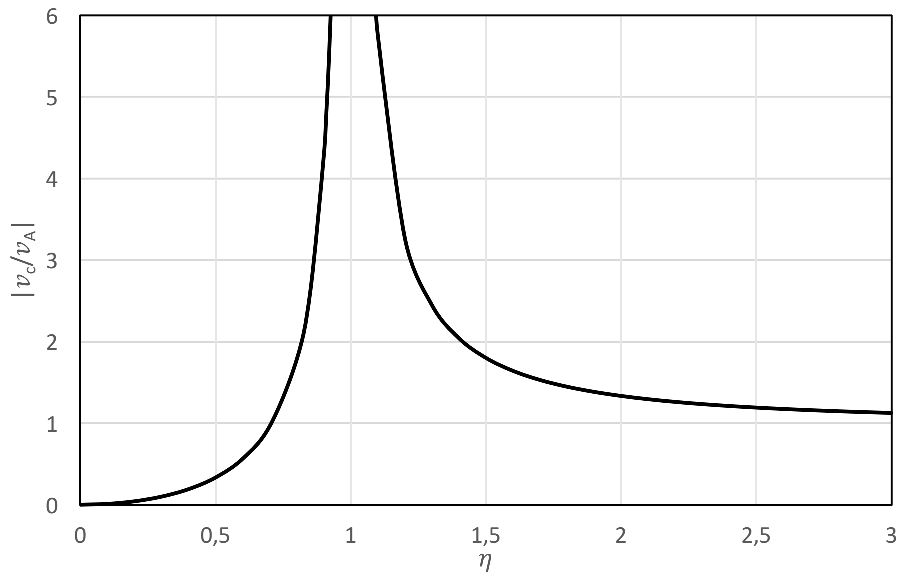


Fig. 5.5: frequency response for the undamped measuring system

As can be seen in Fig. 5.5, the deviations between the oscillation value v_A to be measured and the relative value v_c showed on the display are extremely high at small frequency ratios. Only if the frequency ratio is big, then, sufficiently accurate matches can be obtained. If the measuring errors don't exceed a prescribed value, then the frequency of the measured variable must not fall below a certain lower limit frequency and the device must be tuned as low as possible.

The frequency profile can be substantially flattened in the resonance range $\eta = 1$, if a speed-proportional attenuation element is arranged between the housing and the auxiliary mass. The quotient is calculated as follows:

$$\left|\frac{v_c}{v_A}\right| = \frac{\eta^2}{\sqrt{(1-\eta^2)^2 + (2\vartheta\eta)^2}}$$

with ϑ = the attenuation.

Fig. 5.6 shows the frequency response with different attenuation. For $\vartheta = 0{,}5$ in the resonance range $\eta = 1$, there is sufficient exact matches of the relative size and the variable to be measured.

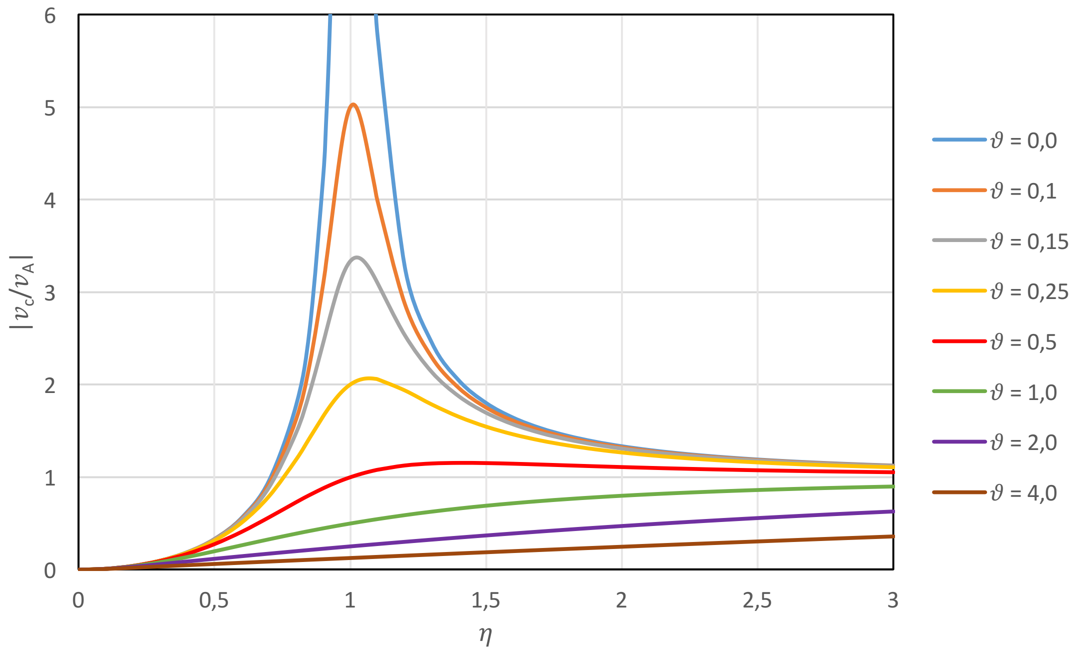


Fig. 5.6: frequency response for the damped measuring system

Many devices operate with an attenuation between 0,5 and 0,6, in order to suppress interfering natural oscillations of the reference mass due to random vibrations.

5.3. Acceleration measurement

It is shown that the effective relative path amplitude w_c at the measuring sensor is proportional to the amplitude of the acceleration $\ddot{w}_A$ of the housing and thus also of the measuring object. This is especially true when the frequency ratio η is much smaller than 1. The function:

$$\frac{v_c}{v_A} = \frac{1}{1 - \frac{1}{\eta^2}} = \frac{w_c}{w_A}$$

can be rewritten as follows:

$$\frac{\mathrm{w_c}}{\mathrm{w_A}} = \frac{\eta^2}{\eta^2 - 1}$$

If $\eta \ll 1$, then you get:

$$\frac{w_c}{w_A} = \frac{\eta^2}{\eta^2 - 1} \sim -\eta^2 = -\left(\frac{\Omega}{\omega_0}\right)^2$$

The expression $-\Omega^2 w_A$ represents the amplitude of the acceleration. For this reason the following formula is obtained:[31]

$$w_c = \frac{\ddot{w}_A}{\omega_0^2}$$

An absolute measuring sensor is, for example, a strain gauge acceleration sensor. This consists of a seismic mass and a spring, which is usually designed as a bending beam. The strain gauge is mounted on the beam to measure its deformations. The problem with these sensors is that they are temperature-sensitive, which influences the long-term consistency of the measurement. In addition, the application of the strain gauge to the small bar is very complex, which leads to these sensors being quite expensive and only used in special cases.

Another absolute measuring sensor is the capacitive acceleration sensor. These measure the acceleration of an object via a change in the capacitance of a plate capacitor. The auxiliary mass or seismic mass is resiliently suspended and formed as an electrode of the capacitor. The mass moves and changes the air gap. Their motion is measured by this movement. Use of two counter electrodes results in a capacitance increase of the capacitor to which the mass moves and a reduction in the capacitance of the second one to which the mass moves away. The impedance at the output is relatively low to allow the connection of long cables. Before each measurement, a zero offset calibration must be performed so that an unavoidable zero drift can occur. These types of sensors are very sensitive to interferences. The capacitive bridge circuit, which provides a high transmission coefficient, causes the sensor to operate at a high frequency as a carrier frequency.

An important sensor is the piezoelectric accelerometer. Depending on the design, a force such as pressure or shear induced an electrical charge displacement on the surface of piezoelectric materials, which is dissipated to the outside. This process is indicated as a piezoelectric effect. The charge, which is generated by the force and is proportional to a piezoelectric constant k_p to it, cannot flow off immediately due to the low conductivity of the piezoelectric materials and is therefore tapped via electrodes. The force on the piezoelectric element is due to the action of the acceleration $\ddot{x}$ to be measured on the seismic mass m of the sensor. On the basis of the Coulomb law, the voltage can be determined as the ratio of the charge Q and the total capacitance C, which is the sum of the capacitance of the piezoelectric element C_P used and of the scattering capacity of the connected cable C_{St}. Thus, an output voltage U_q proportional to the acceleration $\ddot{x}$ is obtained:

[31] Dr. Ing. Erhard Hübner; Technische Schwingungslehre in ihren Grundzügen; p. 273 - 290

$$U_q = \frac{Q}{C} = \frac{k_p}{C_P + C_{St}} m\ddot{x}$$

The piezoelectric constant k_p takes into account the characteristics of the piezoelectric materials used, the direction of the load, and the design of the sensor. The springs of the sensors consist of such a piezoelectric element, which is produced from a single-crystalline material, such as quartz or polycrystalline piezoceramics, for example lead zirconate – lead titanate ceramics. This spring has a high modulus of elasticity so that a high spring constant and a high natural frequency are obtained.

The output voltage must be edited before the further signal processing for two reasons:

- The voltage is a function of the capacity of the cable.
- The high impedance results in a high sensitivity to the interference of disturbances from the neighbourhood.

For this reason, special amplifier circuits have to be used which, among other things, invert the sensor signal and lead to the fact that the output voltage is no longer depending on the capacity of the cables and the cable length, but only on the capacity of the charge amplifier.[32]

5.4. A measuring device of the KIT

The "Materialprüfungs- und Forschungsanstalt Karlsruhe" (MPA) of the Karlsruhe Institute of Technology (KIT), has been using the MR2002 of SYSCOM (Fig. 5.7) for a long time to measure vibrations on structures. This consists of two red boxes: a small vibration sensor and the vibration recorder, where the vibrations are stored and where the system is controlled. However, these settings must be specified using a software that must be installed on the computer. In a second moment, a modem can be connected to it, in order to access from the office to check its functionality and to change certain parameters.

With this measuring principle one can measure general oscillations, determine the natural oscillation of the structures or record a vibration event. In the latter case, the limit values for the oscillation speed can be specified and, as soon as these are exceeded, a certain number of values are stored. In addition, a text message can be sent over the modem to a mobile phone or to an e-mail as an alarm.

Before the actual measurement is started, a "baseline correction" must be carried out. This correction determines that the recorded signal is centred around the zero.

In the case of long-term vibration monitoring, for example during demolition work, the background recording mode is often set. This enables the storage of the maximum measured value, which is achieved within a set period of time.[33]

[32] Thomas Kuttner; Praxiswissen Schwingungsmesstechnik; p. 101 - 140
[33] User Manual MR2002 – CE; SYSCOM Instruments SA; Switzerland

Fig. 5.7: measurement system MR2002 of SYSCOM

5.5. A new measuring device of the KIT

With the measuring system MR2002 from SYSCOM, an alarm can only be sent by a text message. The disadvantage of this variant is that the amount of mobile phone numbers which can be specified is strongly limited. Besides, not all construction workers have a mobile phone on the construction site or they don't hear it. Therefore, it was necessary to find a measuring system which has similar measuring functions as the MR2002 and which emits an acoustic and visual alarm during certain vibrational events. For this reason, the author of this work has worked out a measurement system with the software DIAdem™ and measurement card from National Instrument. This is an interactive software that can be used to measure, find and manage data, analyse it mathematically and graphically, and display it in reports. It also consists of a script to automate repetitive applications with some of its own programming language and some VBScript language.

The goal was to store individual vibration events during the measurement in different files and simultaneously to output an acoustic signal. An event includes measured values that are recorded one second before and five seconds after the limit value is exceeded. To solve this problem, selected circuit diagrams have been linked in DIAdem - DAC in a certain way.

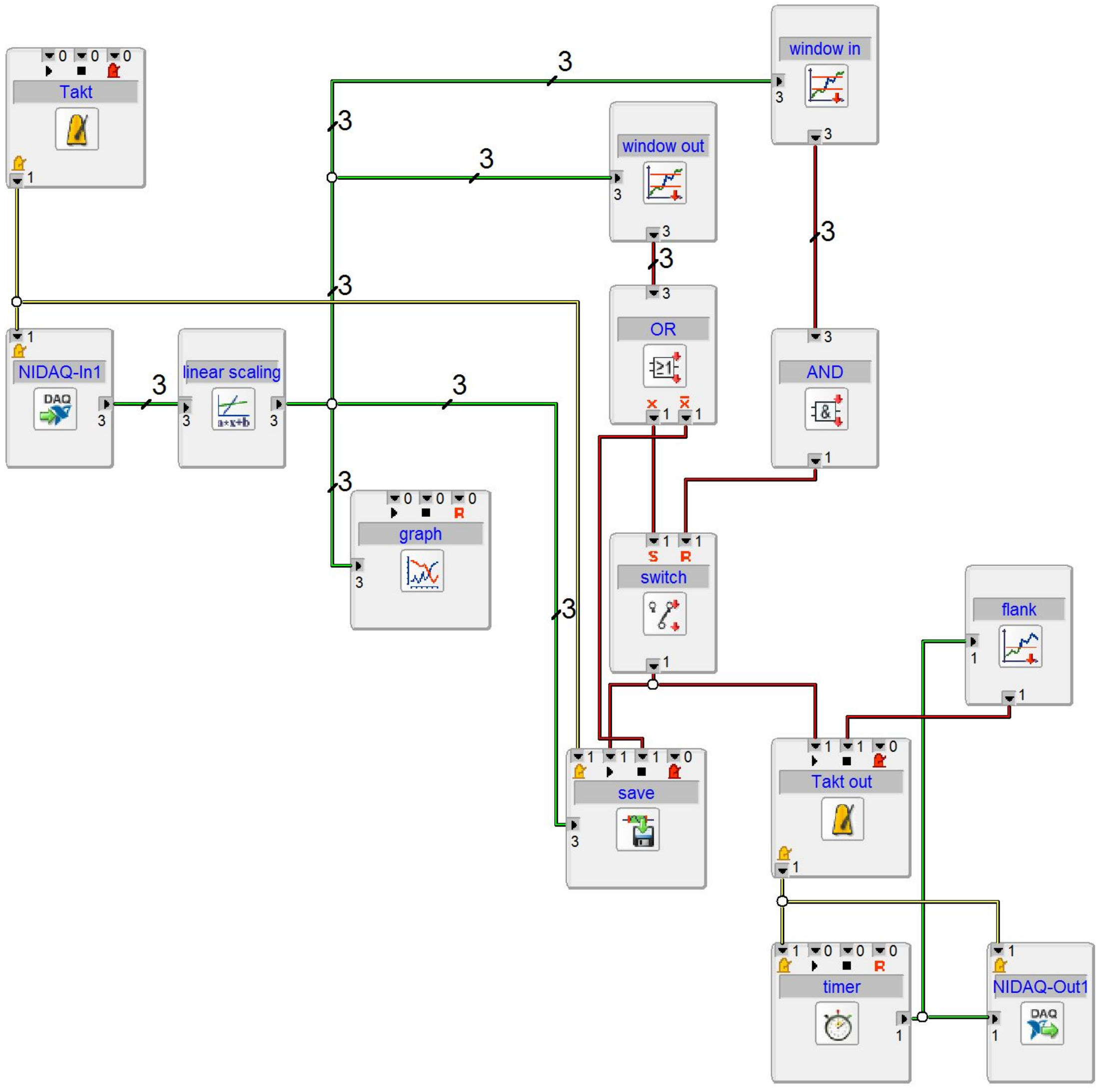


Fig. 5.8: circuit diagram in DIAdem – DAC for three channel

As can be seen in Fig. 5.8, the circuit diagram can be connected to data lines (green), system lines (yellow) and control lines (red), depending on its typ. In "Takt", the sampling rate is set, so it is determined how many values in one second must be measured. This is connected to "NIDAQ-In1" via a system cable, in order to route the information to it. It is related to the external measurement card. That means the measuring channels are set here. The data are fed via a data line to the "linear scaling", which multiplies the values by a factor of 34,7, in order to convert the measured voltages into oscillation speeds. From here, they are sent to four buttons. "Graph" is only used for the immediate display of the long-term measurement. In "window out" and "window in", the limit value is defined and compared with the entered values. "Window out" sends a control signal to "OR" as soon as the limit value in a channel is exceeded. On the other hand, "window in" sends a control signal to "AND" if the limit value in a channel isn't exceeded.

"OR" is a button where the incoming channels (in this case 3) are investigated. As soon as one of these channels sends a signal, it redirects a new signal to the "switch". If this condition isn't met, a signal is sent to the stop switch of the "save" button to indicate when DIAdem should stop the storage.

All individual channels are examined in the "AND" circuit diagram. As soon as all send a signal, it sends a new signal to the "switch". The latter checks the incoming signals: a signal which comes from "OR", is sent to the "takt out" and to the start command of the "save", to specify from which values DIAdem should store. If the signal comes from the "AND", no signal is forwarded.

So, recapitulate, the "save" button starts to save as soon as a signal comes from the "switch" and stops once the signal comes from "OR". In this circuit diagram, the number of values, which must be recorded before (pre-trigger) and after (post-trigger) the limit value must also be stored, is set. In this case, it was set to start recording one second before the event and end five seconds after the border crossing. After these five seconds, the measurement is stopped completely.

The "takt out" starts sending a system signal to the "timer" as soon as the "switch" sends a signal to it. The "timer" delivers a signal to the "NIDAQ-Out1", which causes the acoustic alarm. At the same time, the "flank" receives a signal, which serves to stop the "clock out", and thus the "timer" after a certain time.

Now we had to find a method to restart the measurement after each event. The solution is to program a loop in the script (Fig. 5.9).

```
21 DIM i
22
23   i=1
24   ' Do not load DAC file with user function for automatic filename generation
25   Call SCHEMELOAD(sPathDocuments & "Auto_Filename")
26   Call SCHEMECHECK("normal")
27
28
29 Do While (i<3)
30   select Case MsgBoxDisp("Nein klicken um Messung abbrechen","MB_YESNO",,,1)
31     Case "IDNo"
32       exit do
33     Case "IDYes"
34     call SCHEMEMEASSTART()
35   end select
36 loop
```

Fig. 5.9: script to restart the measurement

Lines 25 and 26 of the script open the DIAdem – DAC file and check for any errors in it. Lines 29 to 36 represent an infinite loop. Before the start of each measurement, a message (line 30) appears for one second, which is used to get out of the infinite loop as soon as the "NO" button is pressed. If the "Yes" button is pressed or after 1 second, there isn't a response, the measurement starts again.

6. Effects on buildings - DIN 4150 – 3

The DIN 4150-3: 2016-12 is the German standard, which deals with the method for the determination and evaluation of the impacts caused by vibrations on building structures. The assessment is based on the examination of maximum values, which are recorded by distance, speed and acceleration sensors. The standard distinguishes between permanent shocks and short-term shocks. On the basis of previous measurements, the DIN 4150-3 gives indications of the vibration speed, where no damage is caused. If, nevertheless, some cracks are formed, so there are other reasons, which must be found. However, the exceedance doesn't mean that a damage will occur at the same time. In this case further investigations of stresses and resistances could be necessary.

The stresses can be determined by calculation, or by measuring the strain on the vibrating component and by using the constitutive law. The amplitude and frequency of the measured displacements, velocities and accelerations are the starting variables for their calculation. When the measurement is made close to the location of the largest amplitude, the stresses of oscillating beams or plates, which are close to the resonant, can be determined approximately from the vibrating velocity amplitude.

The load bearing capacity must also be proved against fatigue failure, with taking into account the corresponding safety. This exact verification may be neglect in the proof of the bearing capacity, if the dynamic load components are multiplied by the fatigue coefficient 3. If the dynamic stress ratio is less than 10% of the statically permissible stress, the test against fatigue failure must not be carried out. For residential buildings and structures which are particularly sensitive to shocks, the value in use can be reduced if:

- Cracks appear in the plaster of walls;
- Existing cracks increases;
- The partition walls aren't attached with the load-bearing walls or ceilings anymore.

These "minor damages" affect the stability of the buildings and the load-bearing capacity of components. In the case of commercial buildings, these minor damages don't affect the value in use.

6.1. Short-term shocks

Such shocks are those whose frequency of occurrence isn't sufficient to cause a material fatigue phenomenon and whose duration isn't suitable to cause an increase in the vibrations due to resonance phenomenon.

6.1.1. Evaluation of an overall construction

The measured vibrational velocities in the uppermost ceiling level or on the foundations are decisive for the evaluation of the overall structures. In the case of the top ceiling, the largest value of the two horizontal individual components i = x, y is taken as a basis. For the foundations, on the other hand, the maximum value of all three components i = x, y, z is used. The values for $v_{i,\,max}$ at the foundation and on the top ceiling are listed in table 6.1 for different building types. The values of the foundations are frequency-dependent and take into account the transfer behaviour from the foundation to the top ceiling. That means, for the classification in the frequency ranges, the frequency that occurs in the range of the maximum oscillation speed value must be used.

building types	reference value for $v_{i,max}$ [mm/s]			
	foundation i = x, y, z frequency range			top ceiling horizontal i = x, y
	1 – 10 [Hz]	10 – 50 [Hz]	50 – 100 [Hz]	all frequency
commercial and industrial buildings and similarly structured buildings	20	0,5·f+15	0,2·f+30	40
residential buildings and buildings with similar construction	5	0,25·f+2,5	0,1·f+10	15
buildings with special vibration sensitivity and which are particularly worthy of preservation	3	0,125·f+1,75	0,04·f+6	8

Table 6.1: reference value for $v_{i,max}$ for evaluating the effects of short-term shocks

It should be noted that the values given in table 6.1 for commercial and industrial buildings cannot rule out, that minor damages are formed. For values measured with a frequency higher than 100 Hz, the reference values of 100 Hz can be used. In Fig. 6.1, the course of the absolute maximal values at the foundation is shown as a function of the frequency. [34]

[34] DIN 4150-3:2016-12 Vibrations in buildings – Part 3: Effects on structures; p. 5 – 9

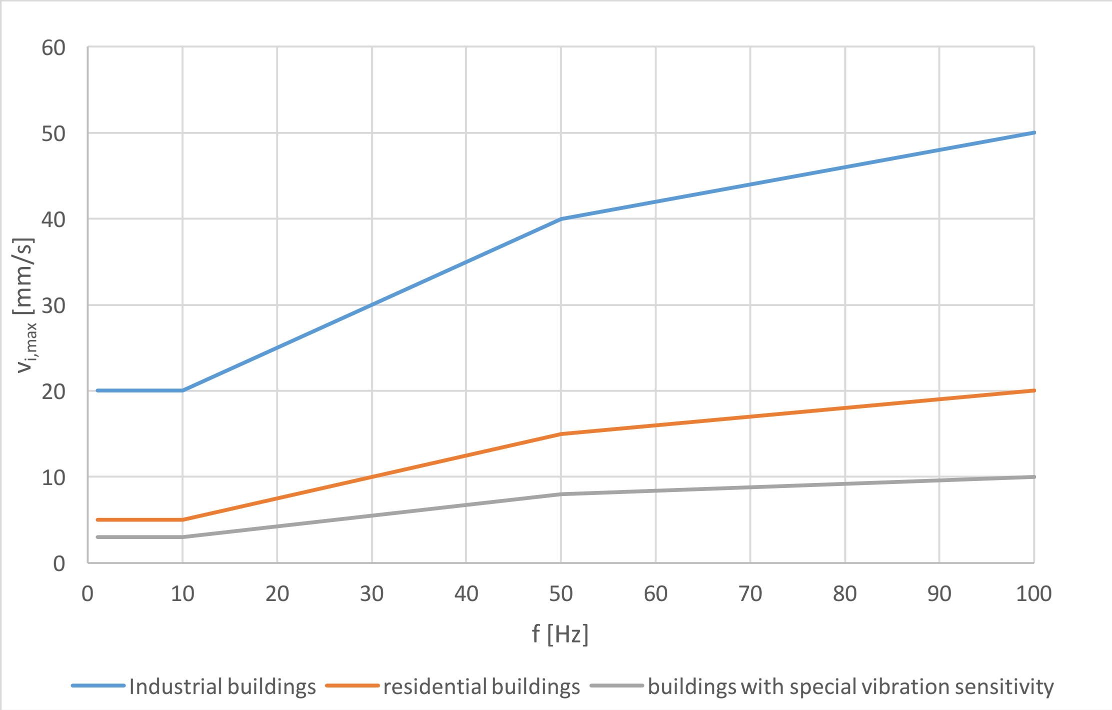


Fig. 6.1: Graphical representation of the reference values on the foundation

The problem is that the methods used to determine the frequency produce different results. Thus the determination of the frequency to the relevant reference value plays an important role. Therefore, they can sometimes influence the assessment result.

An alternative evaluation method is proposed in DIN 45669-1: 2010-09, in which the assessment with frequency-independent reference values was carried out by multiplying the measured unvalued oscillation speeds with the filter H_{vBi}. They are then compared with the reference values of the foundations between 1 and 10 Hz of table 6.1. The target transfer functions of the evaluation filters H_{vBi} are specified in table 6.2 as a function of the frequency range and building type:

building types	frequency range			
	< 10 Hz	10 – 50 Hz	50 – 100 Hz	> 100 Hz
commercial and industrial buildings and similarly structured buildings	1	$\frac{20}{0{,}5 \cdot f + 15}$	$\frac{20}{0{,}2 \cdot f + 30}$	$\frac{20}{50}$
residential buildings and buildings with similar construction	1	$\frac{5}{0{,}25 \cdot f + 2{,}5}$	$\frac{5}{0{,}1 \cdot f + 10}$	$\frac{5}{20}$
buildings with special vibration sensitivity and which are particularly worthy of preservation	1	$\frac{3}{0{,}125 \cdot f + 1{,}75}$	$\frac{3}{0{,}04 \cdot f + 6}$	$\frac{3}{10}$

Table 6.2: target transfer functions $H_{vb,i}(f)$

The target transfer functions are the inverses of the functions of the reference values in table 6.1, then multiplied with the reference values of the frequency range 1 - 10 Hz. Furthermore, the standard specifies that the exact determination of these functions with a digital filter can be very complex. Therefore, the determined transfer function doesn't have to deviate more than ± 5% of the target function. In Fig. 6.2, the functions indicated in the table 6.2 are graphically represented. It is important to note that the phase frequency response of an evaluation filter must be linear in order not to influence the evaluating result. [35]

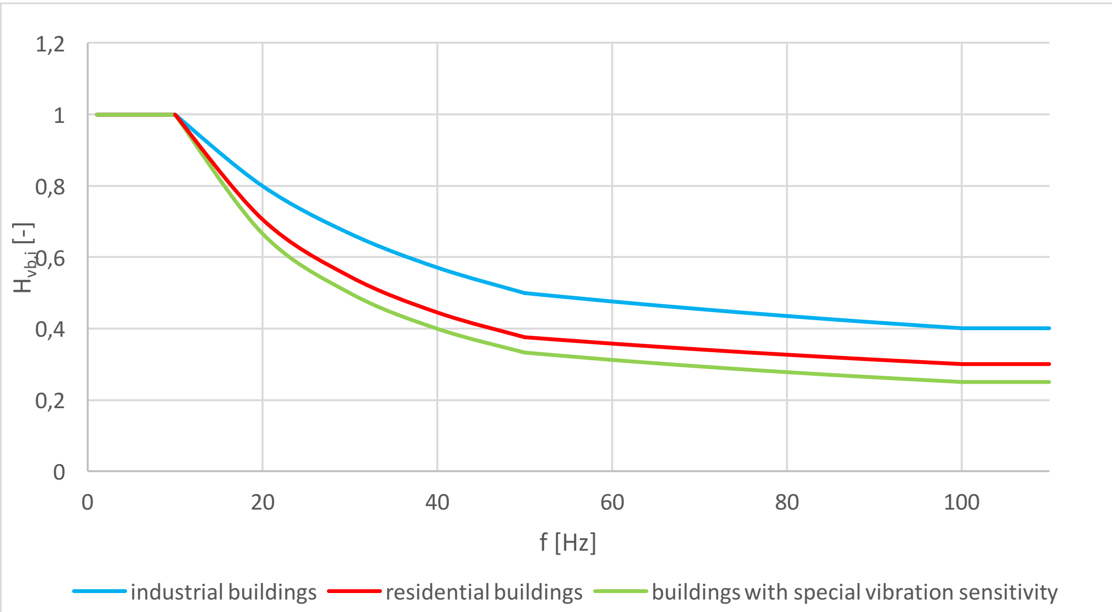


Fig. 6.2: Graphical representation of the target transfer functions

[35] DIN 45669-1:20120-09 Measurement of vibration immission – Part 1: Vibration meters – Requirements and tests; p.49 – 51

6.1.2. Assessment of ceilings

If vibrations are generated in ceilings due to short-term vibrations, the maximum oscillation velocities in the vertical direction must be less than 20 mm/s for all building types. The maximum value often occurs in the middle of the roof. In the case of structures which are particularly sensitive to vibration or which are particularly worthy of preservation, it is sometimes necessary to reduce the 20 mm/s in order to prevent minor damages.

6.1.3. Assessment of massive components and subterranean structures

For steel reinforced concrete, the standard specifies a maximum reference value of 80 mm/s, as long as there is no risk of soil mechanical processes.

If lining of underground cavities is carried out according to the state of the art, the reference values for different building materials are as follows:

- Steel reinforced concrete and shotcrete: $v_{i,\ max}$ = 80 mm/s
- Concrete, natural stone: $v_{i,\ max}$ = 60 mm/s
- Masonry: $v_{i,\ max}$ = 40 mm/s

If the work isn't carried out properly, these speeds must be reduced.

6.1.4. Assessment of underground pipelines

If the pipes are laid according to the state of the art, depending on the pipeline construction material, the oscillation speeds can be maximally as follows:

- Steel, welded: $v_{i,\ max}$ = 100 mm/s
- Metal with or without flanges: $v_{i,\ max}$ = 80 mm/s
- Stoneware, concrete, steel reinforced and prestressed concrete: $v_{i,\ max}$ = 80 mm/s
- Masonry, plastic: $v_{i,\ max}$ = 50 mm/s

Correspondingly, these values must be reduced if the pipes haven't been produced and laid according to the current state of the art, and if there are consequences from soil mechanical processes. For house connection lines closer than 2 m to the foundation, the reference values of table 6.1 must be used.

6.2. The permanent vibrations

As the name implies, permanent vibrations lead to longer oscillation events, with the risk that the entire structure will resonate.

6.2.1. Evaluation of an overall construction

The maximum horizontal vibration velocities of the overall structure must be known for the assessment. As a rule, these are measured in the top ceiling level. The greatest speed between the two horizontal components is taken and compared with the reference values which depend on the building type. These values are as follows:

- For commercial buildings, industrial buildings and similar structures:
  $v_{i,\,max} = 10$ mm/s
- For residential buildings and buildings with similar construction:
  $v_{i,\,max} = 5$ mm/s
- For buildings with special vibration sensitivity and which are particularly worthy of preservation: $v_{i,\,max} = 2{,}5$ mm/s

In the case of industrial buildings, it cannot be excluded that for $v_{i,\,max} < 10$ mm/s, there are minor damages.

Alternatively, the speeds can also be measured in the foundation area. In this case, the transfer behaviour from the foundation to the top ceiling level must first be determined sufficiently precisely and taken into account in the assessment. This behaviour can be carried out mathematically, on the basis of experience and metrological, whereby the latter should always be preferred. The excitation frequency range and the signal characteristics of the oscillation source must be observed for the determination.

6.2.2. Assessment of ceilings

For all building types the maximum vertical oscillation speed must not exceed 10 mm/s. In commercial, industrial and residential buildings, no damage occurs in compliance with this information, even if the components of the structure are fully exploited in the static design. Even if the vibrations don’t form a crack, they are very noticeable. In structures that are sensitive to them, it isn’t always possible to guarantee that no damage occurs, therefore any reductions are necessary.

It is possible to determine the dynamic additional stress at resonance-induced bending vibrations. The maximum bending stress for beams and uniaxially tensioned plates with a constant stiffness and even mass coverage can be calculated independently of the system dimensions as follows:

$$\sigma_{max} = \frac{y_{max}}{i}\left(\frac{E_{dyn}G_{tot}\rho}{G_{beam}}\right)^{0,5} k_n v_{i,max}$$

Where:

- y_{max} distance of the extreme fiber
- i radius of gyration
- E_{dyn} dynamic modulus of elasticity
- ρ building material density
- G_{tot} dead load and additional loads on the beam

- G_{beam} dead load of the beam
- k_n eigenmode parameter. It depends on the boundary conditions and on the order number of the eigenmode. The value is between 1.0 and 1.3.

6.2.3. Assessment of underground pipelines

When evaluating permanent vibration, the reference values in chapter 6.1.4 are reduced to 50%. In addition, the restrictions listed in 6.1.4 must be applied.[36]

[36] DIN 4150-3:2016-12 Vibrations in buildings – Part 3: Effects on structures; p. 9 – 12

7. Monitoring and evaluation of a brick chimney

Bretten is a small town in the western Kraichgau and about 23 km north-east of the city of Karlsruhe in Baden-Württemberg. On the B35 and south of Steinzeugstraße is the long-standing Harsch-Steinzeugwerk (Fig. 7.1). In 2016, a new master plan was established for this zone: the new "stone park" with new residential and commercial areas is to be built on the foundation walls of the old plant. The plan provides for the demolition of the mighty stonework on the approximately 26.000 m^2 of land, and only a high brick chimney is reminiscent of the glorious past of the company and its surroundings.[37]

Fig. 7.1: Satellite image of the „Harsch-Steinzeugwerk" with the chimney (yellow circle)

During demolition work by means of hydraulic rock chisels and tongs, shocks occur at the 30 m high chimney, which at a certain intensity affects the load bearing capacity of the same. For this reason, an oscillation measuring system on the chimney is installed during the entire demolition intervention in order to measure the vibration velocities and,

[37] http://hügelhelden.de/brettener-steinzeugwerk-wird-zu-steinzeugpark/

as soon as a critical value is exceeded, the workers at the construction site are to be alarmed.

This chapter attempts to determine the values of the oscillation speed at the foundation and at the head of the chimney, where a risk to the structural stability of the chimney due to vibrations is improbable. These are then compared with the measured data. Finally, an example is explained, how one can predict the oscillation speed at the foundation as a function of distance and mass as a result of falling components and to compare it with the measured data.

7.1. Building description

According to the plans of 1963 the 54-year-old chimney rises 30 m out of the ground. Its diameter is 2,56 m on the ground and 1,36 m on the head. The wall has four varying thicknesses over the height. They remain constant between the jumps and amounts at the bottom to 50 cm and at the head to only 18 cm. In Table 7.1, the diameters, wall thicknesses and the cross-sectional area, which are derived from the execution plans, are summarized as a function of the height.

section	high [m]	Segment length [m]	ϕ_{out} [m]	Wall thickness [m]	Cross-section area [m]
1	0,00 1,00	1,00	2,56 2,56	0,50	3,236 3,236
2	1,00 4,00	3,00	2,56 2,40	0,38	2,602 2,411
3a	4,00 9,00	5,00	2,40 2,20	0,31	2,035 1,841
3b	9,00 22,00	5,00	2,20 2,00	0,31	1,841 1,646
4a	14,00 18,00	4,00	2,00 1,84	0,25	1,374 1,249
4b	18,00 22,00	4,00	1,84 1,68	0,25	1,249 1,123
5a	22,00 26,00	4,00	1,68 1,52	0,18	0,848 0,758
5b	26,00 30,00	4,00	1,52 1,36	0,18	0,758 0,667

Table 7.1: Sectional description of the chimney

The base is conical and 3,55 m embedded in the ground. It is mounted on a 0,8 m thick plate with a diameter of 4,8 m. In the base there is a 1,00 m and 2,25 m high flow hole.

In the lower part, a 20 cm thick and 2,30 m high cuff was attached at a later time to reinforce the tower. Only in the lower half, this cuff is weakly reinforced in the circumferential direction. For this reason, the steel reinforcement cannot sufficiently absorb the possible circular tensile stresses.

No subsoil expertise, static calculation and material investigations were found. Therefore, there is no detailed information on the loads, the compressive strength of the masonry, the permissible base compression, the soil and material stiffness.

The chimney was renovated in 1995 after cracks have formed due to storms. It is believed that the cuff described above was built after this event. In addition, the tower was enclosed with an in-situ concrete slab. As of 1995, the chimney also served as a carrier for a mobile communications system, which has now been dismantled.

The chimney isn't used at the moment and is therefore closed at the head by a concrete plug to a small opening for ventilation. The supply air duct is closed by dirt and debris. About 3,70 m above the cuff some tie rods were installed about every 110 cm to reinforce the tower. 23 pieces were counted. In Fig. 7.2, these three amplification regions are shown.

With the "Schmidt – Hammer" [38] the tile resistance was estimated. It has been found that the bricks have a relatively high compressive strength of more than 10 N/mm^2. The mortar is most likely a cement mortar with an equally high strength. As mentioned, there is no secured information on the materials used. For this reason, a conservative masonry strength of f_k = 4.8 N/mm^2 is estimated.

[38] The rebound hammer („Schmidt – Hammer") is an instrument for the non-destructive testing of materials, with which the compressive strength, e.g. of concrete can be measured point by point

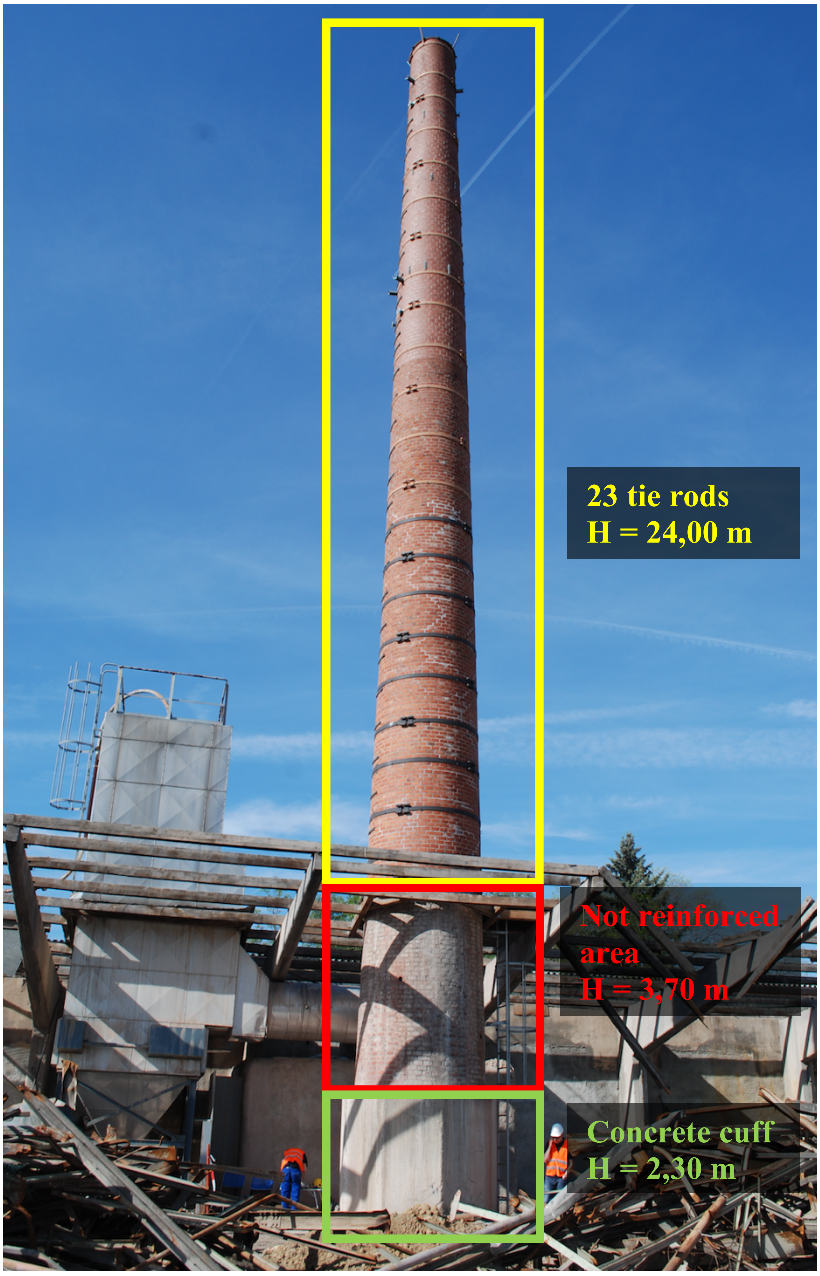


Fig. 7.2: Areas of reinforcement of the chimney

Several pronounced cracks can be found on the reinforced concrete cuff. Above the cuff, there are also cracks in various position which point to an expansion of the chimney. A pronounced vertical crack with a width of about 10 mm is noticeable. (see Fig. 7.3 and Fig. 7.4). They indicate that the circular tensile strength has been exceeded at any time. Apparently, the chimney also exhibits an oblique position in the direction of the pronounced crack. It is possible that the above-mentioned mobile communication system, which was installed on this side, is also responsible for the extent of the vertical crack.

Fig. 7.3: Crack width of the vertical crack

Fig. 7.4: Spread vertical crack

7.2. The demolition works

The chimney is surrounded by several buildings of the old stone factory. These are demolished with demolition hammer. The destruction of components is taken place by means of transmission of impact energy. In the hammer housing is the striking mechanism, which is fed with the necessary hydraulic fluid of the excavator. By the piston stroke, the generated impact energy is transmitted to the impact piston, which then passes the same to the chisel. A damping system must be installed for vibration isolation and reduction of the rebound force. The lifetime of the hammer can be increased by means of special low-voltage fuses and devices for energy recovery, as well as oil cushions for the attenuation of low-frequency oscillations. A special equipment for hydraulic hammers is a control device for the adjustment of the stroke rate and the impact energy at the local conditions of use. These hammers have a significantly higher impact capacity in relation to their weight. Depending on the type of material, different types of chisels are used; there are: pointed, flat, stump and upsilon types. For soft material a high stroke rate and low impact energy is necessary. For hard material this must be exactly the opposite. The impact energy can be between 130 and 14,000 joules and the performance is very high even at higher altitudes. Disadvantages of this process are, among other things, the risk of fragmentation and the considerable wear and tear costs caused by chisel and socket wear. In order to counteract the first case, all workers in the immediate vicinity must protect themselves with appropriate clothing.

In addition, demolition tongs were also used. They break the material structure and separate it from the building, since they can cut the reinforcement. They also comminute different materials. The internal structure can be destroyed by the mechanical pressure of the crushing jaws. There are different types: combi-demolition tongs, concrete tongs, primary pulverizing tongs and secondary-pulverizing tongs. It is important to avoid ramming, pushing and pulling. The breaking force is 600 kN for mechanical up to 5,000 kN for hydraulic demolition tongs. However, the mechanical means have the advantage that no additional hydraulic lines and control device are needed. They also have low susceptibility to failure and low repair and wear costs. Hydraulic demolition tongs, on the other hand, can be rotatable in order to achieve the highest performance in any position, but people with good experience are needed. Moreover, noise pollution is low. It is disadvantageous that an additional hydraulic circuit is required in order to bring the forceps into motion. The maintenance effort is also increased here.

Both devices are used to create component materials which, in the case of the old stonework, can drop down to a height of 4/5 m and cause short-term vibrations. Their size can vary greatly: they can be from a few cubic centimetres up to half cubic meters. [39]

7.3. Approach

In the case of a vibration, oscillations always occur, which can lead to a loss of stability on reaching a certain amplitude maximum. In order to estimate the oscillation amplitudes at the chimney head, which don't lead to a failure yet, one calculation variant is carried out: the maximum occurring displacement amplitude at the chimney head is estimated, which has appeared with great probability during the 54-year service life and which hasn't led to any complete loss of stability. The deformation is determined by the wind according to DIN EN 1991-1-4. This effect is the 50-year-old event, which has probably already occurred with a high percentage.

[39] Dr. Ing. Dietrich Korth, Dipl. Ing. Jürgen Lippok; Abbrucharbeiten – Vorbereitung und Durchführung; p. 293 - 305

7.3.1. Calculation model

The chimney was modelled using the FE program LUSAS. This was as a firmly clamped cantilever from the top edge of the terrain. The diameter changes over the height. Approximately, the tower is divided into several sections of constant diameter and a height of 1 m. Table 7.2 lists the sections with the characteristic values. The soil-building interactions are neglected.

The following material parameters were defined in the calculation model:

- Masonry compressive strength f_k = 4,8 N/mm^2
- Elasticity modulus E = 4,8 × 1100 = 5280 N/mm^2
- Brickwork density ρ = 1800 kg/m^3

Outer diameter [m]	thickness [m]	weight [kg]	Outer diameter [m]	thickness [m]	weight [kg]
2,56	0,50	5825	1,90	0,25	2333
2,53	0,38	4620	1,86	0,25	2276
2,48	0,38	4513	1,82	0,25	2220
2,43	0,38	4405	1,78	0,25	2163
2,38	0,31	3629	1,74	0,25	2106
2,34	0,31	3559	1,70	0,25	2050
2,30	0,31	3488	1,66	0,18	1506
2,26	0,31	3418	1,62	0,18	1466
2,22	0,31	3348	1,58	0,18	1425
2,18	0,31	3278	1,54	0,18	1384
2,14	0,31	3208	1,50	0,18	1344
2,10	0,31	3138	1,46	0,18	1303
2,06	0,31	3068	1,42	0,18	1262
2,02	0,31	2998	1,38	0,18	1221
1,98	0,25	2446		total	81388
1,94	0,25	2389			

Table 7.2: characteristic values for the calculation model

The weight of the individual sections was also calculated in Table 7.2. The total mass of the chimney above the ground is 81,4 t.

In Fig. 7.5 to Fig. 7.7, the first three eigenforms are shown. The fundamental mode is the first one. In Table 7.3 the natural frequency and the effective modal mass are summarized.

Eigenmode	Natural frequency [Hz]	Effective modal mass [%]
First	1,13	44,1
Second	4,60	20,9
Third	10,85	10,0

Table 7.3: Natural frequency and the effective modal mass

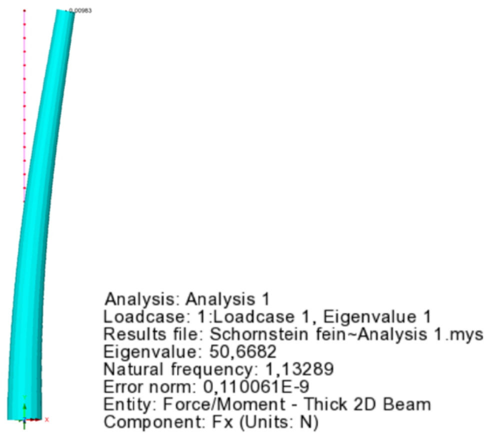


Fig. 7.5: First eigenmode

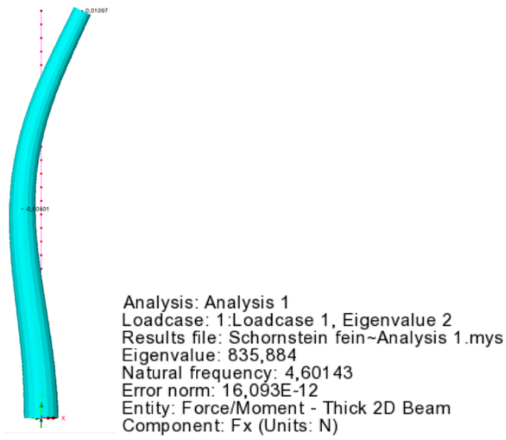


Fig. 7.6: Second eigenmode

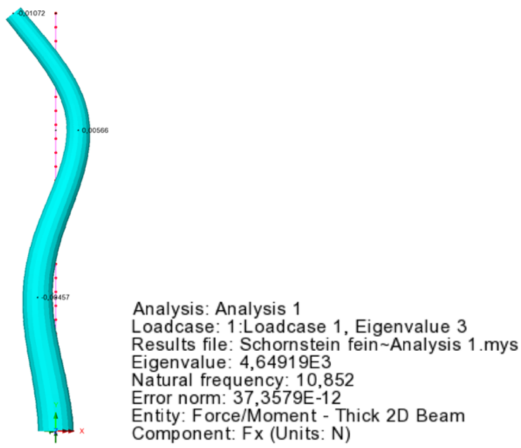


Fig. 7.7: Third eigenmode

Before the actual vibration measurement has begun, the natural frequency of the chimney has been determined. Two acceleration sensors were positioned as high as possible. It was intended to be fixed on the head of the chimney. When the MPA staff climbed the tower, he had to stop for security reason on a height of 19 m. The Problem was, that the steel bars, which were installed as ladders in the masonry, were no longer properly fixed in it. For this reason, the sensor was attached to a steel bar at this height using a wooden plate. After a short calibration the vibrations of the tower were measured for about 4 minutes. During the measurement he was repeatedly excited with a hammer.

This gave the answer in Fig. 7.8. In Fig. 7.9, the response spectrum is shown in which a Fast Fuorier Transformation has been implemented. The measurement was divided into 10 sections; each part was overlapped by 10%. Subsequently, an arithmetic mean value was performed. The spectrum is continuous since the oscillation is not periodic. Thus, the eigenfrequencies are the following: 1,10 Hz, 4,55 Hz and 11,46 Hz. These are almost identical to the results from the FE-program. It follows that the assumed material parameters aren't very different from the effective ones.

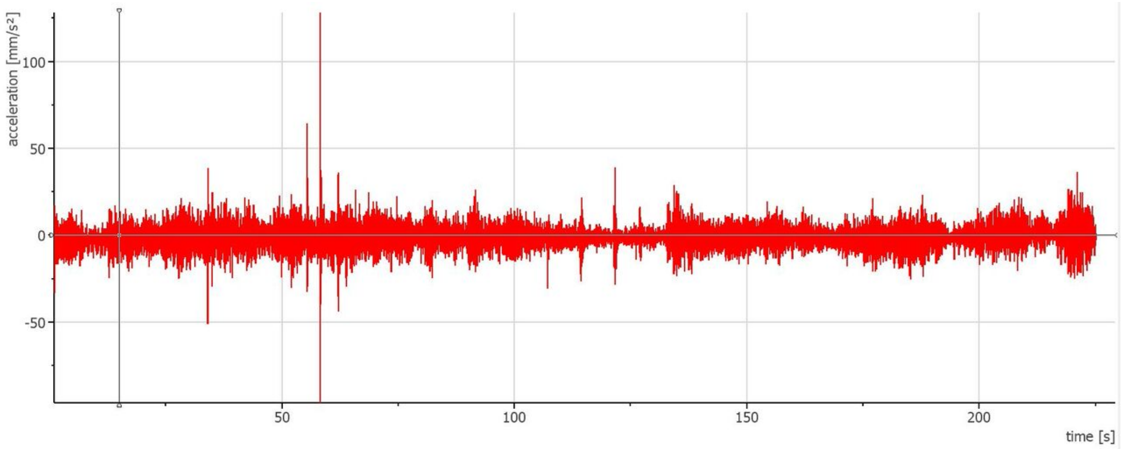


Fig. 7.8: vibration measurement for determining the natural frequency

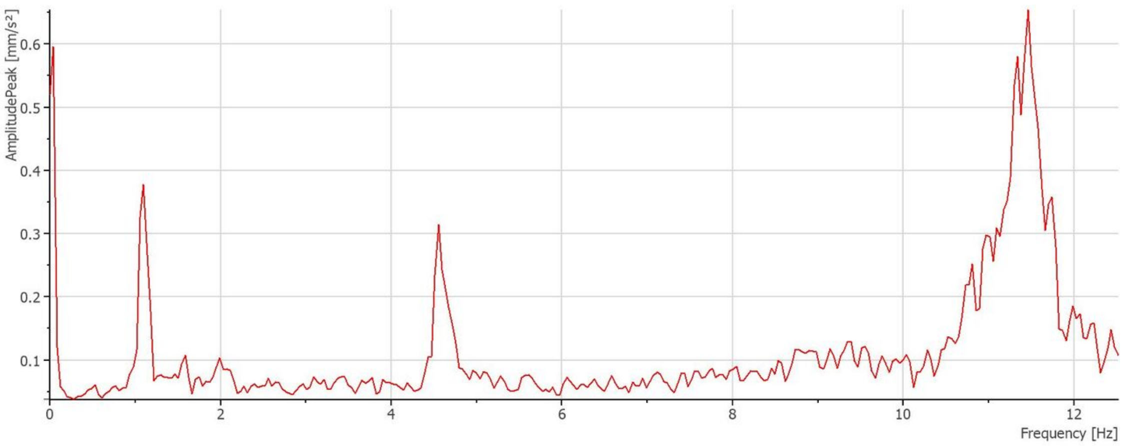


Fig. 7.9: amplitude spectrum of the vibration measurement

7.3.2. Reference value on the chimney head with wind load calculation

Based on the current wind load standard DIN EN 1991-1-4, the wind pressure to be applied is determined. The chimney is located in the wind load zone 1. This results in the fundamental value of the basic wind speed $v_{b,0}$ = 22,5 m/s. The direction factor c_{dir} and the seasons coefficient c_{season} are determined according to DIN EN 1991-1-4 as 1. Thus the fundamental value $v_{b,0}$ corresponds to the basic wind speed v_b, defined as a function of the wind direction and the season, at a height of 10 m above the ground level and for a terrain category II. The chimney can also be classified as the rule profile "Binnenland" (= Inland). According to table NA.B.4 of the National Annex DIN EN 1991-1-4/NA, the gust velocity pressure can be defined as a function of the height z. Up to the minimum height z_{min} = 7 m, the ratio of the gust speed v_p and the basic wind speed v_b can be set to 1,23. For $z > z_{min}$ = 7 m the following equation can be used:

$$\frac{v_p}{v_b} = 1{,}31 \cdot \left(\frac{z}{10}\right)^{0{,}185}$$

Thus, the gust velocity pressure q_p can be determined:

$$q_p = \frac{1}{2} \cdot \rho \cdot v_p^2$$

with the air density $\rho = 1{,}25 \frac{kg}{m^3}$.

To determine the wind load w_k as a function of the height, the force coefficient c_f must be defined. This factor is determined by multiplying the basic coefficient $c_{f,0}$ by a reduction factor ψ_λ to take the slenderness into account. This attenuation factor ψ_λ is determined by means of Fig. 7.36 and the basic coefficient $c_{f,0}$ by Fig. 7.28 of DIN EN 1991-1-4. The required Reynold number Re is calculated as follows:

$$Re = \frac{b \cdot v(z_e)}{\nu}$$

Where $v(z_e) = \sqrt{2 \cdot \frac{q_p}{\rho}}$, b is the diameter and ν the kinematic toughness of the air of $15 \cdot 10^{-6}\, m^2/s$.

The required surface roughness k for the masonry is set as k = 3 mm according to Table 7.13 of the same standard. In Table 7.4, the wind loads, used like a line-shaped load in the FE model, have been calculated for each section. This was determined as follows:

$$w_k = q_p \cdot b \cdot c_f$$

hight [m]	b [m]	v_p [m/s]	q_p [kN/m^2]	Re [-]	k/b [-]	$c_{f,0}$ [-]	ψ_λ [-]	c_f [-]	w_k [kN/m]
1	2,56	27,675	0,479	4723200	0,0012	0,926	0,700	0,648	0,795
2	2,53	27,675	0,479	4667850	0,0012	0,927	0,705	0,653	0,791
3	2,48	27,675	0,479	4575600	0,0012	0,927	0,710	0,658	0,781
4	2,43	27,675	0,479	4483350	0,0012	0,927	0,715	0,663	0,771
5	2,38	27,675	0,479	4391100	0,0013	0,928	0,720	0,668	0,761
6	2,34	27,675	0,479	4317300	0,0013	0,928	0,725	0,673	0,754
7	2,30	27,675	0,479	4243500	0,0013	0,929	0,730	0,678	0,747
8	2,26	28,283	0,500	4261305	0,0013	0,930	0,735	0,684	0,772
9	2,22	28,906	0,522	4278095	0,0014	0,931	0,735	0,685	0,794
10	2,18	29,475	0,543	4283700	0,0014	0,933	0,740	0,690	0,817
11	2,14	29,999	0,562	4279903	0,0014	0,934	0,740	0,691	0,832
12	2,10	30,486	0,581	4268059	0,0014	0,935	0,745	0,696	0,849
13	2,06	30,941	0,598	4249221	0,0015	0,936	0,745	0,697	0,859
14	2,02	31,368	0,615	4224231	0,0015	0,937	0,750	0,703	0,873
15	1,98	31,771	0,631	4193770	0,0015	0,938	0,750	0,703	0,879
16	1,94	32,153	0,646	4158402	0,0015	0,939	0,750	0,704	0,883
17	1,90	32,515	0,661	4118596	0,0016	0,940	0,755	0,709	0,891
18	1,86	32,861	0,675	4074749	0,0016	0,941	0,755	0,710	0,892
19	1,82	33,191	0,689	4027201	0,0016	0,942	0,755	0,711	0,891
20	1,78	33,508	0,702	3976245	0,0017	0,943	0,755	0,712	0,889
21	1,74	33,811	0,715	3922134	0,0017	0,943	0,760	0,717	0,891
22	1,70	34,104	0,727	3865091	0,0018	0,944	0,760	0,718	0,887
23	1,66	34,385	0,739	3805312	0,0018	0,945	0,760	0,718	0,881
24	1,62	34,657	0,751	3742973	0,0019	0,946	0,760	0,719	0,875
25	1,58	34,920	0,762	3678227	0,0019	0,947	0,765	0,725	0,873
26	1,54	35,174	0,773	3611215	0,0019	0,948	0,765	0,725	0,864
27	1,50	35,421	0,784	3542062	0,0020	0,949	0,765	0,726	0,854
28	1,46	35,660	0,795	3470880	0,0021	0,950	0,765	0,727	0,844
29	1,42	35,892	0,805	3397774	0,0021	0,951	0,770	0,733	0,837
30	1,38	36,118	0,815	3322837	0,0022	0,952	0,770	0,733	0,825

Table 7.4: Calculation of wind loads w_k for the FE model

With these values, the head displacement u_1 was calculated using the FE model. It is possible that this displacement amplitude at the chimney head of 17,22 mm has already occurred once and could be recorded without a loss of stability. For this reason, it can be assumed that a dynamic vibration of the chimney with the same oscillation amplitude at the head is possible. In Fig. 7.10 one can see among other things the displacement of the chimney.

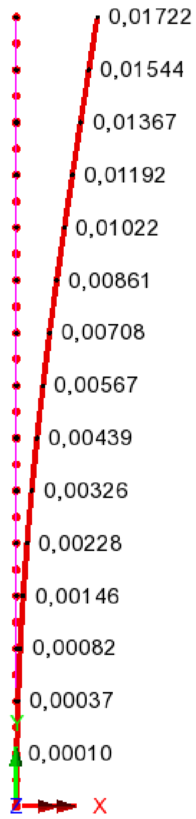


Fig. 7.10: displacement of the chimney

Taking into account the first eigenmode with a natural frequency f = 1,13 Hz, the oscillation speed can be determined as follows:

$$v_1 = 2\pi \cdot f \cdot u_1$$

If the head displacement u_1 without initial oblique position is 17,22 mm, the oscillation speed is:

$$v_1 = 2\pi \cdot f \cdot u_1 = 2\pi \cdot 1{,}13 \cdot 17{,}22 = 122mm/s$$

This determined oscillation speed represents the upper limit value. If this limit is exceeded, it is likely to lead to a failure. Therefore, the reference value from which an alarm is triggered must be set significantly smaller. Due to the existing uncertainties such as the model and any knowledge of the chimney, and as well as the existing cracks, one uses a safety factor of $\gamma = 3{,}0$. It was recommended that the alarm be triggered at 50% of the rated value. This results in the following reference value for the chimney head at which an alarm should be provoked:

$$v_{max} = \frac{122}{3} \cdot 50\% = 20{,}33\frac{mm}{s} \cong 20\frac{mm}{s}$$

7.3.3. Reference value on the chimney foundations

The DIN 4150 part 3 is used to determine the reference value at the chimney base. The chimney was constructed from unreinforced wall bricks, which leads to the structure being classified as particularly sensitive to vibrations. Moreover, demolition works are short-term shocks. The big problem is that nobody knows the vibration frequency before the excitation begins. Therefore, in order to be on the safe side, the smallest reference value of 3 mm/s (see chapter 6.1.1.) is to be considered, which shouldn't be expected to damage the building. Consequently, the alarm is triggered when the 3 mm/s is exceeded.

7.4. Evaluation of the stored measurement data

The demolition work took about three months. During this time, speed sensors were mounted on the concrete cuff at a height of about 2.30 m. The sensors were connected to a measuring card. This, in turn, is connected to a computer and transmits the data to it. If the reference value of 3 mm/s is exceeded, the computer sends to the measuring card a signal which then triggers an acoustic alarm. The acceleration sensors were basically less sensitive than the speed sensors. For this reason and in consultation with the test engineer, only the reference value on the foundation was observed. The speed sensors are "SM6 sensor nederland" which can measure three directions simultaneously: two in the horizontal direction (H1 = v_x and H2 = v_y) and one in the vertical direction (V3 = v_z). To store the data, the new program was used, which is described in chapter 5.5. The sampling rate was 500 values per second. This results in 3000 values for six seconds. Table 7.6 lists the peaks of the oscillation speed, which have triggered an alarm, thus the values which occur exactly at t = 1 sec.

During these months, there were a total of 34 events (see Appendix A and Table 7.6), where the limit value was reached. Of these 34 results, the amplitude spectrum was generated in the region of the vibration. More specifically, a Fast Fourier Transformation was examined for the range between 0.5 sec and 1.5 sec. This was created using the software DIAdem. From these spectra, the frequency with the maximum amplitude was read out, in order to subsequently determine the frequency-dependent limiting speed $v_{max,new}$ according to Table 6.1.

In most cases, the limit value was only very slightly exceeded. More specifically, in 23 cases, the vibration speed didn't reach 4 mm/s. Of these, there were 4 events that surpassed the newly calculated value: these were on May 30 at 7:54 am, on June 6 at 12:57 pm, on June 7 at 11:57 am and on July 14 at 07:51 am. As can be seen in the table 7.6, the oscillation speed of 3 mm/s in the vertical direction is rarely overrun. Only on May 30 at 7:54 am and on 13 June at 08:29 am this case occured. However, the calculated frequency of the second event brought a new limit of 3.2 mm/s, which makes it irrelevant. Vertical vibrations don't always lead to problems because the oscillation acceleration is added with the acceleration due to gravity. Horizontal vibrations could be problematic. In 5 cases, there were a vibration speed over 5 mm/s. There were five that were relevant:

on June 6 at 10:30 am, on June 7 at 02:49 pm, on June 8 at 08:58 am and on July 18 at 08:02 am and at 08:03 am. One important event (June 2 at 09:40 am) had a swing speed between 4 and 5 mm/s.

Event	peak in	H1 [mm/s]	H2 [mm/s]	V3 [mm/s]	f [Hz]	$v_{max,new}$ [mm/s]	exceeded?
May 23, 09:37 am	H2	1,017	5,432	-0,233	195,313	10,000	No
May 23, 02:29 pm	H2	1,239	-3,389	-1,631	17,578	3,947	No
May 30, 07:54 am	V3	2,086	0,095	-3,579	11,719	3,215	Yes
May 30, 07:57 am	H2	0,985	3,579	0,074	147,461	10,000	No
June 02, 09:40 am	H1	-4,575	-4,246	-0,371	9,766	3,000	Yes
June 06, 10:30 am	H2	0,752	-5,549	1,673	15,625	3,703	Yes
June 06, 12:57 pm	H1	3,092	0,752	0,011	9,766	3,000	Yes
June 06, 02:30 pm	H2	1,112	3,420	1,303	26,367	5,046	No
June 07, 11:34 am	H1	3,357	-1,355	-2,245	24,414	4,802	No
June 07, 11:53 am	H1	-3,018	-0,053	1,472	26,367	5,046	No
June 07, 11:57 am	H1	3,304	-0,413	-0,752	10,742	3,093	Yes
June 07, 02:49 pm	H1	-5,951	2,404	-1,334	17,578	3,947	Yes
June 08, 08:58 am	H2	-4,501	8,620	-3,304	16,602	3,825	Yes
June 08, 10:33 am	H2	-2,213	4,204	-0,932	26,367	5,046	No
June 08, 11:26 am	H2	-1,991	4,088	0,720	249,023	10,000	No
June 08, 02:15 pm	H2	1,101	-4,024	-0,286	239,258	10,000	No
June 08, 02:33 pm	H2	-0,085	-3,537	-0,540	238,281	10,000	No
June 08, 03:03 pm	H2	-1,461	3,050	0,318	238,281	10,000	No
June 09, 09:05 am	H2	-1,048	3,240	-0,201	241,211	10,000	No
June 09, 09:08 am	H2	-0,529	-4,268	0,921	245,117	10,000	No
June 09, 09:13 am	H2	0,127	-3,791	-0,064	244,141	10,000	No
June 12, 09:49 am	H2	-1,101	3,071	0,572	248,047	10,000	No
June 12, 10:00 am	H2	-0,074	-3,050	-0,402	248,047	10,000	No
June 12, 10:28 am	H2	0,116	-3,293	-0,042	249,023	10,000	No
June 12, 10:32 am	H2	-0,180	-3,029	-0,339	242,188	10,000	No
June 12, 10:47 am	H2	0,868	-3,156	-0,191	242,188	10,000	No
June 12, 10:50 am	H2	-1,567	3,103	0,286	249,023	10,000	No
June 12, 10:55 am	H2	0,064	3,431	0,413	249,023	10,000	No
June 13, 08:29 am	V3	-0,265	1,535	3,018	11,719	3,215	No
June 13, 08:33 am	H1	-3,929	-1,059	-2,287	17,578	3,947	No
July 14, 07:51 am	H1	3,918	-1,779	2,203	9,766	3,000	Yes
July 17, 10:16 am	H1	-3,929	-0,127	-0,508	25,391	4,924	No
July 18, 08:02 am	H2	3,908	-6,671	-1,313	35,156	6,145	Yes
July 18, 08:03 am	H1	12,919	-8,588	0,275	222,656	10,000	Yes

Table 7.6: Oscillation speed that triggered the alarm and calculation of the new frequency-dependent reference values

In summary: the alarm was triggered in 34 cases. Of this, only ten were relevant, since the frequency-dependent reference value was exceeded. All others can be classified as false alarms.

In Fig. 7.11, for example, the event of June 07 at 11:57 pm is shown. In this case, the oscillation speed is exceeded in all three directions. Considering the direction H1 with v_x = 3,304 mm/s and only the vibration itself, the decisive excitation frequency of 10,74 Hz is obtained. According to Table 6.1, the following oscillation speed can be determined for vibration-sensitive buildings:

$$v_{max,new} = 0{,}125 \cdot f + 1{,}75 = 0{,}125 \cdot 10{,}74 + 1{,}75 = 3{,}093\,mm/s < 3{,}304\,mm/s$$

As can be seen, the existing oscillation speed is higher than the limit value, even if the difference is only 0,211 mm/s. Theoretically, you have to check the chimney for damage. The limits, which the standard specifies, are always on the safe side, so you can actually assume that with a very high probability no crack has formed for this little difference of 0,211 mm/s.

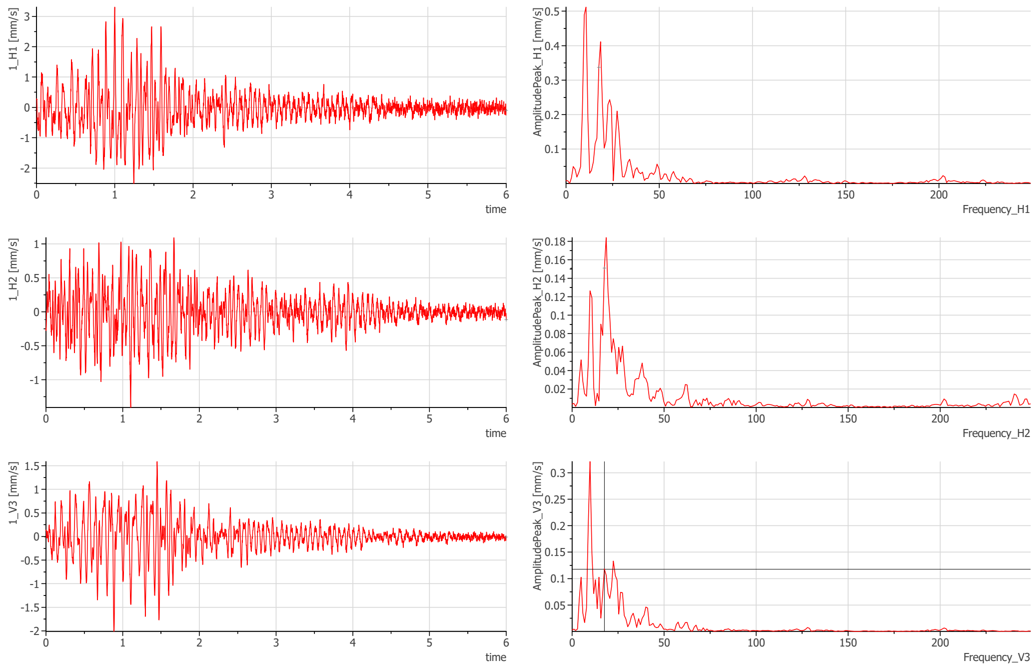


Fig. 7.11: Event from 07.06.17 at 11:57 am with the amplitude spectrum on the right side

Another example is the event of July 17 at 10:16 am. In this case v_x is 3,92 mm/s. The corresponding excitation frequency with the largest amplitude is 25,39 Hz. According to Table 6.1, the new limit value is:

$$v_{max,new} = 0{,}125 \cdot f + 1{,}75 = 0{,}125 \cdot 25{,}39 + 1{,}75 = 4{,}92\,mm/s > 3{,}92\,mm/s$$

As can be seen here, the new limit value is larger than the oscillation speed, so this event doesn't actually have to be considered. In Fig. 7.12 the event is shown, with corresponding amplitude spectrum on the right side.

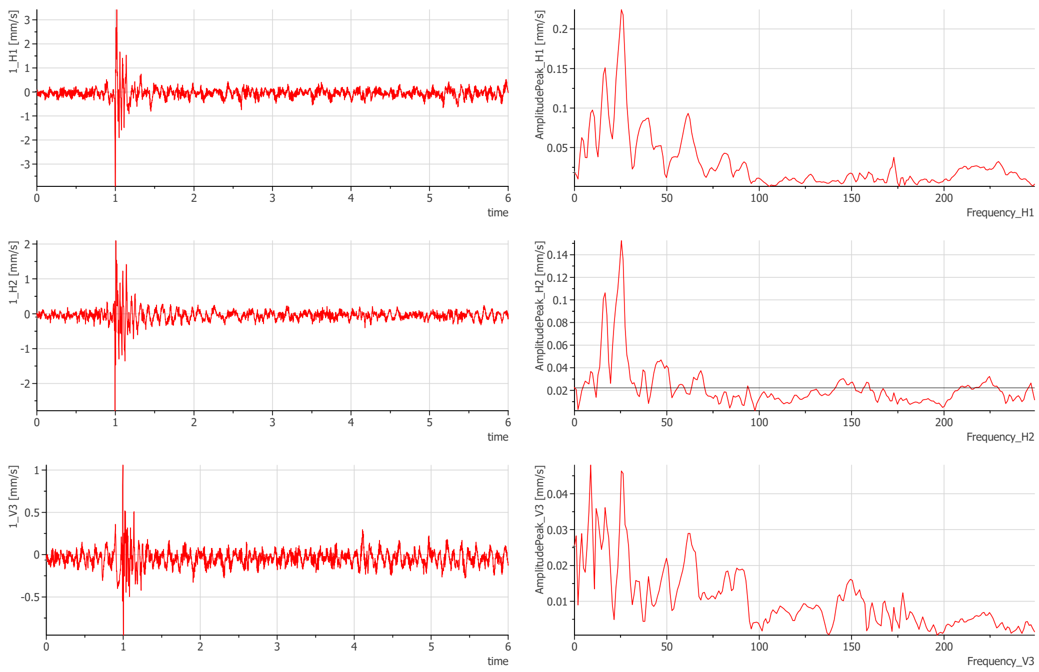


Fig. 7.12: Event from 17.07.2017 at 10:16 am with the amplitude spectrum on the right side

At the end of the demolition, the chimney was examined again. There were no new cracks. This means that the seven decisive events, whose oscillation speed have exceeded the frequency-dependent variable, caused no damage. This shows that overriding the limit values doesn't have to lead to a loss of stability. This only indicates that it is very unlikely to have a damage if the values aren't exceeded. Fig. 7.13 shows the final state of the chimney.

Fig. 7.13: Final state of the chimney

7.5. Example of a forecast model

Before starting the actual demolition work, a prognosis model could have been created to give the construction worker a guideline of the maximum permissible sizes of demolition materials. This wasn't possible in time because the work has already started when installing the devices.

The falling masses convert a part of the drop energy into shock waves. These spread in the ground. The size of the vibrations depends on the potential energy, on the size of the impact surface, on the deformability of the substrate and on the soil properties on the propagation path. For the first prognosis, the formula for mass impact of DIN 4150-1 can be used (see also chapter 3.5):

$$v = k \cdot \frac{\sqrt{E}}{R^m}$$

E is the drop energy and R is the distance. The exponent m takes into account the soil properties on the propagation path and the factor k represents the energy conversion at the impact point. To determine these two constants, a statistical evaluation of existing measurement data must be carried out. In order to simplify the calculation, the following assumptions could be made for this chimney:

- The mass of the components is concentrated at the centre of gravity
- The drop height corresponds to the height of the centre of gravity from the impact surface
- Vibration centre is punctiform

- The ground vibrations propagate from the impact centre in a circular shape
- Soil type and their characteristic values are neglected, since the possible impact surfaces are within a radius of a maximum of 15 m
- Foundation size isn't taken into account

For the evaluation, only measurement data are used for devices that have been positioned on the foundation. For the statistical evaluation of m and k, the maximum measured oscillation velocities are first normalized with the drop energy:

$$v^* = \frac{v}{\sqrt{E}}$$

This means that each mass, with its falling height and the distance from the measurement point can be assigned to each measured value. These data can be illustrated in a diagram. The distance R is indicated on the x-axis and the energy-normalized velocity v^* on the y-axis. By a regression formula a curve of the type

$$v^* = k \cdot R^{-m}$$

could be determined. In Fig. 7.14, an example of blasted chimneys is shown.[40]

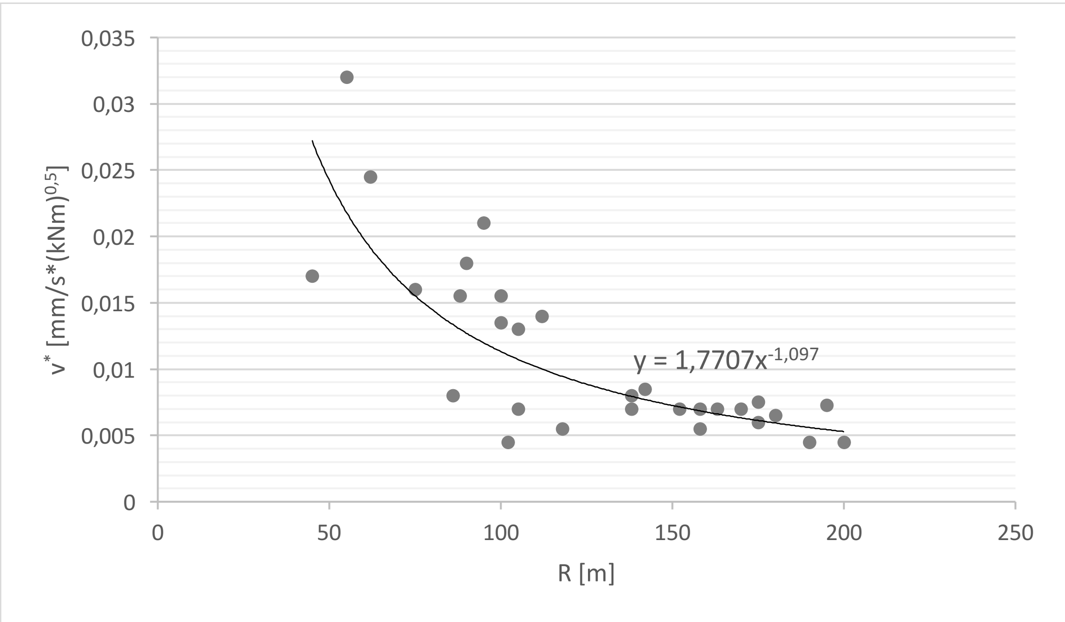


Fig. 7.14: Distance dependence of the energy-normalized oscillation speed

In Fig. 7.14 one can see that for this example k = 1,7707 and m = 1,097. Such a similar procedure could also be applied to the chimney in Bretten to produce a forecast.

[40] The example is derived from VDI reports No. 1754, 2003 by Dr. Ing. P. Lichte, Dr. Ing. R. Melzer and Dipl. Ing. J. Vogel "Erschütterung beim Sprengabbruch von Bauwerken, Prognose von Aufprallerschütterungen". Here 190 measurements of 70 demolitions were taken into account.

8. Conclusion

The aim of this work was to create a new measurement system, which stores specific measured values in different files. This was partly achieved with the application of DIAdem - DAC. The general matter of vibrations and their formation had been discussed before. The introduction of the measuring system during the demolition measures at the Harsch-Steinzeugwerk resulted in two main problems:

- This work shows that the alarm is triggered as soon as a vibration speed of 3 mm/s occured. However, as seen in chapter 6.1, the limit depends on the excitation frequency. Only ten of 34 events were really relevant. In the other cases, the alarm shouldn’t have been triggered. For this reason, a method must be found with DIAdem, where the velocity limit is frequency-dependent. This could be possible by inserting a script in the DIAdem – DAC, which calculates the amplitude spectrum in real time, determines the excitation frequency and the associated limit value. Only then, the alarm may be triggered.
- Another problem with this program is the naming of the stored files. The name of the file is the date and time when the file was created. DIAdem activate the measuring system, when a file has been created and it save six seconds of the event when the limit is reached. Thus, the file name represents the exact time of the previous event. That means the activations time of the file and not the trigger event time. A solution must also be found here.

Furthermore, this study attempts to show that there are a variety of sources of vibration that lead to different excitations. It is intended to draw the reader's attention to the fact that a pure static design is often not sufficient to ensure the well-being of people and the stability of the buildings.

In principle, a subsequent shock protection always costs much more than a good predictive vibration limitation. To avoid or reduce vibrations on buildings, appropriate planning and execution of construction must be satisfied or it can be provided through mitigating measures on the vibration source, along the transmission path or on the building. In order to achieve a suitable planning or an effective reduction measure, the dynamic properties of the source, the ground and their interaction must be known. The indispensable prerequisite for this are prognosis models which can correctly predict the type of wave propagation and the effect of the intervention. In order to create a more accurate model, it is often necessary to use up-to-date measurement data, for example the expert need to know the falling mass and its distance. If these values are given, formulas such as those given in Chapter 6.1 may be applied to have first results. In this work, we couldn’t perform it because we had data, but it wasn’t exactly able to assign the mass and its distance to them, because nobody, except for a few construction workers, were at the site. Thus the two empirical parameters couldn't be determined. Before starting with the demolition, it would have been possible to make a forecast where the permissible size of the demolition parts as a function of the distance between the crash point and the chimney itself can be determined. In addition, there are measurement data with several fluctuations which occur at a constant time interval. These are more reminiscent of the excitation of the chisels than the dropping of masses.

As can be seen, these models depend on various factors. These are often associated with high safety coefficients since they are difficult to determine. Above all the determination of the exact material properties of the propagation path can lead to great uncertainties. In this respect, the standard specifies limit values ensure that no damage occurs. If these are exceeded, there is a high probability of cracks in the structure. In this work, among other things, this is shown: ten times the reference value of the standard has been exceeded, but these haven't caused any damage.

In the case of the Harsch-Steinzeugwerk, slits were sawn around the chimney, so that the vibrations didn't reach directly to it. This disturbed them, and they couldn't release the full energy to it. This is a first measure to protect the building from vibration.

In the field of demolition, controlled explosions can lead to shocks. Experts must determine the exact amount of charge to ensure a safe flow. A big advantage is that the vibration occurs only once by blasting and not more. In addition, the building rubble falls one on top of the other, which means that energy from the drop is consumed and the excitation is reduced. This solution wouldn't have been possible in this case, since the foundations of the chimney were integrated in one of the halls, which had to be demolished. For this reason, one had to work with caution.

A drop bed of deformable masses such as sand can be used to reduce the vibration at the source. Through the deformation of this material, a part of the drop energy can be consumed and the vibration is therefore lower.

These are some of the possible measures to reduce vibration at the immission site, the propagation path and the source. The task of the engineer is to study the planned construction work and its environment and to develop a suitable solution for each case.

9. Symbols

$E_{s,d}$	Dynamic stiffness
G_d	Dynamic shear module
γ_{max}	Shear strain
τ_{max}	Shear stress
ϕ_{out}	Outer diameter
ΔL_{Boden}	Level decrease in the ground
ΔL_{Geb}	Level decrease and increase during the transition from the upper edge of the terrain to the foundations
ΔL_{geo}	Geometrical attenuation
ΔL_{mat}	Material attenuation
ΔL_{Sch}	Level reduction due to possible vibration intervention
A_0	Oscillation displacement amplitude
C_P	Capacity of the piezoelectric element
c_p	Propagation velocity of compression wave
c_R	Surface speed
c_s	Propagation velocity of transversal wave
C_{St}	Scattering capacity
c_u	Undrained shear strength
D	Damping
D_r	Bulk density
E	Impact and vibration energy
E_h	Drop energy
E_{max}	Maximum energy absorbed
f	Frequency
F_0	Centrifugal force
f_{Eigen}	Eigenfrequency
G	Mass of the falling object
G_B	Mass of the ground
G_{dyn}	Oscillating device mass
G_{max}	Maximum shear modulus
G_{Rg}	Mass of the pile
G_U	Unbalanced mass
G_{Vib}	Mass of the vibrator
k	Stiffness value

k_p	Piezoelectric constant
$k^{ü}$	Wave transfer factor
L_{Em}	Emission level
L_{Im}	Immission level on the ceiling
L_{quant}	Charge quantity per time step in an explosion
m	Mass
q_p	Gust velocity pressure
r	Distance from the shock spring to the point under consideration
r_0	Equivalent radius
Re	Reynold number
R_{expl}	Distance detonation - building
r_U	Lever arm
t	Time
T	Period
u	Amplitude
u_0	Displacement of the ground
u_F	Displacement of the vibrating mass
U_q	Output voltage for acceleration sensor
u_r	Relative displacement between the ground and the vibrating mass
v^*	Energy-normalized velocity
V_3	Magnification funktion
v_A	Propagation speed of a wave
$v_{max,expl}$	Maximum expected oscillation velocity during explosion
w	Energy density
w_k	Wind load
ζ	Damping factor
η	Frequency ratio in the measurement
ϑ	Attenuation of the sensor
λ	Length of the oscillation
ν	Transverse contraction number
σ'_m	Mean effective ground tension
φ	Phase shift angle
ω	Angular frequency
Ω	Excitation angular frequency
ω_B	Operating frequency
ω_U	Angular frequency of the unbalanced mass

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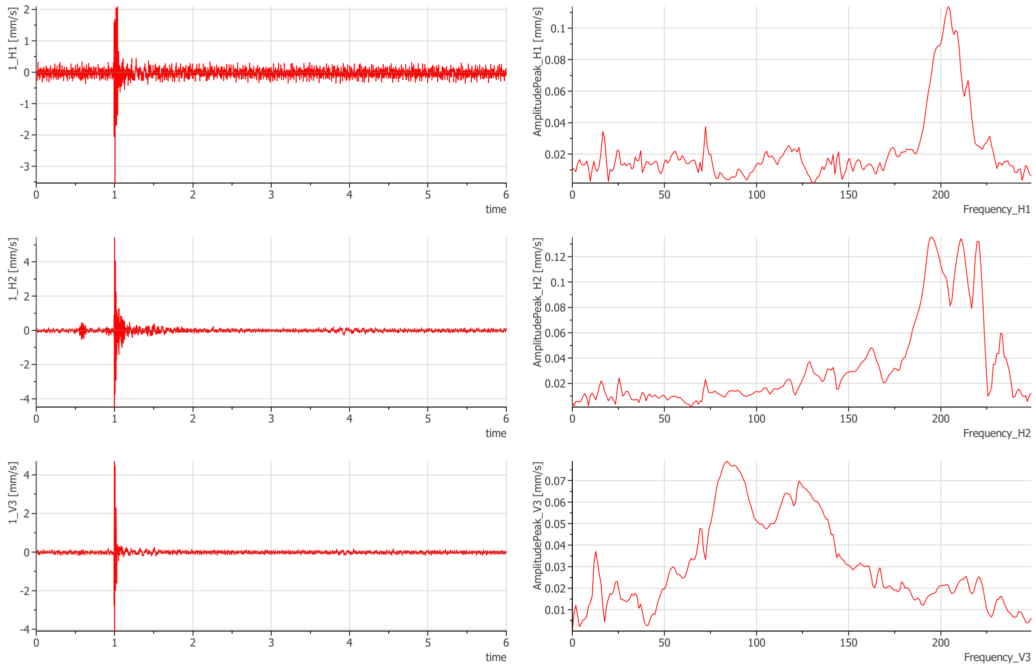
Annex A

Left: oscillation speed; right: amplitude spectrum



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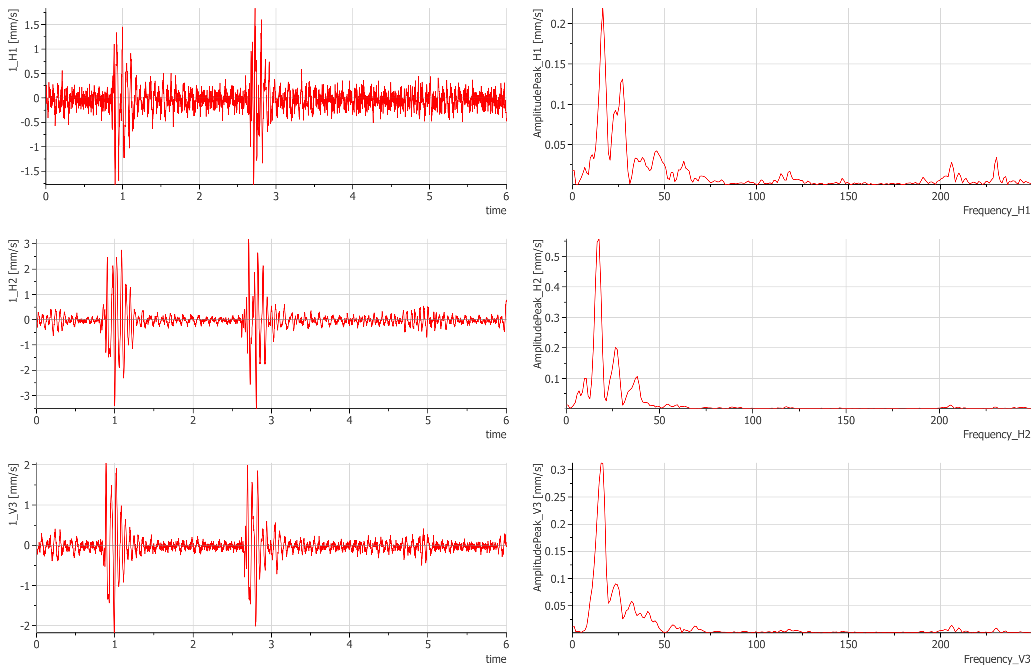

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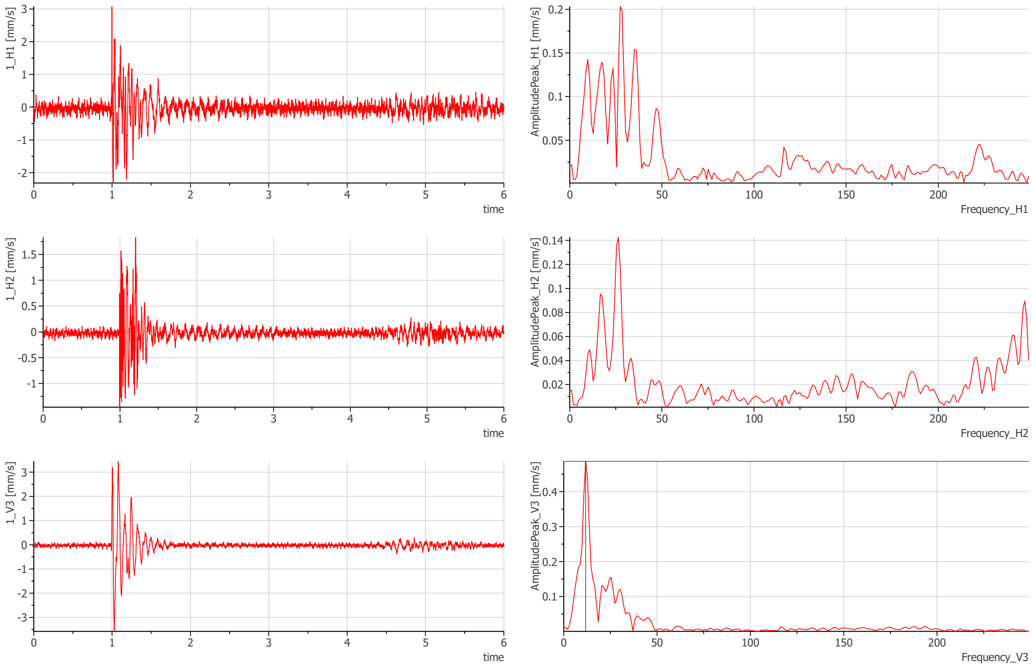

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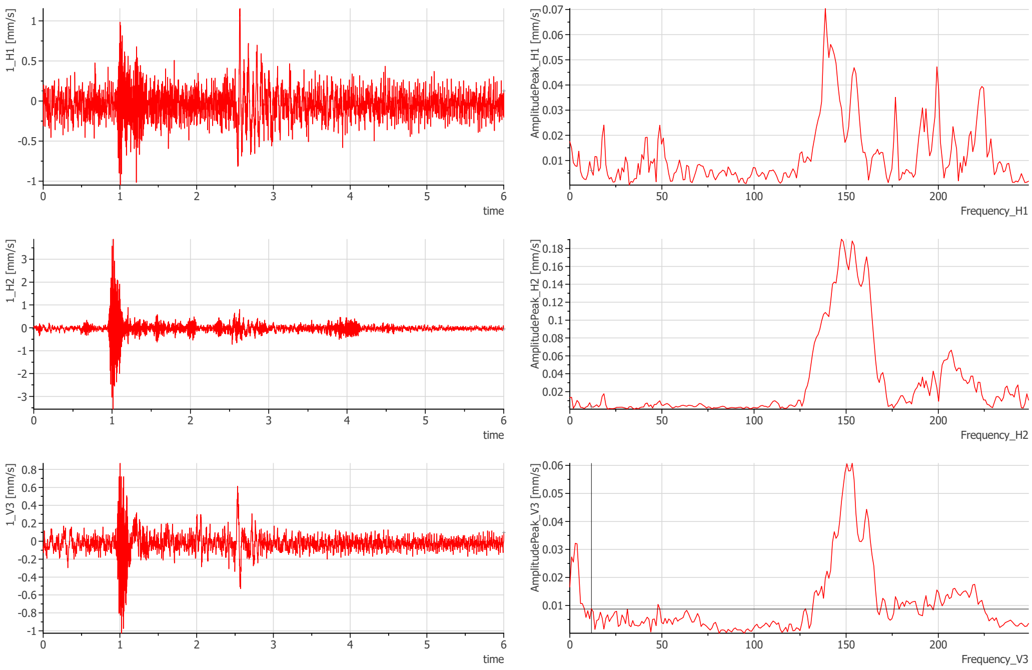

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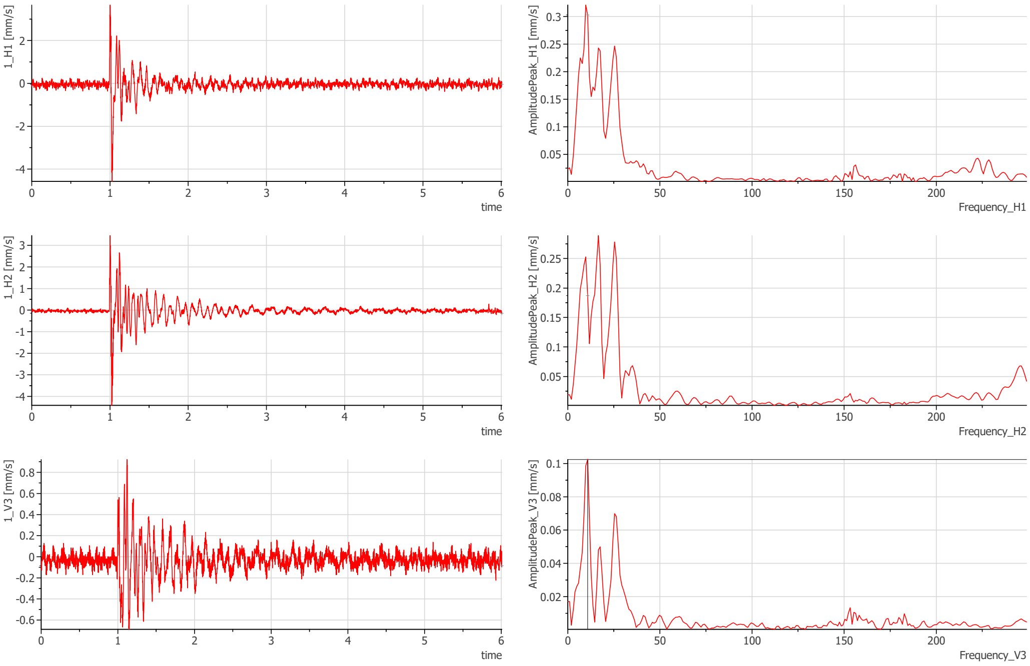

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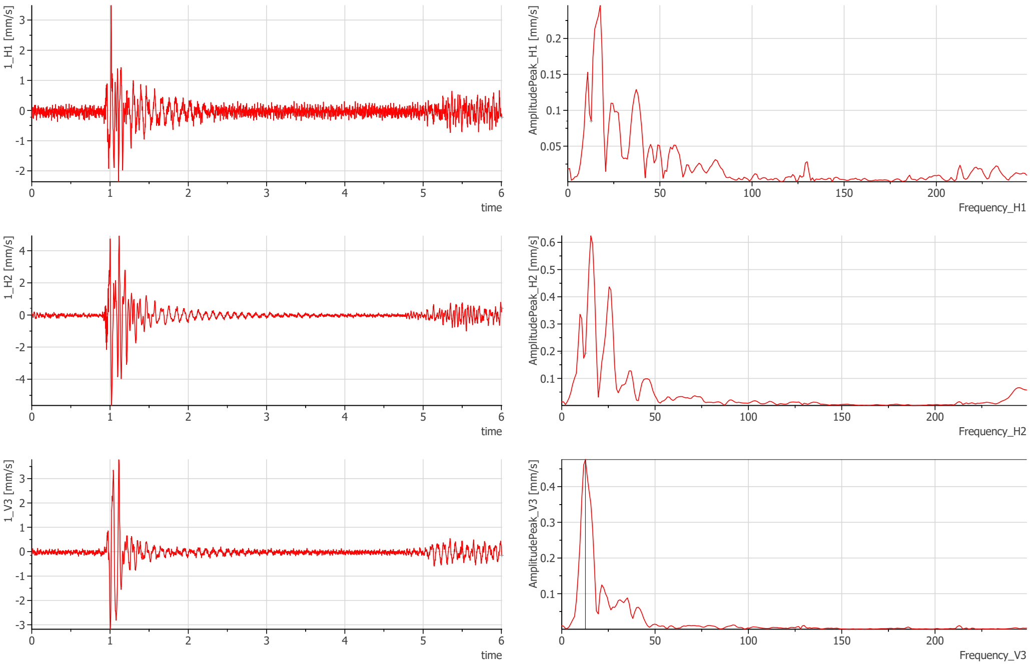

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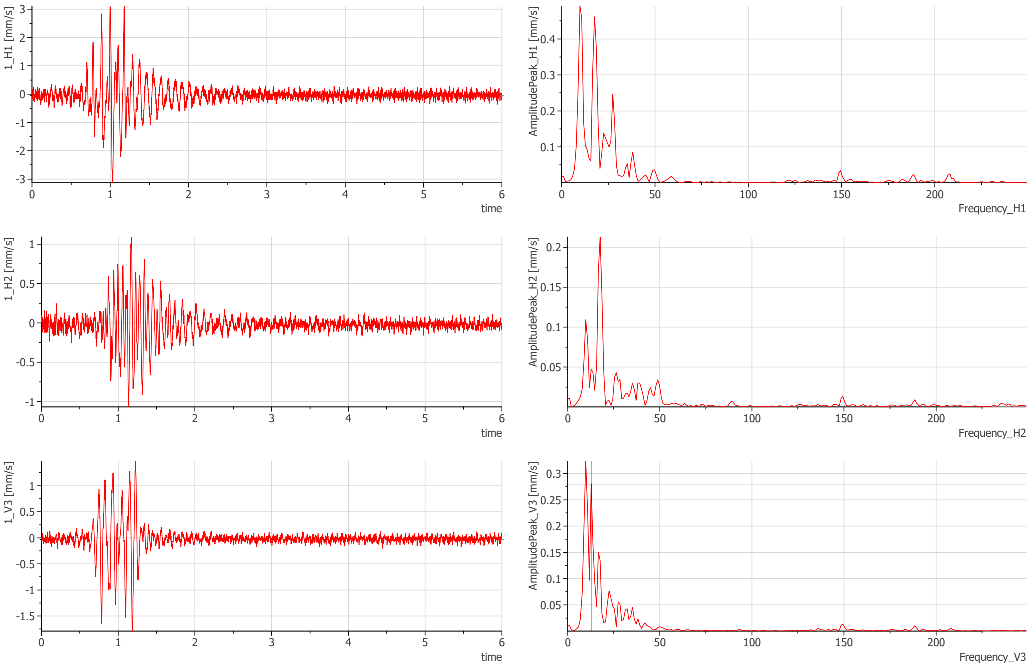

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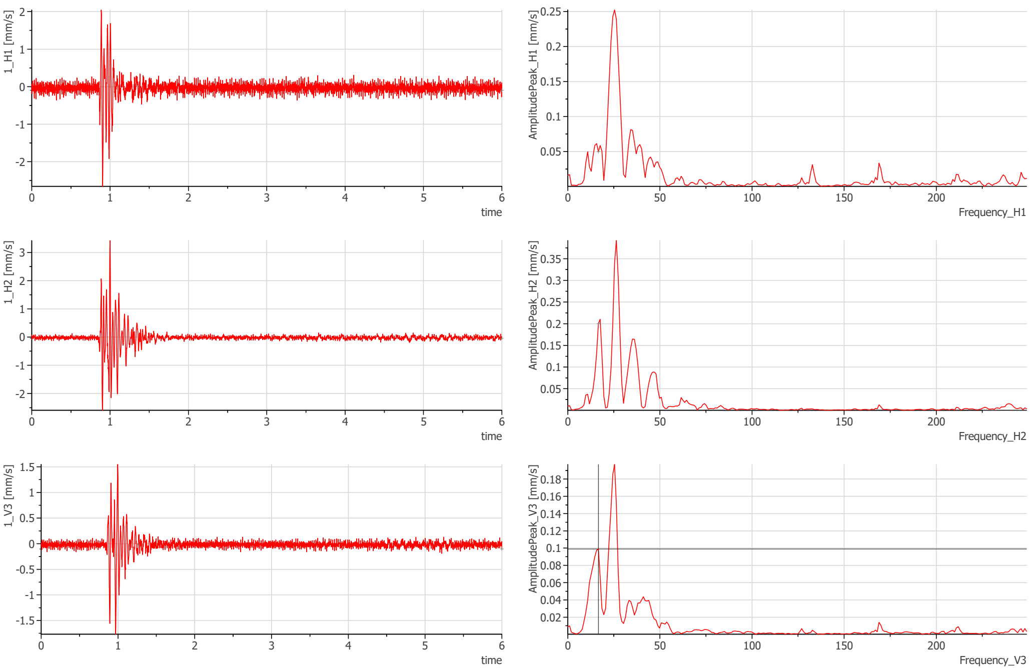

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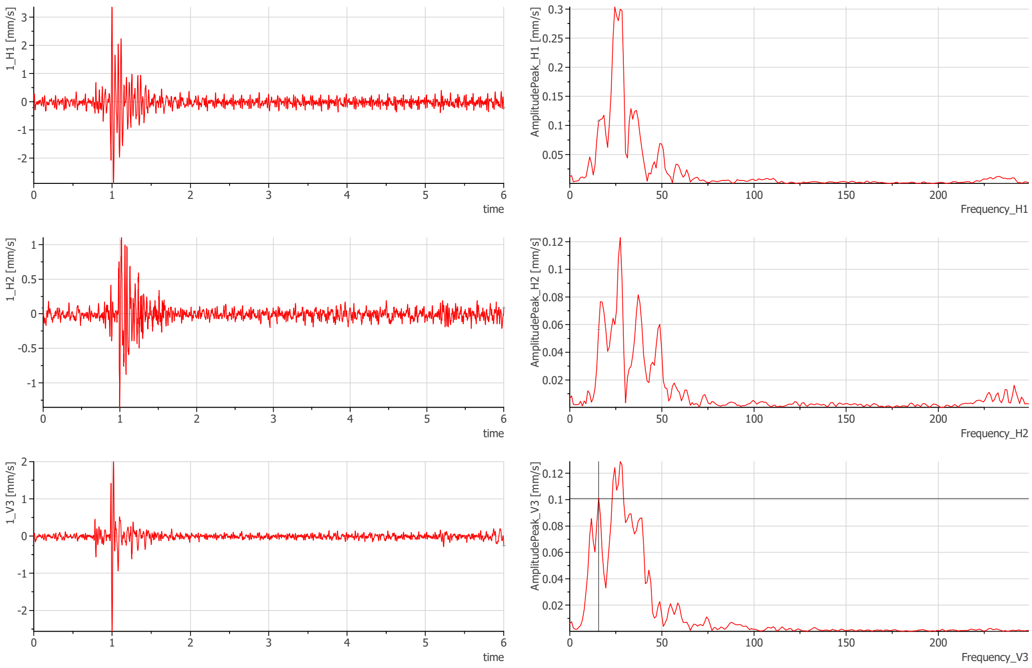

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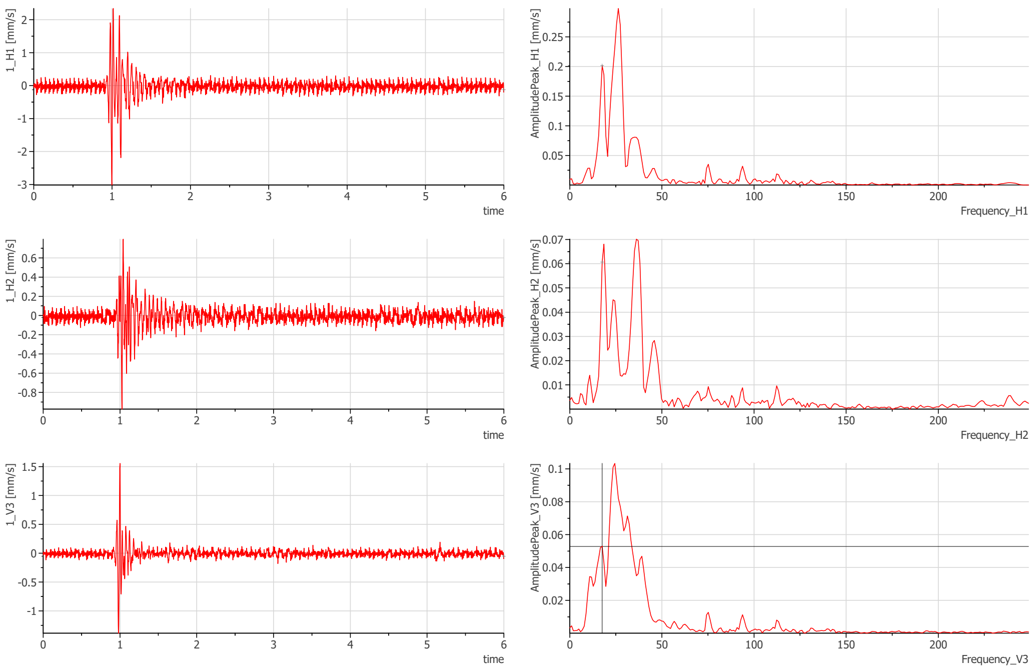

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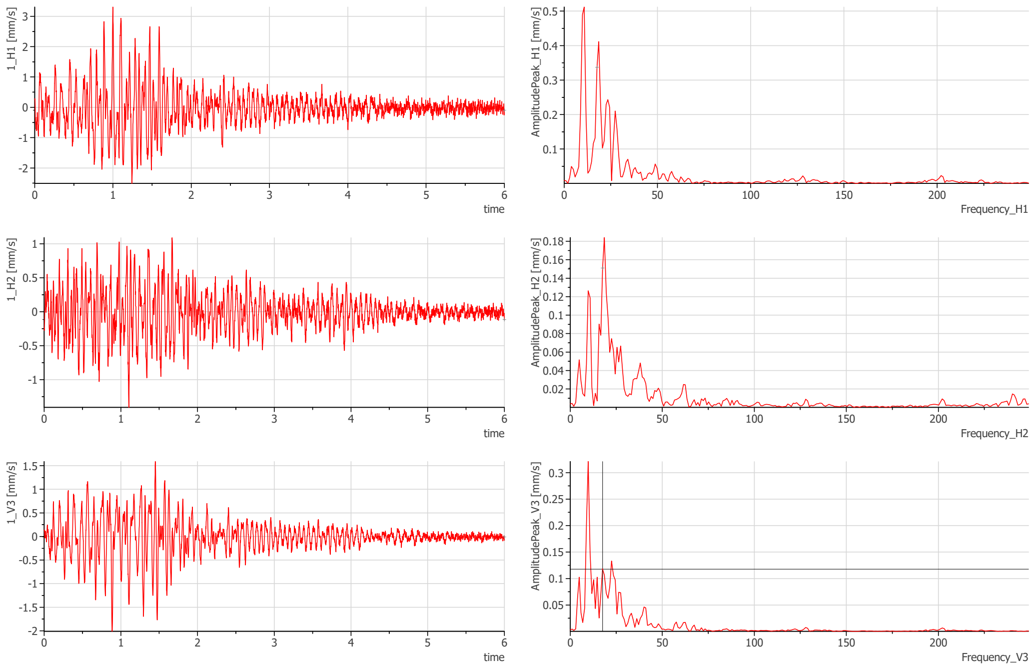

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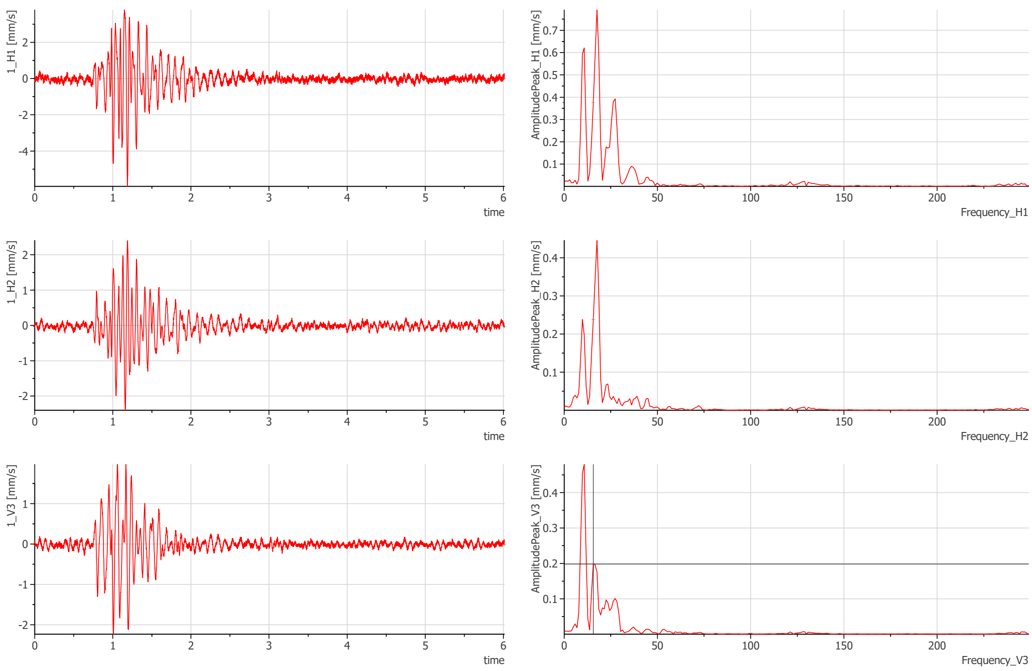

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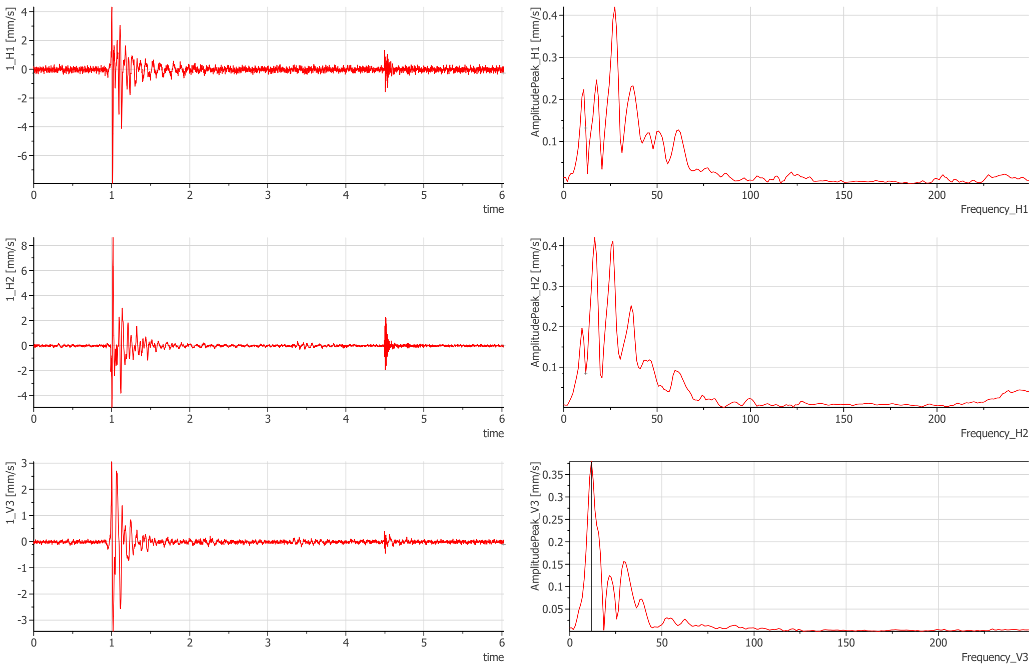

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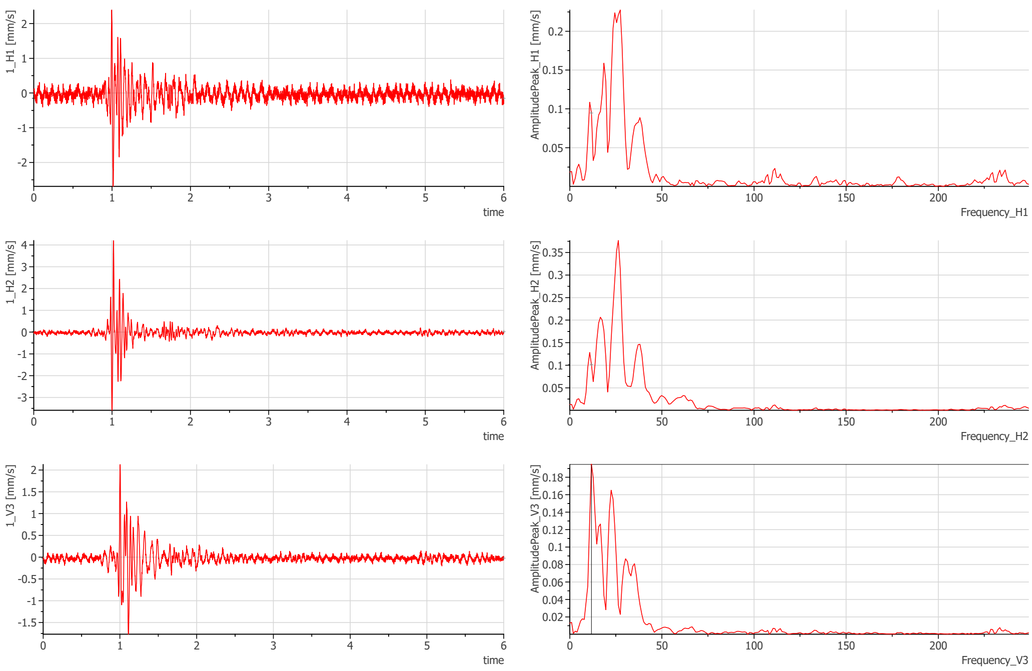

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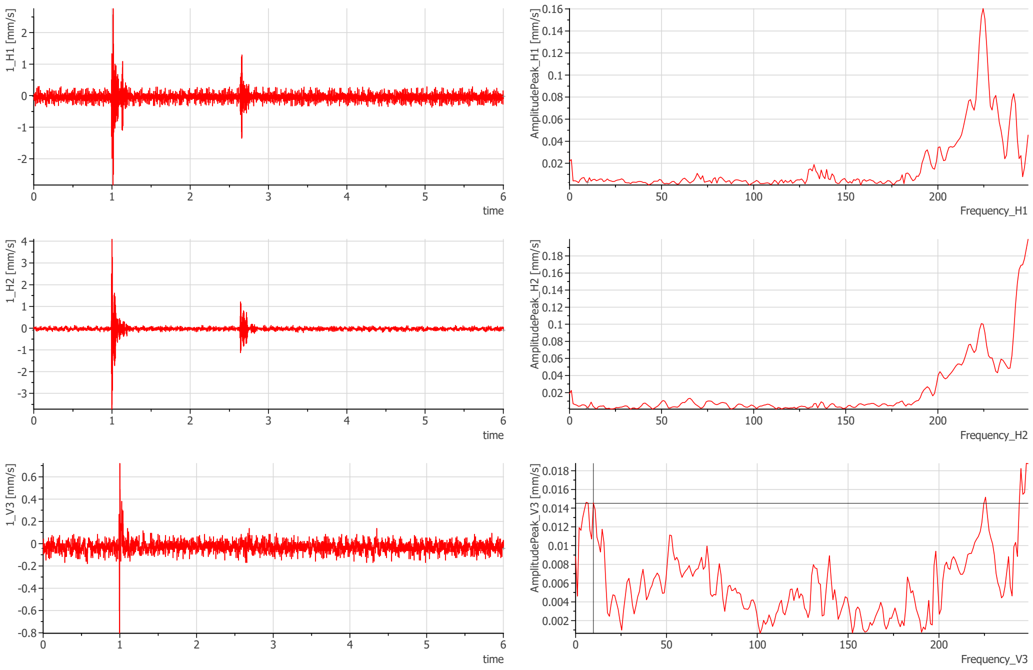

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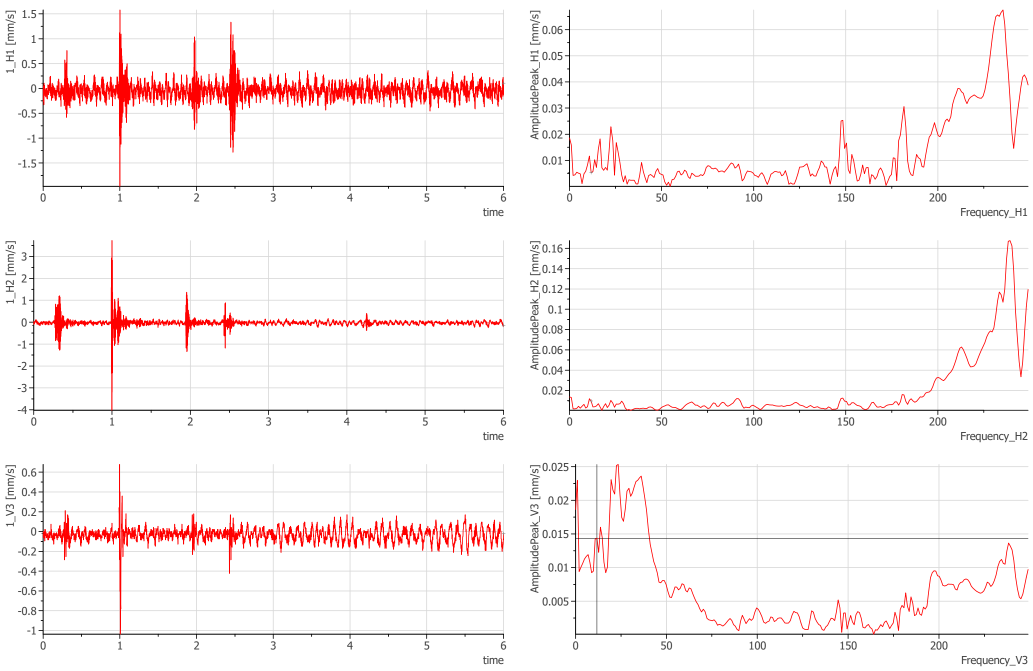

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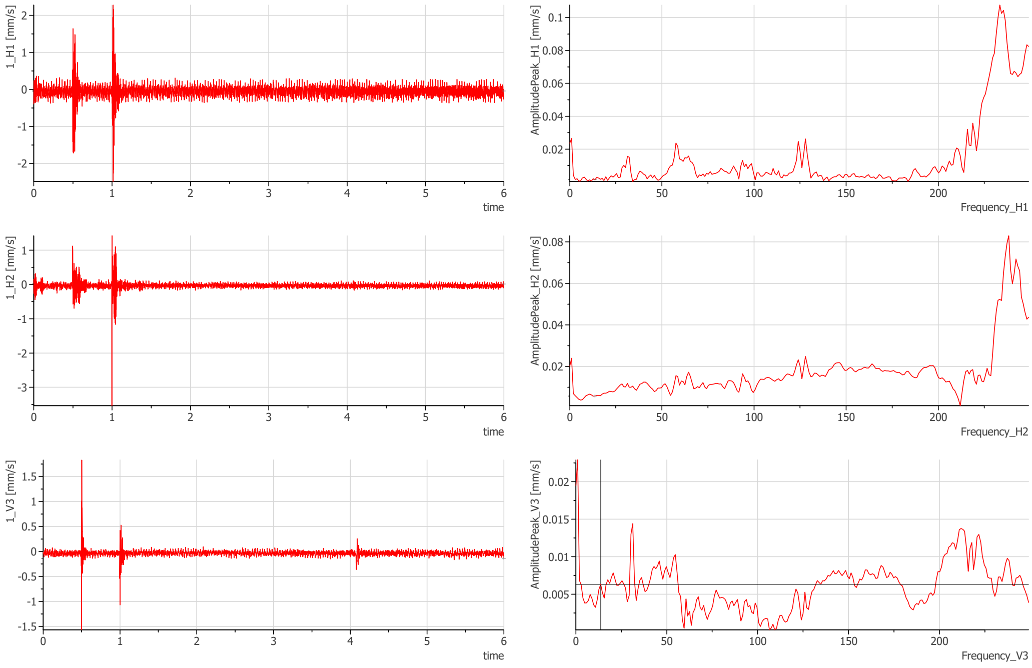

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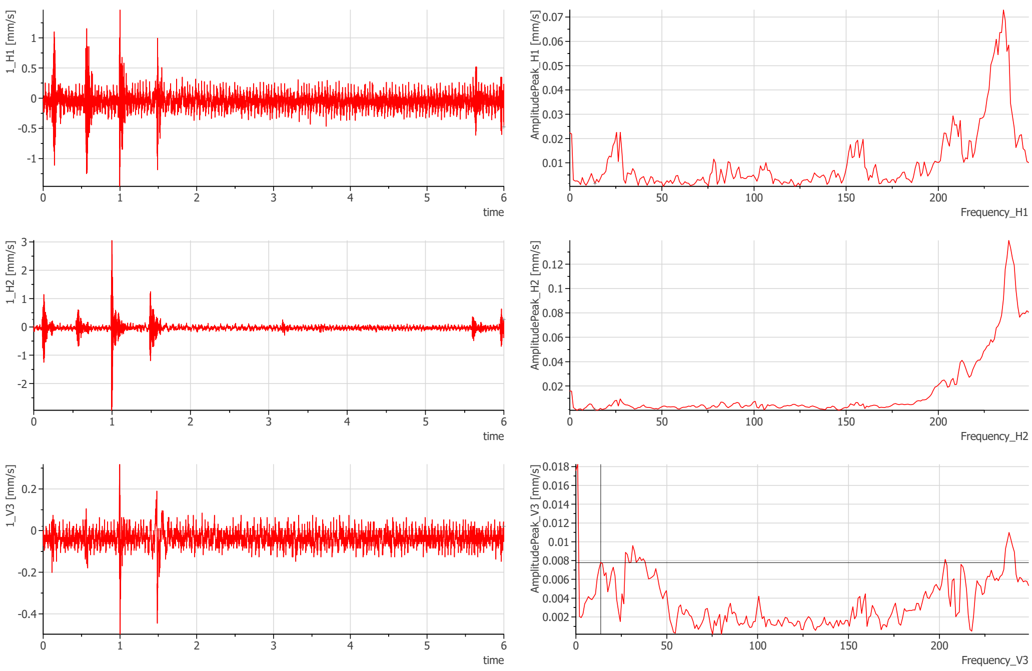

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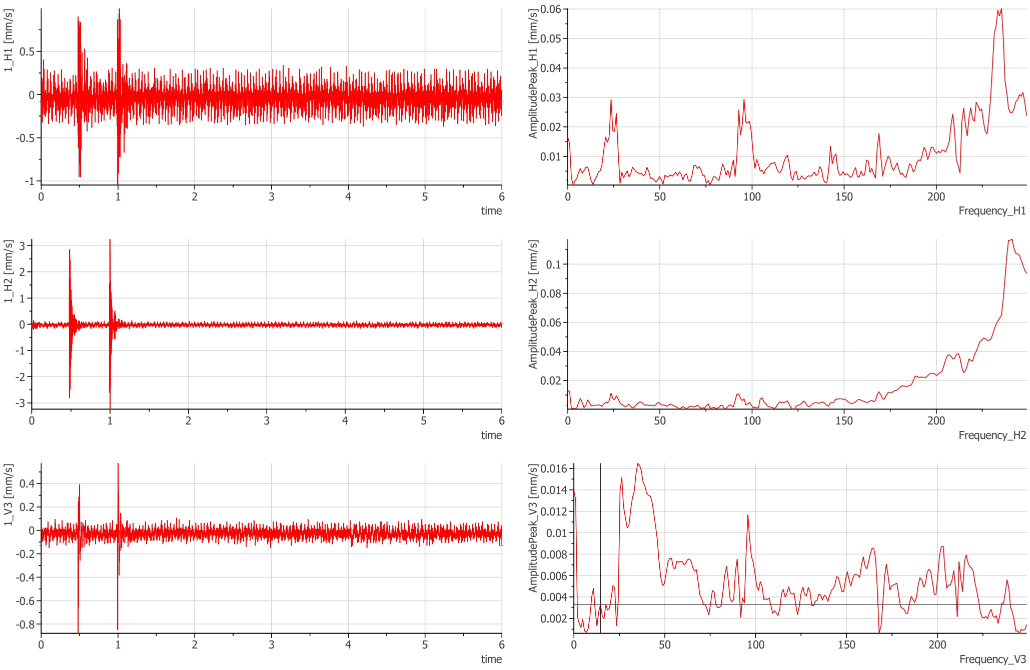

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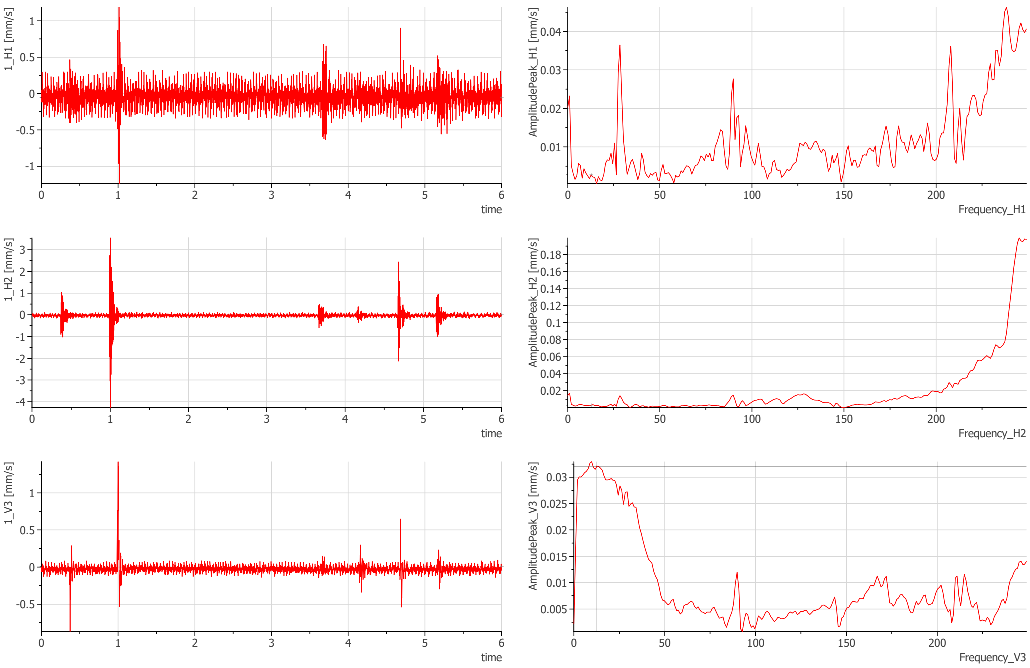

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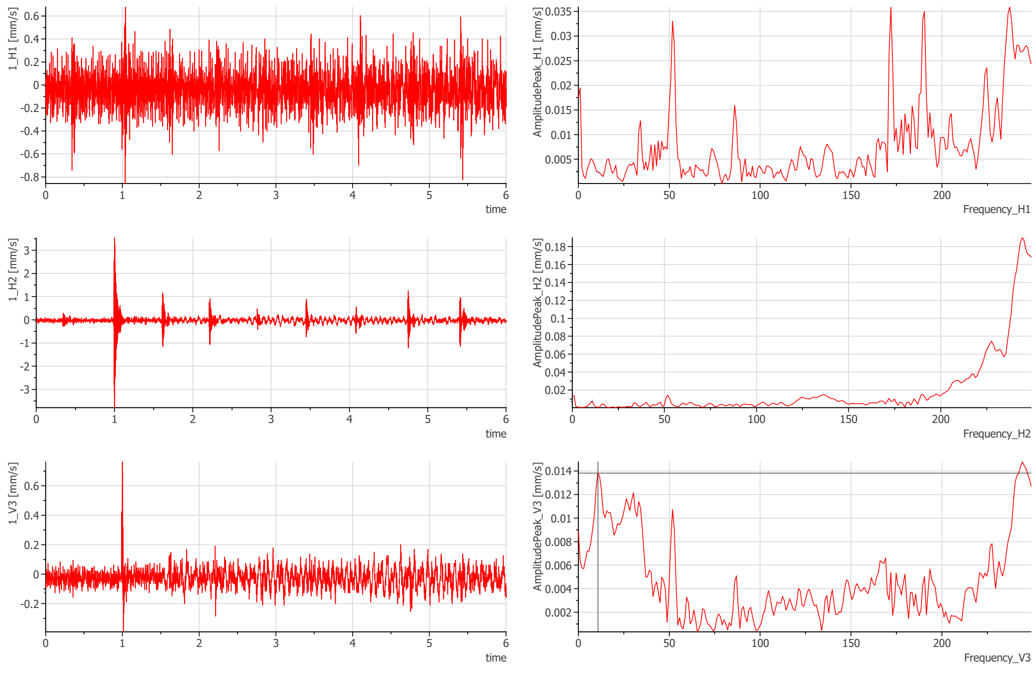

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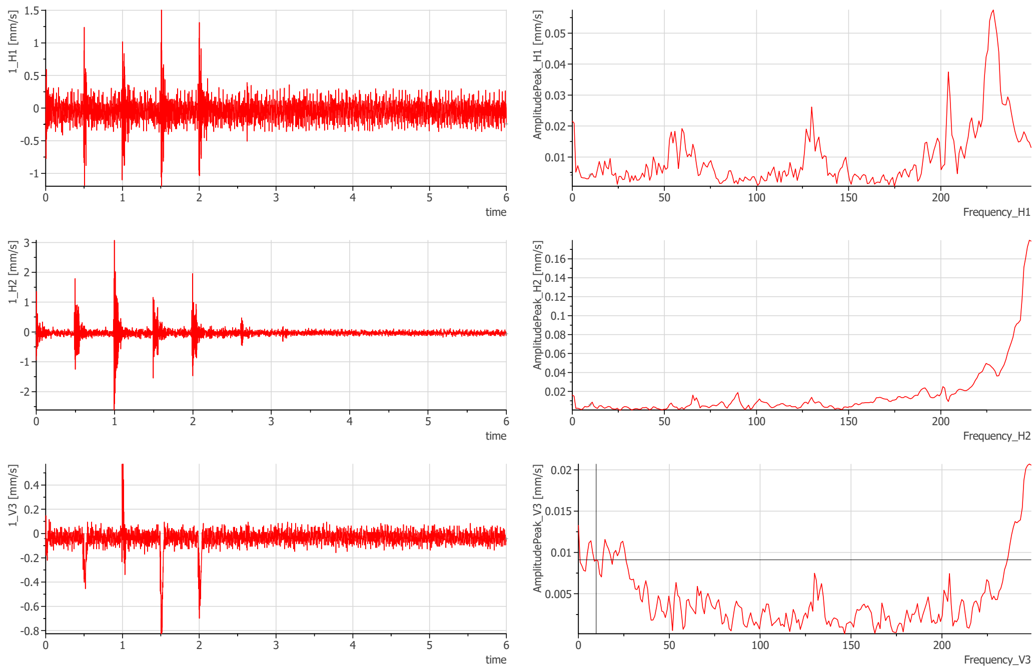

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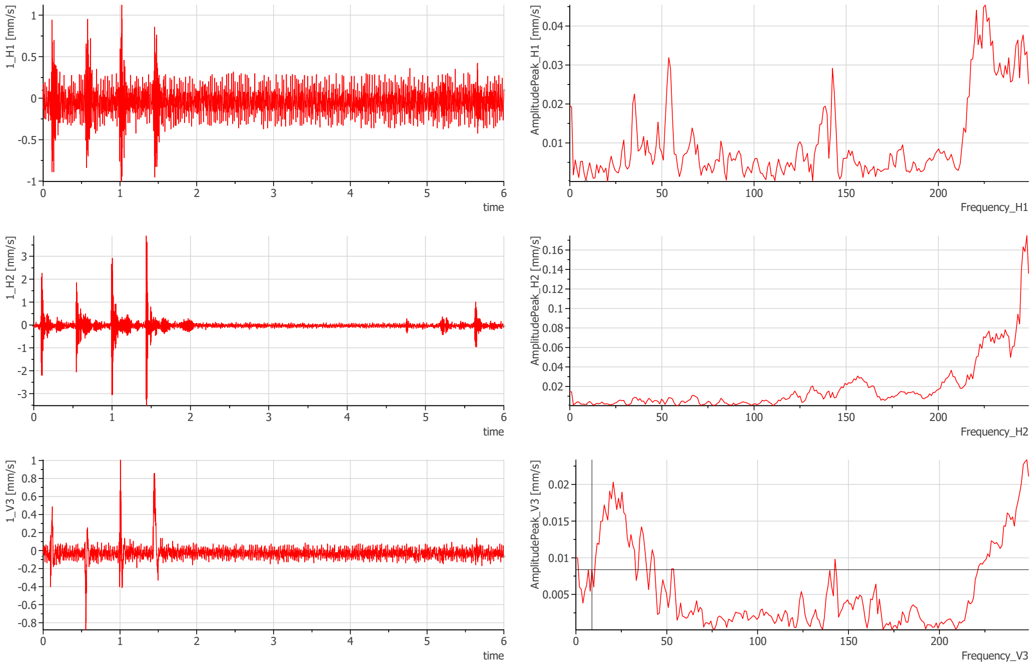
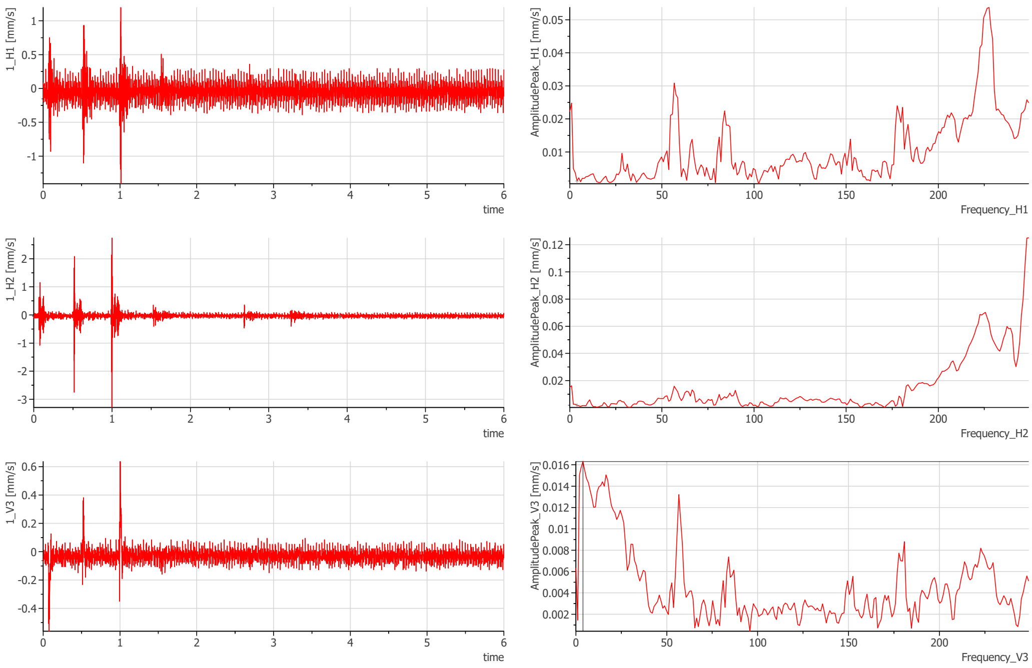
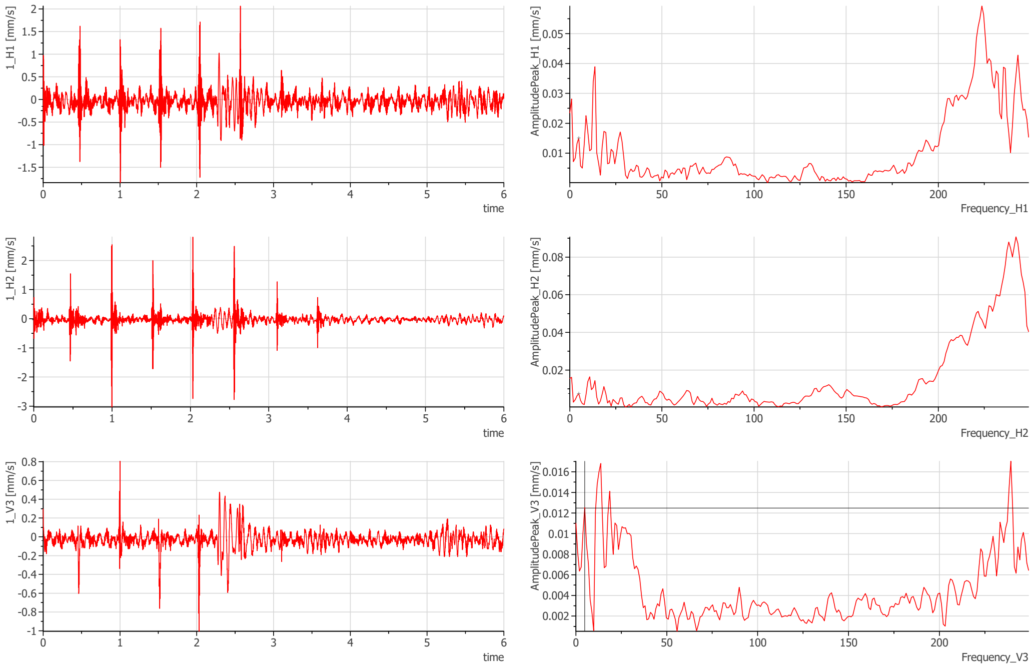


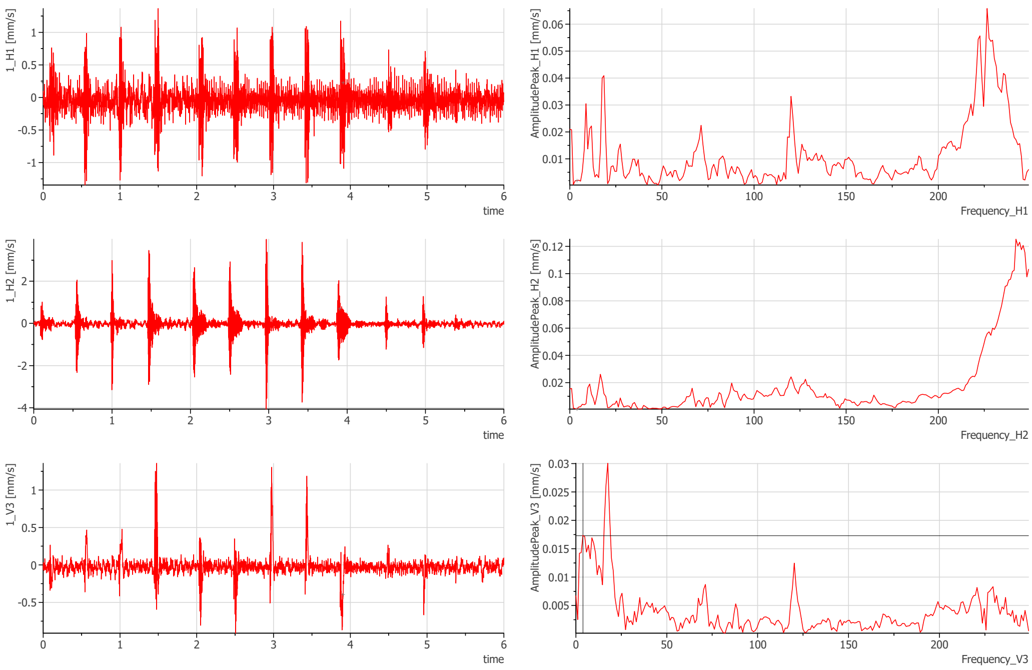
Event 12.06.2017, 10:00 am



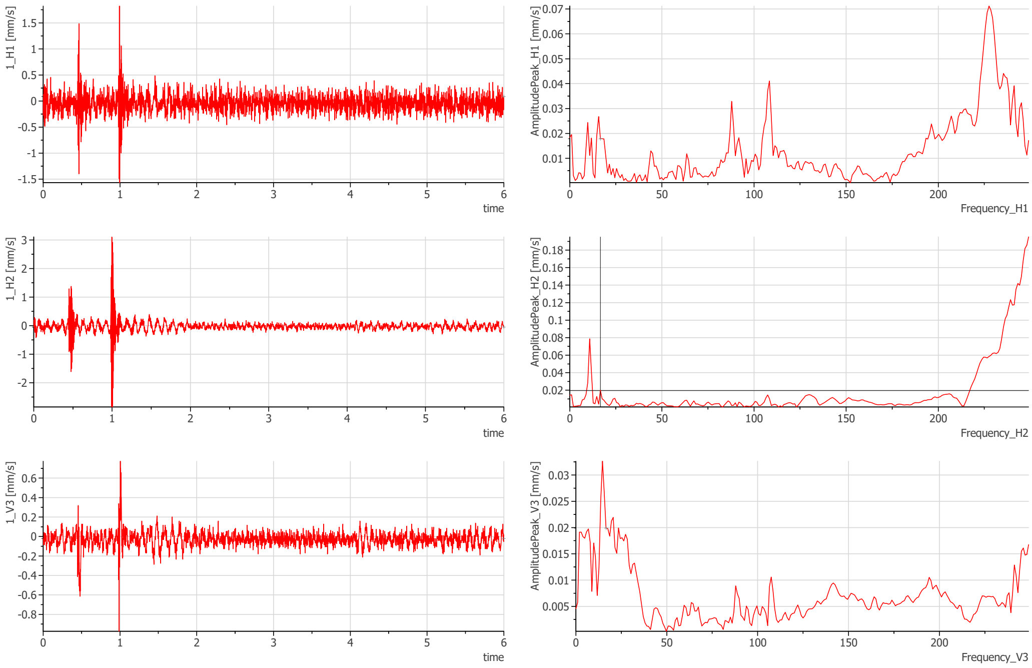
Event 12.06.2017, 10:28 am



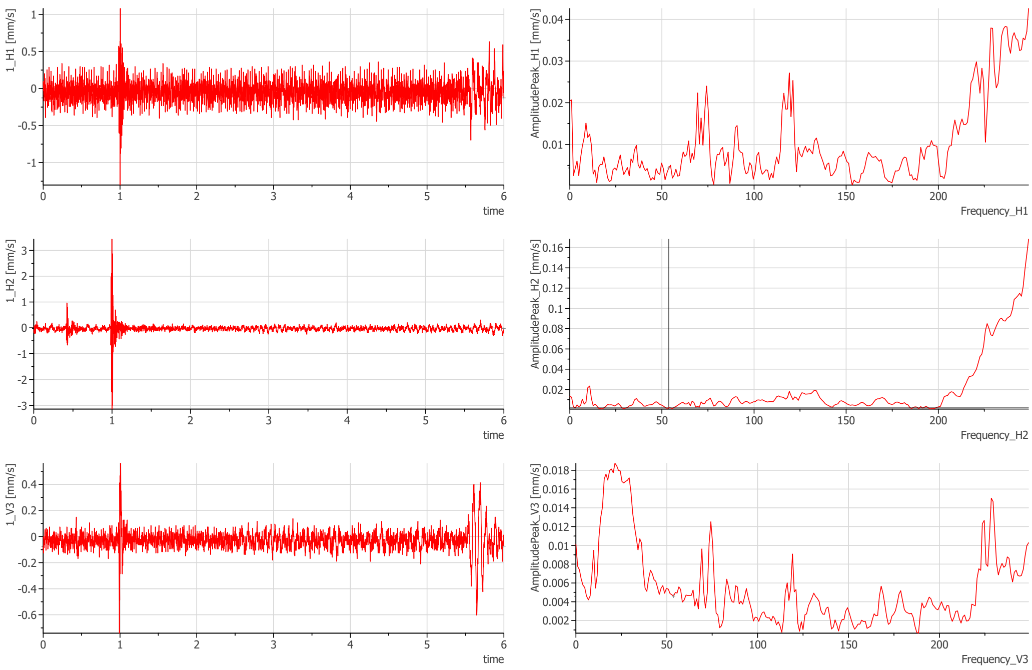
Event 12.06.2017, 10:32 am



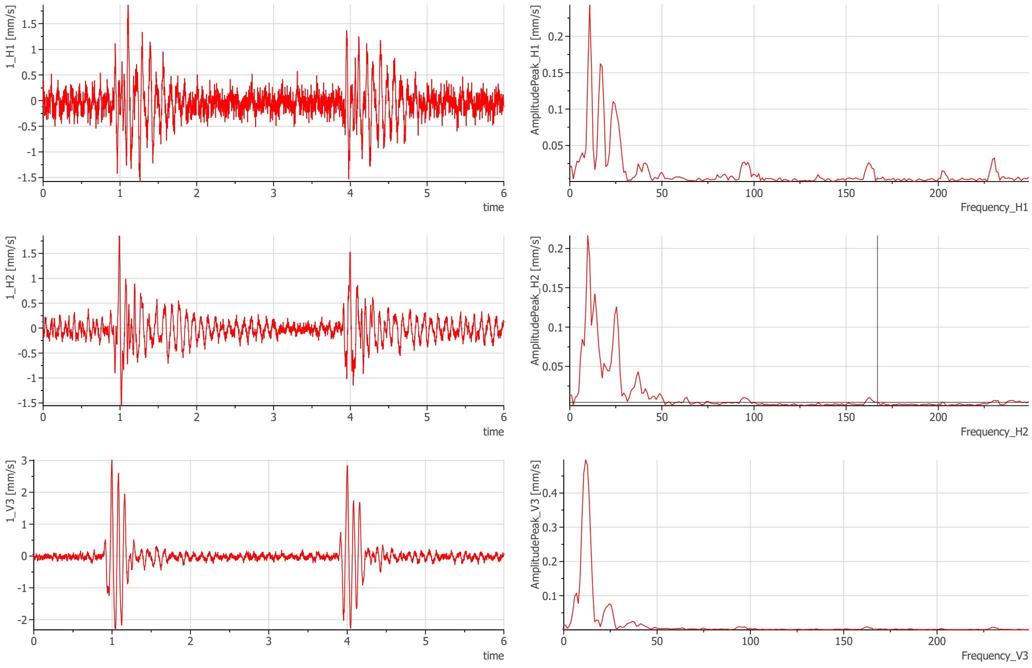
Event 12.06.2017, 10:47 am



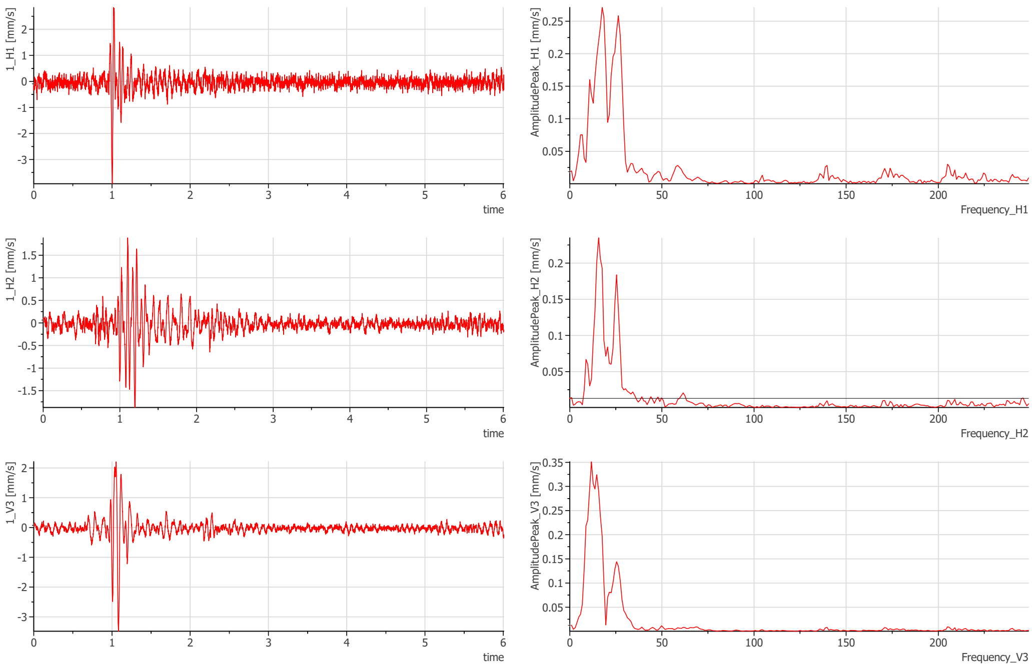
Event 12.06.2017, 10:50 am



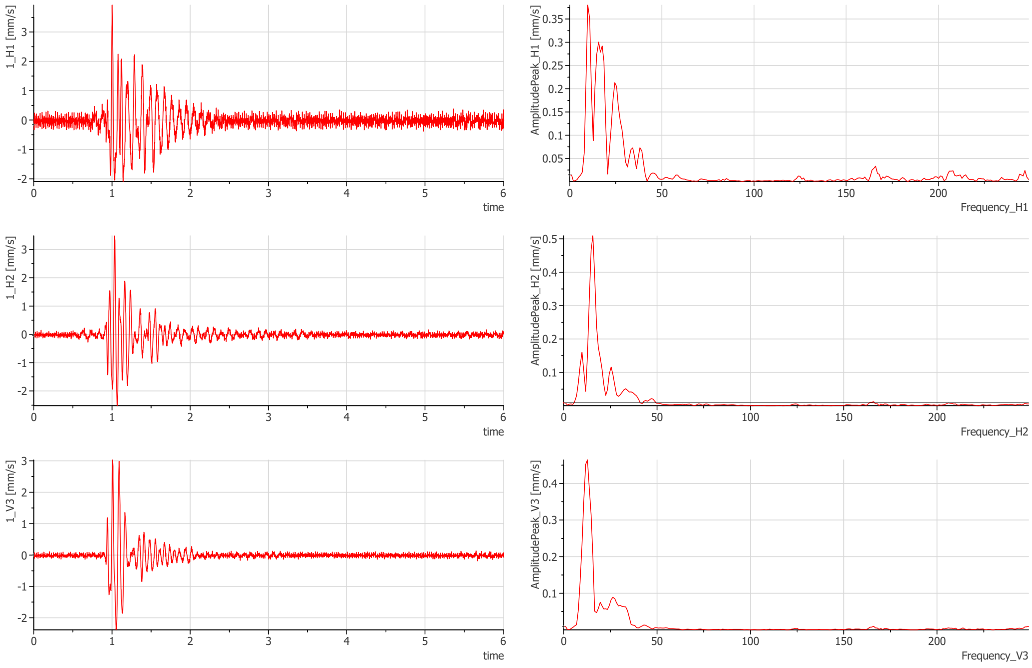
Event 12.06.2017, 10:55 am



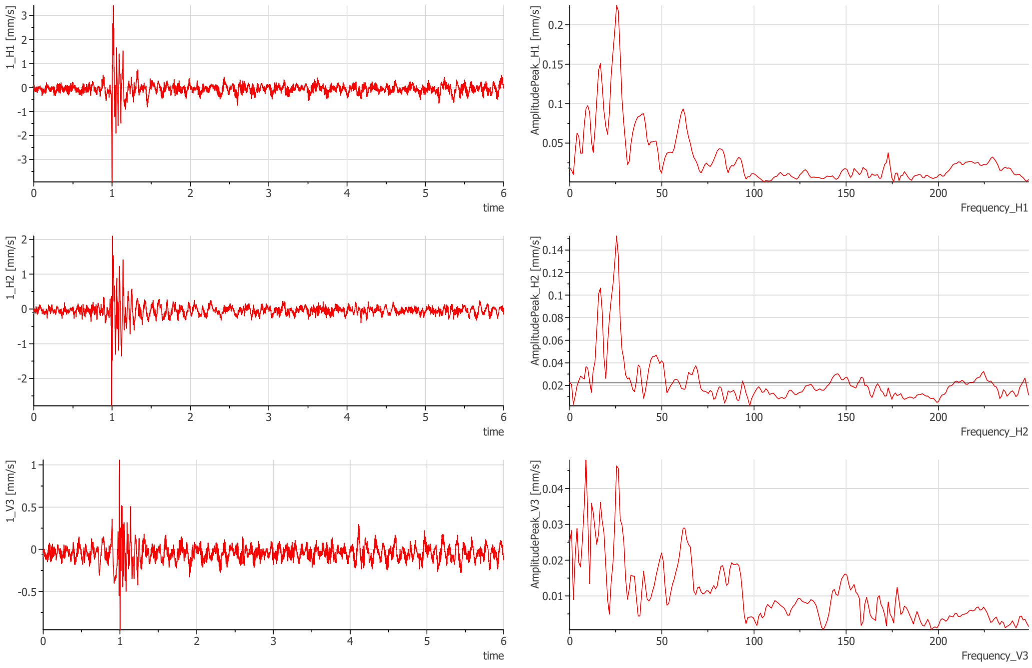
Event 13.06.2017, 08:29 am



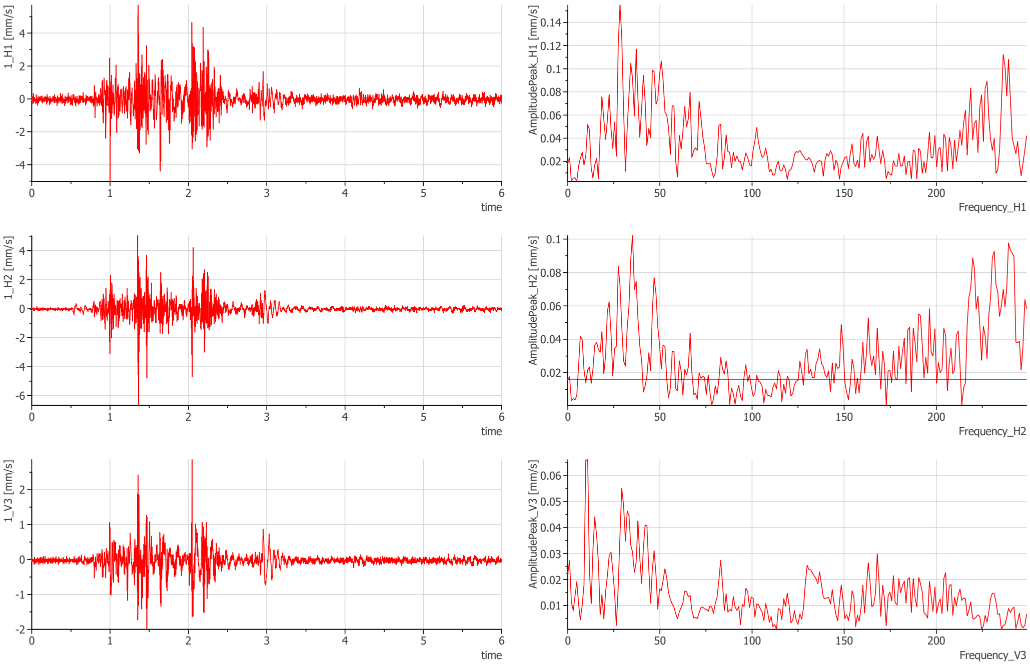
Event 13.06.2017, 08:33 am



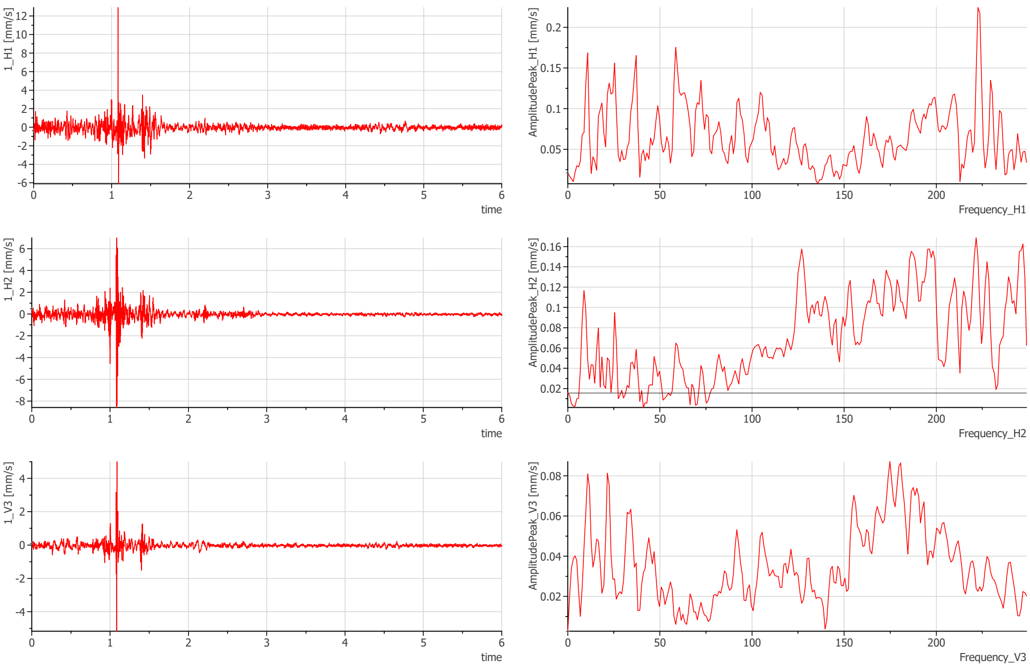
Event 14.07.2017, 07:51 am



Event 17.07.2017, 10:16 am



Event 18.07.2017, 08:02 am



Event 18.07.2017, 08:03 am